Engineering Principles and Practices

for Retrofitting Flood-Prone Residential Structures

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Existing Page(s)	Revised Page(s)	Change(s) Made
i	i	Table of Contents: corrected page numbers in Chapter I; changed "Model Building Codes" to "National Model Building Codes" in Chapter II
ii	ii	Table of Contents: updated page numbers in Chapter IV
v	v	List of Figures: added reference to new Figure III-7; updated page numbers in Chapter III
vi	vi	List of Figures: added references to new Figures IV-27 and IV-31; updated page numbers in Chapter IV
ix	ix	List of Figures: added references to new Figures VI-W6 and VI-W7; updated Figure names and page numbers in Chapter VI-W
xv	xv	List of Tables: added reference to new Table II-5, revised names of Tables II-3 and II-4, and updated paged numbers in Chapter II; revised name of Table III-4; added reference to new Table IV-5, revised name of Table IV-7, and updated page numbers in Chapter IV; revised names of Tables VI-1 and VI-2, and updated page numbers in Chapter VI; added reference to new Table VI-D.6, and updated page numbers in Chapter VI-D; updated page numbers in Chapter VI-F
xvi	xvi	List of Formulas: updated page numbers in Chapter VI-F
xvii	xvii	List of Formulas: replaced Formula IV-25 with references to IV-25 and IV-26; updated page numbers in Chapters IV and V
xviii	xviii	List of Formulas: added reference to new Formula VI-3, deleted Slenderness Ratio Formula VI-4; updated page numbers in Chapters VI, VI-W and VI-F
xix	xix	List of Formulas: updated page numbers in Chapter VI-F
xxi	xxi	Nomenclature: added the word "fresh" to explanations of γ, ρ; added "A_f – Bearing area of the footing"
xxii	xxii	Nomenclature: changed "B – Width of footing" to "b_f, B – Width of footing"
xxiii	xxiii	Nomenclature: added "DFE – Design Flood Elevation" and "DDF – Expected damage by flood depth"; deleted D_s
xxv	xxv	Nomenclature: deleted "FPE – Flood Protection Elevation" and "FPL – Flood Protection Level"; added "FW – Foundation weight"
xxvi	xxvi	Nomenclature: added $h_{f\text{-trans}}$, K_e, K_o; changed K_p to K_b; moved text from bottom of page xxvi to top of page xxvii
xxvi, xxvii	xxvii	Nomenclature: changed "Load" in L_o to lower case; deleted MDDF; added "NES – National Evaluation Service" and "OSW – Overbearing Soil Weight"; moved text from bottom of page xxvii to top of page xxviii
xxviii	xxviii	Nomenclature: changed "Soil" in S_a to lower case; moved text from bottom of page xxviii to top of page xxix
xxix	xxix	Moved text from bottom of page xxix to top of page xxx
xxx	xxx	Nomenclature: added Z, "W_f – Total weight the footing will support" and "W_w – Total weight the wall will support"
		CHAPTER I: Introduction to Retrofitting
I-2	I-2	Updated note to "FEMA-55 (Third Edition): *Coastal Construction Manual*"
I-2	I-2	Added note with reference to updated benefit/cost analysis
I-6	I-6	1st paragraph, 2nd sentence: changed "designated flood protection elevation (FPE)" to "Design Flood Elevation (DFE)"; added note regarding FEMA 347: *Above the Flood: Protecting Your Floodprone House*
I-7	I-7	Updated note resolving different acronym/usage between FEMA Publications: Design Flood Elevation (DFE), Flood Protection Elevation (FPE), or Flood Protection level (FPL)
I-10	I-10	Figure I-5: replaced graphic of structure elevated on posts with photograph
I-11	I-11	Added sentence to end of 3rd paragraph: "Jetting and augering piles reduces the uplift

		capacity compared to driven piles."
I-16	I-16	Table I-2: expanded 4th bullet in the "Advantages" section (define)
I-17	I-17	Added 4th paragraph updating the dry floodproofing research information (NES)
I-20	I-20	Added note regarding FEMA 348: *Protecting Building Utilities from Flood Damage* (reference)
I-22	I-22	Changed caution that levees and floodwalls are allowed under the NFIP, but do not make a non-compliant structure compliant
I-24	I-24	Figure I-14: improved photo quality to clarify figure
		CHAPTER II: Regulatory Framework
		Updated graphic on introduction page to include International Building Code (IBC) and International Residential Code (IRC); changed "Model Building Codes" to "National Model Building Codes"
		Updated chart under Model Building Codes to include National Model Building Codes including the International Building Codes and NFPA Codes
II-i	II-i	Table of Contents: changed "Model Building Codes" to "National Model Building Codes"
II-2	II-2	Added note with references to FEMA 213, *Answers to Questions about Substantially Damaged Buildings*, and FEMA 311, *Guidance on Estimating Substantial Damage*
II-3	II-3	Added paragraphs with brief narratives focused on repetitive loss, and possible funding sources (ICC, HMGP, FMA)
II-5	II-5	Figure II-1: increased scale to clarify figure
II-9	II-9	Figure II-3: increased scale and text size to clarify figure
II-12	II-12	Expanded the X Zone definition to include shaded and un-shaded X
II-17	II-17	Updated text in 4th paragraph: substantially improved pre-FIRM becomes post-FIRM.
II-18	II-18	Expand note to say some states have laws with higher standards than NFIP.
II-18	II-18	Introduced the IBC and ASCE standards in last paragraph.
II-20	II-20	Added new bullet about restrictions on use of fill (higher standard in some states and communities)
II-21	II-21	Replaced page with new text on National Model Building Codes (IBC, IRC, utility codes, and NFPA codes).
II-22	II-22	Updated text on code compliance with the NFIP
II-22	II-22	Replaced Table II-3 with a similar table showing National Model Building Codes and Model Building Codes
II-22	II-22	Converted note to new text on communities that amend the model codes and states that develop their own building codes. Several states have codes pending, and it is unknown whether they will be consistent with the NFIP.
II-23	II-23	Deleted caution; modified text and note on Code Compatibility and the NFIP to reflect I-Codes and need to reconcile with current version of model code.
II-24	II-24	Table II-4: Modified text references and the title of the table to "CODES vs. NFIP REQUIREMENTS: Items of Inconsistency"; correct "BOCO" should be "BOCA"; added section for I-Codes.
	II-25	Added new Table II-5 containing key NFIP provisions compared to IBC, IRC, ASCE 24-98 and other publications (page 1 of 3)
	II-26	Added new Table II-5 containing key NFIP provisions compared to IBC, IRC, ASCE 24-98 and other publications (page 2 of 3)
	II-27	Added new Table II-5 containing key NFIP provisions compared to IBC, IRC, ASCE 24-98 and other publications (page 3 of 3)
		CHAPTER III: Parameters of Retrofitting
III-9	III-9	Deleted note regarding aesthetics concerns
III-10	III-10	Changed caution to text indicating costs derived from FEMA 312. Added note listing additional references for costs to elevate substantially damaged structures.
III-10	III-10	Table III-1: updated costs based on FEMA 312 for year 2000; expanded list of assumptions

III-11	III-11	Table III-2: updated costs based on FEMA 312 for year 2000
III-12	III-12	Tables III-3: combined with Table III-4, updated costs, and added list of assumptions based on FEMA 312 for year 2000. Added new Table III-4 for wet floodproofing costs based on FEMA 312 for year 2000.
III-13	III-13	Moved text from bottom of page III-12 to top of page III-13
III-22	III-22	Added text from top of page III-23 to bottom of page III-22
III-23	III-23	Added new Figure III-7 to illustrate buoyancy forces lifting house off its foundation
III-24	III-24	Changed Figure III-7 to Figure III-8
III-27	III-27	Changed figures, text references from "Figures III-8 and III-9" changed to "Figures III-9 and III-10"
III-32	III-32	Moved text from bottom of page III-32 to top of page III-33
III-32	III-33	Added text from bottom of page III-32 to top of page III-33
III-33	III-33	Changed text reference from "Figure III-10" to "Figure III-11"
III-34	III-34	Changed Figure III-10 to Figure III-11
III-37	III-37	Added text to Erosion Forces section regarding FEMA's erosion mapping feasibility study
		CHAPTER IV: Determination of Hazards
IV-i	IV-i	Table of Contents: added sections on "Time/Duration of Impact" and "Combining Forces"; updated page numbers
IV-ii	IV-ii	Table of Contents: updated page numbers
IV-2	IV-2	Updated note to refer to ASCE 7-98 and ASCE 24-98
IV-3	IV-3	Added note with language discussing the inaccuracies of converting feet to inches
IV-6	IV-6	Revised note to include reference to *Coastal Construction Manual*
IV-8	IV-8	Changed "Flood Protection Elevation (FPE)" to "Design Flood Elevation (DFE)" in Formula IV-2 and text
IV-9	IV-9	Changed "FPE" to "DFE" in Formula IV-3
IV-9	IV-9	Figure IV-7: changed FPE to DFE and adjusted the lowest adjacent ground surface
IV-11	IV-11	Figure IV-9: changed "Flood Protection Elevation" to "Design Flood Elevation"
IV-11	IV-11	Formula IV-4: added the word "fresh" in the γ explanation
IV-12	IV-12	Formula IV-5: added reference to Table IV-2 (Column A) under definition of "S"; added text from page IV-13 to bottom of page IV-12
IV-13	IV-13	Figure IV-10: changed "Flood Protection Elevation" to "Design Flood Elevation"; moved text to bottom of page IV-12
IV-13	IV-13	Table IV-2: increased "S" values for sandy soils (Column A), added notes
IV-15	IV-15	Formula IV-6: added reference to Table IV-2 (Column B) under definition of "S"; added the word "fresh" in the γ explanation
IV-16	IV-16	Changed wording in note to say "may not" instead of "do not"
IV-16	IV-16	Figure IV-11: changed "Flood Protection Elevation" to "Design Flood Elevation"
IV-17	IV-17	Formula IV-8: added the word "fresh" in the γ explanation
IV-18	IV-18	Figure IV-12: added the word "fresh" in the γ explanation
IV-19	IV-19	Figure IV-13: added the word "fresh" in γ explanation; fixed computation error in F_{dif}, F_H
IV-20	IV-20	Added note regarding assumption of slab-on-grade construction for hydrodynamic force discussion
IV-20	IV-20	Figure IV-14: changed FPE to DFE and "h" to "H" in figure
IV-21	IV-21	Table IV-4: changed "b/h" to "b/H"
IV-21	IV-21	2nd paragraph text: changed "b/h" to "b/H" and "(h)" to "(H)"; clarified definition of H
IV-22	IV-22	Formula IV-10: added the word "fresh" in the γ explanation
IV-22	IV-22	Expanded note to include sketch showing application of forces
IV-23	IV-23	Figure IV-15: added the word "fresh" in the γ explanation; deleted "h" from list of variables; changed "b/h" to "b/H"
IV-24	IV-24	Figure IV-16: added the word "fresh" in the γ explanation; deleted "h" from list of variables; changed "b/h" to "b/H"; fixed error in computation of b/H, F_H

IV-25	IV-25	Formula IV-12: added the word "fresh" in the ρ explanation
IV-27	IV-27	Figure IV-17: added the word "fresh" in the ρ explanation; changed "b/h" to "b/H"
IV-28	IV-28	Figure IV-18: added the word "fresh" in the ρ explanation; changed "b/h" to "b/H"
IV-29	IV-29	Added note Reference to *Coastal Construction Manual* for information on impact loads
IV-30	IV-30	Changed text in 1st sentence from "ice blocks, logs, or floating objects" to "debris or floating objects"; added note on reduction of velocities for large debris objects
	IV-31	Added new text from *Coastal Construction Manual* regarding time/duration of debris impact
	IV-31	Added new Table IV-5 on impact durations for use in Formulas IV-14 and IV-15
IV-31	IV-32	Changed page number
IV-32	IV-33	Changed page number
IV-33	IV-34	Changed page number
IV-34	IV-35	Changed page number
IV-35	IV-36	Changed page number
IV-36	IV-37	Changed page number
IV-37	IV-38	Changed page number
IV-38	IV-39	Changed page number
IV-39	IV-40	Changed page number
IV-40	IV-41	Changed page number
IV-41	IV-42	Changed page number
IV-42	IV-43	Changed page number
IV-43	IV-44	Changed page number
IV-44	IV-45	Changed note reference from "Table IV-5" to "Table IV-6"
IV-45	IV-46	Changed Table IV-5 to Table IV-6
IV-46	IV-47	Changed page number
IV-47	IV-48	Changed page number
IV-48	IV-49	Added note referencing *Coastal Construction Manual* and ASCE 7-98
	IV-50	Added new text from the *Coastal Construction Manual* to expand discussion on wind design
IV-49	IV-51	Modified text in 1st paragraph to discuss IBC, ASCE 7-98; added text to last paragraph referencing BPAT reports from Hurricanes Andrew, Iniki, Opal, Fran and Georges
	IV-52	Added new Figure IV-27 containing wind speed map from ASCE 7-98 (page 1 of 2)
	IV-53	Added new Figure IV-27 containing wind speed map from ASCE 7-98 (page 2 of 2)
IV-50	IV-54	Expand top note referencing BPAT reports from Hurricanes Andrew, Iniki, Opal, Fran and Georges; update bottom note to include ASCE 7-98 reference
IV-50	IV-54	Changed text references from "Figures IV-27 and IV-28" to "Figures IV-28 and IV-29"
IV-51	IV-55	Changed Figure IV-27 to Figure IV-28
IV-52	IV-56	Changed Figure IV-28 to Figure IV-29
IV-52	IV-56	Updated note to include reference to ASCE 7-98 and ASCE 24-98
IV-52	IV-56	Moved 1st paragraph to bottom of page IV-54, and inserted new text and note on combining forces
IV-53	IV-57	Changed page number
IV-54	IV-58	Changed page number
IV-55	IV-59	Changed page number
IV-56	IV-60	Changed text reference from "Figure IV-29" to "Figure IV-30"; added text referencing new Figure IV-31
IV-57	IV-61	Changed Figure IV-29 to Figure IV-30: changed "Potential Front Action" to "Potential Frost Action"
	IV-62	Added new Figure IV-31 showing an example of a filled out geotechnical decision matrix
IV-58	IV-63	Added the word "allowable" before the words "bearing capacities" to section title and 1st sentence of 2nd paragraph; expanded note to include reference to FEMA 347; changed text reference from "Table IV-6" to "Table IV-7"
IV-59	IV-64	Changed Table IV-6 to Table IV-7 and "Bearing Pressure" to "Allowable Bearing Capacity"
IV-60	IV-65	Changed text references from "Figures IV-30 and IV-31" to "Figures IV-32 and IV-33"

IV-61	IV-66	Changed Figure IV-30 to Figure IV-32
IV-61	IV-66	Changed Figure IV-31 to Figure IV-33
IV-62	IV-67	Changed page number
IV-63	IV-68	Changed figure, text reference from “Figure IV-32” to “Figure IV-34”
IV-64	IV-69	Replaced scour equation (Formula IV-25) with Formulas IV-25 and IV-26, note and caution from *Coastal Construction Manual*; changed “Figure IV-33” to “Figure IV-35”
IV-65	IV-70	Last paragraph of existing page IV-64 now moved to revised page IV-70; changed Table IV-7 to Table IV-8; changed text reference from “Figure IV-34” to “Figure IV-36”
IV-66	IV-71	Changed figure, text reference from “Figure IV-34” to “Figure IV-36”
IV-67	IV-72	Changed figure, text reference from “Figure IV-35” to “Figure IV-37”
IV-68	IV-73	Changed page number
IV-69	IV-74	Changed formula, text references from “Formula IV-26” to “Formula IV-27”; changed text references from “Table IV-8” to “Table IV-9”
IV-70	IV-75	Changed page number
IV-71	IV-76	Changed Table IV-8 to Table IV-9
		CHAPTER V – Benefit/Cost Analysis and Alternative Selection
V-1	V-1	Updated text to reflect new BCA module on CD ROM and changed “maximum present value” to “net benefit”
V-2	V-2	Delete 1st paragraph, and change “whether or not” to “whether” in 2nd sentence of 3rd paragraph
V-3	V-3	1st paragraph: deleted reference to FIRM in 2nd sentence and delete 3rd sentence. 3rd paragraph: changed “presented in Appendix E” with “included in this publication”
V-4	V-4	Updated note to reflect new computer requirements and reference BCA modules; simplified last paragraph of text on ranking of alternatives
V-5	V-5	Updated last sentence of 2nd paragraph on reference to FEMA’s BCA program
V-6	V-6	2nd paragraph: changed end of last sentence from “elevation relationship.” to “elevation.”
V-7	V-7	Changed top note to referencing FEMA’s Map Service Center to include website address.
V-7	V-7	2nd paragraph: changed “gaging” to “gage” in 3rd sentence. Updated last two paragraph references to riverine and coastal flood zones, added reference to NAVD88.
V-8	V-8	Edited 1st paragraph and deleted 2nd paragraph on discussion of annual exceedence probabilities
V-9	V-9	Replaced 2nd paragraph regarding flood depth to damage curves derived from FIA data (Figure V-5)
V-10	V-10	Replaced Figure V-5 with new combined table containing building and contents damage functions from Excel BCA modules
V-11	V-11	Updated text reference to flood depth (-2 to >8 feet vs. –2 to 18 feet); edited last paragraph text on expected annual damages
V-12	V-12	3rd paragraph: deleted “the damage before mitigation and on” from 1st sentence; changed “benefit/cost ratio” to “benefit/cost” in last sentence
V-13	V-13	1st paragraph: deleted “designer’s/homeowner’s” from last sentence. 4th bullet: changed “benefit/cost ratios” to “net benefits (benefits minus costs)”. 5th bullet: “study”, “alternative” and “estimate” should be plural. 6th bullet: “model”, “figure” and “estimate” should be plural; change “ratios” to “net benefits”.
V-14	V-14	1st paragraph: updated reference to BCA software. 1st bullet: updated RENT description
V-15	V-15	Formula V-2: changed MDDF to DDF in formula and 1st bullet text
V-15	V-15	Formula V-3: modified wording of ECD description
V-16	V-16	Formula V-4: changed wording of DIS and DD in formula and 1st bullet text
V-16	V-16	Formula V-5: modified wording of DD description
V-17	V-17	Formula V-7: changed AVD from “(SCD)(EAE)(EFF)” to “(AD)(EFF)” in formula and second bullet text
V-18	V-18	Updated bullet reference to Appendix E and flood depth (-2 to >8 feet vs. –2 to 18 feet)
V-19	V-19	Updated note on reference to BCA program. 5th paragraph: deleted “maximum present

		value" from 1st sentence.
V-22	V-22	1st paragraph: changed 1st sentence from "fairly evaluate a number of" to "evaluate"
V-24	V-24	Alternative 1 Results, 2nd paragraph: deleted "or not" from last sentence.
V-25	V-25	1st paragraph: deleted "or not" from 1st sentence. 2nd paragraph: changed "cost and benefit/cost ratio" to "economic analysis" in last sentence. Deleted 1st bullet; changed 2nd bullet from "Benefit/Cost Ratio" to "Net Benefits"
V-25	V-25	Figure V-8: deleted 1st bullet; changed 3rd bullet from "Benefit/Cost Ratio" to "Net Benefits"; added bullet labeled "Aesthetics" after "Technical Feasibility"
V-26	V-26	Moved "Technical Feasibility" bullet from top of page V-26 to bottom of page V-25; updated preference scale factor descriptions in 1st paragraph.
V-27	V-27	Figure V-9: deleted "PW Benefits" column, changed "B/C Ratio" column to "Net Benefits", and updated total scores
V-30	V-30	2nd paragraph: changed "the extreme case" to "an extreme case" in last sentence; added text from top of page V-31
V-31	V-31	Moved text from top of pages V-32 and V-33 to bottom of page V-31
V-32	V-32	Figure V-11: increased size of figure; moved text above figure to bottom of page V-31
V-33		Deleted page – text moved to bottom of page V-31
		CHAPTER VI – General Design Principles
VI-i	VI-i	Table of Contents - General Design Practices: changed "Footings" to "Bearing Capacity of Footings" and "Bearing Capacity" to "Bearing Capacity of Foundation Wall"
VI-ii	VI-ii	Table of Contents - Elevation: updated page numbers in Construction Considerations Section
VI-v	VI-v	Table of Contents – Wet Floodproofing: revised section headings and page numbers
VI-vi, vii, viii	VI-vi	Table of Contents – Wet Floodproofing: revised section headings and page numbers; Floodwalls: moved section contents from pages VI-vii and VI-viii to VI-vi
VI-viii	VI-vii	Table of Contents – Levees: moved section contents from page VI-viii to VI-vii
VI-viii		Deleted page
VI-3	VI-4	Moved text from bottom of page VI-3 to top of page VI-4
VI-6	VI-6	Figure VI-2: clarified figure to include water, electrical, cable/TV features; changed "BFE" to "DFE"
VI-7	VI-8	Moved text from bottom of page VI-7 to top of VI-8
VI-16	VI-16	Added note to identify modes of building failure (footings/connections) and to explain how this section should be used
VI-16	VI-17	Moved text from bottom of page VI-16 to top of page VI-17
VI-19	VI-19	Figure VI-5: revised figure to more clearly show loads and reactions
VI-20	VI-20	Updated first note; heading: changed "Footings" to "Bearing Capacity of Footings"; Formula VI-1: added "See Table VI-2"; Formula VI-2: changed "A" to A_f"; changed "S" to "S_{bc}"; added the word "allowable" before the words "soil bearing capacity"; added words "See Table VI-2"
VI-21	VI-21	Added new Formula VI-3 defining bearing capacity of existing strip footing; changed Formula VI-3 to Formula VI-4; added note showing footing and downward (gravity) loading; added note referencing ACI 530 and local codes for design; heading: changed "Bearing Capacity" to "Bearing Capacity of Foundation Wall"; Formula VI-4: corrected formula by removing slenderness ratio; changed "W" to "W_w"; clarified definition of "A"
VI-22	VI-22	Updated note; deleted Formula VI-4; added an example
VI-23	VI-23	Added note with reference to ACI 530; added note with reference to the 2000 International Building Code; revised Table VI-1 to show bearing capacity for masonry walls; revised Table VI-2 to show soil bearing capacity
VI-24	VI-24	Step 2, 1st paragraph: deleted the introductory clause of 1st sentence, deleted last sentence
VI-24, 25	VI-24	Step 2, 2nd paragraph: deleted paragraph from body of the text, revised and added the information as a new note
VI-25	VI-24	Step 2, last paragraph: deleted paragraph from body of the text and placed as a note to reduce confusion

VI-27	VI-27	Added note with reference to ASCE 7
VI-28	VI-28	Moved "Total Weight" at bottom of column (4) to column (1) and changed to bold type
VI-30	VI-29	Moved text from top of page VI-30 to bottom of page VI-29
VI-30	VI-30	Revised paragraph above Formula VI-6 to remove confusion
VI-31	VI-30	Moved Formulae VI-7 , VI-8, and the paragraph between them from page VI-31 to VI-30
VI-31	VI-31	Enlarged Figures VI-7 and VI-8 to show column, wall, girder tributary areas more clearly
VI-33	VI-33	Revised Formula VI-10 to address overbearing soil weight and foundation weight mentioned in Step 4
VI-34	VI-34	Updated "ASCE 7-95" to "ASCE 7-98" in note and in 1st sentence of "Load Combination Scenarios"; 2nd paragraph: changed 1st word from "Model" to "International" and deleted "(BOCA, ICBO, SBCCI, CABO)" from 1st sentence and added "(IBC and IRC)"
VI-34	VI-35	Moved the word "designer" from the bottom of page VI-34 to the top of page VI-35
VI-35	VI-35	Updated "ASCE 7-95" to "ASCE 7-98" in 1st and last paragraphs; changed symbol for Roof Live Load from L_f to L_r
VI-36	VI-36	Updated the six equations listed under "Strength Design Method" using seven equations from ASCE 7-98; 2nd paragraph: added "**1**" after the word "**Exception**"; added paragraph discussing a 2nd exception; updated last paragraph to add ASCE 7-98 flood load information
VI-37	VI-37	Added sentence to the end of 1st paragraph for reference; updated the four equations listed under "Allowable Stress Method" using the five equations from ASCE 7-98; added paragraph after the 2nd paragraph discussing ASCE 7-98 flood load information
VI-37	VI-38	Moved text from bottom of page VI-37 to top of page VI-38; separated this paragraph to create a new paragraph for clarity
		CHAPTER VI-E – Elevation
VI-E.i	VI-E.i	Table of Contents - Elevation: updated page numbers in Construction Considerations Section
VI-E.6	VI-E.6	Figure VI-E4: changed "100-year flood level" to "DFE"; added anchor bolts; corrected color block in legend
VI-E.9	VI-E.9	Figure VI-E7: changed "100-year flood level" to "DFE"; added anchor bolts; corrected color block in legend
VI-E.13	VI-E.13	Figure VI-E10: changed "100-year flood level" to "DFE"; added anchor bolts
VI-E.14	VI-E.14	Figure VI-E11: changed "100-year flood level" to "DFE"; added anchor bolts; corrected color block in legend
VI-E.21	VI-E.21	Figure VI-E16: changed "100-year flood level" to "DFE"; corrected color block in legend; updated shading in figure
VI-E.24	VI-E.24	Figure VI-E19: changed "100-year flood level" to "DFE"; moved "Existing concrete slab" text and arrow from shaded area
VI-E.27	VI-E.27	Figure VI-E22: changed "100-year flood level" to "DFE"
VI-E.30	VI-E.30	Figure VI-E24: added the word "Combinations" after the words "Calculate Gravity Load" and after the words "Calculate Lateral Load" in second graphic box; changed text from "Truss" to "Roof Framing" in third graphic box
VI-E.34	VI-E.34	Updated note for IBC/IRC and ASCE 7-98
VI-E.35	VI-E.35	Updated note for ASCE 7-98
VI-E.38	VI-E.38	Updated "American Plywood Association" name to "Engineered Wood Association"; 4th paragraph, heading: changed "Truss" to "Framing"; 4th paragraph, 1st sentence: changed "trusses and truss" to "framing"
VI-E.39	VI-E.39	Updated "American Plywood Association" name to "Engineered Wood Association"; updated note for the National Evaluation Service (NES)
VI-E.40	VI-E.40	Updated "American Plywood Association" name to "Engineered Wood Association"
VI-E.42	VI-E.42	Revised last paragraph to discuss appropriate load combination and design methods used.
VI-E.46	VI-E.46	Sample Elevation Calculation: changed flood zone "A4" to "AE"; added requirement for 1' freeboard; labeled 1' roof overhang and adjusted elevations on drawing
VI-E.47	VI-E.47	Sample Elevation Calculation: updated wind and seismic load requirements per ASCE 7-98 and 2000 IBC; adjusted elevations on drawing

VI-E.48	VI-E.48	Sample Elevation Calculation: changed calculation page number
VI-E.49	VI-E.49	Sample Elevation Calculation: changed H to 5' for freeboard; revised buoyancy calculation
VI-E.50	VI-E.50	Sample Elevation Calculation: changed calculation page number
VI-E.51	VI-E.51	Sample Elevation Calculation: changed calculation page number
VI-E.52	VI-E.52	Sample Elevation Calculation: updated dead load calculation to include footing and slab weight; revised buoyancy force vs. structure weight check
VI-E.53	VI-E.53	Sample Elevation Calculation: revised hydrostatic forces to account for freeboard
VI-E.54	VI-E.54	Sample Elevation Calculation: updated total flood force on building face upstream; deleted sliding check
VI-E.55	VI-E.55	Sample Elevation Calculation: revised normal impact force based on impact duration; revised wind load calculations per ASCE 7-98
VI-E.56 thru 59	VI-E.56 thru 59	Sample Elevation Calculation: revised wind load calculations and sketches per ASCE 7-98 (MWFRS and Components and Cladding)
VI-E.59 thru 62	VI-E.60, 61, 62	Sample Elevation Calculation: revised seismic load calculations per 2000 IBC and wind load calculations per ASCE 7-98
VI-E.63 thru 66	VI-E.63, 64	Sample Elevation Calculation: updated wind and seismic forces perpendicular and parallel to the long direction
	VI-E-65, 66	Sample Elevation Calculation: added check of load combinations per ASCE 7-98 (ASD); added checks for overturning, sliding, and uplift buoyancy
VI-E.67, 68	VI-E.67, 68	Sample Elevation Calculation: updated check of existing foundation based on revised dead load and load combinations from ASCE 7-98
VI-E.68 thru 78	VI-E.69 thru 76	Sample Elevation Calculation: updated design of new foundation wall per ACI 530-99 and IBC
VI-E.78 thru 81	VI-E.76, 77	Sample Elevation Calculation: updated design of top wall connection per ACI 530-99
VI-E.81 thru 89	VI-E.78, 79, 80	Sample Elevation Calculation: updated sample details for revised design
VI-E.90	VI-E.81	Changed page number
VI-E.91	VI-E.82	Changed page number
VI-E.92	VI-E.83	Changed page number
VI-E.93	VI-E.84	Changed page number
VI-E.94	VI-E.85	Changed page number
		CHAPTER VI-R – Relocation
VI-R.3, 4	VI-R.3	Updated note to include web site address; moved text from top of page VI-R.4 to bottom of page VI-R.3
		CHAPTER VI-D – Dry Floodproofing
VI-D.6	VI-D.6	Figure VI-D2: changed "Flood Protection Elevation (includes freeboard)", to "Design Flood Elevation", added depiction of maximum height and the words "Maximum Height = 3'"
VI-D.7	VI-D.7	Figure VI-D3: deleted "1' Freeboard" and "Flood Elevation", changed "Flood Protection Elevation (Includes Freeboard)" to "Design Flood Elevation", added depiction of maximum height and the words "Maximum Height = 3'"
VI-D.8	VI-D.8	Figure VI-D5: deleted "Flood Elevation" and "1' Freeboard", changed "Flood Protection Elevation" to "Design Flood Elevation", added depiction of maximum height and the words "Maximum Height = 3'"
VI-D.9	VI-D.9	Figure VI-D6: added depiction of maximum height and the words "Max. Height = 3'", added words "Design Flood Elevation"; Figure VI-D7: added depiction of maximum height and the words "Maximum Height = 3'", added the words "Design Flood Elev."
VI-D.10	VI-D.10	Figure VI-D.8: changed floor drain to connect to sewer line, changed "FPE" to "DFE"
VI-D.11	VI-D.11	Figure VI-D.9: changed "FPE" to "DFE"
VI-D.12	VI-D.12	Changed "FPE" to "DFE" in 6th bullet
VI-D.13	VI-D.13	Added the words "or NES" at end of note

VI-D.17	VI-D.17	Figure VI-D.11: adjusted figure, added note
VI-D.20	VI-D.20	Step 6, b.5: changed the L in Loads to lower case, clarified pathway out of structure
VI-D.33	VI-D.34	Moved the words "of the" from bottom of page VI-D.33 to top of page VI-D.34
VI-D.51	VI-D.51	Added note regarding the building code requirements for maximum height of unbalanced fill
VI-D.53	VI-D.53	Separated paragraph into two paragraphs; edited 2nd sentence in 1st paragraph for clarity
VI-D.59	VI-D.59	Revised last sentence in 1st paragraph to say "1/4 horsepower" instead of "1/6 horsepower"
VI-D.66	VI-D.66	Edited text in second bullet to include pipe transitions
VI-D.66	VI-D.66	Formula VI-D1: changed "D_s" to "Z" and added "$h_{f\text{-trans}}$" to formula, descriptions
VI-D.67	VI-D.67	Edited text in first paragraph include pipe transitions; added caution listing assumptions used in formula
VI-D.67	VI-D.67	Formula VI-D2: changed formula to compute "$h_{f\text{-fitting}} + h_{f\text{-trans}}$"; changed "$K_p$" to "$K_b$"; added entrance and exit loss coefficients, descriptions
VI-D.68	VI-D.68	Updated sample calculations to account for revised formulas
VI-D.76	VI-D.76, 77	Moved text from bottom of page VI-D.76 to top of page VI-D.77
VI-D.80	VI-D.81	Moved the word "chief" from the bottom of page VI-D.80 to top of page VI-D.81
VI-D.84	VI-D.84	Added clarification in the note at the bottom of Table VI-D3 to identify that many appliances cycle on and off
		CHAPTER VI-W – Wet Floodproofing
VI-W.i, ii	VI-W.i	Table of Contents – Wet Floodproofing: revised section headings and page numbers
VI-W.1	VI-W.1	Changed "FPE" to "DFE"
VI-W.3	VI-W.3	Changed "#7-93" to "7-93"
VI-W.4	VI-W.4	Changed "#1-93" to "1-93" in note; changed "FPE" to "DFE" in heading, 1st paragraph of text
VI-W.4	VI-W.4	Figure VI-W1: changed "BFE" to "DFE"
VI-W.5, 6	VI-W.5	Moved entire text and notes from page VI-W.6 to VI-W.5
VI-W.7, 8	VI-W.6	Changed page number; moved entire text from pages VI-W.7 and W.8 to VI-W.6; deleted the word "FLOOD" in the heading before the last paragraph
VI-W.9	VI-W.7	Changed "SERVICE EQUIPMENT" to "UTILITY SYSTEMS" in the main heading and note; added new note to reference FEMA 348; changed "RELOCATION" heading to "ELEVATION"; edited the two paragraphs under the new "ELEVATION" heading
VI-W.10	VI-W.8	Deleted bullet item from list; replaced Figure VI-W4 with a more clear graphic; added new note referencing local code requirements
VI-W.11	VI-W.9	Deleted "Wet Floodproofing" from 2nd paragraph; changed "FPE" to "DFE" in second bullet; edited text for 3rd bullet; added new note showing basic protection process
VI-W.12, 13	VI-W.10, 11, 12	Replaced Figure VI-W5 with an updated worksheet
VI-W.14	VI-W.13	Edited 1st paragraph; added graphical representation of design process; revised bullet list items
VI-W.15		Deleted page
VI-W.16		Deleted page
VI-W.17		Deleted page
VI-W.18		Deleted page
VI-W.19	VI-W.14	Deleted bullet items at top of page; changed the words "energized", "de-energized", and "overcurrent" to "active", "inactive", and "overload", respectively, in 1st paragraph; changed "FPE" to "DFE" in 1st bullet; deleted clause at end of 3rd bullet; replaced "ground fault circuit interceptors" with "Ground Fault Interrupting Circuit (GFIC) breakers" in last bullet item; replaced "de-energize" with "deactivate" in last bullet item
VI-W.20, 21	VI-W.15	Replaced "FPE" with "DFE" in 1st paragraph and in bullet items; edited 4th and 7th bullet items; moved bullet items from bottom of page VI-W.20 and top of page VI-W.21 to bottom of page VI-W.15; added new note referencing telephone and cable TV systems
VI-W.21		Deleted 2nd bullet item listed; deleted 1st paragraph and all bullet items following it

VI-W.22	VI-W.16	Changed heading name from "CENTRAL HEATING SYSTEM ALTERNATIVES" with "HEATING, VENTILATING, AND AIR CONDITIONING (HVAC) SYSTEMS"; edited 1st paragraph; revised bullet items following 1st paragraph; deleted note; added new note regarding ductwork
VI-W.23	VI-W.16	Moved note from page VI-W23 to page VI-W16 and edited it
VI-W.23		Deleted page
VI-W.24		Deleted page
VI-W.25		Deleted page
VI-W.26		Deleted page
VI-W.27		Deleted page
VI-W.28		Deleted page
VI-W.29		Deleted page
VI-W.30	VI-W.17	Deleted sentence at top of page and the following five bullet items; changed heading name from "Fuel Supply/Storage Applications" to "FUEL SUPPLY/STORAGE SYSTEMS"; edited 1st paragraph; revised bullet items following 1st paragraph
VI-W.32	VI-W.17	Moved "WATER SYSTEMS" heading from page VI-W.32 to page VI-W.17; revised 1st paragraph following the heading
VI-W.31		Deleted page
VI-W.32		Deleted page
VI-W.33		Deleted page
VI-W.34	VI-W.18	Added bullet items at top of page; added new Figure VI-W6; moved "SEWER SYSTEMS" heading from page VI-W.34 to page VI-W.18; revised 1st paragraph following the heading; added note referencing pipe and tank protection
VI-W.35	VI-W.18	Moved note from page VI-W.35 to page VI-W.18
VI-W.35		Deleted page
VI-W.36	VI-W.19	Deleted the two paragraphs; added new Figure VI-W7; deleted page VI-W.36
VI-W.36	VI-W.20	Moved note from page VI-W.36 to page VI-W.20 and edited it; moved Formula VI-W1 from page VI-W.36 to VI-W.20 and edited it; added new note regarding "topping off" tanks; added new note referencing FEMA 348; added "Calculation of Buoyancy Forces" heading at top of page
VI-W.37	VI-W.21	Edited Formula VI-W2; deleted "TELEPHONE SYSTEMS" heading and the two paragraphs following it; added "CONSTRUCTION/IMPLEMENTATION" heading and two paragraphs following it
VI-W.38		Deleted page
VI-W.39		Deleted page
VI-W.40		Deleted page
		CHAPTER VI-F – Floodwalls
VI-F.1	VI-F.1	Revised note about floodwalls to address compliance
VI-F.2	VI-F.2	Revised note to reference NFIP regulations
VI-F.2	VI-F.2	Figures VI-F1 and VI-F2: lightened images for clarity
VI-F.3	VI-F.3	Figures VI-F3 and VI-F4: changed "FPE" to "DFE"
VI-F.4	VI-F.4	Figure VI-F5: changed "FPE" to "DFE"
VI-F.5	VI-F.5	Figure VI-F6: changed "FPE" to "DFE"
VI-F.6	VI-F.6	Figure VI-F7: changed "FPE" to "DFE"
VI-F.8	VI-F.8	Figure VI-F9: changed "FPE" to "DFE"
VI-F.12	VI-F.12	Figure VI-F11: changed "FPE" to "DFE"
VI-F.13	VI-F.13	Figure VI-F12: changed "Flood Protection Elevation" to "DFE"
VI-F.13	VI-F.13	Updated text in 1st paragraph, second sentence on reducing seepage pressures
VI-F.16	VI-F.16	Figures VI-F14, VI-F15 and VI-F16: changed "FPE" to "DFE"
VI-F.19	VI-F.19	Figure VI-F17: changed "FPE" to "DFE"
VI-F.35	VI-F.35	Figure VI-F18: Added "DFE"
VI-F.52	VI-F-52	Revised caution to reference the Engineered Wood Association guidelines

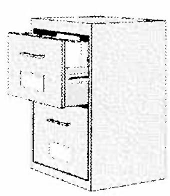
VI-F.56	VI-F.56	Figure VI-F21: relocated roof downspout outside of patio drainage area
VI-F.57	VI-F.57	Figure VI-F22: relocated roof downspout outside of patio drainage area
VI-F.59	VI-F.59	Figure VI-F26: changed "FPE" to "DFE"
		CHAPTER VI-L – Levees
VI-L.1	VI-L.1	Revised note about levees to address compliance
VI-L.12	VI-L.12	2nd paragraph: changed "(based on 1985 prices)" to "(in 2000 dollars)", added the words "in 1985" before the words "to protect"; Table VI-L.2: updated costs, changed row name from "Strip Topsoil" to "Strip Topsoil & Clear & Grub", changed row name from "Total First Cost" to "Total Cost"
VI-L.13	VI-L.13	Changed header name from "Seepage Concerns" to "Cost"; revised Table VI-L3 to clarify items and to update costs and units
		APPENDIX A - National Flood Insurance Program
A-2	A-2	Revised date referenced in note; updated cost information in the hypothetical example
A-3	A-3	Updated cost information in text
A-4	A-4	Figure A-1: updated costs and the date referenced at bottom of the figure
A-5	A-5	Figure A-2: updated costs and the date referenced at bottom of the figure; 1st paragraph, 1st sentence: updated cost
		APPENDIX B – Glossary of Terms
B-1, 2	B-1	Moved "Caulking" from top of page B-2 to bottom of page B-1
B-2	B-2	Added "Design Flood Elevation (DFE)"
B-3, 4	B-3	Moved "Flood Depth" from top to page B-4 to bottom of page B-3
B-5	B-5	Deleted "Flood Protection Elevation"
B-5, 6	B-5	Moved "Interior Grade Beam" from top to page B-6 to bottom of page B-5
		APPENDIX C - Glossary of Resources
C-3	C-3	Deleted hyphens in "FEMA-114" and "FEMA-54" in 3rd and last text paragraphs, respectively
C-4	C-4	Deleted hyphen in "FEMA-102" in 1st paragraph; updated publication description of "Coastal Construction Manual", FEMA 55, in 2nd paragraph; added paragraphs for publication descriptions of FEMA 312, FEMA 347 and FEMA 348
C-4	C-5	Moved text from bottom of page C-4 to top of page C-5
C-16	C-16	Changed heading from "FEMA REGIONAL OFFICES" to "FEMA OFFICES" and updated list of offices based on information from the *Coastal Construction Manual*
	C-17	Added new page to list "INFORMATION SOURCES" taken from the *Coastal Construction Manual*
	C-18	Added new page to list "INFORMATION SOURCES" taken from the *Coastal Construction Manual* including link to NFPA
C-17	C-19	Changed page number
C-18	C-20	Changed page number
C-19	C-21	Changed page number
C-20	C-22	Changed page number, moved text from bottom of page C-20 to top of C-23
C-21	C-23	Changed page number, added reference to FEMA 347-VT
C-22	C-24	Changed page number
C-23	C-25	Changed page number
C-24, 25	C-26	Changed page number; moved text from top of page C-25 to bottom of page C-26
C-25, 26	C-27	Changed page number; moved text from top of page C-26 to bottom of page C-27
C-26	C-28	Changed page number
C-27	C-29	Changed page number
C-28	C-30	Changed page number

		APPENDIX D - Alluvial Fan Flooding
D-8	D-8, 9	Moved text from bottom of page D-8 to top of page D-9
D-19	D-18	Moved text from top of page D-19 to bottom of page D-18
D-20	D-19	Moved text from top of page D-20 to bottom of page D-19

Table of Contents

Page

Page



LIST OF FIGURES

LIST OF FIGURES (continued) **Page**

LIST OF FIGURES (continued) **Page**

LIST OF FIGURES (continued) **Page**

LIST OF FIGURES (continued) **Page**

LIST OF FIGURES (continued) **Page**

LIST OF FIGURES (continued) **Page**

LIST OF FIGURES (continued) **Page**

LIST OF FIGURES (continued) **Page**

LIST OF TABLES

LIST OF TABLES (continued)

LIST OF FORMULAS

LIST OF FORMULAS (continued) **Page**

LIST OF FORMULAS (continued)

NOMENCLATURE

α	Deflection coefficient
β	Moment coefficient
γ	Specific weight of fresh water
γ_s	Specific weight of the water/sediment mixture
γ_{soil}	Unit weight of soil
σ	Allowable stress for closure plate material
Δ_b	Computed deflection
Δ_{ba}	Allowable deflection
ρ	Mass density of fresh water
a	Diameter of post
A	Area
a,b	Span lengths between the walls and center girder
AB	Expected Annual Benefits
$A_{a,b,c}$	Specific Area enclosed by a floodwall/levee
A_c	Column tributary Area
AD	Expected Annual Damages
A_f	Bearing area of the footing
A_g	Center girder tributary Area
A_h	Width of footing heel
A_s	Cross-sectional Area of reinforcing steel required per foot width of wall
A_t	Column tributary Area
AVD	Expected Avoided Damages
A_w	Wall tributary Area

NOMENCLATURE (continued)

b	Width of object (structure) perpendicular to flow
b_f, B	Width of footing
BCR	Benefit/Cost Ratio
BD	Scenario Building Damages
BF	Bulking Factor
BFE	Base Flood Elevation
BRV	Building Replacement Value
c	Residential terrain runoff coefficient
C	Width of footing toe
C_d	Drag Coefficient
CD	Scenario Contents Damages
C_f	Coefficient of friction
CF	Cubic Foot
CFR	Code of Federal Regulations
cfs	Cubic feet per second
CR	Building Contents Replacement value
C_{rs}	Rolling shear Constant
C_s	Allowable soil Cohesion value
C_v	Concentration of sediment in the fluid mixture by percent of volume
d	Depth of flooding
D	Depth of saturated soil over which hydrostatic forces are considered

NOMENCLATURE (continued)

$d_{1.5Q\ design}$	Depth of flooding from a discharge 50% greater than the design discharge
D_a	Height of vertical foundation member above grade
D_b	Depth of burial of vertical foundation member
DD	Displacement Days
DDF	Expected damage by flood depth
d_f	Distance between reinforcing steel and floodwall face opposite retained material
DFE	Design Flood Elevation
dh	Equivalent head due to low velocity flood flow
D_h	Depth of soil above the floodwall heel
DIS	Scenario Displacement costs
DL	Dead Load
$d_{Q\ design}$	Depth of flooding from the design discharge
DRR	Daily Rental Rate
D_t	Depth of soil above the floodwall toe
e	Eccentricity
E	Modulus of Elasticity
EAB_{PV}	Present value of Estimated Annual Benefits
EAE	Expected annual number of floods of a given depth
EAC_{PV}	Present value of Estimated Annual Costs
ECC_{PV}	Present value of Engineering and Construction Costs associated with a retrofitting measure

NOMENCLATURE (continued)

ECD	Expected Contents Damage
EFF	Effectiveness of mitigation measure in reducing expected damages from a flood of a given depth
FA	Floor Area of building in square feet
f	Freeboard (factor of safety)
f_b	Bending stress
F_b	Vertical hydrostatic Force (buoyancy)
FBFM	Flood Boundary and Floodway Map
F_{b1}	Buoyancy Force acting on a floodwall heel
F_{b2}	Buoyancy Force acting on a floodwall toe
f_c	Bearing capacity of masonry
F_c	Cohesion Force between the footing and the soil
F_d	Hydrodynamic Force
F_{dh}	Equivalent hydrostatic Force due to low velocity flood flow
F_{dif}	Differential soil/water Force
FE	Flood Elevation for a specific flood frequency
FEMA	Federal Emergency Management Agency
F_{fr}	Frictional Force between the bottom of the footing and the soil
F_h	Lateral hydrostatic Force from standing water
F_H	Cumulative lateral Hydrostatic Force
FIA	Federal Insurance Administration
FIRM	Flood Insurance Rate Map

NOMENCLATURE (continued)

FIS	Flood Insurance Study
F_n	Normal impact load
F_p	Saturated soil Force over the toe of the footing
fps	Feet per second
F_R	Sum of Resisting Forces to sliding
f_s	Shear stress
F_s	Special impact load
F_{ss}	Maximum Shear Stress
FS	Factor of Safety
F_{sat}	Lateral hydrostatic Force from saturated soil
$FS_{(OT)}$	Factor of Safety against Overturning
$FS_{(SL)}$	Factor of Safety against Sliding
F_v	Net Vertical Force
FW	Foundation weight
g	Acceleration of gravity
gpm	Gallons per minute
GS	Lowest Ground Surface elevation (grade) or other reference feature (slab or footing) adjacent to structure
h	Distance from bottom of structure to water level
H	The floodproofing design depth over which flood forces are considered
h_c	Height of closure

NOMENCLATURE (continued)

$h_{f\text{-}fittings}$	Head loss through the pipe fitting(s)
$h_{f\text{-}pipe}$	Head loss due to pipe friction
$h_{f\text{-}trans}$	Head losses due to pipe transitions
H_1	Height of unbraced foundation wall
i	Interest rate
I	Effective moment of Inertia
i_r	Rainfall intensity
i_{hg}	Hydraulic gradient between two points
I_{PD}	Post-Disaster Inflation
k	Coefficient of permeability for soils
K	Scour factor based upon flow angle of attack
K_b	Loss coefficient of the pipe fitting(s)
K_e	Loss coefficient of pipe entrance
K_o	Loss coefficient of pipe exit/outlet
k_p	Passive soil pressure coefficient
KS	Effective section modulus
l	Length
lbs	Pounds
lbs/ft^3	Pounds per cubic foot
LF	Linear Feet
LL	Live Load

NOMENCLATURE (continued)

L_o	Minimum uniformly distributed live load
M	Mass of object
max	Maximum flood depth considered above zero flood depth
M_b	Bending Moment
min	Minimum damaging flood considered above zero flood depth
M_O	Sum of Overturning Moments
M_R	Sum of Resisting Moments
MSL	Mean Sea Level
n	Assumed life of a structure
NAVD	North American Vertical Datum
NES	National Evaluation Service
NFIP	National Flood Insurance Program
NGVD	National Geodetic Vertical Datum
NOAA	National Oceanic and Atmospheric Administration
NPV	Net Present Value or benefit of a mitigation measure
NRCS	Natural Resources Conservation Service
NWS	National Weather Service
OSW	Overbearing Soil Weight
P	Load
P_d	Hydrodynamic Pressure due to high velocity flow flood
P_D	Lateral hydrostatic Pressure from saturated soil
P_{dh}	Hydrostatic Pressure due to low velocity flood flows

NOMENCLATURE (continued)

P_h	Hydrostatic Pressure from standing water
psi	Pounds per square inch
PWF	Present Worth Factor
q	Soil pressure
Q	Discharge in a given unit of time
$Q_{a,b,c}$	Runoff Quantity (discharge) from a defined area
Q_{BC}	Allowable Bearing Capacity
Q_{sp}	Minimum discharge for sump pump installation
Q_u	Ultimate bearing capacity
RENT	Scenario rental income losses
R_f	Resistance due to foundation friction
RF	Flood depth considered above zero flood depth
s	Slenderness ratio
S_a	Allowable soil bearing pressure (capacity)
SA	Section Area of component
S_{bc}	Soil bearing capacity
S_c	Effective (unit) weight of concrete
SCD	Total Scenario Damages (per event)
s_d	Potential scour depth
SF	Square Foot (feet)
SFHA	Special Flood Hazard Area
S_g	Unit weight of wall material

NOMENCLATURE (continued)

SL	Snow Load
s_{max}	Maximum potential depth of scour hole
S_p	Specific gravity of sediment
Sq. Mi.	Square Mile
sr	Seepage rate
SW	Self Weight of component
t	Time of impact
TA	Total Area occupied (SF)
TDC	Displacement Costs per day (per SF)
t_{ftg}	Footing thickness
TH	Total Head
TL	Total Load
TL_{dls}	Total Load due to dead, live, and snow loads
TVA	Tennessee Valley Authority
t_w	Foundation wall thickness
t_{wall}	Floodwall thickness
UC_{FEMA}	FEMA Unit Cost at specific location
UC_{local}	Unit Cost for a locality
USACE	U.S. Army Corps of Engineers
USGS	United States Geological Survey
V	Velocity of floodwater
V_c	Volume of concrete required to offset tank buoyancy

NOMENCLATURE (continued)

V_s	Shear force
V_t	Volume of tank
w	Span lengths between walls or wall and girder
W_a	Total gravity forces
W_c	Width of closure shield
W_f	Total weight the footing will support
W_{ftg}	Weight of the footing
W_g	Total gravity forces per linear foot of wall
w_n	Weight of object for normal impact loads
W_1	Weight
WPI_{FEMA}	FEMA Wholesale Price Index for a locality
WPI_{local}	Wholesale Price Index for a locality
w_s	Weight of object for special impact loads
W_{sh}	Weight of soil over floodwall heel
W_{st}	Weight of soil over floodwall toe
W_t	Weight of tank
W_u	Unit weight of component
W_w	Total weight the wall will support
W_{wall}	Weight of floodwall
W_{wh}	Weight of water above floodwall heel
y	Support width factor
Z	Difference in elevation between the bottom of the sump and the point of discharge

FOREWORD

The riverine and coastal floodplains of the United States are among the most highly desirable areas in the nation for habitation and construction. Unfortunately, many of these areas are very susceptible to flooding, which is the single most expensive and persistent natural disaster the country experiences. Flooding causes millions of dollars in property damage each year, despite concentrated efforts of government and the private sector to mitigate flood hazards.

The National Flood Insurance Program (NFIP) was created in 1968 by the Congress not only to provide federally-backed flood insurance to those who generally were not able to obtain it from private-sector companies, but also to promote sound floodplain management practices in flood-prone areas. The floodplain management aspects of the program are administered by the Mitigation Directorate and the insurance aspects are administered by the Federal Insurance Administration (FIA), both parts of the Federal Emergency Management Agency (FEMA), under the authority of the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973, U.S.C. 4001-4128, as amended.

Figure v-1: Flooding along major rivers can create widespread damage.

One NFIP mission is to work with communities to reduce future flood losses by establishing guidelines for protecting existing and new development in flood-prone areas. The program makes flood insurance coverage available for structures in those communities that adopt and enforce floodplain management ordinances and regulations that meet or exceed the minimum NFIP requirements as provided for in Section 44 of the Code of Federal Regulations (44 CFR). Coverage is available for walled and roofed structures that are principally above ground and not entirely over water, including manufactured homes that are anchored to permanent foundations. Flood insurance is available for all structures in a participating community, whether the structures are located inside or outside the floodplain identified by FEMA.

Owners who have experienced flooding know that complete recovery is often impossible. In addition to the time and money spent repairing or replacing damaged items, they must also deal with cleaning property, alleviating health risks and safety hazards, losing time from work, finding alternative housing, and the emotional toll of the experience. Responding to flood events also depletes resources at every level of government. Human resources and capital must be diverted to providing emergency services, rebuilding public facilities, financing individual assistance for uninsured victims, and to other efforts. In the Great Midwest Flood of 1993, for example, FEMA estimated damage costs exceeded $10 billion.

Many of the flood insurance claims received by the NFIP are for structures that have previously incurred flood damage. Structures for which two or more claims of more than $1,000 each have been paid during the previous ten-year period are considered to be repetitive loss structures according to the NFIP. Most repetitive loss claims are for small amounts and involve structures built before NFIP-compliant floodplain management regulations were adopted by the community. However, owners have the option of taking steps to reduce the likelihood of serious future flood damage. Retrofitting individual flood-prone structures is a proven technology that has been in use for many years.

If a flood-prone structure is substantially damaged, certain criteria established by the NFIP must be met prior to the initiation of any repair activity. Specifically, NFIP regulation 44 CFR 60.3(c)(2) requires communities to ensure that substantially damaged or improved residential structures be elevated so that the lowest floor is at or above the Base Flood Elevation, (BFE), also known as the 100-year flood level. "Substantially damaged" is defined as damage of any origin sustained by a structure whereby the cost of restoring the structure to its before-damaged condition would equal or exceed 50% of the value of the structure before the damage occurred.

Given the potential cost of recovering from a serious flood event and meeting the NFIP's criteria for restoring substantially damaged property, the owner of a flood-prone home has an incentive to undertake retrofitting measures to limit future flood damages. FEMA and the other contributing agencies and organizations have developed this manual to provide engineering and related economic guidance to professional designers and local officials about what constitutes technically feasible and cost-effective retrofitting techniques.

However, the guidance provided in this manual should be considered generic in nature, subject to final refinement in accordance with local regulations and specific site and structural conditions. It is not intended to be used as a code or specification, nor as a replacement for the engineer's or architect's standard of performance. Through the information and analyses presented in this manual, local officials, and design professionals will gain a better understanding of the advantages of retrofitting and may choose to take steps that could ultimately save the nation millions of dollars each year.

ACKNOWLEDGMENT

FEMA acknowledges the following agencies and organizations for their contributions to this manual:

- Brudis & Associates, Inc.
- Dewberry & Davis LLC
- Flo Engineering, Inc.
- French & Associates, Ltd.
- National Association of Home Builders, (NAHB), Research Center
- National Institute of Building Sciences (NIBS)
- Soza & Company, Ltd.

FEMA also acknowledges Greenhorne & O'Mara, Inc., and Dewberry & Davis LLC for their contributions to the Second Edition of this manual.

METRIFICATION

FEMA is committed to the federal government's transition to metr units remain the standard of practice for residential construction. T has been prepared using English units.

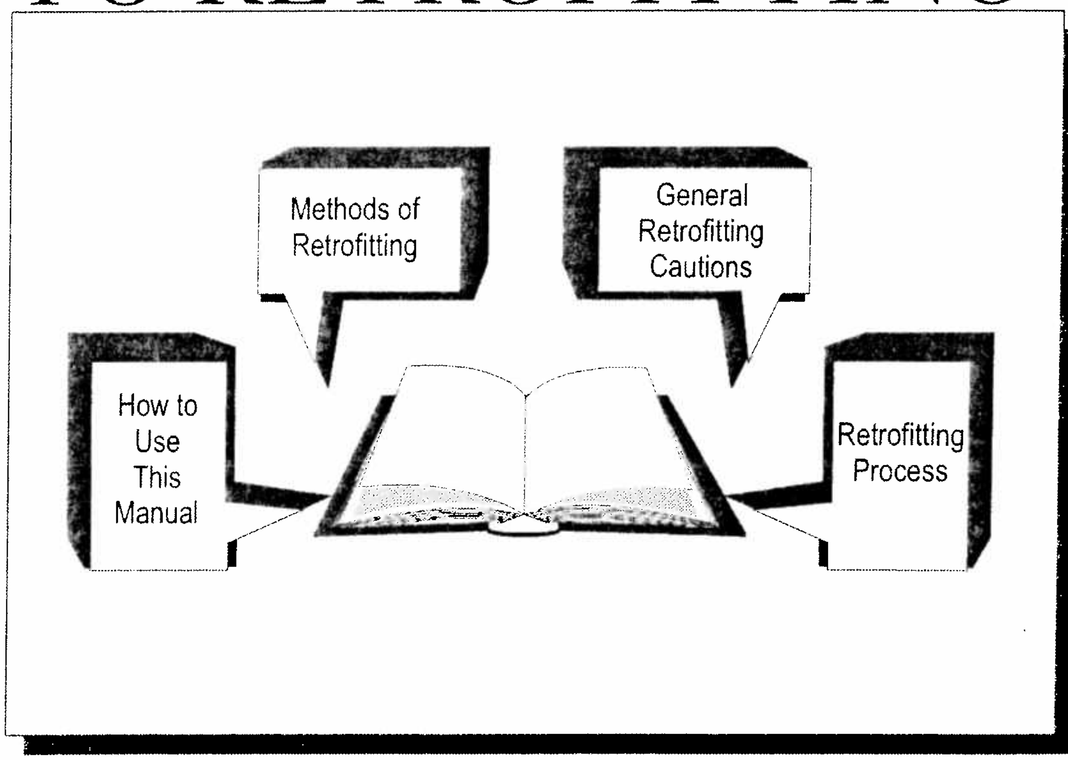
However, it is foreseeable that the metric system will be the standa this country within the next few years. With this in mind, soft met been provided to promote familiarity with the metric system.

A critical component of unit conversion is rounding. Designers sh that rounding does not exceed allowable tolerances for design or fa

Metric Conversion Factors			
Quantity	**From English Units**	**To Metric Units**	
Length	foot inch	(m) (mm)	
Area	square foot acre	m^2 m^2	
Volume	gallon cubic foot	L m^3	
Pressure	psf psi	Pa kPa	
Power	horsepower	kW W	
Weight	pounds	kg	
Flow	cfs	lps	
Velocity	fps	mps	

CHAPTER I

INTRODUCTION TO RETROFITTING



Featuring:

How to Use This Manual
Methods of Retrofitting
General Retrofitting Cautions
Retrofitting Process

INTRODUCTION TO RETROFITTING

- HOW TO USE THIS MANUAL
  - Goals and Intended Users
  - Organization of the Manual
- METHODS OF RETROFITTING
  - Elevation
  - Relocation
  - Dry Floodproofing
  - Wet Floodproofing
  - Floodwalls and Levees
- GENERAL RETROFITTING CAUTIONS
- RETROFITTING PROCESS
  - Homeowner Motivation
  - Parameters of Retrofitting
  - Determination of Hazards
  - Benefit/Cost Analysis
  - Design
  - Construction
  - Operation and Maintenance

Chapter I: Introduction to Retrofitting

Table of Contents

HOW TO USE THIS MANUAL

GOALS AND INTENDED USERS

Other flood-related technical resources are available through federal agencies such as FEMA, the U.S. Army Corps of Engineers, and the Natural Resources Conservation Service, as well as state, regional, and local agencies. See Appendix C, *Glossary of Resources*.

This manual will provide valuable assistance to the design professional. It is not intended to be used as a code or specification, nor as a replacement for the engineer's or architect's standard of performance.

This manual has been prepared by the Federal Emergency Management Agency with assistance from other agencies and organizations involved in the nationwide effort to assist local governments, engineers, architects, and property owners involved in retrofitting flood-prone residential structures. Its objective is to provide engineering design and economic guidance to engineers, architects, and local code officials about what constitutes technically feasible and cost-effective retrofitting measures for flood-prone residential structures.

The focus of this manual is the retrofitting of one- to four-family residences subject to flooding situations without wave action. The manual presents various retrofitting measures that provide both active and passive efforts and employ both wet and dry floodproofing measures. These include elevation of the structure in place, relocation of the structure, construction of barriers (levees and floodwalls), dry floodproofing (sealants, closures, sump pumps, and backflow valves), and wet floodproofing (flood-resistant materials and protection of utilities and contents).

The goal of this manual is to capture state-of-the-art information and present it in an organized manner. To the maximum extent possible, existing data and modern research have been utilized as the cornerstone of this document. Detailed sections covering the evaluation, planning, and design of retrofitting measures are included along with case studies of completed retrofitting efforts. Methods for performing economic analyses of the various alternatives are presented.

> Coastal situations subject to wave action are not addressed in this manual. For information on that area the reader is referred to FEMA 55 (Third Edition): *Coastal Construction Manual*, and the U.S. Army Corps of Engineers (USACE) *Shore Protection Manual.*

The architect, engineer, or code official must recognize that retrofitting a residential structure influences how that structure reacts to hazards other than those associated with floodwaters. Flood-related hazards such as water-borne ice and debris impact forces, erosion forces, and mudslide impacts, as well as non-flood-related hazards such as earthquake and wind forces, should be considered in the retrofitting process. Retrofitting a structure to withstand only floodwater-generated forces may impair the structure's ability to withstand the multiple hazards mentioned above. Thus, it is important to approach the retrofitting method selection and design process with a multi-hazard perspective.

ORGANIZATION OF THE MANUAL

This manual has seven chapters and five appendixes.

Chapters I, II, and III

- Introduction to Retrofitting
- Regulatory Framework
- Parameters of Retrofitting

Chapters IV and V

- Determination of Hazards
- Benefit/Cost Analysis and Alternative Selection

These chapters give detailed guidance on how to focus on the specific retrofitting solution that is most applicable for the residential structure being evaluated.

> FEMA's new Benefit/Cost Analysis software has been upgraded to operate in a runtime version of Microsoft Access® included with the program.

The balance of the design manual encompasses the following:

Chapter VI

- Design Practices

This chapter provides step-by-step design processes for each retrofitting measure. (Note: Each retrofitting measure has its own tab and is organized as a subchapter.)

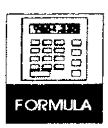
Chapter VII

- Case Studies

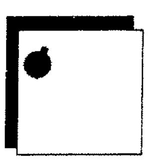
This chapter is a collection of information on the actual retrofitting of specific residential structures.

Throughout this manual, the following icons are used, indicating:

Special Note: Significant or interesting information



Formula: Use of a mathematical formula



Bomb: Special cautions need to be exercised

METHODS OF RETROFITTING

Retrofitting involves a combination of adjustments or additions to features of existing structures that are intended to eliminate or reduce the possibility of flood damage. Retrofitting measures includes the following:

Elevation: The elevation of the existing structure on fill or foundation elements such as solid perimeter walls, piers, posts, columns, or pilings.

Relocation: Relocating the existing structure outside the identified floodplain.

Dry Floodproofing: Strengthening of existing foundations, floors, and walls to withstand flood forces while making the structure watertight.

Wet Floodproofing: Making utilities, structure components, and contents flood- and water-resistant during periods of flooding within the structure.

Floodwalls/Levees: The placement of floodwalls or levees around the structure.

See page I-26 for general cautions to consider in the implementation of a retrofitting measure.

Retrofitting measures can be passive or active in terms of necessary human intervention. Active or emergency retro-fitting measures are effective only if there is sufficient warning time to mobilize labor and equipment necessary to implement the measures. Therefore, every effort should be made to design retrofitting measures that are passive and do not require human intervention.

ELEVATION

Elevating a structure to prevent floodwaters from reaching damageable portions is an effective retrofitting technique. The structure is raised so that the lowest floor is at or above a Design Flood Elevation (DFE). Heavy-duty jacks are used to lift the existing structure. Cribbing supports the structure while a new or extended foundation is constructed below. In lieu of building new support walls, open foundations such as piers, columns, posts, and piles are often used. Elevating a structure on fill is also an option in some situations.

While elevation may provide increased protection of a structure from floodwaters, other hazards must be considered before implementing this strategy. Elevated structures may encounter additional wind forces on wall and roof systems, and the existing footings may experience additional loading. Extended and open foundations (piers, piles, posts, and columns) are also subject to undermining, movement, and impact failures caused by seismic activity, erosion, ice or debris flow, mudslide, and alluvial fan forces, among others.

Cost is an important factor to consider in elevating structures. As an example, lighter wood-frame structures are easier and often cheaper to raise than masonry structures. Masonry structures are not only more expensive to raise, but are also susceptible to cracks.

Base Flood is defined as the flood having a 1% chance of being equaled or exceeded in any given year. The Base Flood Elevation (BFE) is the elevation to which floodwaters rise during a Base Flood.

Refer to FEMA 347: *Above the Flood: Elevating Your Floodprone House* for details on the elevation of residential structures.

Elevation on Solid Perimeter Foundation Walls

The Design Flood Elevation (DFE), also referred to as the Flood Protection Elevation (FPE) or Flood Protection Level (FPL), is the regulatory flood elevation adopted by a local community. Typically, the DFE is the BFE plus any freeboard adopted by the community.

Elevation on solid perimeter foundation walls is normally used in areas of low to moderate water depth and velocity. After the structure is raised from its current foundation, the support walls can often be extended vertically using materials such as masonry block or cast-in-place concrete. The structure is then set down on the extended walls. While this may seem to be the easiest solution to the problem of flooding, there are several important considerations.

Depending on the structure and potential environmental loads (such as flood, wind, seismic, and snow), new, larger footings may have to be constructed. It may be necessary to reinforce both the footings and the walls using steel reinforcing bars to provide needed structural stability.

Deep floodwaters can generate loads great enough to collapse the structure regardless of the materials used. Constructing solid foundation walls with openings or vents will help alleviate the danger by allowing hydrostatic forces to be equalized on both sides. For new and substantially damaged or improved buildings, openings are required under the NFIP.

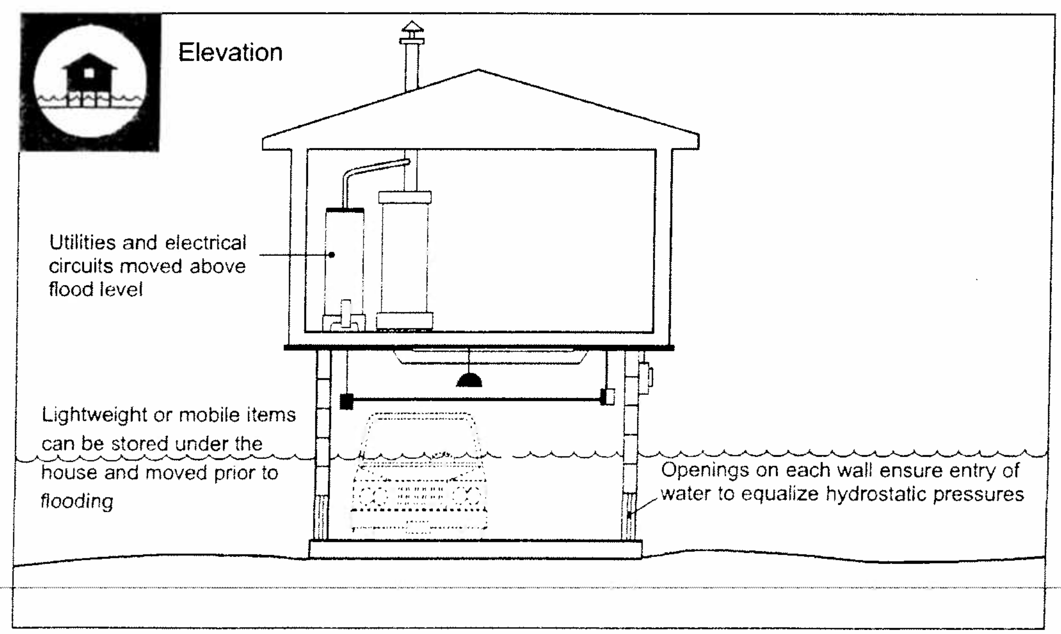


Figure I-1: Elevation on Solid Perimeter Foundation Walls

Figure I-2: Elevation of Existing Residence on Extended Foundation Walls

Elevation on Open Foundation Systems

Open foundation systems are vertical structural members that support the structure at key points without the support of a continuous foundation wall. Open foundation systems include piers, posts, columns, and piles.

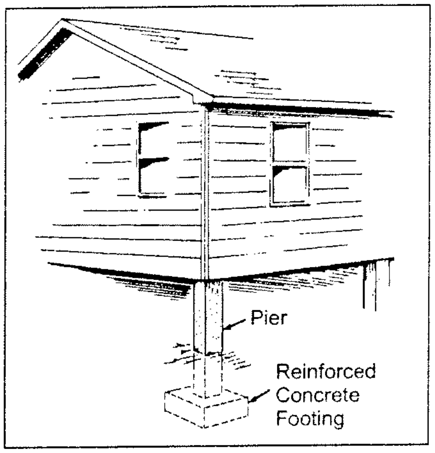


Figure I-3: Elevation on Piers

ELEVATION ON PIERS

The most common example of an open foundation is piers, which are vertical structural members that are supported entirely by reinforced concrete footings. Despite their popularity in construction, piers are often the elevation technique least suited for withstanding significant horizontal flood forces. In conventional use, piers are designed primarily for vertical loading; when exposed to flooding, they may also experience horizontal loads due to moving floodwater or debris impact forces. Other environmental loads, such as seismic loads, can also create significant horizontal force. For this reason, piers used in retrofitting must not only be substantial enough to support the vertical load of the structure, but also must be sufficient to resist a range of horizontal forces that may occur.

Piers are generally used in shallow depth flooding conditions with low-velocity ice, debris, and water flow potential, and are normally constructed of either masonry block or cast-in-place concrete. In either case, steel reinforcing should be used for both the pier and its support footing. The reinforced elements should be tied together to prevent separation. There must also be suitable connections between the superstructure and piers to resist seismic, wind, and buoyancy forces.

ELEVATION ON POSTS OR COLUMNS

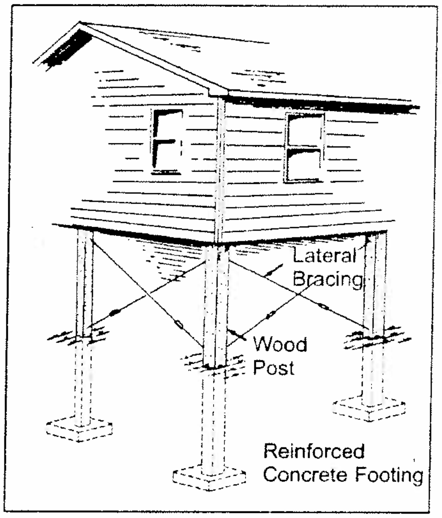


Figure I-4: Elevation on Posts

Columns differ from posts in the size of their application. Posts are small columns.

Elevation on posts or columns is frequently used when flood conditions involve moderate depths and velocities. Made of wood, steel, or precast reinforced concrete, posts are generally square-shaped to permit easy attachment to the house structure. However, round posts may also be used. Set in pre-dug holes, posts are usually anchored or embedded in concrete pads to handle substantial loading requirements. Concrete, earth, gravel, or crushed stone is usually backfilled into the hole and around the base of the post.

While piers are designed to act as individual support units, posts normally must be braced. There are a variety of bracing techniques such as wood knee and cross bracing, steel rods, and guy wires. Cost, local flood conditions, loads, the availability of building materials, and local construction practices frequently influence which technique is used.

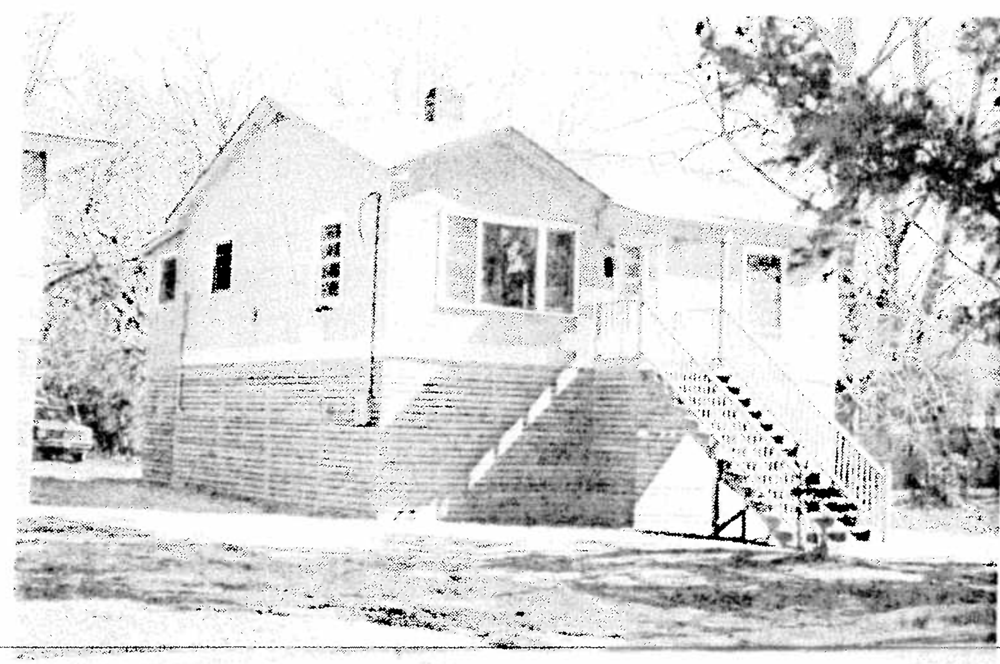


Figure I-5: Structure Elevated on Posts

ELEVATION ON PILES

Piles differ from posts in that they are generally driven, or jetted, deeper into the ground. As such, they are less susceptible to the effects of high-velocity floodwaters, scouring, and debris impact. Piles must either rest on a support layer, such as bedrock, or be driven deep enough to create enough friction to transfer anticipated loads to the surrounding soil. Piles are often made of wood, although steel and reinforced precast or prestressed concrete are also common in some areas. Similar to posts, they may also require bracing.

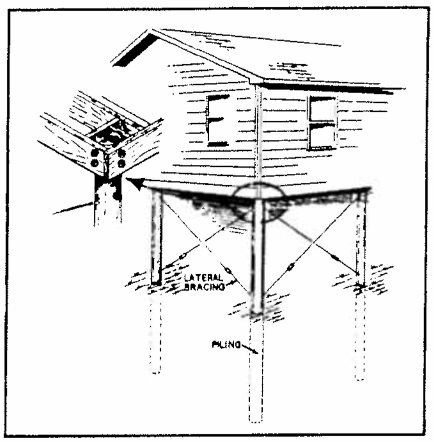


Figure I-6: Elevation on Piles

Because driving piles generally requires bulky, heavy construction machinery, an existing house must normally be moved aside and set on cribbing until the operation is complete. The additional cost and space needs often preclude the use of piles in areas where alternative elevation methods for retrofitting are technically feasible.

Several innovative methods have been developed for setting piles. These include jetting exterior piles in at an angle using high-pressure water flow, and trenching, or auguring, holes for interior pile placement. Augured piles utilize a concrete footing for anchoring instead of friction forces. This measure requires that the existing home be raised several feet above its final elevation to allow room for workers to install the piles. Jetting and auguring piles reduces the uplift capacity compared to driven piles.

Figure I-7: Structure Elevated on Piles

Table I-1 **Advantages and Disadvantages of Elevation**

Advantages	Disadvantages
• **If elevated to the BFE, allows for a substantially damaged or improved structure to be brought into compliance with the NFIP**	• Cost may be prohibitive
• Reduces flood risk to the structure and its contents	• The appearance of the structure may be adversely affected
• Eliminates the need to relocate vulnerable items above the flood level in the house during conditions of flooding	• The structure should not be occupied during a flood
• Often reduces flood insurance premiums	• Access to the structure may be adversely affected
• Techniques are well-known and qualified contractors are often readily available	• Not appropriate in areas with high-velocity water flow, fast-moving ice or debris flow, or erosion unless special measures are taken
• Reduces the physical, financial, and emotional strain that accompanies flood events	• Additional costs may be incurred to bring the structure up to current building codes for plumbing, electrical, and energy systems
• Does not require the additional land that may be needed for floodwalls or levees	• Forces due to wind and seismic hazards must be considered

RELOCATION

Another retrofitting method is to move the structure to a location that is less prone to flooding and flood-related hazards such as erosion. This method is commonly referred to in retrofitting literature as relocation. The structure may be relocated to another portion of the current site or to a different site. The surest way to eliminate flood damage to a structure is to remove it from the floodplain and relocate it to a flood-free location. The procedure normally involves placing the structure on a wheeled vehicle. The structure is then transported to a new location and set on a new foundation.

Relocation is an appropriate measure in high hazard areas where continued occupancy is unsafe and/or owners want to be free from flood worries. It is also a viable option in communities that are considering using the resulting open space for more appropriate floodplain activities. Relocation may offer an alternative to elevation for substantially damaged structures that are required under local regulations to meet NFIP requirements.

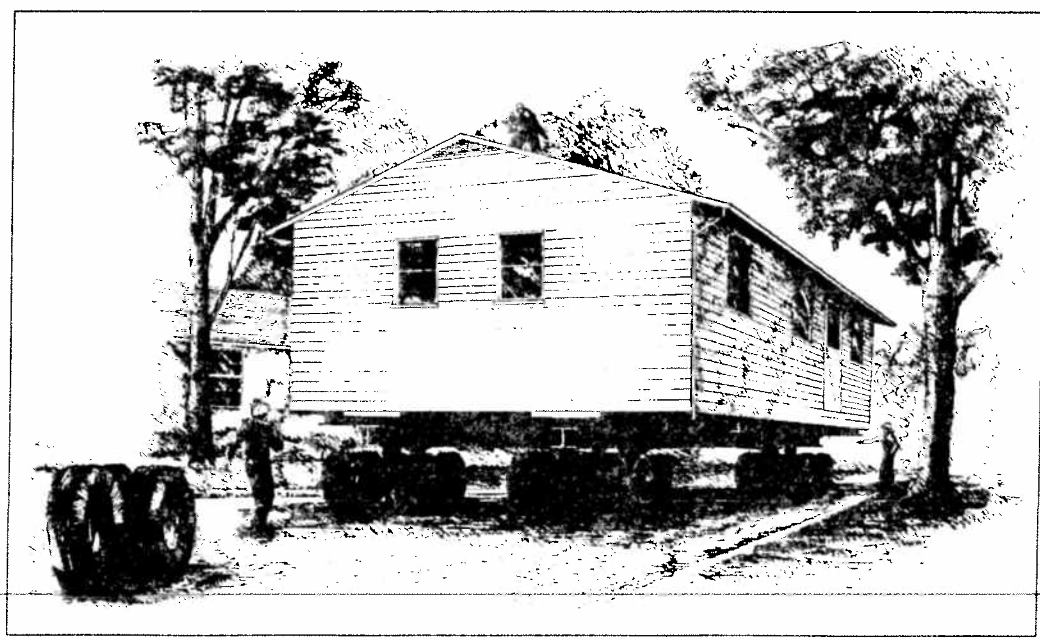


Figure I-8: Structure Placed on a Wheeled Vehicle for Relocation to a New Site

While similar to elevation, relocation of a structure requires additional steps that normally increase the cost of this retrofitting method. These additional costs include moving the structure, purchase and preparation of a new site to receive the structure (with utilities), construction of a new foundation, and restoration of the old site.

Most types and sizes of structures can be relocated either as a unit or in segments. One-story wood-frame houses are usually the easiest to move, particularly if they are located over a crawl space or basement that provides easy access to floor joists. Smaller, lighter wood-frame structures may also be lifted with ordinary house-moving equipment and often can be moved without partitioning. Houses constructed of brick, concrete, or masonry are also movable, but usually with more difficulty and increased costs.

Structural relocation professionals should help owners to consider many factors in the decision to relocate. The structural soundness should be thoroughly checked and arrangements should be made for temporary housing and storage of belongings. Many states and communities have requirements governing the movement of structures in public rights-of-way.

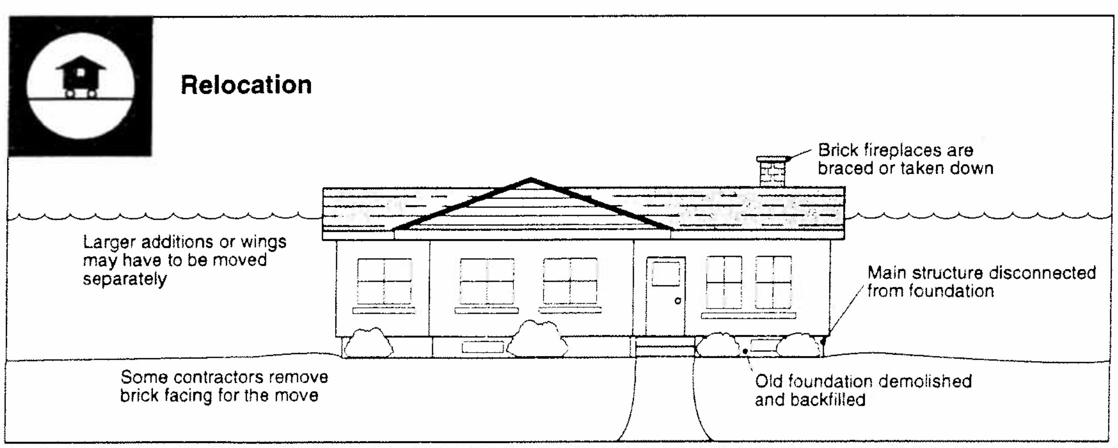


Figure I-9: Structure to be Relocated

Table I-2 **Advantages and Disadvantages of Relocation**

Advantages	Disadvantages
• **Allows for substantially damaged or improved structure to be brought into compliance with the NFIP**	• Cost may be prohibitive
• Significantly reduces flood risk to the structure and its contents	• A new site must be located
• Relocation techniques are well-known and qualified contractors are often readily available	• Disposition of the flood-prone lot must be addressed
• Can eliminate the need to purchase flood insurance or reduce the premium because the house is "moved out" of the floodplain	• Additional costs may be incurred to bring the structure up to current building codes for plumbing, electrical, and energy systems
• Reduces the physical, financial, and emotional strain that accompanies flood events	

DRY FLOODPROOFING

Another approach to retrofitting is to seal that portion of a structure below the flood protection level, making that area watertight. The objective of this approach is to make the walls and other exterior components impermeable to the passage of floodwaters. Creating an impervious membrane, such sealant systems can include wall coatings, waterproofing compounds, impermeable sheeting, or supplemental impermeable wall systems, such as cast-in-place concrete. Doors, windows, sewer and water lines, and vents are closed with permanent or removable shields or valves.

Dry floodproofing is not allowed under the NFIP for new and substantially damaged or improved residential structures located in a Special Flood Hazard Area. Additional information on dry floodproofing can be obtained from FEMA Technical Bulletin 3-93, entitled *Non-Residential Floodproofing Requirements and Certification for Buildings Located in Special Flood Hazard Areas in Accordance with the NFIP*. Non-residential techniques are also applicable in residential situations.

The expected duration of flooding is extremely critical when using sealing systems because seepage can increase over time, rendering the floodproofing ineffective. Waterproofing compounds, sheeting, or sheathing may fail or deteriorate if exposed to floodwaters for extended periods. Sealant systems are also subject to damage (puncture) in areas that experience water flow of significant velocity, or ice or debris flow.

Dry floodproofing is usually appropriate only where floodwaters are less than three feet deep, since most walls and floors in residential structures may collapse or buckle under higher water levels. Research in this area has been conducted by the U.S. Army Corps of Engineers and is available in a document entitled *Floodproofing Tests*, August 1988.

FEMA tasked the National Evaluation Service (NES) to develop an evaluation protocol for flood resistance of materials to be utilized in construction below the BFE. NES has issued a National Evaluation Protocol which can be used in determining which products meet the flood resistant standards. Information on the evaluation protocol is available at http://nateval.org

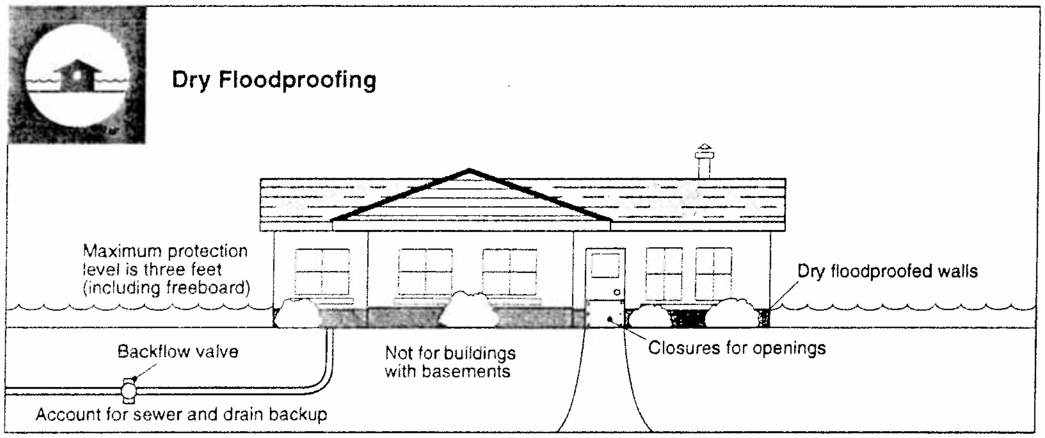


Figure I-10: Dry Floodproofed Structure

Even brick or concrete block walls should not be floodproofed above a height of three feet (without an extensive engineering analysis) due to the danger of structural failure from excessive hydrostatic and other flood-related forces.

The designer should consider incorporating freeboard into the three-foot height constraint as a factor of safety against structural failure. Other factors of safety might include additional pumping capacity and stiffened walls.

Dry floodproofing is also not recommended for structures with a basement. These types of structures can be susceptible to significant lateral and uplift, or buoyancy, forces. When dry floodproofing a wood-frame superstructure, only buildings constructed of concrete block or faced with brick veneer should be considered. Weaker construction materials, such as wood-frame superstructure with siding, will often fail at much lower water depths from hydrostatic forces.

Table I-3 **Advantages and Disadvantages of Dry Floodproofing**

Advantages	Disadvantages
• Reduces the flood risk to the structure and contents if the design flood level is not exceeded • May be less costly than other retrofitting measures • Does not require the extra land that may be needed for floodwalls or levees • Reduces the physical, financial, and emotional strain that accompanies flood events • Retains the structure in its present environment and may avoid significant changes in appearance	• **Does not satisfy the NFIP requirement for bringing substantially damaged or improved residential structures into compliance** • Requires ongoing maintenance • Flood insurance premiums are not reduced for residential structures • Usually requires human intervention and adequate warning time for installation of protective measures • Measures can fail or be exceeded by large floods, in which case the effect will be as if there were no protection at all • If design loads are exceeded, walls may collapse, floors may buckle, and the structure may even float, potentially resulting in more damage than just letting the house flood • The structure should not be occupied during a flood • Shields are not always aesthetically pleasing • The damage to the exterior of the structure and other property may not be reduced • May be subject to leakage, which could cause damage to the structure and its contents

WET FLOODPROOFING

Another approach to retrofitting involves modifying a structure to allow floodwaters to enter it in a way that will minimize damage to the structure and its contents. This type of protection is classified as wet floodproofing.

Wet floodproofing is not allowed under the NFIP for new and substantially damaged or improved structures located in a Special Flood Hazard Area. Refer to FEMA's Technical Bulletin #7-93, entitled *Wet Floodproofing Requirements for Structures Located in Special Flood Hazard Areas in Accordance with the NFIP.*

Wet floodproofing is often used when all other techniques are not technically feasible or are too costly. It is generally appropriate if a structure has available space in which to relocate and temporarily store damageable items. Utilities and furnaces may also need to be relocated or protected along with other non-movable items by using flood-resistant building materials. Wet floodproofing may also be appropriate for structures with basements and crawl spaces that cannot be protected technically or cost-effectively by other retrofitting measures.

For additional information, refer to FEMA 348, *Protecting Building Utilities From Flood Damage: Principles and Practices for the Design and Construction of Flood Resistant Building Utility Systems.*

Compared with the more extensive flood protection measures described in this manual, wet floodproofing is generally the least expensive. The major costs of this measure involve the rearrangement of utility systems, installation of flood-resistant materials, acquisition of labor and equipment to move items, and organization of cleanup when floodwaters recede. Major disruptions to structure occupancy often result during conditions of flooding.

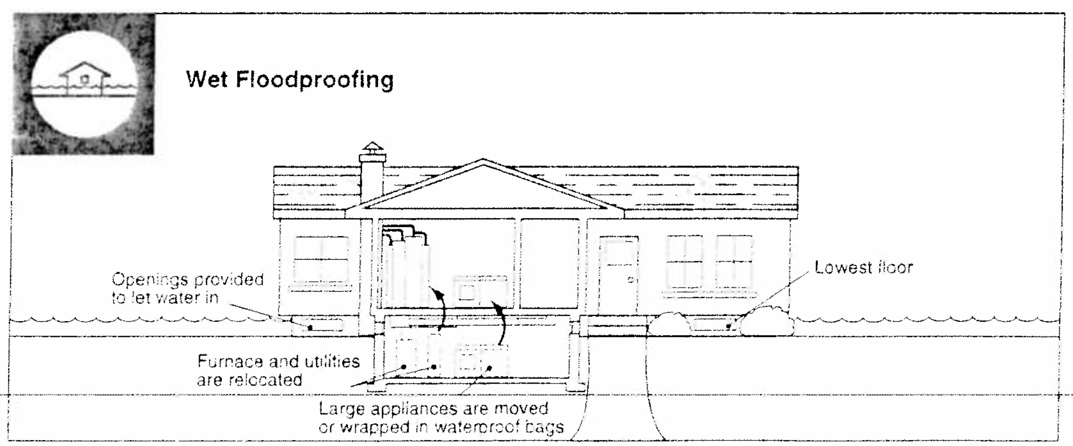


Figure I-11: Wet Floodproofed Structure

Table I-4 **Advantages and Disadvantages of Wet Floodproofing**

Advantages	Disadvantages
• No matter how small the effort, wet floodproofing can, in many instances, reduce flood damage to a building and its contents • Compared to a dry floodproofing measure, loads placed on the walls and floors of a building may be greatly reduced due to equalized hydrostatic pressure • Costs for relocating or storing contents (except basement contents) after a flood warning is issued are covered by flood insurance under certain conditions • Wet floodproofing measures are often less costly than other measures • Does not require extra land, which may be needed for floodwalls or levees • Reduces the physical, financial, and emotional strain that accompanies flood events	• **Does not satisfy the NFIP requirement for bringing substantially damaged or improved structures into compliance** • Flood warning is usually needed to prepare the building and contents for flooding • The evacuation of contents from the flood-prone area is dependent on human intervention • The structure will get wet inside, and possibly be contaminated by sewage, chemicals, and other materials borne by floodwaters. Extensive cleanup may be necessary • The structure should not be occupied during a flood • The structure may be uninhabitable for a time after flooding • There may be a need to limit the uses of the floodable area of the building • There may be some ongoing maintenance requirements • Additional costs may be incurred to bring the structure up to current building codes for plumbing, electrical, and energy systems • To avoid foundation wall collapse, care must be taken when pumping out basements

FLOODWALLS AND LEVEES

Another retrofitting approach is the construction of localized barriers between the structure and the source of flooding. There are two basic types of barriers: levees and floodwalls. They can be built to any height but are usually limited to four feet for floodwalls and six feet for levees due to cost, aesthetics, access, water pressure, and space. Local zoning and building codes may also restrict use, size, and location.

A levee is typically a compacted earthen structure that blocks floodwaters from coming into contact with the structure. To be effective over time, levees must be constructed of suitable materials (i.e., impervious soils) and with correct side slopes for stability. Levees may completely surround the structure or tie to high ground at each end. Levees are generally limited to homes where floodwaters are less than five feet deep. Otherwise, the cost and the land area required for such barriers usually make them impractical for the average owner.

While floodwalls and levees are allowed under NFIP regulations, they do not make a non-compliant structure compliant with the NFIP.

Floodwalls are engineered barriers designed to keep floodwaters from coming into contact with the structure. Floodwalls can be constructed in a wide variety of shapes and sizes but are typically built of reinforced concrete and/or masonry materials.

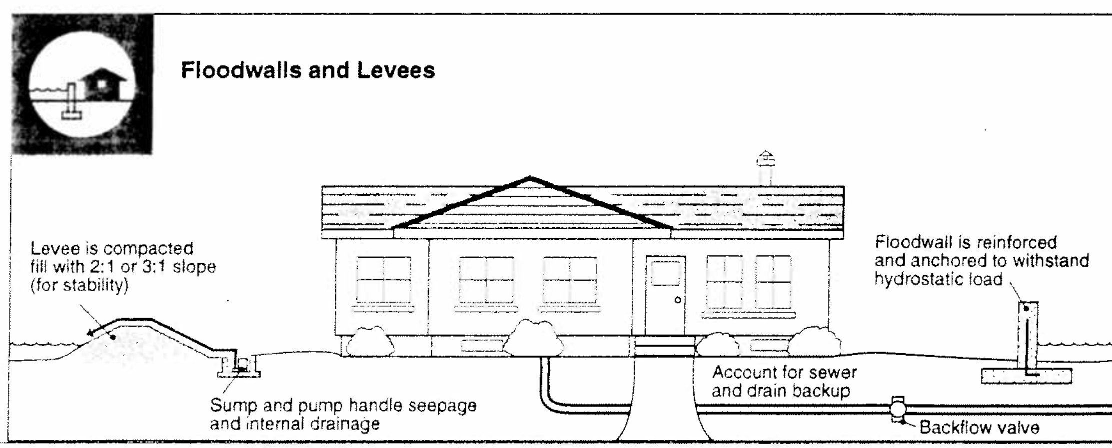


Figure I-12: Structure Protected by Levee and Floodwall

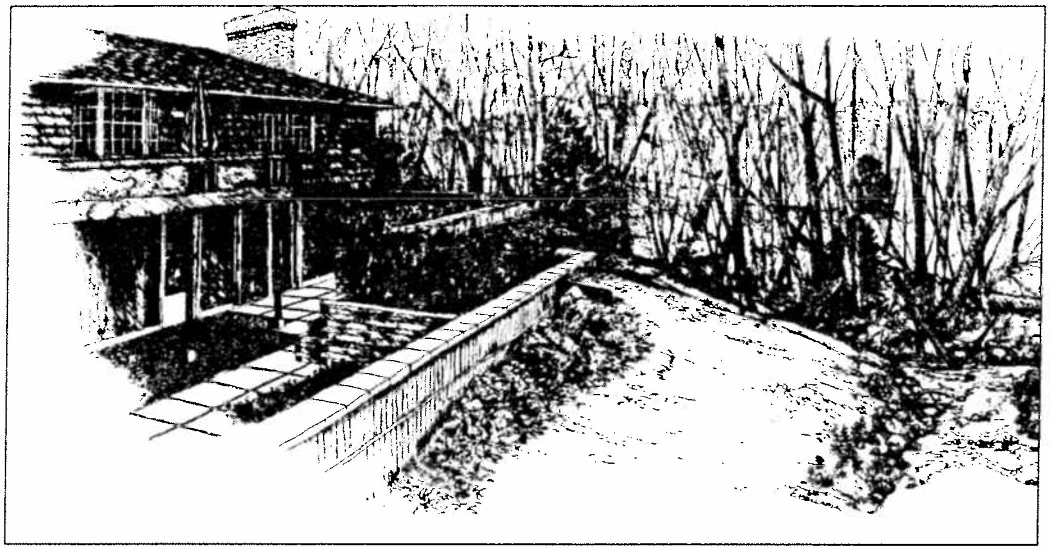


Figure I-13: House Protected by a Floodwall

> Generally, residential floodwalls are only cost-beneficial at providing protection up to four feet and levees up to six feet, including one foot of freeboard.

A floodwall can surround an entire structure or, depending on the flood levels, site topography, and design preferences, it can protect isolated structure openings such as doors, windows, or basement entrances. Floodwalls can be designed as attractive features to a residence, utilizing decorative bricks or blocks, landscaping, and garden areas, or they can be designed for utility at a considerable savings in cost.

Because their cost is usually greater than that of levees, floodwalls would normally be considered only on sites that are too small to have room for levees or where flood velocities may erode earthen levees. Some owners may believe that floodwalls are more aesthetically pleasing and allow preservation of site features, such as trees. Special design considerations must be taken into account when floodwalls or levees are used to protect homes with basements because they are susceptible to seepage that can result in hydrostatic and saturated soil pressure on foundation elements.

Provisions for closing access openings must be included as part of the floodwall or levee design.

The costs of floodwalls and levees can vary greatly, depending on height, length, availability of construction materials, labor, access closures, and the interior drainage system. A levee could be constructed at a lower cost if the proper fill material is available nearby.

Figure I-14: House Protected by a Levee

Table I-5 **Advantages and Disadvantages of Floodwalls and Levees**

Advantages	Disadvantages
• The area around the structure will be protected from inundation without significant changes to the structure • There is no pressure from floodwater to cause structural damage to the home or other structures in the protected area • These barriers are usually less expensive to build than elevating or relocating the structure would be • Occupants do not have to leave the structure during construction • Reduces flood risk to the structure and its contents • Reduces the physical, financial, and emotional strain that accompanies flood events	• **Does not satisfy the NFIP requirements for bringing substantially damaged or improved structures into compliance** • Levees and floodwalls can fail or be overtopped by large floods or floods of long duration, in which case the effect will be as if there were no protection at all • May be expensive • Both floodwalls and levees need periodic maintenance • Interior drainage must be provided • Local drainage can be affected, possibly resulting in water problems for others • No reduction in flood insurance rates • May restrict access to structure • Levees require considerable land area • Floodwalls and levees do not eliminate the need to evacuate during floods • May require warning time and human intervention for closures • Floodplain management requirements may make floodwalls and levees violations of applicable codes and/or regulations

GENERAL RETROFITTING CAUTIONS

Appropriately applied retrofitting measures have several advantages over other damage reduction methods. Individual owners can undertake retrofitting projects without waiting for government action to construct flood control projects. Retrofitting may also provide protection in areas where large structural projects, such as dams or major waterway improvements, are not feasible, warranted, or appropriate. Some general cautions should always be considered in implementing a retrofitting strategy. These include:

- Substantial damage or improvement requirements under the NFIP, local building codes, and floodplain management ordinances render some retrofitting measures illegal.
- Codes, ordinances, and regulations for other restrictions, such as setbacks and wetlands, should be observed.
- Retrofitted structures should not be used nor occupied during conditions of flooding.
- Most retrofitting measures should be designed and constructed by experienced professionals (engineers, architects, or contractors) to ensure proper consideration of all factors influencing effectiveness.
- Most retrofitting measures cannot be installed and forgotten. Maintenance must be performed on a scheduled basis to ensure that the retrofitting measures adequately protect the structure over time.
- Floods may exceed the level of protection provided in retrofitting measures. In addition to implementing these protective measures, owners should consider continuing— and may be required—to purchase flood insurance. In some cases, owners may be required by lending institutions to continue flood insurance coverage.

- When human intervention is most often needed for successful flood protection, a plan of action must be in place and an awareness of flood conditions is required.

RETROFITTING PROCESS

A good retrofitting project should follow a careful path of exploration, fact finding, analysis, detailed design, and construction steps. The successful completion of a retrofitting project will require a series of homeowner coordination and design input meetings. Ultimately, the homeowner will be living with the retrofitting measure, so every effort should be made to incorporate the homeowner's concerns and preferences into the final product. The primary steps in the overall process are shown in Figure I-15 and include:

HOMEOWNER MOTIVATION

The decision to consider retrofitting options usually stems from having experienced or witnessed a flooding event in or near the structure in question; having experienced substantial damage from a flood or an event other than a flood; or embarking on a substantial improvement, which requires adherence to local floodplain regulations. The homeowner may contact other homeowners, community officials, contractors, or design professionals to obtain information on retrofitting techniques, available technical and financial assistance, and other possible options.

PARAMETERS OF RETROFITTING

The goal of this step is to conduct the necessary field investigations, regulatory reviews, and preliminary technical evaluations to select applicable and technically feasible retrofitting techniques that warrant further analysis.

DETERMINATION OF HAZARDS

This step involves the detailed analysis of flood, flood-related, and non-flood-related hazards and the evaluation of specific sites and structures to be retrofitted.

BENEFIT/COST ANALYSIS

This step is critical in the overall ranking of technically feasible retrofitting techniques, and it combines an objective economic analysis of each retrofitting measure considered with any subjective decision factors introduced by the homeowner or others.

DESIGN

During this phase, specific retrofitting measures are designed, construction details developed, cost estimates prepared, and construction permits obtained.

> Within each of these steps, homeowners are involved in providing input into the evaluations, analyses, decisions, and design concepts to ensure that the final product meets their requirements. Finally, maintenance of the constructed retrofitting measure is the responsibility of the homeowner.

CONSTRUCTION

Upon final design approvals, a contractor is selected and the retrofitting measure is constructed.

OPERATION AND MAINTENANCE

The development of a well-conceived operation and maintenance plan is critical to the overall success of the project.

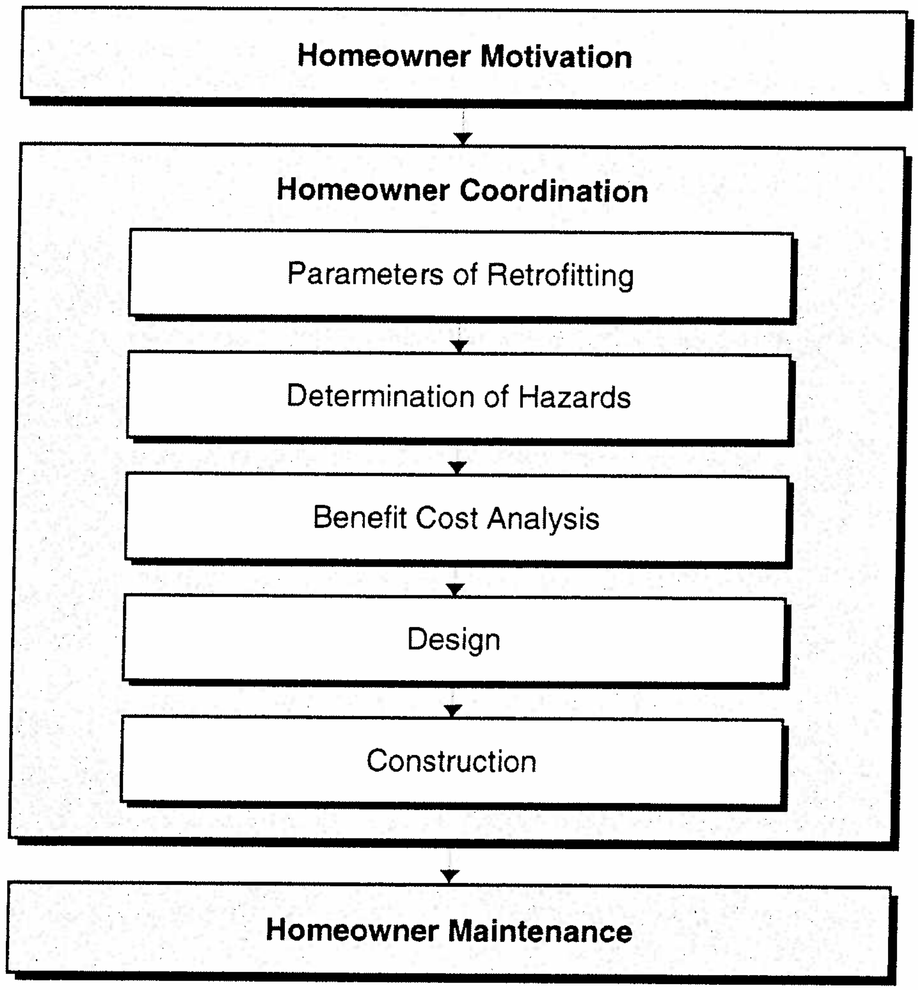
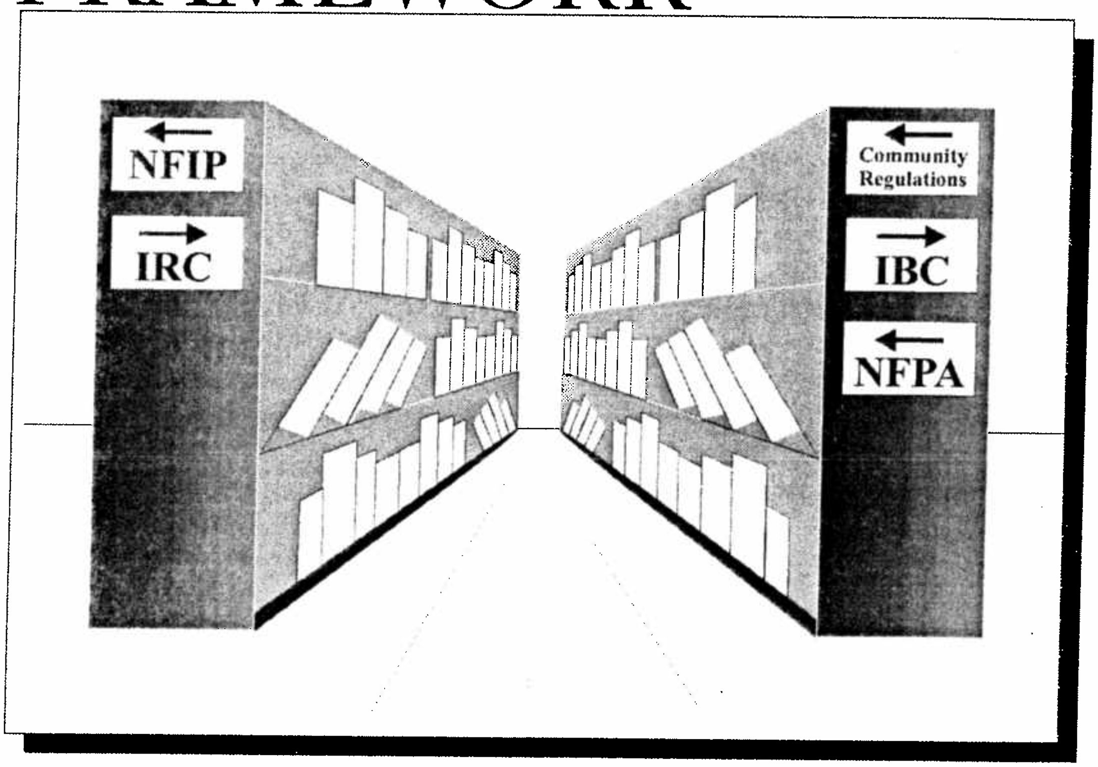


Figure I-15: Primary Steps in the Retrofitting Process

CHAPTER II

REGULATORY FRAMEWORK



Featuring:

National Flood Insurance Program (NFIP)
Community Regulations and the Permitting Process
International Building Codes
Code Compatibility with the NFIP

REGULATORY FRAMEWORK

NATIONAL FLOOD INSURANCE PROGRAM (NFIP)

- Flood Hazard Information
- Floodplain Management Regulations
- Insurance Program
- NFIP Flood-Prone Building Performance Standards

COMMUNITY REGULATIONS AND THE PERMITTING PROCESS

NATIONAL MODEL BUILDING CODES

I - Codes

- International Building Code (IBC)
- International Residential Code (IRC)
- International Plumbing Code
- International Mechanical Code
- International Fuel Gas Code
- International Private Sewage Disposal Code

NFPA Codes

- National Fire Protection Association (NFPA) Life Safety Code
- NFPA Building Code (NFPA 5000) (under development)

CODE COMPATIBILITY WITH THE NFIP

Chapter II: Regulatory Framework

Table of Contents

REGULATORY FRAMEWORK

Most retrofitting projects are regulated by local floodplain, zoning, and building code ordinances. In addition to governing the extent and type of activities allowable in the regulatory floodplain, these codes set construction standards that must be met both by new construction and by substantial improvement and repair of damaged buildings. The portions of these ordinances dealing with retrofitting are generally derived from guidance issued by FEMA under the NFIP and the U.S. Army Corps of Engineers (USACE).

This chapter discusses the typical community floodplain management and building code environment, including:

- the role of local officials in a retrofitting project,
- the various tenets of the NFIP, and
- the compatibility of items covered in international building codes with the NFIP.

Each jurisdiction may adopt standards that are more restrictive than the minimum NFIP requirements, but this section will examine only the minimum federal regulations governing construction in a Special Flood Hazard Area. Local building codes and construction standards vary widely across the country.

In individual communities, local regulations are the mechanism by which NFIP requirements are enforced. The reader is encouraged to contact local floodplain management and building code officials to determine if more restrictive requirements are in place.

NATIONAL FLOOD INSURANCE PROGRAM (NFIP)

The creation of the National Flood Insurance Program was a major step in the evolution of floodplain management. During the 1960s, Congress became concerned with problems related to the traditional methods of dealing with flood damage. It concluded:

- Flood protection structures are expensive and cannot protect everyone.
- People are still building in floodplains and therefore are risking disaster.
- Disaster relief is inadequate and expensive.
- The private insurance industry cannot sell affordable flood insurance because only those at significant risk will buy it.
- Federal flood control programs are funded by all taxpayers, but they primarily help only those who live in the floodplains.

In 1968, Congress passed the National Flood Insurance Act to correct some of the shortcomings of the traditional flood control and flood relief programs. The Act created the National Flood Insurance Program (NFIP) to:

- Guide future development away from flood hazard areas;
- Require that new and substantially improved buildings be constructed to resist flood damage;
- Provide floodplain residents and owners with financial assistance after floods, especially after smaller floods that do not warrant federal disaster aid; and
- Transfer some of the costs of flood losses from the taxpayers to floodplain property owners through flood insurance premiums.

> Guidance on substantial improvement and substantial damage may be found in FEMA Publications 213, *Answers and Questions About Substantially Damaged Buildings*, and 311, *Guidance on Estimating Substantial Damage.*

To obtain information on repetitive loss structures in your community, contact your state CRS program coordinator or your FEMA regional office.

Congress originally charged the Department of Housing and Urban Development's (HUD's) Federal Insurance Administration (FIA) with responsibility for the program. In 1979, the FIA and the NFIP were transferred to the newly created Federal Emergency Management Agency (FEMA).

Currently, the floodplain management aspects of the program are administered by the Mitigation Directorate and the insurance aspects are administered by the Federal Insurance Administration, both parts of FEMA.

FEMA has focused particular attention on mitigating buildings and facilities subject to repetitive losses. A building is considered to be a repetitive loss structure when a building has had at least two losses of $1,000 or more within any 10-year period.

These buildings represent significant losses for the NFIP each year. FEMA is continuing to focus NFIP and retrofitting mitigation efforts on properties that have sustained or are likely to sustain repetitive losses. Possible funding sources for these activities include:

- ICC - Increased Cost of Compliance
- HMGP - Hazard Mitigation Grant Program
- FMA - Flood Mitigation Assistance Program

For additional information on other FEMA and non-FEMA sources of funding, the reader is encouraged to contact their state NFIP and HMGP coordinators.

FLOOD HAZARD INFORMATION

FEMA has developed a home study course on how to use a Flood Insurance Study (FIS). Contact your local FEMA regional office (telephone numbers listed in Appendix C) for further information.

Communities that participate in the NFIP's Regular Program typically have a detailed Flood Insurance Study (FIS), which presents flood elevations of varying intensity, including the base (100-year) flood, areas inundated by the various magnitudes of flooding, and floodway boundaries. This information is presented on a Flood Insurance Rate Map (FIRM) and on a Flood Boundary and Floodway Map (FBFM).

Riverine Floodplains

The FIS report for riverine floodplains describes in detail how the flood hazard information—including floodways, discharges, velocities, and flood profiles for major riverine areas—was developed for each community.

The area of the 100-year riverine floodplain is often divided into a floodway and a floodway fringe. The floodway is the channel of a watercourse plus any adjacent floodplain areas that must be kept free of encroachment so that the cumulative effect of the proposed encroachment, when combined with all other existing or proposed encroachments, will not increase the 100-year flood elevation more than one foot at any point within the community.

The area between the floodway and 100-year floodplain boundaries is termed the floodway fringe. The floodway fringe encompasses the portion of the floodplain that could be completely obstructed without increasing the water-surface elevation of the 100-year flood by more than one foot at any point. Many states and communities limit the allowable increase to less than one foot.

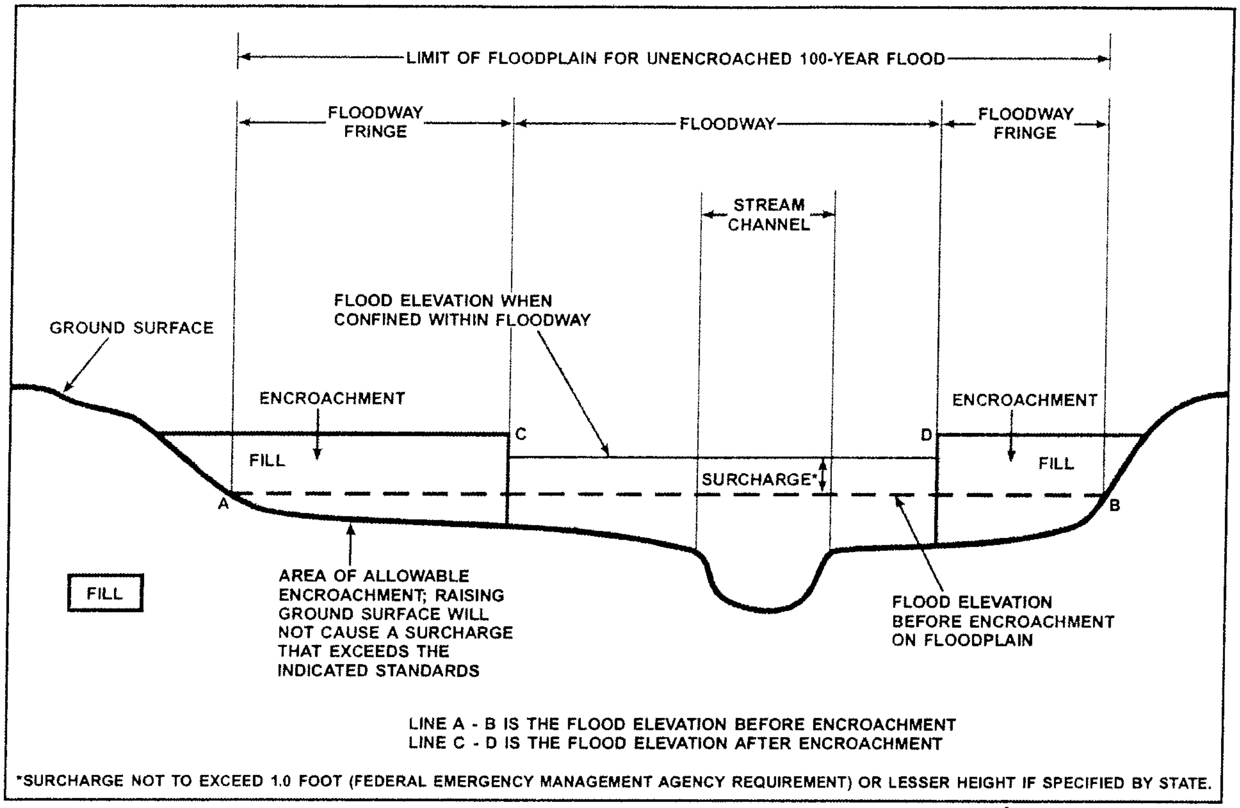


Figure II-1: Typical Floodplain Cross Section

Discharges are determined for various locations and flood frequencies along the stream and are presented in a summary table in the FIS report, as shown in Table II-1. Flood profiles depict various flood frequency and channel bottom elevations along each studied stream. Figure II-2 illustrates a flood profile included in a typical FIS. For most streams with significant flood hazards, the FIS for riverine floodplains normally contains discharges and water-surface elevations for the 10-, 50-, 100-, and 500-year floods, which have annual exceedence probabilities of 10%, 2%, 1%, and 0.2%, respectively.

Table II-1 **Typical Summary of Discharges Table**

Flooding Source and Location	Drainage Area (Sq. Mi.)	Peak Discharges (CFS) 10-Yr	50-Yr	100-Yr	500-Yr
Overpeck Creek					
• Upstream of the confluence of Flat Rock Brook	8.1	910	1,310	1,490	1,960
• Upstream of the confluence of Tributary to Overpeck Creek	5.7	760	1,090	1,200	1,600
• Upstream of the confluence of Metzlers Creek	3.0	530	750	830	1,100
Tributary to Overpeck Creek					
• At its confluence with Overpeck Creek	1.0	275	445	545	810
Metzlers Creek					
• At its confluence with Overpeck Creek	2.4	453	625	704	995
Flat Rock Brook					
• At its confluence with Overpeck Creek	2.5	665	1,075	1,315	1,980

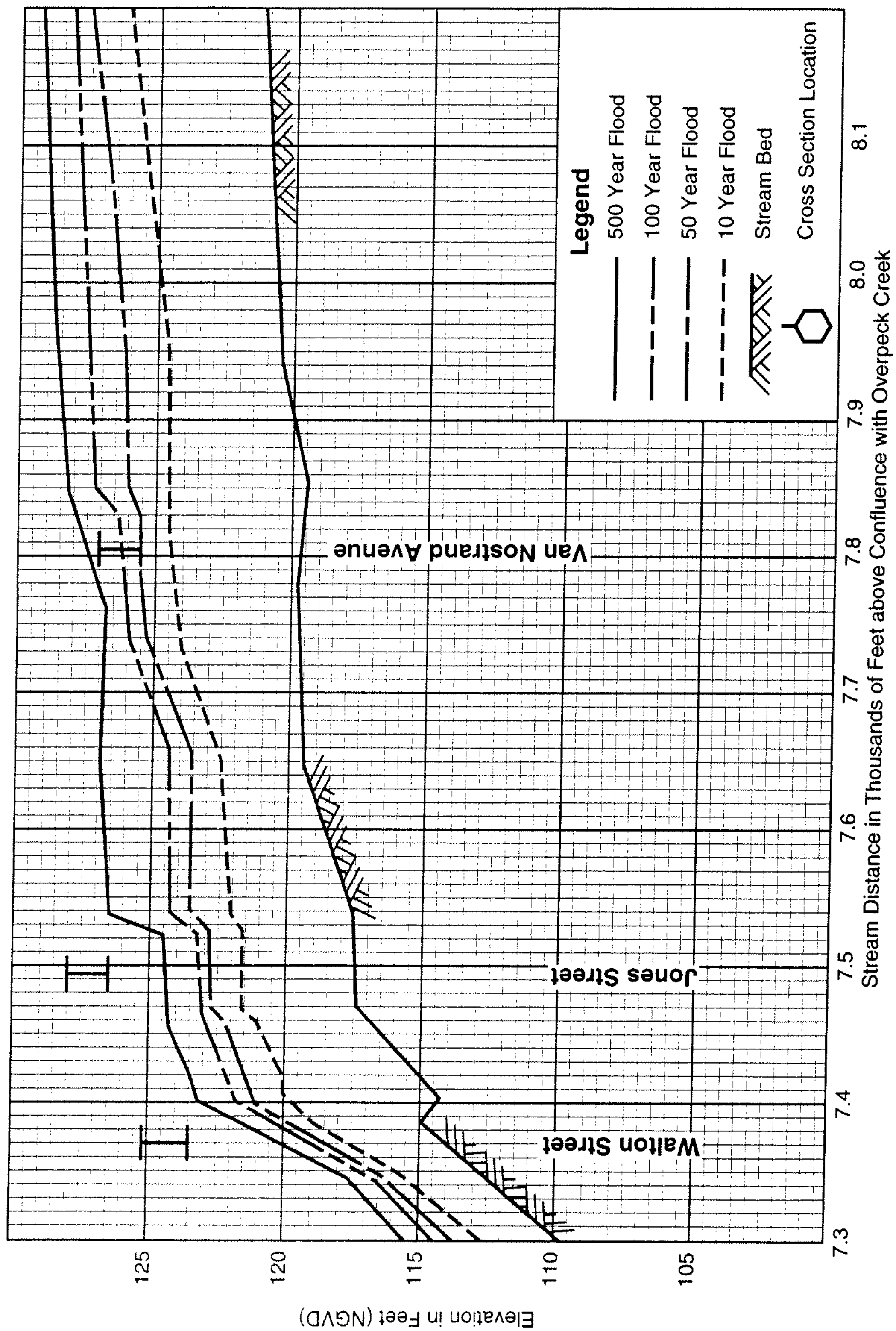


Figure II-2: Typical Flood Profile for Riverine Floodplains

Coastal Floodplains

In coastal communities that contain both riverine and coastal floodplains, the FIS may contain information on both coastal and riverine hazards. These analyses include the determination of the storm surge stillwater elevations for the 10-, 50-, 100-, and 500- year floods as shown in Table II-2.

Table II-2 **Typical Summary of Coastal Stillwater Elevations**

Flooding Source and Location	Elevation (feet) Above NGVD			
	10-Yr	50-Yr	100-Yr	500-Yr
ATLANTIC OCEAN Entire shoreline within Floodport	8.2	8.9	9.2	9.8
MERRIMACK RIVER Entire shoreline within Floodport	5.9	7.2	8.2	8.9

These stillwater elevations represent the potential flood elevations from tropical storms (hurricanes and typhoons), extra-tropical storms (northeasters), tsunamis, or a combination of any of these events. The FIS wave analysis includes an estimate of the expected beach and dune erosion during the 100-year flood and the increased flood hazards from wave heights and wave runup.

> This manual does not cover design issues in Coastal High Hazard Areas (V Zones).

The increases from wave heights and runup are added to the stillwater elevations to yield the regulatory base flood elevation. Figure II-3 illustrates the typical wave height transect showing the effects of physical features on the wave heights and corresponding base flood elevation.

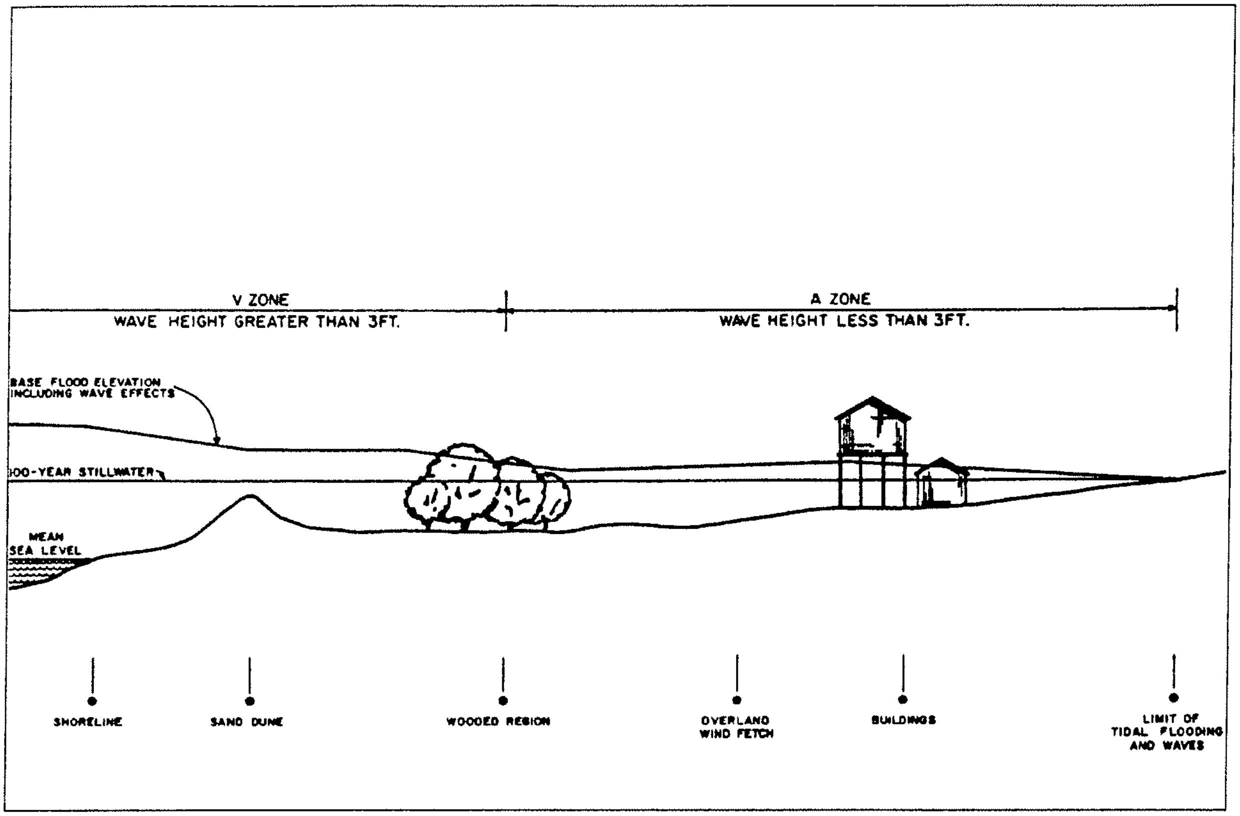


Figure II-3: Typical Wave Height Transect

A FIRM generally shows areas inundated during a 100-year flood as either A Zones or V Zones. An example of a FIRM for riverine flooding is shown in Figure II-4, while a FIRM for coastal flooding is shown in Figure II-5. Retrofitting designers may use data from FIS materials to determine floodplain limits, flood depth, flood elevation, and flood frequency.

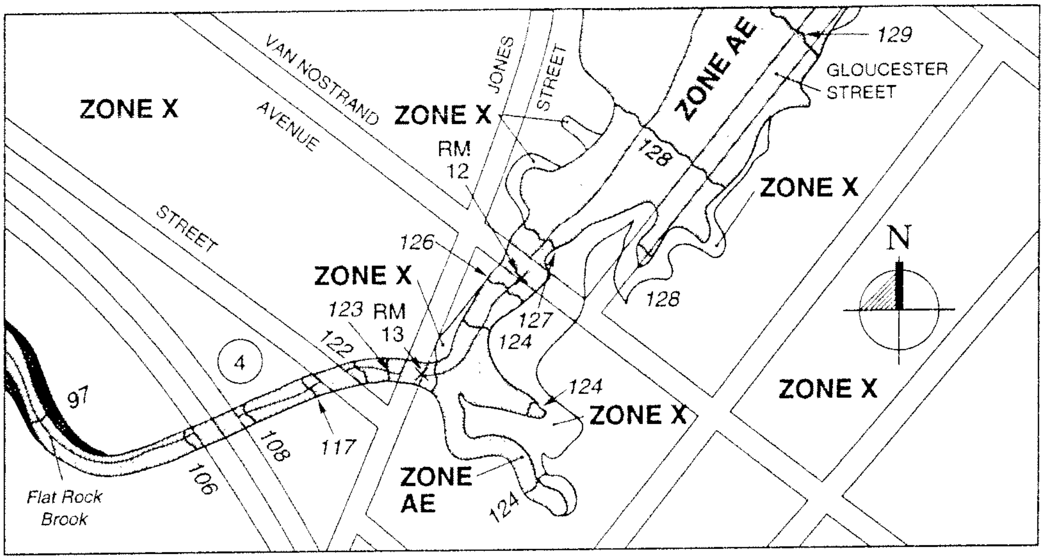


Figure II-4: Typical FIRM for Riverine Flooding

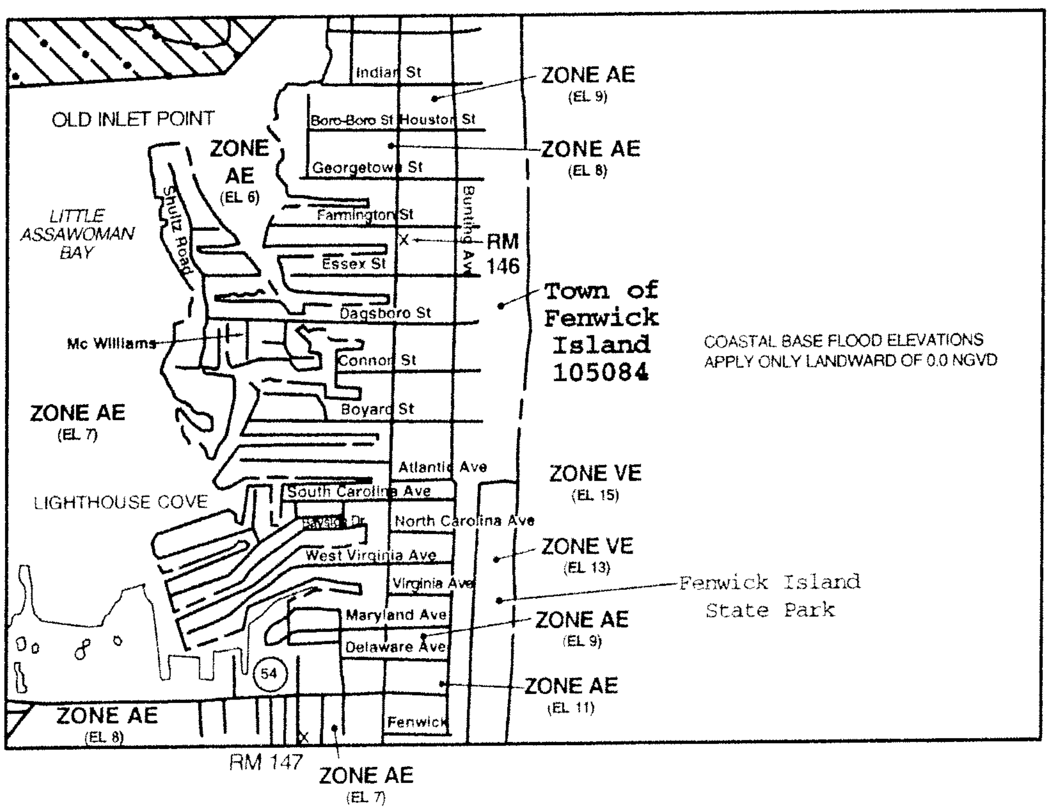


Figure II-5: Typical FIRM for Coastal Flooding

FEMA is in the process of converting from use of the National Geodetic Vertical Datum (NGVD) to the North American Vertical Datum (NAVD). Both datum references will be in use until the transition is completed.

Zone Definitions

A Zones: are the Special Flood Hazard Areas (except coastal V Zones) shown on a community's FIRM. There are six types of A Zones:

A: SFHA where no base flood elevation is provided.

A#: (Numbered A Zones; e.g., A7 or A14) SFHA where the FIRM shows a base flood elevation in relation to National Geodetic Vertical Datum (NGVD) or North American Vertical Datum (NAVD).

AE: SFHA where base flood elevations are provided. AE Zone delineations are used on new FIRMs instead of A# Zones.

AO: SFHA with sheet flow, ponding, or shallow flooding. Base flood depths (feet above grade) are provided.

AH: Shallow flooding SFHA. Base flood elevations in relation to NGVD or NAVD are provided.

AR: Area of special flood hazard that results from the decertification of a previously accredited flood protection system that is determined to be in the process of being restored to provide a 100-year or greater level of flood protection.

B Zones: Areas of moderate flood hazard, usually depicted on FIRMs as between the limits of the base and 500-year floods. B Zones are also used to designate base floodplains of little hazard, such as those with average depths of less than one foot.

C Zones: Areas of minimal flood hazard, usually depicted on FIRMs as above the 500-year flood level. B and C Zones may have flooding that does not meet the criteria to be mapped as a Special Flood Hazard Area, such as ponding and local drainage problems.

D Zones: Areas of undetermined but possible flood hazard.

V Zones: Special Flood Hazard Areas subject to coastal high hazard flooding. There are three types of V Zones, which correspond to the A Zone designations:

V: SFHA where no base flood elevation is provided.

V#: (Numbered V Zones; e.g.,V7 or V14) SFHA where the FIRM shows a base flood elevation in relation to NGVD or NAVD.

VE: SFHA where base flood elevations are provided. VE Zone delineations are now used on new FIRMs instead of V# Zones.

X Zones: Appear on newer FIRMs and incorporate areas previously shown as B and C Zones.

Shaded X: Areas of moderate flood hazard usually depicted on FIRMs as between the limits of the base and 500-year floods. Previously shown as a B Zone.

Unshaded X: Areas of minimal flood hazard usually depicted on FIRMs as above the 500-year flood level. Previously shown as a C Zone.

FLOODPLAIN MANAGEMENT REGULATIONS

The floodplain management aspects of the NFIP are implemented by communities. A "community" is a governmental body with the statutory authority to enact and enforce development regulations. The authority of each unit of government varies by state. Eligible communities can include cities, villages, towns, townships, counties, parishes, states, and Indian tribes. In 1994, more than 18,350 communities participated in the NFIP.

To participate in the NFIP, communities must, at a minimum, regulate development in their floodplains in accordance with the NFIP criteria and state regulations. To do this, communities must require a permit before any development proceeds in the regulatory floodplain. Before the permit is issued, the community must ensure that two basic criteria are met:

- All new buildings and substantial improvements to existing buildings will be protected from damage by the base flood, and

- New floodplain development will not aggravate existing flood problems or increase damage to other properties.

Several definitions are needed to guide the designer through floodplain management regulations. The NFIP definition of key terms is provided below:

Structure: For floodplain management purposes, a walled and roofed building, including a gas or liquid storage tank that is principally above ground, as well as a manufactured home.

Basement: Any area of the structure having its floor subgrade (below ground level) on all sides.

Lowest Floor: The lowest floor of the lowest enclosed area (including basement). An unfinished or flood-resistant enclosure, usable solely for parking, building access, or storage in an area other than a basement is not considered a building's lowest floor, provided that such enclosure is not built so as to render the structure in violation of the applicable non-elevation design requirement of 44 Code of Federal Regulations (CFR) Ch. 1 (60.3).

Enclosed Area Below BFE: An unfinished or flood-resistant enclosure, usable solely for parking, building access, or storage in an area other than a basement that has an elevation below the BFE.

Substantial Damage: Damage of any origin sustained by a structure whereby the cost of restoring the structure to its before-damaged condition would equal or exceed 50 percent of the value of the structure before the damage occurred.

Substantial Improvement: Any reconstruction, rehabilitation, addition, or other improvement of a structure, the cost of which equals or exceeds 50 percent of the value of the structure before the "start of construction" of the improvement. This term includes structures that have incurred "substantial damage," regardless of the actual repair work performed. The term does not, however, include either:

1. any project to correct existing violations of state or local health, sanitary, or safety code specifications that have been previously identified by the local code enforcement official and that are the minimum necessary to assure safe living conditions, or

2. any alteration of a "historic structure," provided that the alteration will not preclude the structure's continued designation as a "historic structure."

> The definitions of pre-FIRM and post-FIRM are different for insurance and floodplain management purposes.

Pre FIRM: A pre-FIRM building (for floodplain management purposes) is a building for which the start of construction occurred before the effective date of the community's NFIP-compliant floodplain management ordinance.

Post-FIRM: A post-FIRM building (for floodplain management purposes) is a building for which the start of construction post-dates the effective date of the community's NFIP-compliant floodplain management ordinance.

Under NFIP criteria, all new (post-FIRM) and substantially damaged/substantially improved construction of residential structures located within Zones A1 - A30, AE, and AH must have the lowest floor at or above the BFE. Therefore, elevation and relocation are the retrofitting alternatives that enable a post-FIRM or substantially damaged/substantially improved structure to be brought into compliance with the NFIP.

Utilizing the aforementioned definitions and local codes, the designer can begin to determine which retrofitting measures may be acceptable for each specific home.

INSURANCE PROGRAM

Federally-backed flood insurance is made available in communities that agree to implement NFIP-compliant floodplain management programs that regulate future floodplain development. Communities apply to participate in the program in order to make flood insurance and certain forms of federal disaster assistance available in their community.

Everyone in a participating community can purchase flood insurance coverage, even for properties not located in mapped floodplains. Insurance provides relief for all floods, including those that are not big enough to warrant federal disaster aid, as long as a general condition of flooding exists.

The federal government makes flood insurance available only in communities that adopt and enforce floodplain management regulations that meet or exceed NFIP criteria. Because the communities will ensure that future development will be resistant to flood damage, the federal government is willing to support insurance and help make it affordable.

The Flood Disaster Protection Act of 1973 expanded the program to require flood insurance coverage as a condition of federal aid or loans from federally-insured banks and savings and loans for buildings located in identified flood hazard areas. Most communities joined the NFIP after 1973 in order to make this assistance available for their flood-prone properties.

NFIP flood insurance is available through many private flood insurance companies and independent agents, as well as directly from the federal government. All companies offer identical coverage and rates as prescribed by the NFIP.

Pre-FIRM Versus Post-FIRM (Insurance Purposes)

Please refer to Appendix A—*The National Flood Insurance Program*—for general information and an example of the costs of insurance coverage for structures subject to various flooding scenarios.

For flood insurance rating purposes, residential buildings are classified as being either pre-FIRM or post-FIRM.

Pre-FIRM construction is defined as construction or substantial improvement begun on or before December 31, 1974, or before the effective date of the community's initial FIRM, whichever is later.

Post-FIRM construction includes construction or substantial improvement that began after December 31, 1974, or on or after the effective date of the community's initial FIRM, whichever is later.

Insurance rates for pre-FIRM buildings are set on a subsidized basis; while insurance rates for post-FIRM structures are set actuarially on the basis of designated flood hazard zones on the community's FIRM and the elevation of the lowest floor of the building in relation to the BFE. This rate structure provides owners an incentive to elevate buildings in exchange for receiving the financial benefits of lower insurance rates. Subsequent to substantial improvements, a pre-FIRM building will become a post-FIRM building for flood insurance rating purposes. Only elevation or relocation techniques may result in reduced flood insurance premiums or in eliminating the need for flood insurance.

NFIP FLOOD-PRONE BUILDING PERFORMANCE STANDARDS

Communities often adopt floodplain regulations that exceed the NFIP minimum requirements. Some states have also enacted laws that exceed the NFIP minimum requirements.

The NFIP has established minimum criteria and design performance standards that communities participating in the NFIP must enforce for structures located in Special Flood Hazard Areas. These standards specify how a structure should be constructed in order to minimize or eliminate the potential for flood damage.

FEMA, the U.S. Army Corps of Engineers (USACE), the Tennessee Valley Authority (TVA), the Natural Resources Conservation Service (NRCS), and several states and local government entities have developed technical guidance manuals and information for public distribution to assist in the application of these requirements by the building community (i.e., building code and zoning officials, engineers, architects, builders, developers, and the general public). These publications, which are listed in Appendix C, *Glossary of Resources*, contain guidelines for the use of certain techniques and materials for design and construction that meet the intent of the NFIP's general design criteria. These publications also contain information on the generally accepted practices for flood-resistant design and construction.

FEMA has also been involved in a multi-year effort to incorporate the NFIP flood-damage-resistant design standards into the nation's model building codes and standards, which are then adopted by either states or communities. This effort has resulted in the inclusion of the standards in the new International Building Code Series and in ASCE 7-98, *Minimum Design Loads for Buildings and Other Structures* and ASCE 24-98, *Flood Resistant Design and Construction*.

COMMUNITY REGULATIONS AND THE PERMITTING PROCESS

> The floodway is the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height.

Regulation of the use of floodplain lands is a responsibility of state and local governments and, in limited applications, the federal government (wetlands, navigable waterways, federal lands, etc.). It can be accomplished by a variety of procedures, such as establishment of designated floodways and encroachment lines, zoning ordinances, subdivision regulations, special use permits, floodplain ordinances, and building codes. These land-use controls are intended to reduce or eliminate flood damage by guiding and regulating floodplain development.

As was explained in Chapter I, flood-prone communities that participate in the NFIP are required to adopt and enforce, at a minimum, NFIP-compliant floodplain regulations to qualify for many forms of federal disaster assistance and for the availability of flood insurance.

Many states and communities have more restrictive requirements than those established by the NFIP. In fact, state and community officials, using knowledge of local conditions and in the interest of safety, may set higher standards, the most common of which are listed below.

- Freeboard is the elevation difference between the flood protection elevation and the anticipated flood elevation. Freeboard requirements provide an extra measure of flood protection above the design flood elevation to account for waves, debris, hydraulic surge, or insufficient flooding data.

- Restrictive standards prohibit building in certain areas, such as the floodplain, conservation zones, and the floodway.

- The use of building materials and practices that have previously proven ineffective during flooding may be prohibited.

- The use and type of construction fill material may be further restricted by the higher standards adopted by some states and communities.

Before committing a significant investment of time and money in retrofitting, the design professional should contact the local building official for building code and floodplain management requirements and information on obtaining necessary permits.

NATIONAL MODEL BUILDING CODES

Designers should consult *Reducing Flood Losses Through the International Code Series: Meeting the Requirements of the National Flood Insurance Program* to gain a thorough understanding of how the requirements of the NFIP have been included in the I-Codes.

The National Model Building Codes currently include the International Building Codes and the NFPA code. The series of International Building Codes (I-Codes) has recently been developed and are now available for adoption and use by governmental jurisdictions. Representatives of the existing code groups (BOCA, ICBO, and SBCCI) drafted this code series. The intent of the I-Codes was to include a comprehensive set of regulations for building systems consistent with and inclusive of the scope of the existing model codes. The new I-Codes include:

- The 2000 International Building Code (IBC) meets the minimum design and construction requirements of the NFIP for all buildings and structures. Appendix G of the IBC addresses other NFIP requirements such as floodplain management issues;

- The 2000 International Residential Code (IRC) meets the minimum requirements for flood resistant design and construction of one- and two-family dwellings;

- The 2000 International Plumbing Code meets the minimum requirements for flood resistant design and construction of plumbing systems;

- The 2000 International Mechanical Code meets the minimum requirements for flood resistant design and construction of mechanical systems;

- The 2000 International Fuel Gas Code meets the minimum requirements for flood resistant design and construction of fuel gas systems;

- The 2000 International Private Sewage Disposal Code meets the minimum requirements for flood resistant design and construction of private sewage disposal systems; and

- The NFPA Life Safety Codes, developed by the National Fire Protection Association (NFPA), are used as a standard for fire protection in various parts of the country. The NFPA Building Code (NFPA 5000) is currently under development.

Most communities, at the time of this revision, have adopted model codes from BOCA, ICBO, or SBCCI. Many of these locally adopted model codes have incorporated provisions of the NFIP floodplain management regulations pertaining to building standards.

The National Model Building Codes are available for adoption by local communities; some states and communities have already adopted the new codes. With the IBC, it is possible to integrate building codes into a single administrative process. In order to participate in the NFIP using this approach, all of the IBC, including Appendix G, must be adopted. Otherwise, not all "development," as defined by the NFIP, is regulated adequately. If Appendix G is not adopted, then provisions regulating "development" would need to be included in a stand-alone ordinance. The IRC includes flood-resistant construction requirements as part of the code and thus are adopted when the IRC is adopted. For more information about the IBC adoption process, contact the state or local building and permitting officials.

Some states, local governments, and communities, however, make their own amendments to the above code. In these cases, it may be unclear if the adopted code is still consistent with NFIP floodplain management regulations.

Table II-3 **Building Code Groups**

National Model Building Codes:	• International Building Code (IBC)
International Building Codes (I-Codes)	• International Residential Code (IRC)
	• International Plumbing Code
	• International Mechanical Code
	• International Fuel Gas Code
	• International Private Sewage Disposal Code
NFPA Codes	• NFPA Life Safety Codes (NFPA)
	• NFPA Building Code (NFPA 5000)
Model Building Codes:	• National Building Code (BOCA)
	• Standard Building Code (SBCCI)
	• Uniform Building Code (IMCO)
	• One- and Two-Family Dwelling Code (CABO)

Note: Model Building Codes to be phased out and replaced by International Building Codes

CODE COMPATIBILITY WITH THE NFIP

Designers should consult *Reducing Flood Losses Through the International Code Series: Meeting the Requirements of the National Flood Insurance Program* to gain a thorough understanding of how the requirements of the NFIP have been included in the I-Codes.

Until a state or community adopts the I-Codes, the basis for administering building code and floodplain regulations are the current regulations and locally adopted building code.

The use of one of the model codes (BOCA, ICBO, SBCCI) may still require that the flood-resistant construction standards be compared to the appropriate model code so that all of the required flood standards are included in the local ordinances. Table II-4 compares the I-Codes and the model codes to selected NFIP requirements.

The application of flood-resistant construction methods is also included in the engineering standards ASCE 7-98 *Minimum Design Loads for Buildings and Other Structures* and ASCE 24-98 *Flood Resistant Design and Construction* in addition to the I-Codes. Table II-5 is a summary of selected key NFIP provisions, citations from the I-Codes and other publications, including the ASCE engineering standards.

Table II-4	CODES vs. NFIP REQUIREMENTS: Items of inconsistency				
NFIP ITEM	I-Codes	SBCCI	ICBO	BOCA	CABO
Use of Registered Professionals			X	X	
Wind, Seismic & Snow Loads		X	X	X	X
Footing & Slab Design		X	X	X	
Standards for Use of Wood Materials		X	X	X	X
Geotechnical Reports and Requirements for Open Foundations		X	X	X	X
Corrosion Protection			X	X	
Hydrostatic and Hydrodynamic Load Considerations and Computations				X	
Occupancy in Basements Below the BFE		X	X	X	
Consistency of Criteria for Residential and Non-Residential Buildings		X			
Anchorage Requirements		X			
Exposed Ductwork		X			
Utility Clearances		X			
Standards for Sealants			X		
Standards for Breakaway Walls			X		
Design Tables Based on Materials			X		
Design Considerations for Floodwalls			X		
Protection of Electrical Systems Below the BFE					
Grounded and Labeled Power Outlets for Pumps and Motors					
Maintenance of Interior Finishes for Different Occupancies					
Complete Flood Design Criteria		X			X
Alternate Forms or Means of Construction					X
Site Preparation Requirements					X
Vapor Barrier Requirements					X
Walls, Floor & Roof Sheathing Design		X	X	X	X

X=Item that may not be consistent with the NFIP.

Table II-5 SUMMARY OF SELECTED KEY NFIP PROVISIONS, CODE CITATIONS, AND REFERENCE DOCUMENTS

KEY PROVISIONS OF THE NFIP	2000 IBC	2000 IRC	ASCE 24-98	OTHER PUBLICATIONS
60.3(a)(3)(i) new construction and substantial improvements to be designed and adequately anchored to prevent flotation, collapse, or lateral movement	**1605.2.2** and **1605.3.1.2** flood loss and load combinations (reference ASCE 7) **1612.4** design and construction (reference ASCE 24)	**R301.1** construction to support all loads, including flood loads **R327.1.1** structural systems designed, connected, and anchored	**Section 5.6** anchorage and connections to resist effects of vertical and lateral loads	ASCE 7-98, *Minimum Design Loads for Buildings and Other Structures*
60.3(a)(3)(ii) new construction and substantial improvements to be constructed with materials resistant to flood damage	**801.1.3** interior finishes, trim, and decorative materials to be in accordnace with FEMA FIA-TB#2 **1403.7** exterior walls to be resistant to water damage	**R327.1.7** and **R501.3** building materials to be flood resistant, installation methods for flooring and walls to conform to FEMA FIA-TB#2	**Chapter 6** exposed structural and non-structural materials, including connections, to be resistant to damage, deterioration, corrosion or decay due to direct and prolonged contact with floodwater	National Evaluation Service, Inc., *Evaluation Plan for Determination of Flood-Resistance of Building Elements* Technical Bulletin FEMA FIA-TB#2: *Flood-Resistant Material Requirements for Buildings Located in Special Flood Hazard Areas* Technical Bulletin FEMA FIA-TB#8: *Corrosion Protection for Metal Connectors in Coastal Areas for Structures Located in Special Flood Hazard Areas*
60.3(a)(3)(iv) electrical, heating, ventilation, plumbing, and air conditioning equipment and other service facilities to be designed and/or located to protect components	**1612.4** design and construction of buildings and structures (including utility support systems) to be in accordance with ASCE 24	**R327.1.5** new and replacement mechanical and electrical systems to be elevated **IFGC R301.5** appliance installations to be elevated or otherwise protected **R1601.3.8** ducts and ducts systems to be elevated	**Chapter 8** utilities and attendant equipment to be elevated or designed, constructed and installed to prevent floodwaters from entering or accumulating within the components; utilities not to be mounted on breakaway walls	FEMA 348, *Protecting Building Utilities From Flood Damage: Principles and Practices for the Design and Construction of Flood Resistant Building Utility Systems* Technical Bulletin FEMA FIA-TB#4: *Elevator Installation for Buildings Located in Special Flood Hazard Areas*
60.3(a)(6)(i) new/replacement sanitary sewage system designed to minimize/eliminate infiltration/discharges **(ii)** onsite waste disposal systems located to avoid impairment or contamination	**Appendix G 401.3** Sewer facilities	**R327.1.6** general performance, refer to Chapter 3 of the *International Private Sewage Disposal Code*	**Section 8.3** buried and exposed plumbing systems, systems below flood level, and sanitary systems, including septic tanks	FEMA 348, *Protecting Building Utilities From Flood Damage: Principles and Practices for the Design and Construction of Flood Resistant Building Utility Systems*
60.3(b)(1) require permits for all development, including placement of manufactured homes	**Appendix G 101.3** Scope (and definition of Development)	**R101.2** Scope **R105.3.1.1** specifically addresses substantial improvement and substantial damage of existing buildings **Appendix E** Manufactured Housing Used as Dwellings AE101, Exception, refers to IRC Section R327 **Appendix J** Existing Buildings AJ102.5 work in existing buildings in flood hazard areas per R105.3.1.1	**Section 1.1** defines the scope to be new structures, including subsequent work and substantial repair or substantial improvement	ASFPM and Federal Interagency Floodplain Management Task Force, *Addressing Your Community's Flood Problems: A Guide for Elected Officials* FEMA EMI IS-9, *Managing Floodplain Development Through the NFIP* (independent study course)

Table II-5 SUMMARY OF SELECTED KEY NFIP PROVISIONS, CODE CITATIONS, AND REFERENCE DOCUMENTS (cont'd)

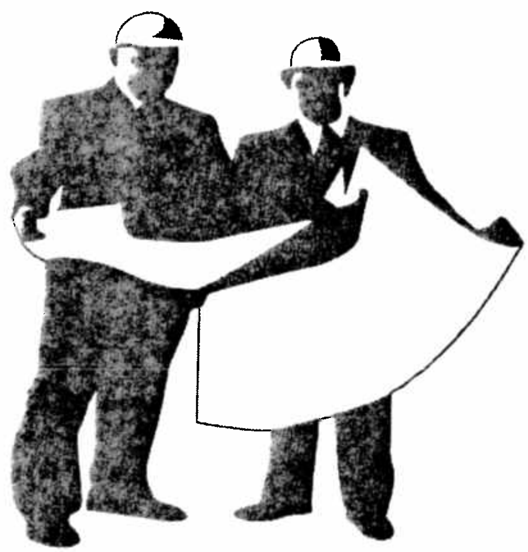
KEY PROVISIONS OF THE NFIP	2000 IBC	2000 IRC	ASCE 24-98	OTHER PUBLICATIONS
60.3(b)(5) where flood elevation data are provided: **(i)** obtain lowest floor elevation of new and substantially improved structures **(ii)** for floodproofed non-residential structures, obtain elevation to which structure was floodproofed **(iii)** maintain records of elevations	**109.3.3** inspection and submission of Elevation Certificate **1612.5.1** submission of specific certifications, including Elevation Certificate **104.7** and **Appendix G 103.8** retention of Department records	**R109.1.3** inspections and submision of Elevation Certificate **R104.7** retention of Department records	Does not address administrative requirements of submissions of certifications	*Elevation Certificate* (FEMA Form 81-31). [Online]. Available: **http://www.fema.gov/library-/elvcert.pdf** *Floodproofing Certificate* (FEMA Form 81-65) [Online]. Available: **http://www.fema.gov/nfip/ff-81-65.pdf**
60.3(b)(8) require installation of MFH using methods to minimize flood damage, including anchoring, and to resist wind forces	**Appendix G 501.1** elevation requirements **Appendix G 501.2** foundation requirements **Appendix G 501.3** anchoring requirements	**R327.1.8** MFH elevation per R327.2; anchor and tie-down per AE604 and AE605. MFH in Floodways per IBC **Appendix AE101** refers to IRC Section R327	Does not specifically address manufactured housing separate from other buildings. Foundations for MFH to be designed as other foundations and based on location within flood hazard areas (with and without high velocity wave action)	FEMA 85, *Manufactured Home Installation in Flood Hazard Areas*
60.3(c)(2) require all new and substantially improved structures to have the lowest floor elevated to or above the flood elevation	**1603.1.6** Flood load (information in application) **1612.4** design and construction (reference ASCE 24) **3402.1** Exception requires substantial improvement or repair of existing building to be brought into compliance with flood provisions	**R105.3.1.1** specifically addresses substantial improvement and substantial damage of existing buildings **R327.2.1** elevation requirements, except for conforming enclosures **R327.1.4** lowest floor, excluding enclosures that meet certain use limitations and are compliant	**Section 2.4** specifies general elevation requirements **Section 2.5** and Chapter 5 detail foundation design requirements	FEMA 259, *Engineering Principles and Practices for Retrofitting Flood Prone Residential Buildings*
60.3(c)(3) for non-residential structures: **(i)** lowest floor elevated, or **(ii)** floodproofed (including utility and sanitary facilities)	**1612.4** design and construction (reference ASCE 24)	Not applicable to One-and Two-Family Dwellings	**Section 2.4** specifies general elevation requirements **Chapter 7** details restrictions and requirements for dry and wet floodproofing	Technical Bulletin FEMA FIA-TB#3: *Non-Residential Floodproofing - Requirements and Certification for Buildings Located in Special Flood Hazard Areas* FEMA 348, *Protecting Building Utilities From Flood Damage: Principles and Practices for the Design and Construction of Flood Resistant Building Utility Systems*
60.3(c)(4) for floodproofed non-residential structures: **(i)** registered design professional to develop and/or review the structural design and certify **(ii)** certification retained in records	**104.7** retention of Department records **1612.5.1** submission of specific certifications, including Elevation Certificate	Not applicable to One- and Two-Family Dwellings	**Chapter 7** details restrictions and requirements for dry and wet floodproofing, but does not include administrative requirements	*Floodproofing Certificate* (FEMA Form 81-65) [Online]. Available: **http://www.fema.gov/nfip/ff-81-65.pdf**

Table II-5 SUMMARY OF SELECTED KEY NFIP PROVISIONS, CODE CITATIONS, AND REFERENCE DOCUMENTS (cont'd)

KEY PROVISIONS OF THE NFIP	2000 IBC	2000 IRC	ASCE 24-98	OTHER PUBLICATIONS
60.3(c)(5) fully enclosed areas below elevated buildings are to be: limited in use (parking, access, storage); provided with flood openings that meet minimum criteria or are designed and certified by a registered design professional	**1202.3** under-floor ventilation (exception allows flood openings) **1612.4** design and construction (reference ASCE 24) **1612.5.1** Flood hazard certifications (for flood opening designs other than as specified)	**R327.2.2** enclosed area below design flood elevations, use limitations and flood openings specifications **R408.5** Enclosing underfloor spaces to have flood openings Garages allowed if elevated or compliant with provisions for enclosures below elevated buildings	**Section 2.6** details provisions for enclosures below DFE, including engineered and non-engineered openings	Technical Bulletin FEMA FIA-TB#1: *Openings in Foundation Walls for Buildings Located in Special Flood Hazard Areas*
60.3(d)(3) prohibit floodway encroachment unless no impact on flood levels is demonstrated	**Appendix G 103.5** and **G 401.1** floodway development not authorized unless no increase in flood level is demonstrated	**R301.2.4** residential development in floodways to be reviewed under the IBC **R327.1.8** manufactured housing in floodways to comply with the IBC	**Section 2.3** flood elevations and conveyance to be maintained	FEMA EMI IS-9, *Managing Floodplain Development Through the NFIP* (independent study course) FEMA FIA-12, *Appeals, Revisions, and Amendments to NFIP Maps: A Guide for Community Officials*
Additional requirements for buildings and structures in flood hazard areas subject to high velocity wave action (V Zones)				
60.3(e)(4) require all new and substantially improved construction to be elevated on pilings and columns so that: **(i)** bottom of lowest horizontal structural member of the lowest floor is at or above the flood elevation, **(ii)** pile or column foundation and structure are anchored to resist flotation, collapse and lateral movement due to wind and water loads; registered design professional to develop or review the design, specifications and plans and provide certification	**1603.1.6** specifies elevation of the lowest of the bottom of the lowest horizontal structural member **1605.2.2** and **1605.3.1.2** flood loads and combined loads **1612.4** design and construction (reference ASCE 24) **1612.5.2** submission of certifications	**R327.3.1** elevation requirements **R327.3.2** foundation requirements, including wind and water loads **R327.3.5** registered professional to certify design and methods of construction	**Section 2.4** and **Section 4.4** specify elevation requirements **Section 2.5, Chapter 4**, and **Chapter 5** address foundations and designs	FEMA 55, *Coastal Construction Manual* Technical Bulletin FEMA FIA-TB#8: *Corrosion Protection for Metal Connectors in Coastal Areas for Structures Located in Flood Hazard Areas*
60.3(e)(5) enclosed areas, if any, are to be constructed with non-supporting, breakaway walls, lattic, or screening intended to collapse under wind and water loads; uses limited to parking, building access, or storage	**1612.4** requires design and construct in accordance with ASCE 24 **1612.5.2** submission of certification of breakaway wall design under certain circumstances	**R327.3.3** specifications for walls and partitions of enclosures below DFE with breakaway walls, and references ASCE 7 (Section 5.3.2.2) for design criteria		

CHAPTER III

PARAMETERS OF RETROFITTING



Featuring:

Examination of Owner Preferences
Community Regulations and Permitting
Technical Parameters

PARAMETERS OF RETROFITTING

EXAMINATION OF OWNER PREFERENCES

- Initial Homeowner Meeting
- Initial Site Visit
- Aesthetic Concerns
- Economic Considerations
- Risk Considerations
- Accessibility

COMMUNITY REGULATIONS AND PERMITTING

- Local Codes
- Building Systems/ Code Upgrades
- Offsite Flooding Impacts

TECHNICAL PARAMETERS

- Flooding Characteristics
- Site Characteristics
- Building Characteristics
- Historic Preservation
- Multiple Hazards

Chapter III: Parameters of Retrofitting

Table of Contents

PARAMETERS OF RETROFITTING

In this chapter, the factors that influence retrofitting decisions are examined and compared with various methods to determine the viability of specific retrofitting techniques. These factors include:

- homeowner preferences,
- community regulations and permitting requirements, and
- technical parameters.

Factors such as homeowner preference and technical parameters are key elements in identifying appropriate retrofitting measures, while consideration of the multiple flood-related and non-flood-related hazards is critical in designing the retrofitting measure and/or avoiding the selection of a poor retrofitting method.

This selection of alternatives can be streamlined through the use of two generic retrofitting matrices, which are designed to help the designer narrow the range of floodproofing options:

Preliminary Floodproofing/Retrofitting Preference Matrix (Figure III-1), which focuses on factors that influence homeowner preference and those measures allowable under local regulations.

Retrofitting Screening Matrix (Figure III-3), which focuses on the objective physical factors that influence the selection of appropriate retrofitting techniques.

EXAMINATION OF OWNER PREFERENCES

The proper evaluation of retrofitting parameters will require a series of homeowner coordination and design input meetings. Ultimately the homeowner will have to deal with the flood protection environment on a daily basis. Therefore, the functional and cosmetic aspects of the retrofitting measure, such as access, egress, landscaping, appearance, etc., need to be developed by including the homeowner's thoughts and ideas. Most retrofitting measures are permanent and should be considered similar to a major home addition or renovation project. The design should incorporate the concepts of those who will be using the retrofitted structure.

Issues that should be addressed include:

- retrofitting aesthetics,
- economic considerations,
- risk considerations,
- accessibility,
- local code requirements,
- building mechanical/electrical/plumbing system upgrades, and
- offsite flooding impacts.

In order to avoid any future misunderstandings, designers should use their skills and knowledge of retrofitting projects to address technical implications while working with homeowners. Many owners have little or no technical knowledge of retrofitting and naturally look to the designer or local official for guidance and expert advice.

The Preliminary Floodproofing/Retrofitting Preference Matrix, (Figure III-1), assists the designer in documenting the initial consultation with the homeowner. The first consideration, measure allowed by community, enables the designer to screen alternatives that are not permissible and must be eliminated from further consideration. Discussion of the considerations for the remaining measures should lead to a "no" or "yes" for each of the boxes. Examination of the responses will help the homeowner and designer select retrofitting measures for further examination that are both viable and preferable to the owner.

Owner Name: ______________________________ Prepared By: ____________
Address: ______________________________ Date: ____________
Property Location: ______________________________

Floodproofing Measures / Considerations	Elevation on Foundation Walls	Elevation on Fill	Elevation on Piers	Elevation on Posts and Columns	Elevation on Piles	Relocation	Dry Flood-proofing	Wet Flood-proofing	Floodwalls and Levees
Measure Allowed or Owner Requirement									
Aesthetic Concerns									
High Cost Concerns									
Risk Concerns									
Accessibility Concerns									
Code Required Upgrade Concerns									
Off-Site Flooding Concerns									
Total "x's"									

Instructions: Determine whether or not floodproofing measure is allowed under local regulations or homeowner requirement. Put an "x" in the box for each measure which is **not** allowed. Complete the matrix for only those measures that are allowable (no "x" in the first row). For those measures allowable or owner required, evaluate the considerations to determine if the homeowner has concerns which would impact its implementation. A concern is defined as a homeowner issue which if unresolved would make the retrofitting method(s) infeasible. If the homeowner has a concern, place an "x" in the box under the appropriate measure/consideration. Total the number of "x's." The floodproofing measure with the least number of "x's" is the most preferred.

Figure III-1: Preliminary Floodproofing/Retrofitting Preference Matrix

THE INITIAL HOMEOWNER MEETING

The first step in the homeowner coordination effort is the educational process for both the designer and the property owner. This step is a very important one.

The Homeowner Learns:

- How it was determined that the home is in the flood-plain;
- Possible impacts of an actual flood;
- Benefits of flood insurance;
- Physical, economic, and risk considerations, and
- What to expect during each step in the retrofitting process.

The Designer Learns:

- Flood history of the structure;
- Homeowner preferences;
- Financial considerations;
- Special issues, such as accessibility requirements for the disabled, and
- Information about the subject property such as:

  - topographic surveys,

  - site utility information, and

  - critical home dimensions.

During this initial meeting, the designer and homeowner should jointly conduct a preliminary assessment of the property to determine which portions of the structure require flood protection and the general condition of the structure. This initial evaluation will identify the elevation of the lowest floor and the elevation of potential openings throughout the structure through which floodwaters may enter the residence.

INITIAL SITE VISIT

A Low Point of Entry determination, illustrated in Figure III-2, determines the elevation of the lowest floor and each of the structure's openings, and may include:

- basement slab elevation;
- windows, doors, and vents;
- mechanical/electrical equipment and vents;
- the finished floor elevation of the structure;
- drains and other floor penetrations;
- water spigots, sump pump discharges, and other wall penetrations;
- other site provisions that may require flood protection, such as storage sheds, wellheads, and storage tanks; and
- the establishment of an elevation reference mark on or near the house.

Once the Low Point of Entry determination has been completed, the designer/owner can determine the flood protection elevation and/or identify openings that need to be protected (in the case of dry floodproofing).

The evaluation of information obtained during the initial meeting with the homeowner will help the designer and owner address the flood threat to the entire structure and the vulnerability of specific openings to floodwater intrusion.

Sometimes it is necessary for a field survey to be conducted by a professional land surveyor before design documents are developed. However, frequently the homeowner and designer may be able to develop a rough elevation relationship between the expected flood elevation, the elevation of the lowest floor, and the low points of entry to the structure sufficient for an initial evaluation.

A detailed discussion of how to evaluate the costs of different alternatives and the effect of the Low Point of Entry may be found in the chapter on Benefit/Cost Analysis.

The approximate height of the retrofitting measure can be used by the owner and designer as they evaluate each of the parameters of retrofitting discussed in this chapter. In addition to determining the Low Point of Entry, this initial site visit should be used to assess the general overall condition of the structure.

I see no reference to LPE in that chapter

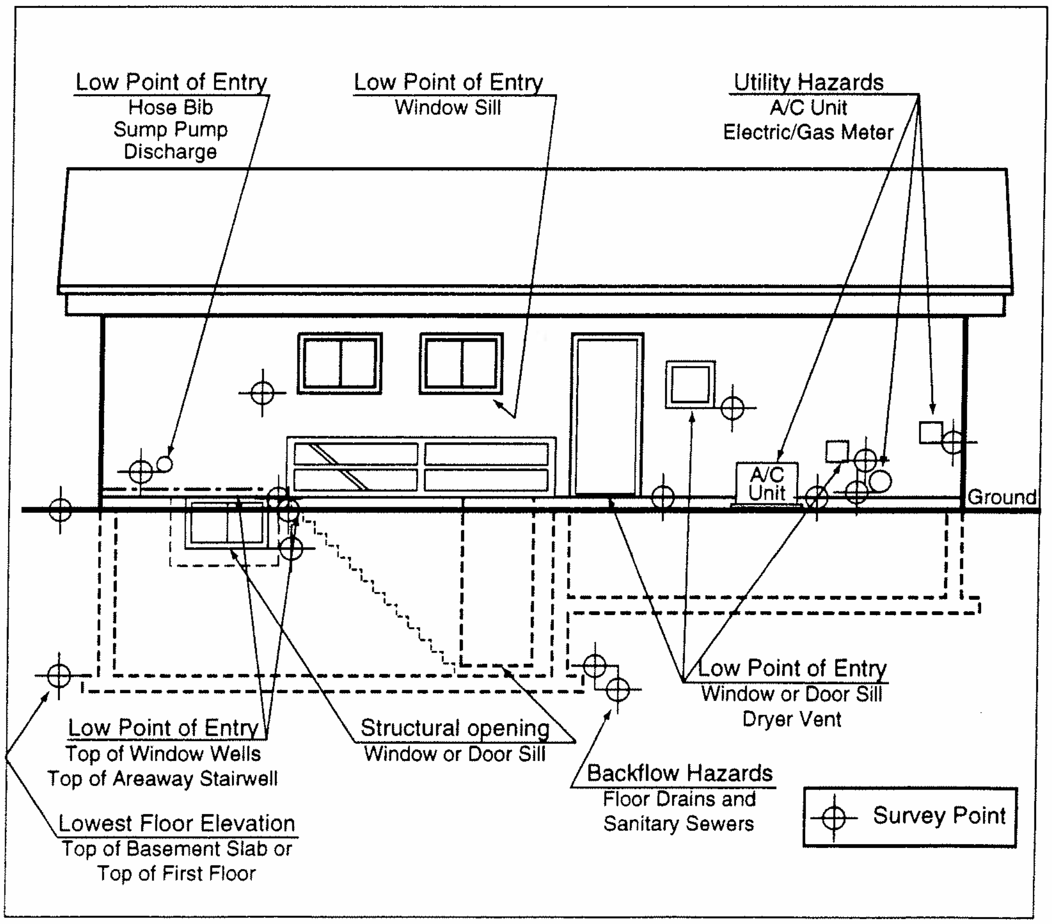


Figure III-2: Low Point of Floodwater Entry Survey for a Typical Residential Structure

AESTHETIC CONCERNS

Although physical and economic considerations may help determine feasible retrofitting measures for individual buildings, the homeowners may consider other factors equally or more important. Aesthetics, for example, is a subjective issue.

The homeowner may reject a measure that scores high for all considerations except aesthetics. On the other hand, what may be aesthetically pleasing to the homeowner may not be technically appropriate for a project. Here, a designer must use skill and experience to achieve a common ground. In doing so, the homeowner's preference should be considered, while not jeopardizing the structural, functional, and overall success of the proposed project.

An aesthetically pleasing solution that also performs well as a retrofitting alternative can be achieved through an understanding of the relationship between the existing and proposed modifications, creative treatment and modification of surrounding landforms, proper landscaping techniques, and preservation of essential and scenic views.

ECONOMIC CONSIDERATIONS

At this point, the designer should not attempt to conduct a detailed cost analysis. Rather, general estimates of the cost of various retrofitting measures should be presented to the homeowner.

As discussed in Chapter I, the cost of retrofitting will depend on a variety of factors including the building's condition, the retrofitting measure to be employed, the design flood elevation, the choice of materials and their local availability, the availability and limitations of local labor, and other site-specific issues (i.e., soil conditions and flooding levels) and other hazards.

> Cost information on elevation of substantially damaged structures is available from a variety of post-disaster documents prepared by FEMA as a result of recent disasters. Two such documents were prepared following Hurricane Andrew in Florida (January 1993) and the Midwest Flood of 1993. Cost data from these two documents suggest that elevation costs for substantially damaged homes may vary significantly from the elevation costs listed in Table III-1.

The following costs are nationwide averages that may need to be adjusted for local economic conditions. They were derived from various sources, including FEMA 312, *Homeowner's Guide to Retrofitting* (June 1998) and the USACE document, *Flood Proofing, How to Evaluate Your Options*. They are provided to assist in economic analysis and preliminary planning purposes.

Table III-1 **Elevation and Relocation Cost Guide**

Construction Type	Foundation Type	Elevation Cost	Relocation Cost	Per
Wood Frame	Basement	$18	$34	square foot
	Crawlspace	$18	$29	square foot
	Slab-on-Grade	$50	$54	square foot
Masonry	Basement	$37	$52	square foot
	Crawlspace	$37	$34	square foot
	Slab-on-Grade	$50	$65	square foot

Table III-1 Assumptions:

1. Elevation costs include foundation, existing utilities, and miscellaneous items, such as extending staircases.
2. Elevation unit cost is based on a 2-foot raise. Add $0.80 per square foot for each additional foot of elevation up to 8 feet. Above 8 feet, add $1.05 per square foot.
3. Relocation costs include off-site relocation (less than 5 miles) and new site development for a 1,000 SF building. Extrapolation of this unit cost to larger buildings may result in artificially high estimates because the costs of relocation do not increase proportionally with building size.
4. Relocation costs do not include the cost of restoring the old site, which would be $12 per square foot of building footprint regardless of construction type or foundation type.
5. For wood frame house with brick veneer on walls, add 10 percent to elevation and relocation costs.

In relocating a structure, the cost of preparing the new site and cleaning up the old site must be considered.

Table III-2 **Floodwalls and Levees Cost Guide**

Type	Cost	Per
Floodwalls, two feet above ground level	$90	linear foot
Floodwalls, four feet above ground level	$132	linear foot
Levees, two feet above ground level	$39	linear foot
Levees, four feet above ground level	$73	linear foot
Levees, six feet above ground level	$122	linear foot
Floodwall costs are based upon typical foundation depth of 30 inches. Levee costs are based upon typical foundation depth of one foot, 5-foot top width, and 1:3 side slopes. Levee costs include seeding and stabilization. Additional costs that may need to be estimated for both floodwalls and levees are as follows:		
Interior Drainage	$4,500	lump sum
Closures	$77	square foot
Riprap	$33	cubic yard
Seeding of disturbed areas	$0.05	square foot

More detailed cost estimating guidance is provided in Chapters V and VI.

Table III-3 **Dry Floodproofing Cost Guide**		
Type	**Cost**	**Per**
Sprayed-on Cement (above grade)	$3.50	square foot
Waterproof Membrane (above grade)	$1.17	square foot
Asphalt (2 coats below grade)	$1.17	square foot
Perimeter drainage	$33	linear foot
Plumbing Check Valve	$660	lump sum
Sump Pump (with backup battery)	$1,060	lump sum
Metal Flood Shield	$77	square foot
Wood Flood Shield	$24	square foot

Table III-3 Assumptions:

1. Cement, membrane, and asphalt are alternative sealant methods.
2. Asphalt costs do not include the cost of excavation.

Table III-4 **Wet Floodproofing Cost Guide**				
Construction Type	**Height of Wet Floodproofing**	**Foundation Type**	**Cost**	**Per**
Wood Frame or Masonry	2 feet	Basement	$1.80	square foot of house footprint
		Crawlspace	$1.40	
	4 feet	Basement	$3.70	square foot of house footprint
		Crawlspace	$3.45	
	8 feet	Basement	$10.60	square foot of house footprint
		Crawlspace	NA	

Table III-4 Assumptions:

1. For house with basement, the height of wet floodproofing is measured in feet above basement floor. For house with crawlspace, the height of wet floodproofing is measured in feet above lowest adjacent grade.
2. Basements are unfinished.
3. "NA" indicates a house would almost never have a crawlspace 8 feet high which is nearly the height of a full story.

Additional costs which may be included:

- temporary living quarters (displacement costs) that may be necessary during construction (estimate: relocation - 3 to 4 weeks; elevation - 2 to 3 weeks),
- professional or architectural design (10% of the costs of selected retrofitting measures),
- contractors' profit (10% of the estimated costs), and
- contingency to account for unknown or unusual conditions.

Table III-5 can serve as a guide for developing the initial planning level estimate for each retrofitting alternative being considered.

Table III-5 **Preliminary Cost Estimating Worksheet**

Owner Name: ______________________ **Prepared By:** ______________________

Address: ______________________ **Date:** ______________________

Property Location: ______________________

Cost Component	Unit	Unit Cost	Quantity	Total
Subtotal Retrofitting Measure				
Contractor's Profit (10%)				
Design Fee (10%) (optional)				
Loss of Income (optional)				
Displacement Expenses (optional)				
Contingency				
Subtotal Other Costs				
Total Costs				

RISK CONSIDERATIONS

Another element that is included in the evaluation of retrofitting measures is the risk associated with a do-nothing approach. Risk can also be established among the various measures by knowing the exceedence probability of floods and the design flood levels for competing measures. Relocation is an example of how retrofitting can eliminate the risk of flood damage. On the other hand, a levee designed to protect against a 10-percent chance annual exceedence probability (10-year) flood would have an 88-percent chance of being overtopped during a 20-year period. Such information will assist the homeowner in evaluating the pros and cons of each measure. Table III-6 provides the probabilities associated with one or more occurrences of a given flood magnitude occurring within a specific number of years.

Table III-6 **Flood Risk**

	Frequency-Recurrence Interval (Year-Event)					
		10	25	50	100	500
Length of Period (Years)	1	10%	4%	2%	1%	0.2%
	10	65%	34%	18%	10%	2%
	20	88%	56%	33%	18%	5%
	25	93%	64%	40%	22%	5%
	30	96%	71%	45%	26%	6%
	50	99+%	87%	64%	39%	10%
	100	99.99+%	98%	87%	63%	18%

The table values represent the probabilities, expressed in percentages, of one or more occurrences of a flood of given magnitude or larger within a specified number of years.

Flood probabilities are also useful in evaluating the home-owner inconvenience aspects of retrofitting. Reducing cleanup and repairs, lost time from work, and average non-use of a building from once in two years to once in ten years could be a powerful incentive for retrofitting even though other aspects may be less convincing.

ACCESSIBILITY FOR THE DISABLED

Accessibility for the disabled is an issue that must be addressed primarily on the specific needs of the owner. Many retrofitting measures can create access problems for a house that was previously fully accessible. The Americans with Disabilities Act (ADA) of 1990 and the Fair Housing Amendment Act (FHA) of 1988 and other accessibility codes and regulations do not specifically address private single-family residences, which are the focus of this manual. However, the above-mentioned regulations contain concepts that may be of assistance to a designer representing a disabled property owner.

It is important for the designer to remember that the term disabled does not refer only to someone who uses a wheelchair. Other disabilities may include:

- limited mobility requiring the use of a walker or cane, which can inhibit safe evacuation;
- a person's limited strength to open doors, climb stairs, install flood shields, or operate other devices; and
- partial or total loss of hearing or sight.

Special considerations such as small elevators may be needed.

Discussion of the above factors with the homeowner and utilization of the **Preliminary Retrofitting Preference Matrix** will allow the designer to rank the retrofitting methods by homeowner preference.

COMMUNITY REGULATIONS AND PERMITTING

LOCAL CODES

Most local governments regulate building activities by means of building codes as well as floodplain and zoning ordinances and regulations. With the intent of protecting health and safety, most local codes are fashioned around the model building codes discussed in Chapter II. The designer should be aware that modifications may be undertaken to make the model codes more responsive to the local conditions and concerns in the area, such as seismic and hurricane activity, extreme cold, or humidity.

A designer should become familiar with the prevailing conditions, codes, and legal restrictions particular to a building's location.

Determination of which retrofitting measures are allowed under local regulations is an important step in compiling the Preliminary Floodproofing/Retrofitting Preference Matrix. Retrofitting measures not allowed under local regulations will be screened and eliminated from further consideration.

BUILDING SYSTEMS/CODE UPGRADES

Other local code requirements must be met by owners building improvements. Most building codes require approval when elevation is considered, especially if structural modification and/or alteration and relocation of utilities and support services are involved.

If more stringent laws have been adopted since a building was constructed, local code restrictions can seriously affect the selection of a retrofitting method because construction may be expected to comply with new building codes.

Some communities require that structures undergoing substantial rehabilitation, either because of previous damage or significant improvements/additions, be brought into compliance with current building codes. In addition to floodplain management requirements, these requirements could include items such as the addition of fire alarms, removal of lead water pipes, upgrades in electrical wiring, etc.

OFFSITE FLOODING IMPACTS

Where a chosen retrofitting measure requires the modification of site elements, a designer shall consider how adjacent properties will be affected.

- Will construction of levees and floodwalls create diversions in the natural drainage patterns?
- Will new runoffs be created that may be detrimental to nearby properties?
- If floodproofing disturbs the existing landscape, will regrading and relandscaping undermine adjacent streets and structures?
- Will the measure be unsightly or increase the possibility of sliding and subsidence at a later date?
- If a building is to be relocated to another portion of the current site, or if it is to be elevated, will it encroach on established easements or rights-of-way?
- Will the relocated building infringe on wetland areas or regulated floodplains?

> Addressing offsite impacts and issues is as much a matter of responsible practice and conscience as it is a requirement of most building codes and floodplain management ordinances.

> NFIP, state, and local regulations do not allow construction within a floodway or, in some cases, within a floodplain that would back up and increase flood levels.

These and other questions must be addressed and satisfactorily answered by the designer and homeowner in selecting the most appropriate retrofitting measure. Both must be aware of the liabilities that may be incurred by altering drainage patterns and other large-scale site characteristics. The designer should insure that any modified runoffs do not cause negative impacts on the surrounding properties. The means necessary to collect, conduct, and dispose of unwanted flood or surface water resulting from retrofitting modifications must be understood and clearly resolved.

TECHNICAL PARAMETERS

Once the designer has resolved preliminary retrofitting preference issues with the owner, a more intensive evaluation of the technical parameters is normally conducted, including flooding, site, and building characteristics. Figure III-3 provides a Retrofitting Screening Matrix (worksheet) that can be used to evaluate which measures are appropriate for individual structures. Instructions for using this matrix are presented in Figure III-4. The remainder of this chapter provides background information on each of the technical parameters, which will be useful to the designer in completing the Retrofitting Screening Matrix.

Owner Name: ______________________ Prepared By: __________

Address: ______________________ Date: __________

Property Location: ______________________

	Parameters \ Measures	Elevation on Foundation Walls	Elevation on Fill	Elevation on Piers, Piles, Posts, and Columns	Relocation	Dry Flood-proofing	Wet Flood-proofing	Floodwalls and Levees
	Measure Permitted by Community or Preferred by Homeowner							
Flooding Characteristics	**Flood Depth**							
	Shallow (<3 feet)							
	Moderate (3 to 6 feet)					N/A		
	Deep (>6 feet)					N/A	N/A	N/A
	Flood Velocity							
	Slow/Moderate (≤5 fps)							
	Fast (>5 fps)	1	1	1		N/A		1
	Flash Flooding							
	Yes (<1 hour)					2	2	2
	No							
	Ice and Debris Flow							
	Yes	6		4		N/A		4
	No							
Site Characteristics	**Site Location**							
	Floodway	5	5	5	5	5	5	5
	Other A Zone							
	Soil Type							
	Permeable					3		3
	Impermeable							
Building Characteristics	**Building Foundation**							
	Slab on Grade							
	Crawl Space					N/A		
	Basement		6	6		6		
	Building Construction (Framing)							
	Concrete or Masonry							
	Wood and Others							
	Building Condition							
	Excellent to Good							
	Fair to Poor	6	6	6	6	6		

Figure III-3: Retrofitting Screening Matrix

The Retrofitting Screening Matrix (Figure III-3) is designed to screen and eliminate retrofitting techniques that should not be considered for a specific situation.	
Step 1:	Screen alternatives which are not permitted nor preferable to the homeowner and are eliminated from further consideration, by inserting N/P (not permitted) in the appropriate box(es) on the Measures Permitted by Community row. If a N/P is placed in a column representing a retrofitting measure, that alternative is eliminated from consideration.
Step 2:	Select the appropriate row for each of the nine characteristics that best reflect the flooding, site, and building characteristics.
Step 3:	Circle the N/A (not advisable) boxes that apply in the rows of characteristics selected. Do not circle any N/A boxes where there is a plan to engineer a solution to address the specific characteristic.
Step 4:	Examine each column representing the different retrofitting measures. If one or more N/A boxes are circled in a column representing a retrofitting measure, that alternative is eliminated from consideration.
Step 5:	The numbers enclosed in the boxes represent special considerations (detailed below) which must be accounted for to make the measure applicable. If the consideration cannot be addressed, the number should be circled and the measure eliminated from consideration.
Step 6:	Retrofitting measures that remain should be further evaluated for technical, benefit-cost, and other considerations. A preferred measure should evolve from the evaluation.
N/A	Not advisable in this situation.
N/P	Not permitted in this situation.
1	Fast flood velocity is conducive to erosion and special features to resist anticipated erosion may be required.
2	Flash flooding usually does not allow time for human intervention; thus, these measures must perform without human intervention. Openings in foundation walls must be large enough to equalize water forces and should not have removable covers. Closures and shields must be permanently in place, and wet floodproofing cannot include last-minute modifications.
3	Permeable soils allow seepage under floodwalls and levees; therefore, some type of subsurface cutoff feature would be needed beneath structures. Permeable soils become saturated under flood conditions, potentially increasing soil pressures against a structure, therefore some type of foundation drain system or structure may be needed.
4	Ice and debris loads should be considered and accounted for in the design of foundations and floodwall/levee closures.
5	Any retrofitting alternative considered for the floodway must meet NFIP, state, local, and community floodplain requirements concerning encroachment/obstruction of the floodway conveyance area.
6	Not advisable in this situation, unless a specific engineering solution is developed to address the specific characteristic or constraint.

Figure III-4: Instructions for Retrofitting Screening Matrix

FLOODING CHARACTERISTICS

Riverine flooding is usually the result of heavy or prolonged rainfall or snowmelt occurring in upstream inland watersheds. In some cases, especially in and around urban areas, flooding can also be caused by inadequate or improper drainage. In coastal areas subject to tidal effects, flooding can result from wind-driven and prolonged high tides, poor drainage, storm surges with waves, and tsunamis.

There are several different flood characteristics that must be examined to determine which retrofitting measure will be best suited for a specific location. These characteristics not only indicate the precise nature of flooding for a given area, but can also be used to anticipate the performance of different retrofitting measures. These factors are outlined below.

Flood Depth

Determining the potential depth of flooding for certain flood frequencies is a critical step because it is often the primary factor in evaluating the potential for flood damage.

A building is susceptible to floods of various depths. Floods of greater depth occur less frequently than those of lesser depths. Potential flood elevations from significant flooding sources are shown in Flood Insurance Studies (FIS) for most participating NFIP communities. For the purpose of assessing the depth of flooding a structure is likely to endure, it is convenient to use the flood levels shown in the study, historical flood levels, and flood information from other sources. The depth of flooding affecting a structure can be calculated by determining the height of the flood above the ground elevation at the site of the structure.

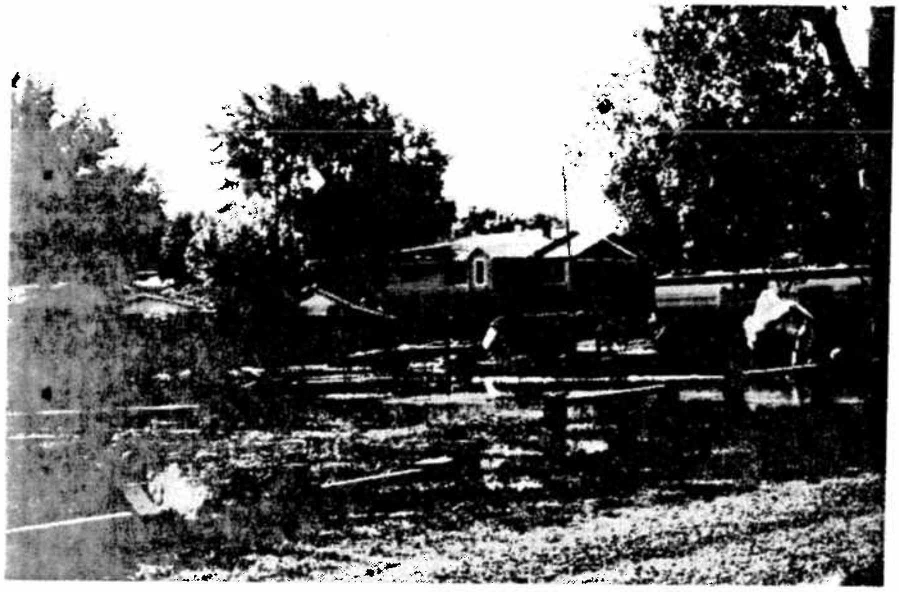


Figure III-5: Photographs showing mud lines on homes are a source of historical information

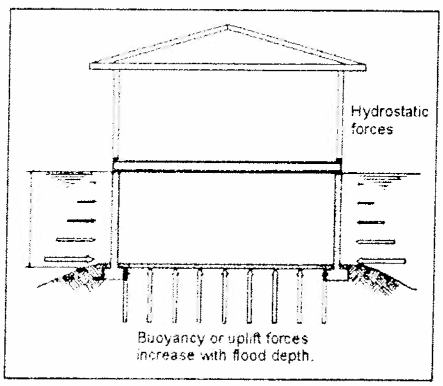


Figure III-6: Hydrostatic Forces

For those areas outside the limits of an FIS or state, community, or privately prepared local floodplain study, determination of flood depth may require a detailed engineering evaluation of local drainage conditions to develop the necessary relationship between flow (discharge), water-surface elevation, and flood frequency. The designer should contact the local municipal engineer, building official, or floodplain administrator for guidance on computing flood depth in areas outside existing study limits.

Floodwaters can impose hydrostatic forces on buildings. These forces result from the static mass of water acting on any point where floodwater contacts a structure. They are equal in all directions and always act perpendicularly (or normally) to the surfaces on which they are applied. Hydrostatic loads can act vertically on structural members such as floors and decks (buoyancy forces) and laterally (hydrostatic forces) on upright structural members such as walls, piers, and foundations. Hydrostatic forces increase linearly as the depth of water increases. Figure III-6 illustrates the hydrostatic forces generated by water depth.

If a well-constructed building is subject to flooding depths of less than three feet, it is possible that unequalized hydrostatic forces may not cause significant damage. Therefore, consideration can be given to using barriers, sealants, and closures as retrofitting measures. If shallow flooding (less than three feet) causes a basement to fill with water, wet floodproofing methods can be used to reduce flood damage to basements.

If a residential building is subject to flooding depths greater than three feet, elevation or relocation are often the most effective methods of retrofitting. Water depths greater than three feet can often create hydrostatic forces with enough load to cause structural damage or collapse if the house is not moved or elevated. One other potential method (provided the cost is not prohibitive) is the use of levees and floodwalls designed to withstand flooding depths greater than three feet.

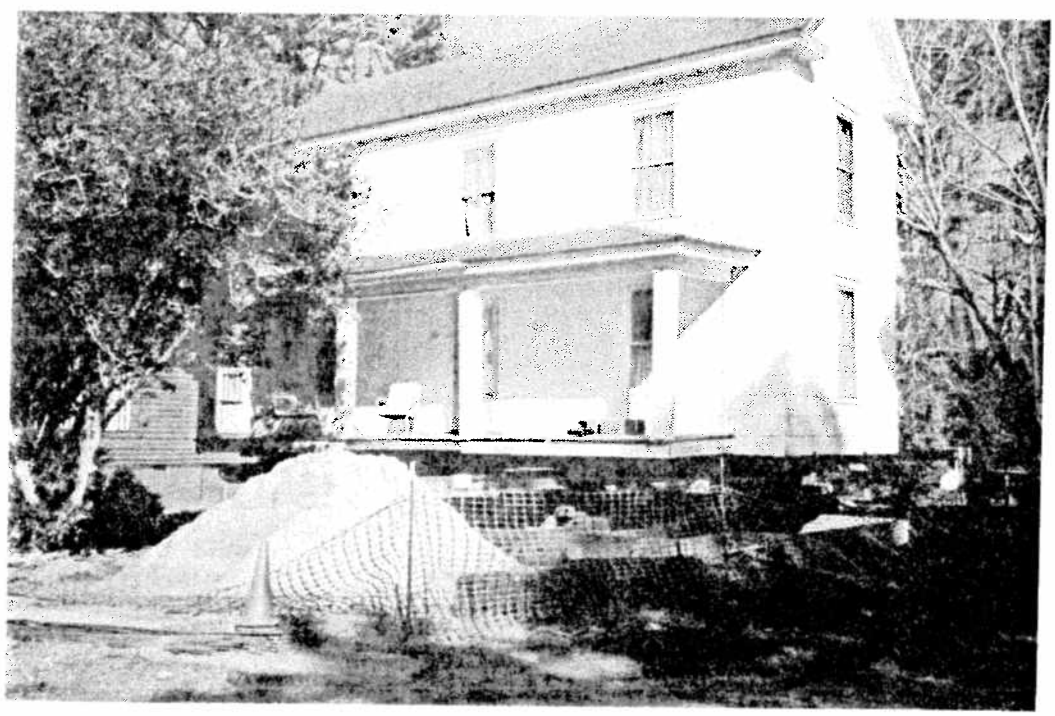


Figure III-7: Buoyancy forces from flood waters caused structure to lift off foundation

Flood Velocity

The speed at which floodwaters move (flood flow velocity) is normally expressed in terms of feet per second (fps). As floodwater velocity increases, hydrodynamic forces imposed by moving water are added to the hydrostatic forces from the depth of still water, significantly increasing the possibility of structural failure. Hydrodynamic forces are caused by water moving around an object and consist of positive frontal pressure against the structure, drag forces along the sides, and negative pressures on the building's downstream face. Greater velocities can quickly erode, or scour, the soil supporting and/or surrounding buildings. Thus, the impact, drag, and suction from these fast-moving waters may move a building from its foundation or otherwise cause structural damage or failure.

> The use of existing and historical data can be very useful in analyzing the flood threat. Through interviews with residents, approximate dates of flooding may be established, as well as remembered depths of flooding, types of velocity (moving or standing water), duration of flooding, etc. Once the dates have been established, the designer can check other sources such as newspapers and the National Weather Service for additional information.

Unfortunately, there is usually no definitive source of information to determine potential flood velocities in the vicinity of specific buildings. Hydraulic computer models or hand computations based on existing floodplain studies may provide flood velocities in the channel and overbank areas. Where current analysis data is not available, historical information from past flood events is probably the most reliable source.

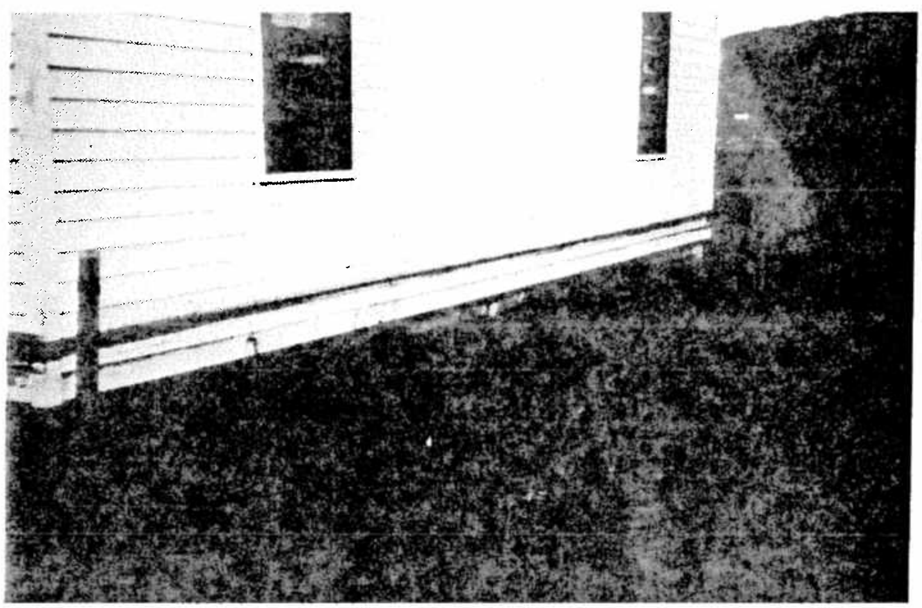


Figure III-8: Fast-moving floodwaters caused scour around the foundation and damage to the foundation wall

Onset of Flooding

Flash flooding will usually preclude the use of any retrofitting measure that requires human intervention.

In areas of steep topography or those areas with a small drainage area, floodwaters can rise very quickly with little or no warning. This condition is known as flash flooding. High velocities usually accompany these floods and may preclude certain types of retrofitting, especially those requiring human intervention. In a flash flooding situation, damage usually begins to occur within one hour after significant rainfall. If a building is susceptible to flash floods, insufficient warning time can preclude the installation of shields on windows, doors and floodwalls, as well as the activation of pump systems and backup energy sources. Temporarily relocating movable contents to a higher level may also be impractical. However, such measures may be effective if a building is not subject to flash flooding and the area has adequate flood warning systems, such as television and radio alerts.

A detailed hydrograph can provide information on duration of flooding. However, such information is usually not available, and the cost of creating a new study is usually prohibitive. One potential source of information is to check **similarly** sized drainage basins in neighboring areas to see if historical data exists.

Flood Duration

In areas of long-duration flooding, certain measures such as dry floodproofing may be inappropriate due to the increased chance of seepage and failure caused by prolonged exposure to floodwaters.

SITE CHARACTERISTICS

Site characteristics such as location, underlying soil conditions, and erosion vulnerability play a critical role in the determination of an applicable retrofitting method.

Site Location

The floodplain is usually defined as the area inundated by a flood having a 100-year flood frequency. The riverine floodplain is often further divided into a floodway and a floodway fringe.

As defined earlier, the floodway is the portion of the floodplain that contains the channel and enough of the surrounding land to enable floodwaters to pass without increasing flood depths greater than a predetermined amount. If there are high flood depths and/or velocities, this area is the most dangerous portion of the riverine floodplain. Also, since the floodway carries most of the flood flow, any obstruction may cause floodwaters to back up and increase flood levels. For these reasons, the NFIP and local communities prohibit new construction or substantial improvement in identified floodways that would increase flood levels. Relocation is the recommended retrofitting option for a structure located in a floodway. Community and state regulations may prohibit elevation of structures in this area. However, elevation on an open foundation will allow for more flow conveyance than a structure on a solid foundation.

The portion of the floodplain outside the floodway is called the floodway fringe. This area normally experiences shallower flood depths and lower velocities. With proper precautions, it is often possible to retrofit structures in this area with an acceptable degree of safety.

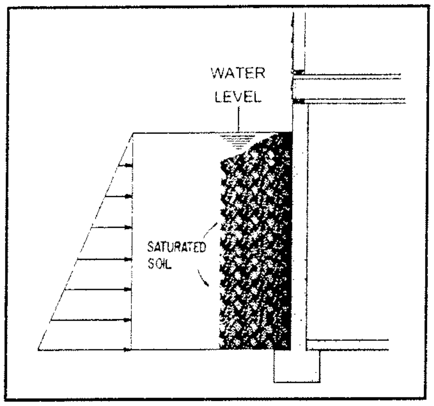


Figure III-9: Lateral Forces Resulting From Saturated Soil

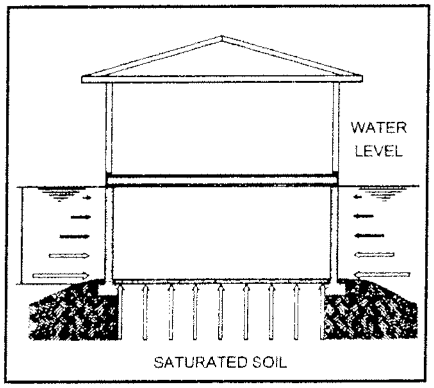


Figure III-10: Buoyancy Forces Resulting From Saturated Soil

Contact the local office of the Natural Resource Conservation Service (NRCS) or a local geotechnical engineering firm to obtain guidance on the permeability or consolidation features of soils native to the area. Because the site may have been backfilled with non-native materials during original construction, NRCS data should be used carefully.

Soil Type

Permeable soils, such as sand and gravel, are those that allow groundwater flow. In flooding situations, these soils may allow water to pass under floodwalls and levees unless extensive seepage control measures are employed as part of the retrofitting measures. Also, saturated soil pressure may build up against basement walls and floors. These conditions cause seepage, disintegration of certain building materials, and structural damage. Levees, floodwalls, sealants, shields, and closures may not be effective in areas with highly permeable soil types.

Saturated soils subject horizontal surfaces, such as floors, to uplift forces, called buoyancy. Like lateral hydrostatic forces, buoyancy forces increase in proportion to the depth of water/saturated soil above the horizontal surface. Figures III-9 and III-10 illustrate the combined lateral saturated soil and buoyancy forces.

For example, a typical wood-frame home without a basement or proper anchoring may float if floodwaters reach three feet above the first floor. A basement without floodwater in it could fail when the ground is saturated up to four feet above the floor. Uplift forces occur in the presence of saturated soil. Therefore, well-designed, high-capacity subsurface drainage systems with sump pumps may be an effective solution and may allow the use of dry floodproofing measures.

Other problems with soil saturated by floodwaters need to be considered. If a building is located on unconsolidated soil, wetting of the soil may cause uneven (differential) settlement. The building may then be damaged by inadequate support and subject to rotational, pulling, or bending forces. Some soils, such as clay or silt, may expand when exposed to floodwaters, causing massive forces against basement walls and floors. As a result, buildings may sustain serious damage even though floodwaters do not enter or even make contact with the structure itself.

BUILDING CHARACTERISTICS

Ideally, a building consists of three different components: substructure, superstructure, and support services. The substructure consists of the foundation system; the superstructure consists of the portion of the building envelope above the foundation system. The support services are those elements that are introduced into a building to make it habitable.

These components are integrally linked together to help a building maintain its habitability and structural integrity. Any action that considerably affects one may have a minimal or sometimes drastic effect on the others. An understanding of building characteristics and types of construction involved is therefore an important consideration in deciding upon an appropriate retrofitting measure.

Substructure

A cracked foundation is one indication of a weak foundation. The use of floodwalls and levees may be the easiest and most practical approach to retrofitting a structure with a poor foundation. Another solution may be an entire relocation of the building's superstructure onto a new foundation.

The substructure of a building supports the building envelope. It includes components found beneath the earth's surface, as well as above-grade foundation elements. This system consists of both the vertical foundation elements such as walls, posts, piles, and piers, which support the building loads and transmit them to the ground, and the footings that bear directly on the soil.

At any given time, there are a number of different kinds of loads acting on a building. The foundation system transfers these loads safely into the ground. In addition to dead and live loads, retrofitting decisions must take into account the buoyant uplift thrust on the foundation, the horizontal pressure of floodwater against the building, and any loads imposed by multiple hazards such as wind and earthquake events.

The ability of a foundation system to successfully withstand these and other loads or forces, directly or indirectly, is dependent to a large extent on its structural integrity. A designer should determine the type and condition of a building's foundation system early in the retrofitting evaluation.

Retrofitting of structures with basements is not covered in this manual.

All foundations are classified as either shallow or deep. Shallow foundations consist of column and wall footings, slab-on-grade, crawl space, and basement substructures; deep foundations include piles. Even though each of these foundation types may be utilized either individually or in combination with others, most residential buildings located outside coastal high hazard areas are supported on shallow foundations. Each type has its own advantages and limitations when retrofitting measures are being evaluated. Whichever is used in a building, a designer should carefully check for the structural soundness of the foundation system.

Basement walls may be subject to increased hydrostatic and buoyancy forces; thus, retrofitting a building with a basement is often more involved and costly.

Superstructure

The superstructure is the portion of the building envelope above the foundation system. It includes walls, floors, roof, ceiling, doors, and other openings. A designer should carefully and thoroughly analyze the existing conditions and component parts of the superstructure to determine the best retrofitting options available. Flood- and non-flood-related hazard effects should also be considered; the uplift, suction, shear, and other pressures exerted on building and roof surfaces by wind and other environmental hazards may be the only reasons needed to rule out elevation as a retrofitting measure.

Support Services

These are elements that help maintain a human comfort zone and provide needed energy, communications, and disposal of water and waste. For a typical residential building, the combination of the mechanical, electrical, telephone, cable TV, water supply, sanitary, and drainage systems provides these services. An understanding of the nature and type of services used in a building is necessary for a designer to be able to correctly predict how they may be affected by retrofitting measures.

For example, the introduction of new materials or the alteration of a building's existing features may require resizing existing services to allow for the change in requirements. Retrofitting may also require some form of relocated ductwork and electrical rewiring. Water supply and waste disposal systems may have to be modified to prevent future damage. This is particularly true when septic tanks and groundwater wells are involved. If relocation is being considered, the designer must consider all these parameters and weigh the cost of repairs and renovation against the cost of total replacement.

Building Construction

For general consideration of retrofitting measures, all construction should be classified as wood material unless all walls and foundations are concrete and masonry.

Modern buildings are constructed with a limitless palette of materials integrated into various structural systems. A building may be constructed with a combination of various materials. Thus, the suitability of applying a specific retrofitting measure may be difficult to assess.

Concrete and masonry construction may be considered for all types of retrofitting measures, whereas other materials may not be structurally sound or flood-damage resistant and therefore not suitable for some measures. When classifying building construction as concrete and masonry, it is important that all walls and foundations be constructed of this material. Otherwise, there may be a weak link in the retrofitting measure, raising the potential for failure when floods exert hydrostatic or hydrodynamic forces on the structure.

Masonry-veneer-over-wood-frame construction must be identified since wood-frame construction is less resistant to lateral loading than a brick-and-block wall section.

Building Condition

Typically, a designer will begin a retrofitting project with an initial analysis of the present conditions. Decisions based on early findings may be revised after a more detailed analysis.

A building's condition may be difficult to evaluate, as many structural defects are not readily apparent. However, careful inspection of the property should provide for a classification of "excellent to good" or "fair to poor." This classification is only for the reconnaissance phase of selecting appropriate retrofitting measures. More in-depth planning and design may alter the initial judgment regarding building condition, thereby eliminating some retrofitting measures from consideration at a later time.

Analysis of a building's substructure, superstructure, and support services may be done in two stages–an initial analysis usually based on visual inspection, and a detailed analysis (discussed in Chapter VI) which is often more informative, involves greater scrutiny, and usually requires detailed engineering calculations.

In the course of an analysis, a designer should visually inspect the walls, floors, roof, ceiling, doors, windows, and other superstructure and substructure components. Walls should be examined for type of material, structural stability, cracks, and signs of distress. A crack on a wall or dampness on concrete, plaster, wood siding, or other wall finishes may be a sign of concealed problems. Doors, windows, skylights, and other openings should be checked for cracks, rigidity, structural strength, and weather resistance.

Metal-clad wood doors or panel doors with moisture-resistant paint, plastic, or plywood exterior finishes may appear fine even though the interior cores may be damaged. Aluminum windows may be checked for deterioration due to galvanic action or oxidation from contact with floodwater. Steel windows may be damage-free if they are well protected against corrosion. Wood windows require inspection for shrinkage and warping, and for damage from moisture, mold, fungi, and insects.

Flooring in a building can include a vast range of treatments. It involves the use of virtually every material that can be walked upon, from painted concrete slabs to elegant, custom-designed wood parquet floors. A designer should investigate the nature of both the floor finishes and the underlying subfloor. Vinyl or rubberized plastic finishes may appear untouched due to their resistance to indentations and water; however, the concrete or wood subfloor may have suffered some damage. Likewise, a damage-free subfloor may be covered with a scarred finish.

An initial analysis of the conditions of the roof and ceiling of a building can be done by observation during the early decision-making stage. An understanding of the materials and construction methods will be necessary at a later date to evaluate fully the extent of possible damage and need to retrofit. The roofs over most residential buildings consist of simple to fairly complex wood framing that carries the ceilings below and plywood roof decks above, over which the roof finishes are placed. Finish materials include asphalt, wood, metal, clay and concrete tile, asbestos, and plastic and are available in various compositions, shapes, and sizes. In some cases, observation may be enough to determine the suitability, structural rigidity, and continuing durability of a roof system. However, it may be necessary to pop up a ceiling tile; remove some shingles, slate, or roof tiles; or even bore into a roof to achieve a thorough inspection.

The inspection also determines if the building materials and component parts are sound enough for the building easily to undergo either elevation, relocation, or wet or dry floodproofing. If not, floodwalls or levees around the structure may be the best alternative if allowable.

Figure III-11 presents a template that a designer can utilize to document findings during the initial building condition survey.

Owner Name:_______________	Prepared By:_______________
Address:_______________	Date:_______________
Property Location:_______________	

Preliminary Building Condition Evaluation Worksheet

Building Components	Condition		Notes and Materials
	Excellent to Good	Fair to Poor	
Substructure Footings Foundation Foundation Walls Other _______________			
Superstructure Floors Walls Ceilings Doors Windows Roof Other _______________			
Support Services Heating System Plumbing System Air Conditioning System Water Supply Sewage Other _______________			
Comments			

Figure III-11: Preliminary Building Condition Evaluation Worksheet

BALANCING HISTORIC PRESERVATION INTERESTS WITH FLOOD PROTECTION

Many historic building features were developed, either deliberately or intuitively, as responses to natural and environmental hazards, and to local climate or topography. Recognizing how and why these features were intended to work can help in designing a program of preventive measures that is historically appropriate and that minimizes incongruous modifications to historic residential properties.

There are retrofitting steps that will not have a negative or even significant impact upon the historic character of a site or its particular features. Preventive measures can be carried out without harming or detracting from historic character, as long as design and installation are carefully supervised by a professional knowledgeable in historic preservation.

There may well be instances, however, when a measure that best protects the site also may result in some loss of historic character. In such a case, the designer and the owner will have to weigh the costs of compromising character or historic authenticity against the benefits of safeguarding the site or a particular site feature against damage or total destruction. One example of such a choice is the decision whether to elevate a historic structure located in a flood hazard area, relocate it out of the area, retrofit it with wet or dry floodproofing techniques, or leave it in its existing state to face the risks of damage or loss. It is difficult to prescribe a formula for such a decision, since each situation will be unique, considering location, structural or site conditions, the variety of preventive alternatives available, cost, and degree of potential loss of historic character. Here are some questions the designer may wish to pose in deliberating such a decision:

- What is the risk that the historic feature or the entire site could be totally destroyed or substantially damaged if

the preventive measure is not taken? If the measure is taken, to what degree will this reduce the risk of damage or total destruction?

- Are there preventive alternatives that provide less protection from flood damage but also detract less from historic character? What are these, and what is the trade-off between protection and loss of character?

- Is there a design treatment that could be applied to the preventive measure to lessen detraction of historic character?

MULTIPLE HAZARDS

The selection of a retrofitting method may expose the structure to additional non-flood environmental hazards that could jeopardize the safety of the structure. These multiple hazards can be accommodated through careful design of the retrofitting measures or may necessitate selection of a different retrofitting method. Multiple hazards include both flood-related and non-flood-related hazards. Information concerning the analysis and design for these multiple hazards is contained in Chapters IV and VI.

The significant flood-related hazards to consider include ice and debris flow, impact forces, erosion forces, and mudslide or alluvial fan impacts. The major non-flood-related hazards to consider include earthquake and wind forces. Less significant hazards addressed in Chapter IV include land subsidence, fire hazards, snow loads, movable bed streams, and closed basin lakes. Multihazards may affect a structure independently, as with flood and earthquakes, or concurrently, as with flood and wind in a coastal area.

Flood-Related Hazards

IMPACT FORCES – ICE AND DEBRIS FLOW

In colder climates, floodwaters may carry chunks of ice that can act as a battering ram on a structure. During a flood, ice can also form around the structure. Rising floodwaters can lift a structure, resulting in severe damage. Flash and high-velocity floodwaters often carry debris such as cars, sheds, boulders, rocks, and trees that can destroy most retrofitting measures as well as the structure itself.

Retrofitting measures suitable for areas of ice and debris flow may include elevation on fill, relocation, levees, and armored floodwalls.

EROSION FORCES

If a soil is highly erodible, fast-moving floodwaters can undermine foundations and cause building, levee, or floodwall failures. The consideration of soil erosion is critical when retrofitting a building located in the floodplain. With the exception of deep foundation systems such as piles, shallow foundation systems generally do not provide sufficient protection against soil erosion without some type of protection or armoring measure of below-grade elements. The local office of the Natural Resources Conservation Service (NRCS) will generally have information concerning the erodibility of the soils native to a specific site. FEMA conducted an erosion mapping feasibility study that concluded that mapping of erosion-prone areas was feasible. Refer to the FEMA website www.fema.gov/mit/tsd/FT_reha.htm for additional information.

It is important to consider these multiple hazards when screening and selecting a retrofitting measure. However, the designer should be aware that structures can be engineered to withstand these multiple hazards, and the existence of these hazards alone may not justify the elimination of specific homeowner-preferred retrofitting methods. The local building codes normally contain additional guidance concerning natural hazard-resistant design and construction practices.

ALLUVIAL FANS

Because of the potential for high flood velocities, significant debris flow, and varying channel locations, alluvial fans present many unique challenges. In the upper portions of the fan, the only feasible retrofitting technique may be relocation. However, on lower portions of the fan where the flood velocities and depths are low, several options may be available. The hazards associated with alluvial fan flooding are discussed in detail in Appendix D of this manual.

Non-Flood-Related Hazards

EARTHQUAKE FORCES

> FEMA is currently involved in an interagency task force developing earthquake-resistant design standards in the wake of recent disasters. For additional information contact FEMA's Mitigation Directorate or the appropriate Regional FEMA office.

Earthquake protection steps can be divided into two categories: steps that deal with the building structure itself, and steps that can be taken with other parts of the building and its contents.

The most important step for the structure is making sure that it is properly bolted down onto its foundation so it will not slide off in an earthquake. Another important step, especially if the foundation is being raised to place the structure above flood levels, is to make sure the foundation can withstand an earthquake. For masonry block foundations, this usually means strengthening key portions of the wall by installing reinforcing bars in the blocks and then filling them with concrete grout.

WIND FORCES

> Strengthening an existing masonry block foundation wall can be complicated and normally requires the expertise of a designer knowledgeable in this type of work. The local building codes may contain additional guidance concerning earthquake-resistant design and construction materials.

High winds impose forces on a home and the structural elements of its foundation. Damage potential is increased when the wind forces occur in combination with flood forces. In addition, as a structure is elevated to minimize the effects of flood forces, the wind loads on the elevated structure may be increased.

A conventional structure is normally built to resist vertical downward loads (its own weight) plus live loads (contents, people) on the floor and snow and wind loads on the roof. Occasionally, structural elements are laid on top of each other with minimal fastening. However wind forces can be upwards, or from any direction exerting considerable pressure on structural components such as walls, roofs, connections, and anchorage. Therefore, wind loads should be considered in the design process at the same time as hydrostatic, hydrodynamic, and impact dead and live loads as prescribed under the applicable codes.

CHAPTER IV

DETERMINATION OF HAZARDS

Featuring:

Analysis of Flood-Related Hazards
Analysis of Non-Flood-Related Hazards
Geotechnical Considerations

DETERMINATION OF HAZARDS

ANALYSIS OF FLOOD-RELATED HAZARDS

- Flood Depth
- Hydrostatic Forces
- Hydrodynamic Forces
- Impact Loads
- Riverine Erosion
- Interior Drainage
- Alluvial Fans
- Closed Basin Lakes
- Movable Bed Steams

ANALYSIS OF NON-FLOOD-RELATED HAZARDS

- Wind Forces
- Seismic Forces
- Land Subsidence

GEOTECHNICAL CONSIDERATIONS

- Bearing Capacity
- Scour Potential
- Frost Zone
- Permeability
- Shrink-Swell Potential

Chapter IV: Determination of Hazards

Table of Contents

DETERMINATION OF HAZARDS

Chapters I through III introduced retrofitting and guided the designer through the technical process of pre-selecting retrofitting techniques for consideration. In this chapter, the analyses necessary to determine the flood- and non-flood-related forces and other site-specific considerations that control the design of a retrofitting measure are presented. This information may be useful in preparing benefit/cost analyses and determining which retrofitting alternatives are infeasible. The analysis of hazards contributes to the design criteria for retrofitting measures, which are described in Chapter VI.

Retrofitting measures must be designed, constructed, connected, and anchored to resist flotation, collapse, and movement due to all combinations of loads appropriate to the situation, including:

- flood-related hazards, such as hydrostatic and hydrodynamic forces, impact forces, interior drainage considerations, and the effects of erosion;
- site-specific flood-related hazards, such as alluvial fans, closed basin lakes, and movable bed streams;
- non-flood-related environmental loads, such as earthquake and wind forces and land subsidence; and
- site-specific soil or geotechnical considerations, such as soil pressure, bearing capacity, scour potential, shrink-swell potential, and permeability.

ANALYSIS OF FLOOD-RELATED HAZARDS

- Flood-Related Hazards
  - Flood Depth
  - Hydrostatic Forces
  - Hydrodynamic Forces
  - Impact Forces
  - Erosion Hazards
  - Interior Drainage
  - Alluvial Fans
  - Closed Basin Lakes
  - Movable Bed Streams

Figure IV-1: Flood-Related Hazards

The success of any retrofitting measure depends on an accurate assessment of the flood-related forces acting upon a structure. Floodwaters surrounding a building exert a number of forces on the structure, including lateral and vertical hydrostatic forces, hydrodynamic forces, impact forces, and erosion effects. Additionally, interior drainage, closed basin lakes, alluvial fans, and movable bed streams pose flood-related hazards that require consideration.

Hydrostatic forces (pressures) are caused by water above the surface of the ground that is either stagnant or moving slowly. Saturated soils beneath the ground surface also impose hydrostatic loads on foundation components.

Hydrodynamic forces (pressures) result from the moderate- or high-velocity flow of water against or around a structure. Impact loads are imposed on the structure by waterborne objects; their effects become greater as the velocity of flow, the weight of the objects, and the duration of the impact increase. The basic equations for analyzing and considering these flood-related forces are provided below.

FLOOD DEPTH

Riverine Areas

The determination of expected flood depth at a site is a critical aspect of the overall determination of flood-related hazards. One method of determining the 100-year water-surface elevation is to look at the Flood Insurance Rate Map (FIRM) panel depicting the location of the structure in question. On most FIRMs, floodplains are delineated for floods of 100- and 500-year frequencies. As an example,

> Additional information concerning the determination of flood-related forces is available in Section 5 of ASCE 7-98, *Minimum Design Loads for Buildings and Other Structures*, and ASCE 24-98, *Flood Resistant Design and Construction Standard*.

Figure IV-2 shows the portion of a community's FIRM where a subject home is located.

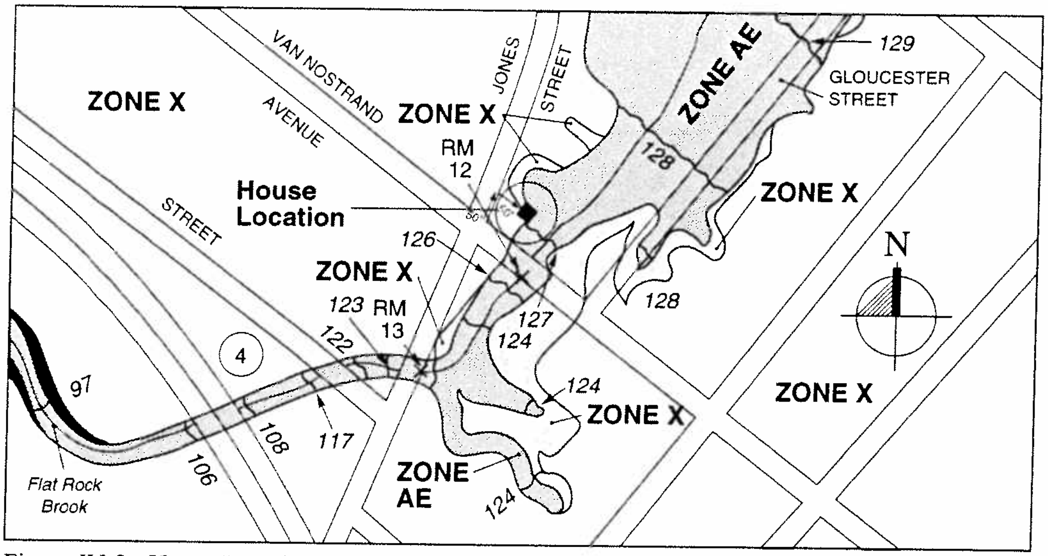


Figure IV-2: House Location on the FIRM

> Converting feet to inches on maps with small scales (greater than 1"=400') can introduce inaccuracies in locating the home and in specifying the flood elevations impacting the site.

In this example, the location of the home was determined by pacing off the distance from the intersection of Van Nostrand Avenue and Jones Street. The house is located approximately 50 feet north of the intersection. Converting this distance to the map's scale (one inch equals 400 feet), the house is 0.125 inches along Jones Street from its intersection with Van Nostrand Avenue, and 0.125 inches from Jones Street.

The darker shaded area on the map is the 100-year floodplain. The lighter shaded area denotes the 500-year floodplain. The house is located in this area between two wavy lines numbered 127 and 128. These are the 100-year flood elevations at those locations on Flat Rock Brook. Therefore, the 100-year flood elevation affecting the home in this example is between 127 and 128 feet, based on the National Geodetic Vertical Datum (NGVD).

Flood elevations for the other frequencies are shown on the stream's water-surface profile in the FIS report. For the above example, the position of the house on Flat Rock Brook was determined by drawing a line on its location on the FIRM (Figure IV-3) perpendicular to the stream. The point where this line crosses the streamline is the location of the house along the stream.

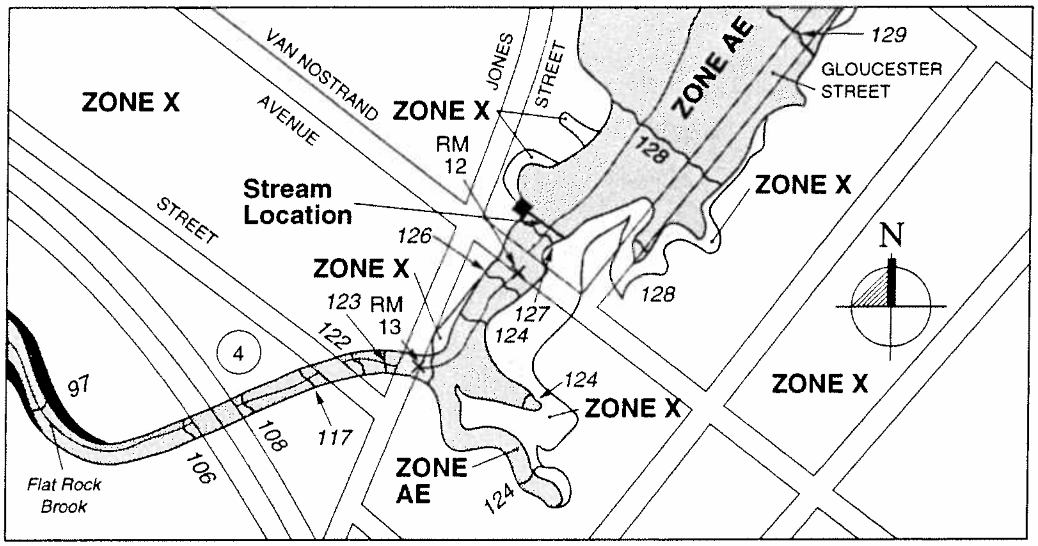


Figure IV-3: Stream Location on the FIRM

The distance along the stream (Figure IV-3) is then measured from the home to Van Nostrand Avenue, the nearest bridge structure across Flat Rock Brook. This distance is 0.11 inches, a measurement that when converted to the map scale is equal to approximately 45 feet (0.11 inches multiplied by 400 feet per inch of map).

The Van Nostrand Avenue bridge is then located on the Flat Rock Brook profile (Figure IV-4) and measured 0.45 inches upstream (45 divided by 100 feet per inch, which is the horizontal scale of the profile). This location is marked as the point on Flat Rock Brook with water-surface elevations equivalent to the house. The elevations on the profile at this point are 124.5, 125.9, 127.1, and 128.1 feet for the 10-, 50-, 100-, and 500-year floods, respectively. The bottom of the Flat Rock Brook channel shown on the profile is at 119.5 feet.

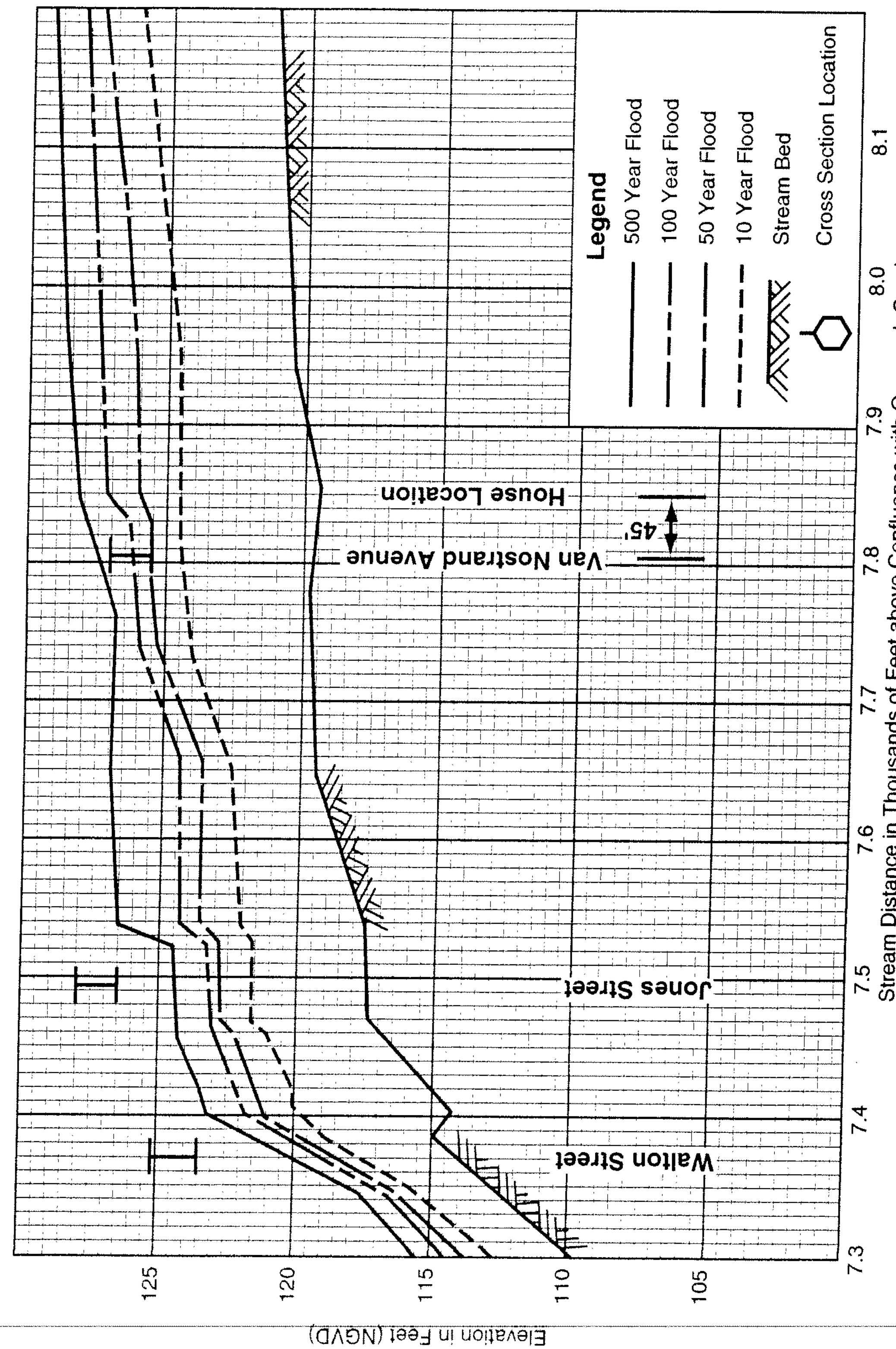


Figure IV-4: House Location on Flood Profile for Flat Rock Brook

Table IV-1 Flood Data Summary	
Frequenc	**Elevation**
Channel Bottom	119.5 ft.
10-yr.	124.5 ft.
50-yr.	125.9 ft.
100-yr.	127.1 ft.
500-yr.	128.1 ft.

Once the flood frequency and associated elevation information is obtained, a summary table can be created and used to calculate the depth of each flood frequency to be considered. Table IV-1 depicts the flood data obtained from the FIS for this example

Coastal Areas

In coastal areas, the determination of the expected water surface elevation for the various recurrence interval floods is made by locating the structure and its flooding source on the FIRM, identifying the corresponding flooding source/ location row on the summary of stillwater elevation table, and selecting the appropriate elevation for the recurrence interval in question.

As an example, consider a building located on Georgetown Street (as depicted on Figure IV-5). From the FIRM we can identify the flooding source as the Atlantic Ocean. Review of the entire area map for the FIS would indicate the Town of Fenwick Island (and Georgetown Street) is located between Bethany Beach and the Delaware-Maryland State Line.

Flood elevations in coastal A and V Zones are based on wave height and runup added to the stillwater elevation. For the 100-year frequency flood (BFE), refer to the FIRM. For other flood frequencies, the flood elevation can be estimated by multiplying 1.55 times the difference between the stillwater elevation and the ground surface elevation. A detailed discussion of the methodologies involved in computing wave heights and runup is beyond the scope of this manual. Refer to FEMA's *Guidelines and Specifications for Wave Elevation Determination and V Zone Mapping*, Third Draft, July 1989, or FEMA 55 (Third Edition): *Coastal Construction Manual*, for more information.

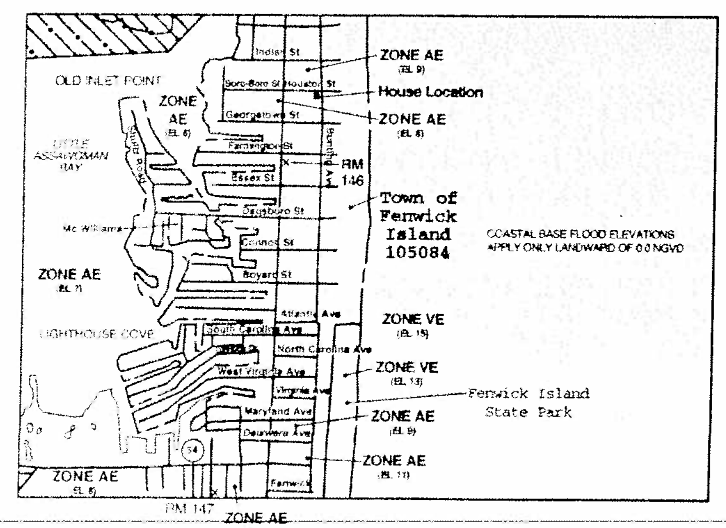


Figure IV-5: Coastal FIRM

This flooding source/location is located on the summary of stillwater elevations table (Figure IV-6). From this table, flood elevations of 6.2, 7.8, 8.6, and 10.2 feet above NGVD are identified for the 10-, 50-, 100- and 500-year frequency floods, respectively.

Summary of Stillwater Elevations

Flooding Source and Location	Elevation (feet NGVD*)			
	10-Year	50-Year	100-Year	500-Year
Atlantic Ocean				
Coastline from Cape Henlopen to just south of Dewey Beach	6.5	8.2	9.2	11.3
Coastline from just south of Dewey Beach to just north of Bethany Beach	6.4	8.0	8.9	10.8
Coastline from just north of Bethany Beach to Delaware-Maryland state line	6.2	7.8	8.6	10.2
Chesapeake Bay				
Coastline at Chance	4.2	5.4	5.8	6.8
Delaware Bay				
Coastline from Kent-Sussex County line to Cape Henlopen	6.6	8.5	9.3	11.3
Indian River Bay				
Entire coastline	4.7	6.4	7.5	10.8
Rehoboth Bay				
Entire coastline	3.9	5.9	7.0	10.8
Assawoman Bay				
Coastline within Sussex County	3.8	5.4	6.0	10.2
Little Assawoman Bay				
Entire Coastline	3.8	5.4	6.0	10.2

* National Geodetic Vertical Datum of 1929

Figure IV-6: Summary of Stillwater Elevations

Flood depth can be computed by subtracting the lowest ground surface elevation (grade) adjacent to the structure from the flood elevation for each flood frequency, as shown in Formula IV-1.

FORMULA

$$\mathbf{d = FE - GS = \_\_\_ \; feet}$$

where: **d** is the depth of flooding (in feet);
FE is the flood elevation for a specific flood frequency (in feet); and
GS is the lowest ground surface elevation (grade) adjacent to a structure (in feet).

Formula IV-1: Flood Depth

For design purposes, a factor of safety (freeboard) is typically added to the flood elevation to develop a retrofitting design level as illustrated in Formula IV-2: Design Flood Elevation.

When computing flood depth, be sure to utilize the lowest ground surface adjacent to the structure in question as shown in Figure IV-7.

FORMULA

$$\mathbf{DFE = FE + f = \_\_\_ \; feet}$$

where: **DFE** is the design flood elevation (in feet);
FE is the flood elevation for a specific flood frequency (in feet); and
f is the factor of safety (freeboard), typically a minimum of 1.0 foot.

Formula IV-2: Design Flood Elevation

The floodproofing design depth (H), which is used to calculate flood-related hazard forces, is the difference between the DFE and the lowest grade adjacent to the structure. This computation is shown in Formula IV-3.

$$H = DFE - GS = \_\_\_ \text{feet}$$

where: **H** is the floodproofing design depth over which flood forces are considered (in feet);

DFE is the design flood elevation for a specific flood frequency (in feet); and

GS is the lowest ground surface elevation (grade) adjacent to the structure (or other reference feature such as a slab or footing) (in feet).

Formula IV-3: Floodproofing Design Depth

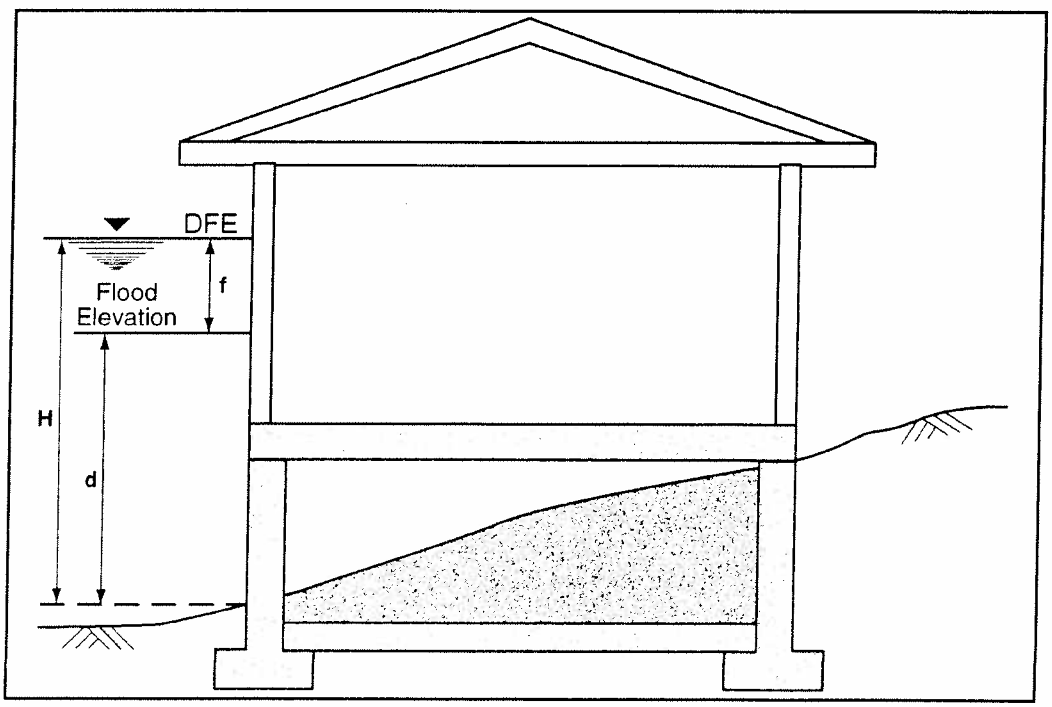


Figure IV-7: Illustration of Flood Depth and Design Depth

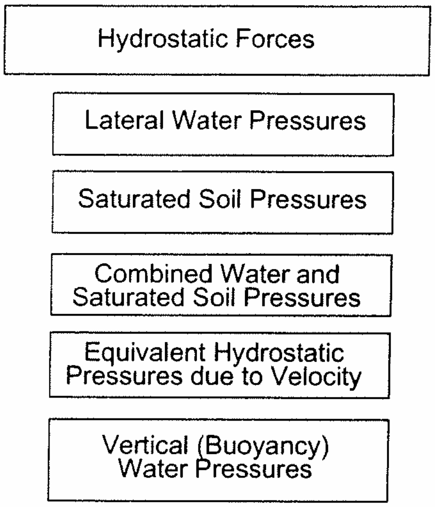


Figure IV-8: Hydrostatic Forces

HYDROSTATIC FORCES

Hydrostatic pressures (loads), at any point of floodwater contact with the structure are equal in all directions and always act in a perpendicular manner to the surface on which they are applied. Pressures increase linearly with depth or "head" of water above the point under consideration. The summation of pressures over the surface under consideration represents the load acting on that surface. For structural analysis, hydrostatic forces, as shown in Figures IV-9 and IV-10, are defined to act:

- vertically downward on structural elements such as flat roofs and similar overhead members having a depth of water above them;

- vertically upward (uplift) from the underside of generally horizontal members such as slabs, floor diaphragms, and footings (also known as buoyancy);

- laterally, in a horizontal direction on walls, piers, and similar vertical surfaces. (For design purposes, this lateral pressure is generally assumed to act on the receiving structure at a point one-third of the water depth above the base of the structure or two-thirds of the altitude from the water surface, which correlates to the center of gravity for a triangular pressure distribution.)

Hydrostatic forces include lateral water pressures, saturated soil pressures, combined water and soil pressures, equivalent hydrostatic pressures due to velocity flows, and vertical or buoyancy pressures. The computation of each of these pressures is illustrated in the sections that follow.

For the purpose of this document, it has been assumed that hydrostatic conditions prevail for stillwater and water moving with a velocity of less than ten feet per second.

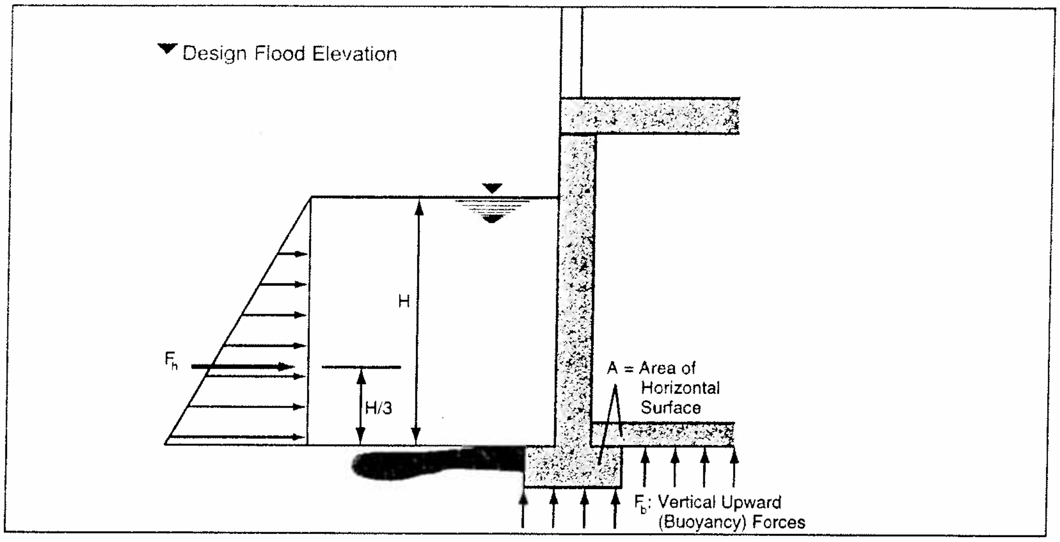


Figure IV-9: Hydrostatic Force

Hydrostatic loads generated by velocities up to 10 feet per second may be converted to an equivalent hydrostatic load using the conversion formula presented later in this chapter.

Lateral Hydrostatic Forces

The basic equation for analyzing the lateral force due to hydrostatic pressure from standing water above the surface of the ground is illustrated in Formula IV-4:

FORMULA

$$F_h = \frac{1}{2} P_h H = \frac{1}{2}\gamma H^2 = \_\_\_\_ \text{lbs/LF}$$

where: F_h is the lateral hydrostatic force from standing water (in pounds per linear foot of surface) acting at a distance H/3 from the point under consideration;

P_h is the hydrostatic pressure due to standing water at the point under consideration (in pounds per square foot), ($P_h = \gamma H$);

γ is the specific weight of fresh water (62.4 pounds per cubic foot); and

H is the floodproofing design depth (in feet).

Formula IV-4: Lateral Hydrostatic Forces

Saturated Soil Forces

If any portion of the structure is below grade, saturated soil forces must be included in the computation in addition to the hydrostatic force. This situation is illustrated in Figure IV-10. The basic equation for analyzing the resultant lateral force due to hydrostatic forces from saturated (non-expansive) soil is:

$$F_{sat} = ½\, S\, D^2 = ½\, P_D D = \_\_\_\_ \text{lb/LF}$$

where: F_{sat} is the lateral force from saturated soil acting at a distance D/3 from the point under consideration (in pounds per linear foot of surface);

P_D is the lateral hydrostatic pressure due to saturated soil at the point under consideration (in pounds per square foot);

S is the equivalent fluid weight of saturated soil (in pounds per cubic foot) as shown in column A of Table IV-2; and

D is the depth of saturated soil (in feet) over which hydrostatic forces are considered.

Formula IV-5: Saturated Soil Hydrostatic Forces

Formula IV-5: Saturated Soil Hydrostatic Forces is not suitable for expansive soils, due to the unpredictable nature of these soils. Due to the continual shrink and swell of expansive soil backfills and the variation of their water content, the stability and elevation of these soils and overlaying soil layers may vary considerably. The analysis of hydrostatic pressure and bearing capacity for expansive soils should be conducted by a qualified soils engineer. Preferably, expansive soils should be removed and replaced by stable soils.

The equivalent fluid pressures for various soil types are presented in Tables IV-2 and IV-3. The equivalent fluid weight of saturated soil is <u>not</u> the same as the effective weight of saturated soil. Rather, the equivalent fluid weight of saturated soil is a combination of the unit weight of water and the effective saturated weight of soil.

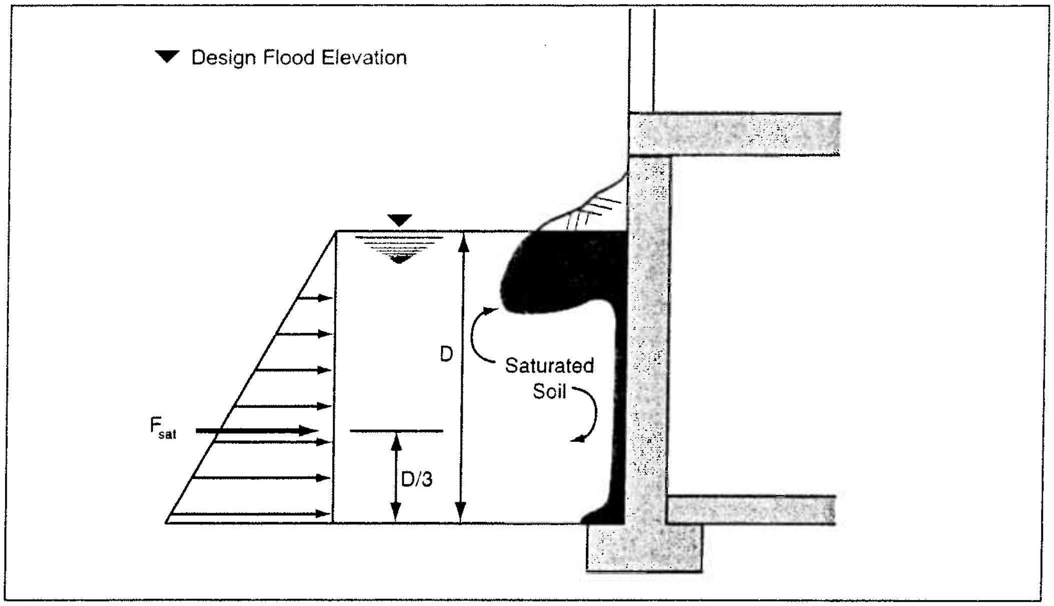


Figure IV-10: Saturated Soil Hydrostatic Forces

Table IV-2 **Effective Equivalent Fluid Weight of Soil (S)**		
	Column A	**Column B**
Soil Type*	**Equivalent Fluid Weight of Saturated Soil (pounds per cubic foot)**	**Equivalent Fluid Weight of Submerged Soil and Water (pounds per cubic foot)**
Clean sand and gravel: GW, GP, SW, SP	35	75
Dirty sand and gravel of restricted permeability: GM, GM-GP, SM, SM-SP	45	77
Stiff residual silts and clays, silty fine sands, clayey sands and gravels: CL, ML, CH, MH, SM, SC, GC	45	82
Very soft to soft clay, silty clay, organic silt and clay: CL, ML, OL, CH, MH, OH	100	106
Medium to stiff clay deposited in chunks and protected from infiltration: CL, CH	120	142

Notes: Use Column A with Formula IV-5,
Use Column B with Formula IV-6,
*See Table IV-3 for soil type definitions.

Table IV-3 **Soil Type Definitions Based on USDA Unified Soil Classification**

Soil Type	Group Symbol	Description
Gravels	GW	Well-graded gravels and gravel mixtures.
	GP	Poorly graded gravel-sand-silt mixtures.
	GM	Silty gravels, gravel-sand-silt mixtures.
	GC	Clayey gravels, gravel-sand-clay mixtures.
Sands	SW	Well-graded sands and gravelly sands.
	SP	Poorly graded sands and gravelly sands.
	SM	Silty sands, poorly graded sand-silt mixtures.
	SC	Clayey sands, poorly graded sand-clay mixtures.
Fine Grain Silt and Clays	ML	Inorganic silts and clayey silts.
	CL	Inorganic clays of low to medium plasticity.
	OL	Organic silts and organic silty clays of low plasticity.
	MH	Inorganic silts, micaceous or fine sands or silts, elastic silts.
	CH	Inorganic clays of high plasticity, fine clays.
	OH	Organic clays of medium to high plasticity.

Combined Water and Saturated Soil Forces

When a structure is subject to hydrostatic forces from both saturated soil and standing water (illustrated in Figure IV-11), the resultant cumulative lateral force, $\mathbf{F_H}$, is the sum of the lateral water hydrostatic force, $\mathbf{F_h}$, and the differential between the water and soil pressures, $\mathbf{F_{dif}}$. The basic equation for computing $\mathbf{F_{dif}}$ is:

$$\mathbf{F_{dif} = \tfrac{1}{2}\,(S - \gamma)\,D^2 = \_\_\_\,lbs/LF}$$

where: $\mathbf{F_{dif}}$ is the differential soil/water force acting at a distance D/3 from the point under consideration (in pounds per linear foot of surface);

$\mathbf{S}$ is the equivalent fluid weight of submerged soil and water (in pounds per cubic foot) as shown in column B of Table IV-2;

$\mathbf{D}$ is the depth of saturated soil (in feet); and

$\boldsymbol{\gamma}$ is the specific weight of fresh water (62.4 pounds per cubic foot).

Formula IV-6: Combined Water and Soil Forces

Note that while F_h and F_{dif} may not act at the same point, we can assume for structural analysis purposes that F_H acts at a distance H/3 above the point under consideration.

FORMULA

$$F_H = F_h + F_{dif} = \_\_\_ \text{lbs/LF}$$

where: F_H is the cumulative lateral hydrostatic force acting at a distance H/3 from the point under consideration (in pounds per linear foot of surface);

F_h is the lateral hydrostatic force from standing water (from Formula IV-4); and

F_{dif} is the differential soil/water force (from Formula IV-6).

Formula IV-7: Cumulative Lateral Hydrostatic Force

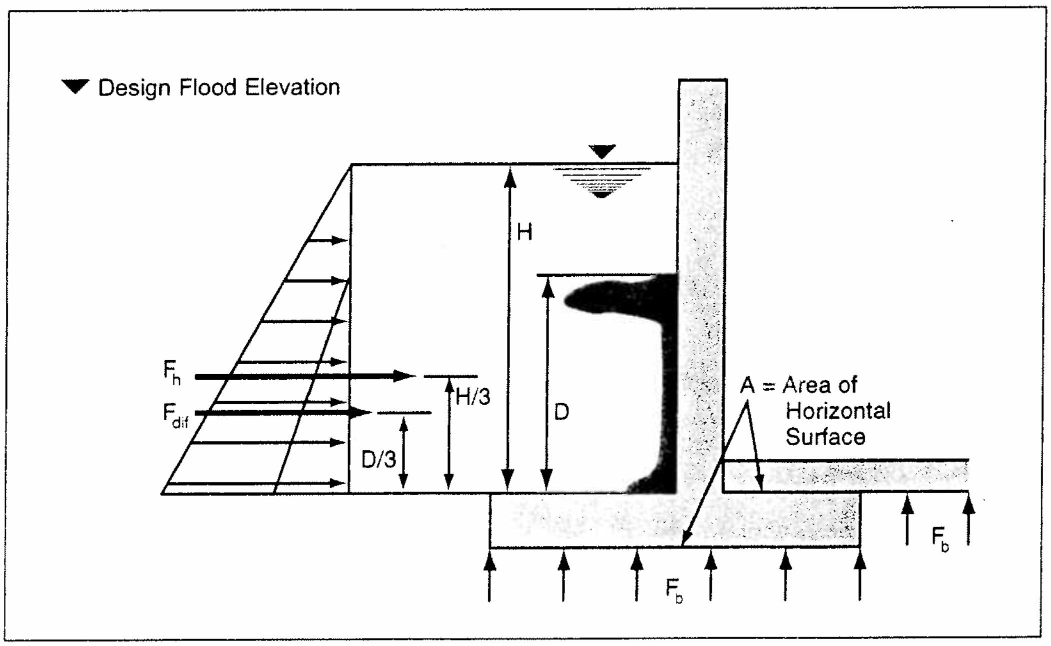


Figure IV-11: Combination Soil/Water Hydrostatic and Buoyancy Forces

Vertical Hydrostatic Force

The basic equation for analyzing the vertical hydrostatic force (buoyancy) due to standing water (illustrated by Figure IV-11) is:

$$F_b = \gamma A H = \_\_\_\_\_ \text{lbs}$$

where:

- F_b is the force due to buoyancy (in pounds);
- γ is the specific weight of fresh water (62.4 pounds per cubic foot);
- A is the area of horizontal surface (floor or slab) being acted upon (in square feet); and
- H is the floodproofing design depth (in feet).

Formula IV-8: Buoyancy Force

The computation of hydrostatic forces is vital to the successful design of floodwalls, sealants, closures, shields, foundation walls, and a variety of other retrofitting measures. The following Hydrostatic Force Computation Worksheet (Figure IV-12) can be utilized to conduct hydrostatic calculations. Figure IV-13, Example Hydrostatic Force Computation, illustrates the use of the worksheet.

HYDROSTATIC FORCE COMPUTATION WORKSHEET

Owner Name:______________________ Prepared By:____________
Address: ______________________ Date:________________
Property Location:_____________________________________

Variables:	Summary of Forces
H (Floodproofing Design Depth)=	F_h =
D (Depth of Saturated Soil) =	F_{sat} =
γ (Specific Weight of Fresh Water) = 62.4 lbs/cubic foot	F_{dif} =
S (Equivalent Fluid Weight of Saturated Soil) =	F_H =
A (Area) =	F_b=

Formula IV-4: Lateral Hydrostatic Force From Freestanding Water

$$F_h = \tfrac{1}{2} P_h H = \tfrac{1}{2}\gamma H^2 =$$

Formula IV-5: Lateral Hydrostatic Force From Saturated Soil

$$F_{sat} = \tfrac{1}{2} S D^2 \text{ or } \tfrac{1}{2} P_D D =$$

Formula IV- 6: Lateral Hydrostatic Force
From Standing Water and Saturated Soil

$$F_{dif} = \tfrac{1}{2} (S-\gamma) D^2 =$$

Formula IV-7: Total Lateral Hydrostatic Force From
Standing Water and Saturated Soil

$$F_H = F_h + F_{dif} =$$

Formula IV-8: Vertical Hydrostatic Force (Buoyancy)

$$F_b = \gamma A H =$$

Note: Formulas IV-4-6 do not account for equivalent hydrostatic loads due to velocity floodwaters (less than 10 fps.). If velocity floodwaters exist, recompute F_H using Formula IV-11.

Figure IV-12: Hydrostatic Force Computation Worksheet

HYDROSTATIC FORCE COMPUTATION WORKSHEET

Owner Name: SMITH — Prepared By: TCV
Address: 12 WATER STREET — Date: 10/31/94
Property Location: TM 38, SECTION 6, LOT 4

Variables:	Summary of Forces
H (Floodproofing Design Depth)= 4'	F_h = 499 lbs/LF
D (Depth of Saturated Soil) = 2'	F_{sat} = 150 lbs/LF
γ (Specific Weight of Fresh Water) = 62.4 lbs/cubic foot	F_{dif} = 25 lbs/LF
S (Equivalent Fluid Weight of Saturated Soil) = 75 lbs/ft³	F_H = 524 lbs/LF
A (Area) = 30'x40' = 1200 ft²	F_b = 299,520 lbs

Formula IV-4: Lateral Hydrostatic Force From Freestanding Water

$$F_h = \tfrac{1}{2} P_h H = \tfrac{1}{2}\gamma H^2 = \tfrac{1}{2}(62.4\ \text{lbs/ft}^3)(4\ \text{ft})^2 = 499\ \text{lbs/LF}$$

Formula IV-5: Lateral Hydrostatic Force From Saturated Soil

$$F_{sat} = \tfrac{1}{2} S D^2 \cancel{\text{ or } \tfrac{1}{2} P_D D} = \tfrac{1}{2}(75\ \text{lbs/ft}^3)(2\ \text{ft})^2 = 150\ \text{lbs/LF}$$

Formula IV- 6: Lateral Hydrostatic Force
From Standing Water and Saturated Soil

$$F_{dif} = \tfrac{1}{2}(S-\gamma) D^2 = \tfrac{1}{2}(75-62.4\ \text{lbs/ft}^3)(2\ \text{ft})^2$$
$$= 25\ \text{lbs/LF}$$

Formula IV-7: Total Lateral Hydrostatic Force From
Standing Water and Saturated Soil

$$F_H = F_h + F_{dif} = 499\ \text{lbs/LF} + 25\ \text{lbs/LF}$$
$$= 524\ \text{lbs/LF}$$

Formula IV-8: Vertical Hydrostatic Force (Buoyancy)

$$F_b = \gamma A H = (62.4\ \text{lbs/ft}^3)(1200\ \text{ft}^2)(4\ \text{ft})$$
$$= 299{,}520\ \text{lbs}$$

Note: Formulas IV-4-6 do not account for equivalent hydrostatic loads due to velocity floodwaters (less than 10 fps.). If velocity floodwaters exist, recompute F_H using Formula IV-11.

Figure IV-13: Example Hydrostatic Force Computation

HYDRODYNAMIC FORCES

Hydrodynamic forces have been shown to act on slab-on-grade houses to maximize their effects.

When floodwaters flow around a structure at moderate to high velocities, they impose additional loads on the structure, as shown in Figure IV-14. These loads consist of frontal impact by the mass of moving water against the projected width and height of the obstruction represented by the structure, drag effect along the sides of the structure, and eddies or negative pressures on the downstream side of the structure.

Low velocity hydrodynamic forces are defined as situations where floodwater velocities do not exceed 10 feet per second, while high velocity hydrodynamic forces involve floodwater velocities in excess of 10 feet per second.

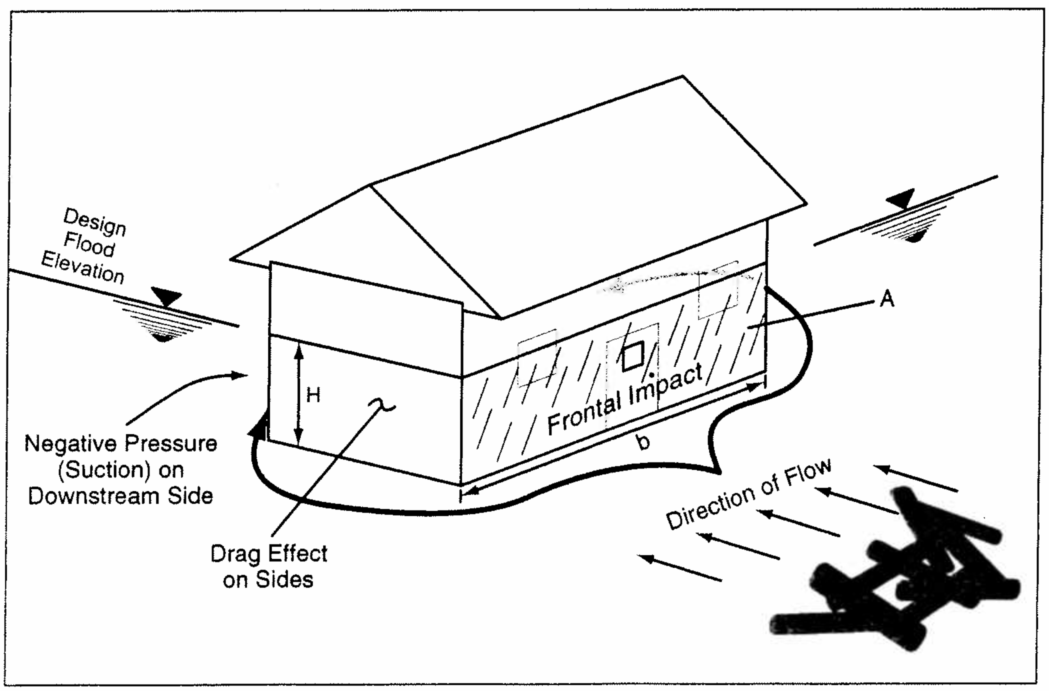


Figure IV-14: Hydrodynamic and Impact Forces

Sources of data for determining flood flow velocity include hydraulic calculations, historical measurements, and rules of thumb. Floodwaters one foot deep moving in excess of five feet per second can knock an adult over and cause erosion of stream banks. Overbank velocities are usually less than stream channel velocities. If no data for flood flow velocity exists for a site, the reader should contact an experienced hydrologist or hydraulic engineer for estimates.

Low Velocity Hydrodynamic Forces

In cases where velocities do not exceed 10 feet per second, the hydrodynamic effects of moving water can be converted to an equivalent hydrostatic force by increasing the depth of the water (head) above the flood level by an amount **dh**, which is:

FORMULA

$$dh = \frac{C_d V^2}{2g} = \_\_\_\_ \text{feet}$$

where: **dh** is the equivalent head due to low velocity flood flows (in feet);

C_d is the drag coefficient (from Table IV-4);

V is the velocity of floodwaters (in ft/sec); and

g is the acceleration of gravity (equal to 32.2 ft/sec²).

Formula IV-9: Conversion of Low Velocity Flow to Equivalent Head

The drag coefficient C_d depends on the proportions of the shape of the object around which the water flows. The value of C_d, unless otherwise evaluated, shall not be less than 1.25 and can be determined from the width-to-height ratio, **b/H**, of the structure in question. The width (**b**) is the side perpendicular to the flow and the height (**H**) is the distance from the floodproofing design depth to the lowest adjacent grade level. Table IV-4 gives C_d values for different width-to-height ratios.

Table IV-4 **Drag Coefficients**

Width to height Ratio b/H	Drag Coefficient C_d
From 1 to 12	1.25
13 to 20	1.3
21 to 32	1.4
33 to 40	1.5
41 to 80	1.75
81 to 120	1.8
160 or more	2.0

The value **dh** is then converted to an equivalent hydrostatic pressure through use of the basic equation for lateral hydrostatic forces introduced earlier in this chapter and modified as shown below:

FORMULA

$$F_{dh} = \gamma\,(dh)H = P_{dh}H = \_\_\_\_ \text{ lbs/LF}$$

where: F_{dh} is the equivalent hydrostatic force due to low velocity flood flows (in pounds per linear foot of surface);

γ is the specific weight of fresh water (62.4 pounds per cubic foot);

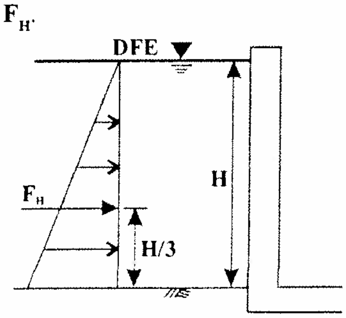
H is the floodproofing design depth in feet;

dh is the equivalent head due to low velocity flood flows in feet; and

P_{dh} is the hydrostatic pressure due to low velocity flood flows (in pounds per square foot) ($P_{dh} = \gamma\,(dh)$).

Formula IV-10: Conversion of Equivalent Head to Equivalent Hydrostatic Force

While F_{dh} acts at a point **H/2**, it is normally a <u>small</u> percentage of F_H, therefore, we can assume that F_{dh} acts at the same point **H/3** as F_H.



The resultant lateral hydrostatic force due to low velocity hydrodynamic pressures is then added to the lateral hydrostatic pressures due to standing water and saturated soil to obtain the total lateral hydrostatic force shown below and illustrated in the **Equivalent Hydrostatic Force Computation Worksheet,** Figures IV-15 and IV-16.

FORMULA

$$F_H = F_h + F_{dif} + F_{dh} = \_\_\_\_\_\_\_\_ \text{ lbs/LF}$$

where: variables were defined previously in Formulas IV-4, IV-6, IV-7, and IV-10.

Formula IV-11: Total Lateral Hydrostatic Force

EQUIVALENT HYDROSTATIC FORCE
COMPUTATION WORKSHEET

Owner Name:______________________ Prepared By:__________
Address: _________________________ Date:______________
Property Location:_____________________________________

Variables:	**Summary of Forces**
b (width of structure perpendicular to flow) =	F_{dh} =
H (floodproofing design depth) =	F_h =
V (velocity of flood water, 10 ft. per second or less) =	F_{dif} =
γ (specific weight of fresh water) = 62.4 lbs/cubic foot	F_H =
g (acceleration of gravity) = 32.2 feet per second squared	

Formula IV-9: Conversion of Low Velocity Flood Flow to Equivalent Head

$$dh = \frac{C_d V^2}{2g} =$$

Develop C_d:
b/H =
From Table IV-4; C_d =

Formula IV-10: Conversion of dh to Equivalent Hydrostatic Force

$$F_{dh} = \gamma\,(dh)\,H =$$

Formula IV-11: Total Lateral Hydrostatic Force

$$F_H = F_h + F_{dif} + F_{dh} =$$

Figure IV-15: Equivalent Hydrostatic Force Computation Worksheet

EQUIVALENT HYDROSTATIC FORCE COMPUTATION WORKSHEET

Owner Name: SMITH Prepared By: JJS
Address: 12 WATER STREET Date: 12/12/00
Property Location: TM 38, SECTION 6, LOT 4

Variables:	**Summary of Forces**
b (width of structure perpendicular to flow) = 30'	F_{dh} = 175 lbs/LF
H (floodproofing design depth) = 4'	F_h = 499 lbs/LF
V (velocity of flood water, 10 ft. per second or less) = 6 fps	F_{dif} = 25 lbs/LF
γ (specific weight of fresh water) = 62.4 lbs/cubic foot	F_H = 699 lbs/LF
g (acceleration of gravity) = 32.2 feet per second squared	

Formula IV-9: Conversion of Low Velocity Flood Flow to Equivalent Head

$$\mathbf{dh} = \frac{C_d V^2}{2g} = \frac{(1.25)(6\ ft/sec)^2}{2(32.2\ ft/sec^2)} = 0.70\ ft$$

Develop C_d:
b/H = 30/4 = 7.5
From Table IV-4; C_d = 1.25

Formula IV-10: Conversion of dh to Equivalent Hydrostatic Force

$$F_{dh} = \gamma\,(dh)\,H = (62.4\ lbs/ft^3)(0.70\ ft)(4\ ft) = 175\ lbs/LF$$

Formula IV-11: Total Lateral Hydrostatic Force

$$F_H = F_h + F_{dif} + F_{dh} = 499 + 25 + 175\ lbs/LF$$
$$= 699\ lbs/LF$$

Figure IV-16: Example Equivalent Hydrostatic Force Computation

High Velocity Hydrodynamic Forces

For special structures and conditions, and for velocities greater than 10 feet per second, a more detailed analysis and evaluation should be made utilizing basic concepts of fluid mechanics and/or hydraulic models. The basic equation for hydrodynamic pressure is:

FORMULA

$$P_d = C_d \rho \frac{V^2}{2} = \_\_\_\_\text{lbs/SF}$$

where: P_d is the hydrodynamic pressure (in pounds per square foot);
ρ is the mass density of fresh water (1.94 slugs/ft^3);
V is velocity of floodwater (in feet per second); and
C_d is the drag coefficient (taken from Table IV-4).

Formula IV-12: High Velocity Hydrodynamic Pressure

After determination of the hydrodynamic pressure (**P_d**), the total force (**F_d**) against the structure (see Figure IV-14) can be computed as the pressure times the area over which the water is impacting:

FORMULA

$$F_d = P_d A = \_\_\_\_\text{lbs}$$

where: F_d is the total force against the structure (in pounds);
P_d is the hydrodynamic pressure (in pounds per square foot); and
A is the submerged area of the upstream face of the structure in question (in square feet).

Formula IV-13: Total Hydrodynamic Force

Figure IV-17, Hydrodynamic Force (High Velocity) Computation Worksheet, can be used in the computation of high velocity hydrodynamic forces, while Figure IV-18 illustrates the computations.

HYDRODYNAMIC FORCE (HIGH VELOCITY) COMPUTATION WORKSHEET	
Owner Name:__________ Prepared By:__________ Address: __________ Date:__________ Property Location:__________	
Variables: **ρ** (mass density of fresh water) = 1.94 slugs/ft³ **V** (velocity of floodwater, ≥ 10 feet per second) $\mathbf{C_d}$ (drag coefficient) = **A** (submerged area of upstream face of structure) =	**Summary of Forces** $\mathbf{P_d}$ = $\mathbf{F_d}$ =
Formula IV-12: High Velocity Hydrodynamic Pressure (Force)	
$\mathbf{P_d = C_d \rho (V^2/ 2) =}$ Develop $\mathbf{C_d}$: b/H = From Table IV-4; $\mathbf{C_d}$ =	
Formula IV-13: Total Force Against the Structure	
$\mathbf{F_d = P_d A =}$	

Figure IV-17: Hydrodynamic Force (High Velocity) Computation Worksheet

HYDRODYNAMIC FORCE (HIGH VELOCITY) COMPUTATION WORKSHEET

Owner Name: SMITH Prepared By: TCV
Address: 12 WATER STREET Date: 10/31/94
Property Location:: TM 38, SECTION 6, LOT 4

Variables:
ρ (mass density of fresh water) = 1.94 slugs/ft³
V (velocity of floodwater, ≥10 feet per second) = 12 fps
C_d (drag coefficient) = 1.25
A (submerged area of upstream face of structure) = 4' x 30' = 120 ft²

Summary of Forces
P_d = 175 lbs/ft²
F_d = 21,000 lbs

Formula IV-12: High Velocity Hydrodynamic Pressure (Force)

$P_d = C_d \rho (V^2/2) =$

$$= \frac{1.25\left(1.94 \frac{slugs}{ft^3}\right)\left(\frac{\frac{lb\text{-}sec^2}{ft}}{slug}\right)\left(12 \frac{ft}{sec}\right)^2}{2}$$

$$= 175 \text{ lbs/ft}^2$$

Develop C_d:
b/H= 30/4 = 7.5
From Table IV-4; C_d = 1.25
Conversion: $1 \text{ slug} = \frac{lb\text{-}sec^2}{ft}$

Formula IV-13: Total Force Against the Structure

$F_d = P_d A = (175 \text{ lbs/ft}^2)(120 \text{ ft}^2)$

$= 21{,}000 \text{ lbs}$

Figure IV-18: Example Hydrodynamic Force (High Velocity) Computation

IMPACT LOADS

FEMA 55 (Third Edition): *Coastal Construction Manual* contains information on computing impact loads in coastal situations.

Impact loads are imposed on the structure by objects carried by the moving water. These loads are the most difficult to predict and define, yet reasonable allowances must be made for these loads in the design of retrofitting measures for potentially affected buildings. To arrive at a realistic allowance, considerable judgment must be used, along with the designer's knowledge of debris problems at the site and consideration of the degree of exposure of the structure. Impact loads are classified as either:

- no impact (for areas of little or no velocity or potential source of debris);
- normal impact;
- special impact; or
- extreme impact.

Normal Impact Forces

The assumption that debris velocity is equal to the flood velocity may overstate the velocities of large debris objects; therefore, engineering judgment may be required in some instances. Designers may wish to reduce debris velocity for larger objects.

Normal impact forces relate to isolated occurrences of typically sized debris or floating objects striking the structure (see Figure IV-14). For design purposes, this can be considered a concentrated load acting horizontally at the flood elevation, or any point below it, equal to the impact force created by a 1,000-pound mass traveling at the velocity of the floodwater acting on a one-square-foot surface of the submerged structure area perpendicular to the flow. The calculation of normal impact forces is shown in Formula IV-14.

FORMULA

$$F_n = \frac{MV}{t} = \frac{w_n V}{gt} = \_\_\_\_\_\_ \text{lbs}$$

where: F_n is the normal impact force (in pounds);
w_n is weight of object (1,000 lbs for normal impact loads);
g is acceleration of gravity (32.2 ft/sec^2);
t is time of impact (generally 1 sec or less);
V is velocity of flow (in feet per second); and
M is the mass of the object computed as w_n/g.

Formula IV-14: Normal Impact Force

Time/Duration of Impact

Uncertainty about the duration of impact (t)--the time from initial impact, to the time the object leaves--is the most likely cause of error in the calculation of debris impact loads. According to physics and dynamics texts, the duration of impact is influenced primarily by the natural frequency of the building, which is a function of the building's "stiffness." This stiffness is determined by the properties of the material being struck by the object, the number of supporting members (columns or piles), the height of the building above the ground, and the height at which the material is struck.

Although little guidance on duration of impact exists, the City of Honolulu Building Code recommends the following durations based on the type of construction being struck:

- wood: 1 second
- steel: 0.5 second
- reinforced concrete: 0.1 second

A complete mathematical analysis of this problem is beyond the scope of this manual; however, Table IV-5 suggests durations (t) to use in Formulas IV-14 and IV-15. These durations were developed with a mathematical model from dynamic theory. They are of approximately the same order of magnitude as those provided in the City of Honolulu Building Code.

Table IV-5 **Impact Durations (t) for Use in Formulas IV-14 and IV-15**

Type of Construction	Duration (t) of Impact (sec)	
	Wall	Pile
Wood	0.7 - 1.1	0.5 - 1.0
Steel	NA	0.2 - 0.4
Reinforced Concrete	0.2 - 0.4	0.3 - 0.6
Concrete Masonry	0.3 - 0.6	0.3 - 0.6

Special Impact Forces

Special impact forces occur when large objects or conglomerates of floating objects, such as ice floes or accumulations of floating debris, strike a structure. In an area where special impact forces may occur, the load considered for design purposes is the impact created by a 100-pound load times the width of building, acting horizontally over a one-foot-wide horizontal strip at the flood elevation or at any level below it. Where stable natural or artificial barriers exist that would effectively prevent these special impact forces from occurring, these forces may not need to be considered in the design.

FORMULA

$$F_s = \frac{MV}{t} = \frac{w_s V}{gt} = \frac{100bV}{32.2t} = \_\_\_\_ \text{lbs}$$

where: F_s is the special impact force (in pounds);

w_s is weight of object (in pounds) (100 lbs/ft x width of structure (**b**) normal to flow); **b** is shown in Figure IV-14;

b is the width of structure normal to flow (in feet);

g is acceleration of gravity (32.2 ft/sec^2);

t is time of impact (generally 1 sec or less);

V is velocity of flow (in feet per second); and

M is the mass of the object computed as w_s/g

Formula IV-15: Special Impact Forces

Whether impact loads should be allowed for depends on data that can be obtained from a number of sources:

- historic records and the FIS;
- interviews with local residents and floodplain management officials;
- floodway versus floodplain location;
- upstream debris potential; and
- climatologic information.

Impact forces are critical design considerations that must be thoroughly evaluated. The following **Impact Force Computation Worksheet**, Figure IV-19, can be used to conduct those calculations, while Figure IV-20 illustrates those calculations.

Where extreme impact loads are a threat, the preferred retrofitting alternative is relocation.

Extreme Impact Forces

Extreme impact forces occur when large, floating objects and masses, such as runaway barges or collapsed buildings and structures, strike the structure (or a component of the structure). These forces generally occur within the floodway or areas of the floodplain that experience the highest velocity flows. It is impractical to design residential buildings to have adequate strength to resist extreme impact forces.

IMPACT FORCE COMPUTATION WORKSHEET

Owner Name:________________________ Prepared By:__________
Address: ___________________________ Date:________________
Property Location: ____

Normal Impact Loads

Variables:

$\mathbf{w_n}$ (weight of object) = typically, 1,000 pounds
g (acceleration of gravity) = 32.2 ft/sec^2
t (time of impact) = typically, 1 sec.
V (velocity of floodwater) =
M (mass of the object computed as $\mathbf{w_n/g}$) =

Summary of Forces

$\mathbf{F_n}$ =
$\mathbf{F_s}$ =

Formula IV-14: Normal Impact Force

$$F_n = \frac{MV}{t} = \frac{w_n V}{gt}$$

Special Impact Loads

Variables:

b (width of structure normal to flow) =
$\mathbf{w_s}$ (weight of object) = 100 (b) =
g (acceleration of gravity) = 32.2 ft/sec^2
t (time of impact) = typically, 1 sec. or less
V (velocity of floodwater) =
M (mass of the object computed as $\mathbf{w_s/g}$) =

Formula IV-15: Special Impact Forces

$$F_s = \frac{MV}{t} = \frac{w_s V}{gt} = \frac{100bV}{32.2t} = \_\_\_\_ \text{lbs}$$

Figure IV-19: Impact Force Computation Worksheet

IMPACT FORCE COMPUTATION WORKSHEET

Owner Name: SMITH Prepared By: TCV
Address: 12 WATER STREET Date: 10/31/95
Property Location: TM 38, SECTION 6

Normal Impact Loads

Variables:
$\mathbf{w_n}$ (weight of object) = typically, 1,000 pounds
g (acceleration of gravity) = 32.2 ft/sec^2
t (time of impact) = typically, 1 sec.
V (velocity of floodwater) = 12 fps
M (mass of the object computed as w_n/g) = $\frac{1000 \text{ lbs}}{32.2 \text{ ft/sec}^2}$

Summary of Forces

F_n = 373 lbs
F_s = 1,118 lbs

Formula IV-14: Normal Impact Force

$$F_n = \frac{M\cancel{V}}{\cancel{t}} = \frac{w_n V}{gt} = \frac{(1000 \text{ lbs})(12 \text{ ft/sec})}{(32.2 \text{ ft/sec}^2)(1 \text{ sec})}$$

$$= 373 \text{ lbs}$$

Special Impact Loads

Variables:
b (width of structure normal to flow) = 30 ft
$\mathbf{w_s}$ (weight of object) = 100 (b) = 100 lbs/ft (30 ft) = 3000 lbs
g (acceleration of gravity) = 32.2 ft/sec^2
t (time of impact) = typically, 1 sec. or less
V (velocity of floodwater) = 12 fps
M (mass of the object computed as w_s/g) = 3000 lbs / 32.2 ft/sec^2 =

Formula IV-15: Special Impact Forces

$$F_s = \frac{M\cancel{V}}{\cancel{t}} = \frac{w_s V}{gt} = \frac{\mathbf{100bV}}{\mathbf{32.2t}} = \frac{(100 \text{ lbs/ft})(30 \text{ ft})(12 \text{ ft/sec})}{(32.2 \text{ ft/sec}^2)(1 \text{ sec})}$$

$$= 1{,}118 \text{ lbs}$$

Figure IV-20: Example Impact Force Computation

RIVERINE EROSION

The analysis of erosion that impacts stream banks and nearby overbank structures is a detailed effort that is usually accompanied by detailed geotechnical investigations. Some of the variables that impact the stability (or erodibility) of the stream banks include the following:

- critical height of the slope;
- inclination of the slope;
- cohesive strength of the soil in the slope;
- distance of the structure in question from the shoulder of the stream bank;
- degree of stabilization of the surface of the slope;
- level and variation of groundwater within the slope;
- level and variation in level of water on the toe of the slope;
- tractive shear stress of the soil; and
- frequency of rise and fall of the surface of the stream.

Both FEMA and the USACE have researched the stability of stream banks in an effort to quantify stream bank erosion. However, concerns over the universal applicability of the research results preclude their inclusion in this manual. It is suggested that when dealing with streambanks susceptible to severe erosion, the designer contact a qualified geotechnical engineer or a hydraulic engineer experienced in channel stability.

INTERIOR DRAINAGE

The drainage system for the area enclosed by a levee or floodwall must accommodate the precipitation runoff from this interior area (and any contributing areas such as roofs and higher ground parcels) and the anticipated seepage through or under the floodwall or levee during flooding conditions.

There are two general methods for removing interior drainage. The first is a gravity flow system, which provides a means for interior drainage of the protected area when there is no floodwater against the floodwall or levee. This is accomplished by placing a pipe(s) through the floodwall or levee with a flap gate attachment. The flap gate prevents flow from entering the interior area through the drainpipe when floodwaters rise above the elevation of the drain.

The second method, a pump system, removes accumulation of water when the elevation of the floodwater exceeds the elevation of the gravity drain system. A collection system composed of pervious trenches, underground tiles, or sloped surface areas transports the accumulating water to a sump area. In the levee application, these drains should be incorporated into the collection system. The anticipated seepage from under and through levees and floodwalls must also be taken into consideration by combining it with flow from precipitation (see Figure IV-21).

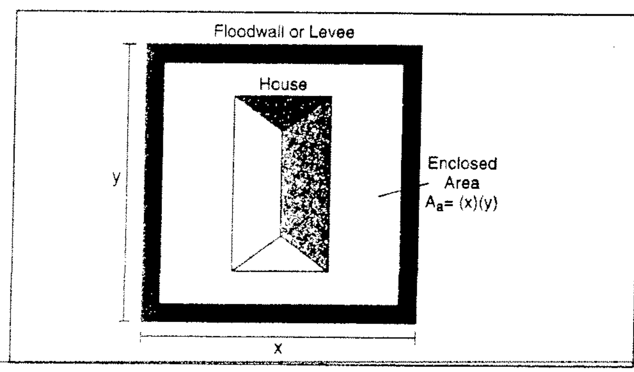


Figure IV-21: Rectangular Area Enclosed by a Floodwall or Levee

Rainfall intensities for the eastern half of the United States are available from *HYDRO-35*, a publication of the National Weather Service (NWS), while rainfall intensities for the western half of the United States can be obtained from *NOAA ATLAS 2*, also published by the NWS. Rainfall intensities are available for a range of storm frequencies including the 2-, 10-, 25-, 50-, and 100-year 60-minute events. The 2- or 10-year intensity rainfall is considered a minimum design value for pumping rates when floodwaters prevent gravity discharge from floodwalls and levees. The 100-year intensity rainfall should be the maximum design value for sizing gravity flow pipes and/or closures.

The rational formula **($Q=c i_r A$)** is used to compute the amount of precipitation runoff from small areas. It is generally not applicable to drainage areas greater than 10 acres in size.

The residential terrain runoff coefficient, **c**, is used to model the runoff characteristics of different land uses. Use the value for the predominant land use within a specific area or develop a weighted average for areas with multiple land uses. The most common coefficients are 0.70 for residential area, 0.90, for commercial area, and 0.40 for undeveloped land.

To determine the amount of precipitation that can collect in the contained area, the rainfall intensity, given in inches per hour, must be determined for a particular location (see note). This value is multiplied by the enclosed area, A_a, in square feet, a residential terrain runoff coefficient (c) of 0.7, and a conversion factor of 0.01. The answer is given in gallons per minute.

FORMULA

$$Q_a = 0.01\, c i_r A_a = \_\_\_\_ \text{gpm}$$

where: Q_a is the runoff from the enclosed area (in gal/min (gpm));

A_a is the area enclosed by the floodwall or levee (in square feet);

c is a residential terrain runoff coefficient of 0.7;

0.01 is a factor converting the answer to gallons per minute; and

i_r is the intensity of rainfall (in inches per hour).

Formula IV-16: Runoff Quantity in an Enclosed Area

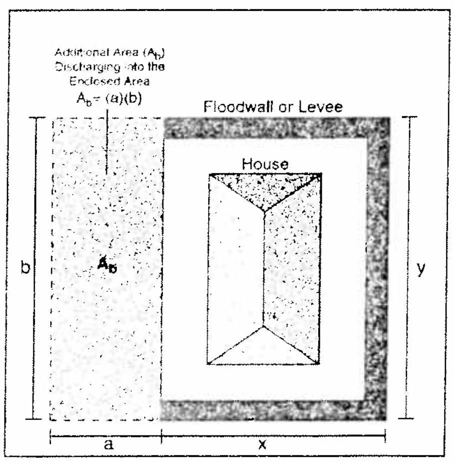


Figure IV-22: Rectangular Area Partially Enclosed by a Floodwall or Levee

In some cases, a levee or floodwall may extend only partially around the property and tie into higher ground (see Figure IV-22). For such cases, the amount of precipitation that can flow downhill as runoff into the protected area, A_a, must be included. To calculate this value, the additional area of land, A_b, that can discharge water into the enclosure should be estimated. This value is then multiplied by the previously determined rainfall intensity, i_r, by the most suitable terrain coefficient, and by 0.01.

When determining the minimum discharge size for sump pumps within enclosed areas, the designer should consider the impacts of lag time between storms that control the gravity flow mechanism (i.e., inside and outside the enclosed area) and the storage capacity within the enclosed area after the gravity discharge system closes. If the designer is not familiar with storm lag time and the computation of storage within an enclosed area, an experienced hydrologist or hydraulic engineer should be consulted.

FORMULA

$$Q_b = 0.01\, c i_r A_b = \_\_\_\_ \text{gpm}$$

where: Q_b is the runoff from additional contributing area (in gal/min (gpm));

A_b is the area discharging to the area partially enclosed by the flood wall or levee (in square feet);

c is the most suitable terrain runoff coefficient;

0.01 is a factor converting the answer to gallons per minute; and

i_r is the intensity of rainfall (in inches per hour).

Formula IV-17: Runoff Quantity from Higher Ground into a Partially Enclosed Area

Seepage flow rates from the levee or floodwall, **Q**, must also be estimated. In general, unless this seepage rate is calculated by a qualified soils engineer, a value of two gallons per minute for every 300 feet of levee or one gallon per minute for every 300 feet of floodwall should be assumed during base 100-year-flood conditions.

FORMULA

$$Q_c = sr(l)$$

where: Q_c is the seepage rate through the levee/floodwall (in gallons per minute);

sr is the seepage rate (in gallons per minute) per foot of levee/floodwall; and

l is the length of the levee/flood wall (in feet).

Formula IV-18: Seepage Flow Rate through a Levee or Floodwall

The values for inflow within the enclosed area, runoff from uphill areas draining into the enclosure, and seepage through the levee/floodwall should be added together to obtain the minimum discharge size, $\mathbf{Q_{sp}}$, in gallons per minute (gpm) for the sump pump.

FORMULA

$$Q_{sp} = Q_a + Q_b + Q_c$$

where: Q_{sp} is the minimum discharge for sump pump installation (in gpm);
Q_a is discharge from an enclosed area (from Formula IV-16) (gpm);
Q_b is the discharge from higher ground to a partially enclosed area (from Formula IV-17) (in gpm); and
Q_c is the discharge from seepage through a floodwall or levee (from Formula IV-18) (in gpm).

Formula IV-19: Minimum Discharge for Sump Pump Installation

Important considerations in determining the minimum discharge size of a sump pump include storage available within the enclosed area and the lag time between storms that impact the enclosed area and the area to which the enclosed area drains. Sump pumps will continue to operate during flooding events (assuming power is constant or backup power is available), but gravity drains will close once the floodwater elevation outside of the enclosed area exceeds the elevation of the drain pipe/flap gate. Therefore, the critical design issue is to determine runoff and seepage that occurs once the flap gate closes. Typical design solutions incorporate a freeboard of several inches or more to control the 10-year flood event safely.

ALLUVIAL FAN FLOODING HAZARDS

A detailed discussion of alluvial fan flooding and techniques for retrofitting under those conditions is presented in Appendix D: Alluvial Fan Flooding.

Alluvial fan floods are natural hazards in the western United States. Alluvial fan flooding is characterized by sudden unpredictable, high-velocity flow that transports dangerous debris down steep mountain drainages to the valley floor below. The type of detailed information available for other flood-prone areas is not yet available for alluvial fan situations, but a profile of this type of flooding and general measures to mitigate its impacts are beginning to emerge.

Alluvial Fans: Hazards and Management (1989) is a FEMA publication that provides an overview of alluvial fans and related management issues, and briefly discusses retrofitting of residential structures. Another FEMA publication entitled *Reducing Losses in High Risk Flood Hazard Areas: A Guidebook for Local Officials* specifically addresses alluvial fan flooding as a regulatory problem and provides outlines for the development of regulations and master plans for communities. This guidebook also summarizes the Dawdy Method for flood frequency estimates on alluvial fans and presents the Colorado Statute HB-1041 as a model geologic hazard ordinance that includes alluvial fan flooding hazards.

Alluvial fans are landforms evolved from a history of flood events debouching from steep-sloped watersheds onto valley floors or piedmonts. Across the western United States alluvial fans are appealing to residential developers for their vistas, and the pressure to construct on fans is increasing as the valley floors become populated. On most fans, there is evidence of past floods, but the history of development is relatively short and the consequences of a 100-year return period flood may not have been fully addressed.

Figure IV-23: Telluride, Colorado, Alluvial Fan

Flood hazards on alluvial fans are compounded by high velocities, hyper-concentrated sediment flows, severe erosion, and massive sediment deposition.

Retrofitting designs are typically dependent on the assessment of flood hazards (specifically flow depth and velocity), but for alluvial fans this information may not be available. FIRMs may provide general information such as the delineation of flood hazard zones and 100-year maximum flow depths. Local ordinances may recommend methods for determining design criteria. Additional available information may include the apex peak discharge and potential sediment concentrations. Retaining a qualified engineer may be necessary to determine design flow conditions at the property location.

Some aspects of alluvial fan flood loads are comparable to riverine flooding. Flow analyses including hydraulic loading and buoyancy are similar in principle to riverine flooding, but several key elements are different. Alluvial fan analyses should consider the severe velocity gradients, the combined effects of water and sediment mixtures, boulder impact pressure, and hydraulic loading on the upstream side of a structure.

Formulas for the computation of sediment-water mixtures, hydrodynamic forces, freeboard, and factor of safety recommendations are provided below.

Bulking Factor

The design flood conditions must be evaluated considering the increased flood discharge related to sediment bulking. The bulking factor, **BF**, is given by Formula IV-20.

Concentration of Sediment (C_v) values are estimated by engineers experienced with this type of analysis and typically range from 0-50% (decimal equivalent).

FORMULA

$$BF = \frac{1.0}{1.0 - C_v} = \_\_\_\_$$

where: **BF** is a dimensionless factor applied to riverine discharge values (Q) to account for sediment bulking; and

C_v is the concentration of sediment of the fluid mixture by percent (decimal equivalent) of volume.

Formula IV-20: Bulking Factor

For semi-arid alluvial fans, typical bulking factors range from 1.1 to 1.2 for sediment concentrations of 0.10 to 0.15 by volume. Bulking factors for mud flows can be as high as 2.0 ($C_v = 0.50$).

Hydrostatic and Hydrodynamic Loads

In hyperconcentrated sediment flows, where the sediment concentrations range from 20 to 45 percent sediment by volume, the hydrostatic pressures can be 30 to 75 percent greater than from clear water.

Hydrostatic loading is the force of the weight of standing water acting in a perpendicular manner on a submerged surface. Sediment suspended in floodwater will increase the specific weight of the fluid as a function of sediment concentration by volume C_v. Water with a high sediment concentration will impose greater hydrostatic pressures than clear water.

Likewise, hydrodynamic loading is related to the density of the fluid, which will increase with sediment loading. The greater mass the fluid has, the more momentum it will transfer when it impinges on an obstacle.

To include the effects of sediment loading in hydrostatic and hydrodynamic calculations, the specific weight of water is replaced with the specific weight of the water-sediment mixture (Formula IV-21).

In alluvial fan situations, hydrostatic and hydrodynamic forces developed using Formulas IV-4 through IV-13 should be recomputed replacing the specific weight of water (γ) with the specific weight of the water-sediment mixture (γ_s).

FORMULA

$$\gamma_s = (1-C_v)\gamma + C_v\, S_p\, \gamma = \_\_\_\_ \text{lbs/ft}^3$$

where: γ_s is the specific weight of the water-sediment mixture, in lbs/ft^3;

C_v is the sediment concentration by volume expressed as a percent (decimal equivalent);

γ is the specific weight of water, 62.4 lbs/ft^3; and

S_p is the specific gravity of sediment (dimensionless).

Formula IV-21: Specific Weight of Water-Sediment Mixture

The additional live load attributed to sediment should be considered in all calculations of hydrostatic loading with volumetric concentration of five percent or greater. This additional hydrostatic load will be most significant near the fan apex where sediment concentrations are higher and will decrease in the downfan direction. The loading factor related to sediment will be negligible in the sheet flow zone.

Freeboard

Prediction of alluvial fan flooding parameters is not an exact science, so safety factors should be considered in retrofitting design. Freeboard is the additional design height of walls, levees, and foundations above the base flood level to account for velocity head, waves, splashes, and surges. The conditions of superelevation and flow runup can be severe for mud, debris, and high velocity flows and should be evaluated separately.

The U.S. Army Corps of Engineers recommends that the freeboard (f) be greater than or equal to 2.0 feet in alluvial fan situations.

The U.S. Army Corps of Engineers (draft report, undated) recommends that the amount of freeboard be based on the velocity head plus the increase in depth caused by a 50% increase in flow rate, with a minimum value of 0.5 feet, expressed by the equation shown in Formula IV-22:

FORMULA

$$f = (d_{1.5Qdesign} - d_{Qdesign}) + V^2/2g = \_\_\_ft$$

where:

- **f** is the recommended freeboard in feet;
- **V** is the velocity of flow in feet per second;
- **g** is the acceleration of gravity (32.2 ft/sec²);
- $d_{1.5Qdesign}$ is the depth of flooding from a discharge 50% greater than the design discharge, in feet; and
- $d_{Qdesign}$ is the depth of flooding from the design discharge (typically the 100-year event), in feet.

Formula IV-22: Recommended Freeboard

Hydrostatic, hydrodynamic, and impact loading design should fall within constraints imposed by local ordinances or building codes. Where local guidelines are not available, factors of safety presented in Table IV-6 should be applied to design loads for structure design.

Safety Factors

A safety factor greater than one is an additional measure of safety to account for unanticipated or unquantifiable factors. In the case of retrofitting on alluvial fans, additional safety should be built into the design, depending on the engineer's perception of the sensitivity of the flow conditions to change. The engineer must also weigh the cost of obsolescence if a retrofitting technique becomes inadequate with continued development. Safety factors are always a compromise between the desire for added protection and the additional costs associated with retrofitting design and construction.

Table IV-6 **Freeboard and Factor of Safety Recommendations**

Type of Flooding	Freeboard (ft.)	Factor of Safety
Shallow Water Flooding, < 1 ft. (FIRM Zones A and B)	1	1.10
Moderate Water Flooding, < 3 ft.	1	1.20
Moderate Water Flooding, < 3 ft. with potential for debris, rocks < 1 ft. diameter and sediment	1	1.20
Mud Floods, Debris Flooding < 3 ft., minor surging and deposition, < 1 ft. boulders	2	1.25
Mud Flows, Debris Flows < 3 ft., surging, mud levees, > 1 ft. boulders, minor waves, deposition	2	1.40
Mud and Debris Flows > 3 ft., surging, waves, boulders > 3 ft., major deposition	3	1.50

Source: 1986 Colorado Floodproofing manual

More information on closed basin lakes, alluvial fan, and movable bed stream hazards can be obtained from the *Community Rating System (CRS) Commentary Supplement for Special Hazards Credit*, dated July 1994. This document is available through Flood Publications, *NFIP/CRS*, P.O. Box 501016, Indianapolis, Indiana 46250-1016. Telephone (317) 845-2898.

CLOSED BASIN LAKES

Two types of lakes pose special hazards to adjacent development: lakes with no outlets, such as the Great Salt Lake and the Salton Sea (California); and lakes with inadequate, or elevated outlets, such as the Great Lakes and many glacial lakes. These two types are referred to as "closed basin lakes." Closed basin lakes are subject to very large fluctuations in elevation and can retain persistent high water levels.

Closed basin lakes occur in almost every part of the United States for a variety of reasons: lakes in the northern tier of states and Alaska were scoured out by glaciers; lakes with no outlets (playas) formed in the west due to tectonic action; oxbow lakes along the Mississippi and other large rivers formed as a result of channel migration; and sinkhole lakes form in areas with large limestone deposits at or near the surface where there is adequate surface water and rainfall to dissolve the limestone (Karst topography).

Determination of the flood elevations for closed basin lakes follows generally accepted hydrological methods, which incorporate statistical data, historical high water mark determinations, stage-frequency analysis, topographical analysis, water balance analysis, and combinations of these methods. While NFIP regulations do not specifically address closed basin lakes, communities that develop mapping and regulatory standards addressing these hazards may receive flood insurance premium credits through the NFIP Community Rating System. The designer should determine if a local community has mapped or enacted an ordinance covering this special hazard.

MOVABLE BED STREAMS

Erosion and sedimentation are factors in the delineation and regulation of almost all riverine floodplains. In many rivers and streams, these processes are relatively predictable and steady. In other streams, sedimentation and erosion are continual processes, often having a larger impact on the extent of flooding and flood damages than the peak flow.

Extreme cases of sedimentation and erosion are a result of both natural and engineered processes. They frequently occur in the arid west, where relatively recent tectonic activity has left steep slopes, where rainfall and streamflow are infrequent, and where recent and rapid development has disturbed the natural processes of sediment production and transport.

Movable bed streams include streams where erosion (degradation of the streambed), sedimentation (aggradation of the streambed), or channel migration cause a change in the topography of the stream sufficient to change the flood elevation or the delineation of the floodplain or floodway.

Analysis of movable bed streams generally includes a study of the sources of sediment, changes in those sources, and the impact of sediment transport through the floodplain. While NFIP regulations do not specifically address movable bed streams, communities that develop mapping and regulatory standards that address these hazards may receive flood insurance premium credits through the Community Rating System. The designer should determine if a local community has mapped or enacted an ordinance covering this special hazard.

ANALYSIS OF NON-FLOOD-RELATED HAZARDS

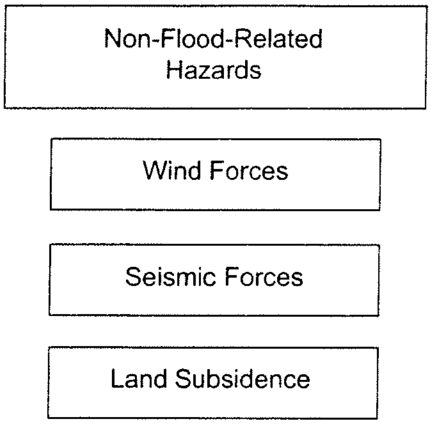


Figure IV-24: Non-Flood-Related Natural Hazards

While floods continue to be a major hazard to homes nationwide, they are not the only natural hazard that causes damage to residential buildings. Parts of the United States are subject to high winds that can accompany thunderstorms, hurricanes, tornadoes, and frontal passages. In addition, many regions are threatened by earthquake fault areas, land subsidence, and fire and snow hazards (Figure IV-24).

Retrofitting measures can be designed to modify structures to reduce the chance of damage from wind and other non-flood-related hazards. Fortunately, strengthening a home to resist earthquake damage can also increase its ability to withstand wind damage and flood-related impact and velocity forces.

> The designer must be aware that retrofitting actions may trigger a threat from multiple natural hazards and be prepared to address these issues.

> Refer to FEMA 55 (Third Edition), *Coastal Construction Manual*, and ASCE 7-98, for a detailed discussion of wind forces.

WIND FORCES

High winds impose significant forces on a home and the structural elements of its foundation. Damage potential is increased when the wind forces occur in combination with flood forces, as often occurs in coastal areas. In addition, as a structure is elevated to minimize the effects of flood forces, the wind loads on the elevated structure may be increased, depending on the amount of elevation and the structure's exposure to wind forces.

Wind forces exert pressure on structural components such as walls, roofs, connections, and foundations. Therefore, wind loads should be considered in the design process at the same time as hydrostatic, hydrodynamic, impact, and building dead and live loads, and loads from other natural hazards such as earthquakes.

A detailed discussion for computation of wind forces is beyond the scope of this publication. However, FEMA 55 (Third Edition), *Coastal Construction Manual* provides the following basic process for determining wind loads:

1. Determine the wind speed from the map (shown in Figure IV-27) from ASCE 7-98.

2. Determine the building as either open, partially enclosed, or enclosed.

3. Determine the Exposure Category: A, B, C, or D (see ASCE 7-98).

4. Determine the Importance Factor I and, if needed, the topographical influence factor, K_{zt}.

5. Determine the velocity pressure at the approximate mean roof height.

6. Select appropriate internal and external pressure coefficients.

7. Determine the design pressures (all pressures should be net pressures; use + to indicate inward-acting pressure and - to indicate outward-acting pressure).

8. Apply the design pressure to the appropriate tributary area for the element or connection to be analyzed.

This 8-step process may be used in cojunction with the wind design process flowchart in Figure IV-25.

Wind Design Process

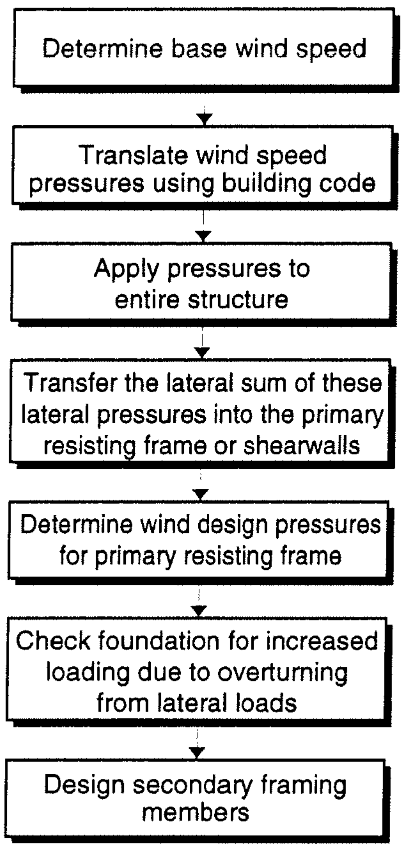


Figure IV-25: Wind Design Process

The concept of wind producing significant forces on a structure is based on the velocity difference of a medium (air) striking an obstruction (the structure). Wind speeds vary depending on the location within the United States and the frequency with which these loads occur. The IBC has wind speed maps (developed by ASCE) showing the wind velocity for an exceedence frequency of approximately 50 years. The design velocity for a particular site can be determined from these maps. If no local code is in force, the designer should refer to the ASCE 7-98, *Minimum Design Loads for Buildings and Other Structures*.

Whatever the governing code or wind load standard in force, the application of the wind loads is primarily the same, and is shown in Figure IV-25 and illustrated in Figure IV-26.

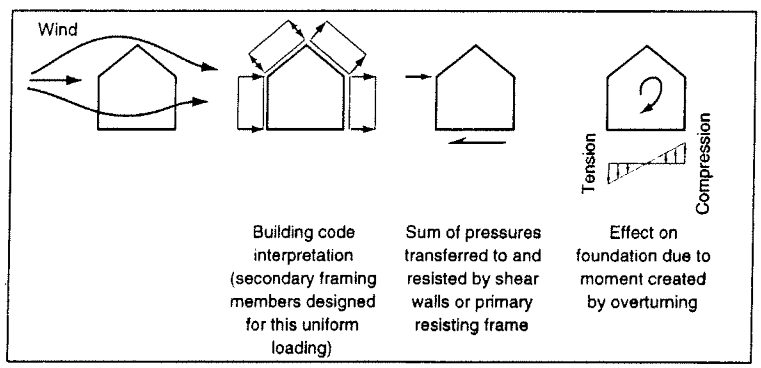


Figure IV-26: Wind Design Process Illustration

FEMA has completed several building performance assessments following Hurricanes including Andrew (1992), Iniki (1992), Opal (1996), Fran (1996), and Georges (1998). FEMA assessed the structural performance of residential building systems damaged by hurricane winds; provided findings and recommendations for enhancing building performance under hurricane wind conditions; and addressed building materials, code compliance, plan review, construction techniques, quality of construction, and construction inspection issues.

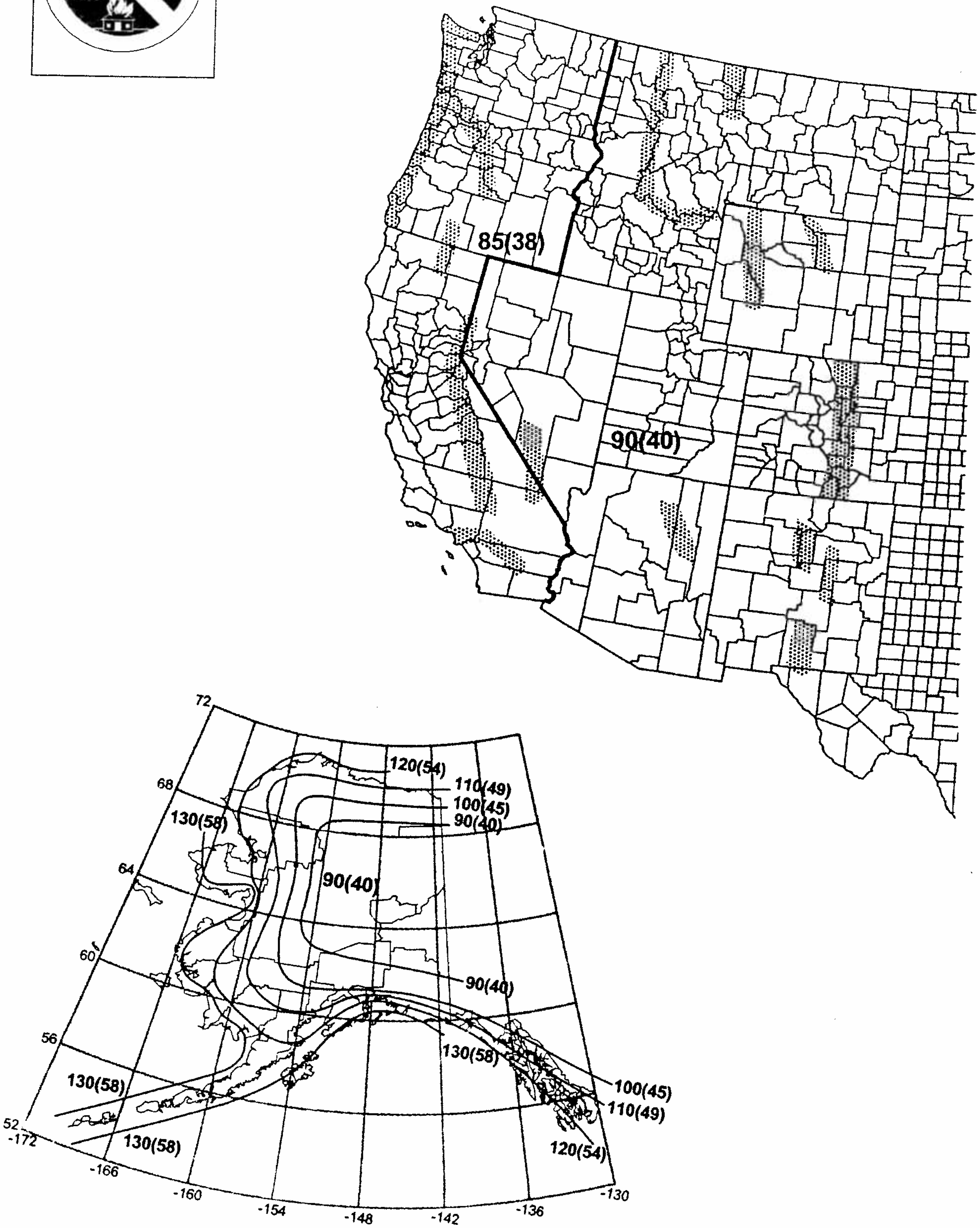
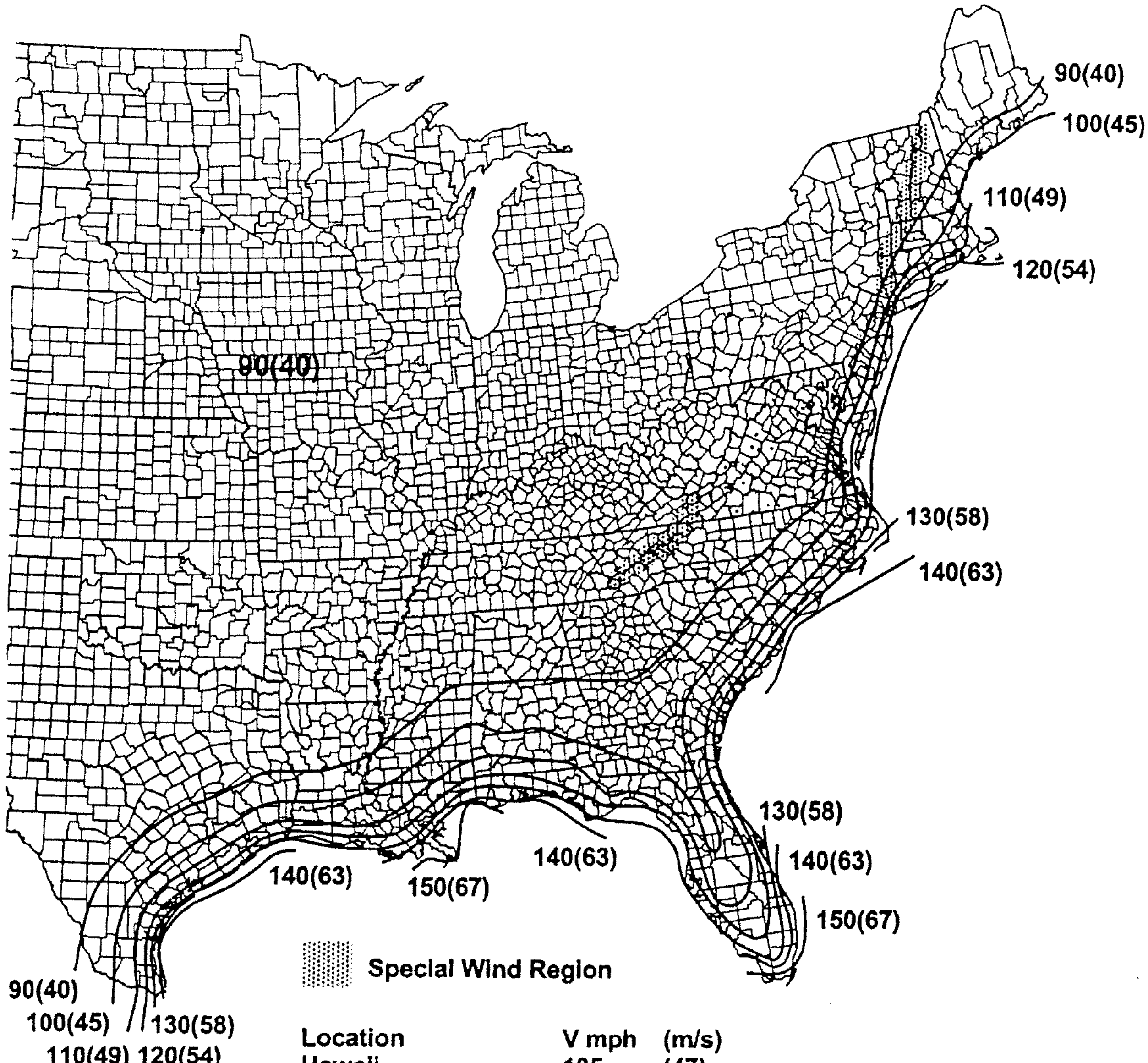


Figure IV-27: Wind Speed Map (ASCE 7-98) (page 1 of 2)



Location	V mph	(m/s)
Hawaii	105	(47)
Puerto Rico	145	(65)
Guam	170	(76)
Virgin Islands	145	(65)
American Samoa	125	(56)

Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10 m) above ground for Exposure C category.
2. Linear interpolation between wind contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

Figure IV-27: Wind Speed Map (ASCE 7-98) (page 2 of 2)

> Copies of these documents can be obtained from FEMA:
>
> FEMA (FIA-22), *Building Performance: Hurricane Andrew in Florida, Observations, Recommendations and Technical Guidance*, 1993.
>
> FEMA (FIA-23), *Building Performance: Hurricane Iniki in Hawaii, Observations, Recommendations and Technical Guidance*, 1993.
>
> FEMA 281, *Hurricane Opal in Florida, A Building Performance Assessment*, 1996.
>
> FEMA 290, *Building Performance Assessment: Hurricane Fran in North Carolina, Observations, Recommendations and Technical Guidance, 1997.*
>
> FEMA 338, *Hurricane Georges in the Gulf Coast - Observations, Recommendations, and Technical Guidance*, 1999.
>
> FEMA 339, *Hurricane Georges in Puerto Rico - Observations, Recommendations, and Technical Guidance, 1999.*

These reports present detailed engineering discussions of building failure modes along with successful building performance guidance supplemented with design sketches. Please refer to these documents for specific engineering recommendations.

SEISMIC FORCES

Seismic forces on a home and the structural elements of a foundation can be significant. Seismic forces may also trigger additional hazards such as landslides and soil liquefaction, which can increase the damage potential on a home. Seismic forces act on structural components such as walls, roofs, connections, and foundations. Similar to wind forces, seismic forces should be considered in the design process at the same time as hydrostatic, hydrodynamic, impact, and building dead and live loads, and loads from other natural hazards such as hurricanes. Design assumptions for seismic loadings are normally based upon local building codes.

Figures IV-28 and IV-29 illustrate the process for estimating seismic hazards and determining the ability of existing structural components to withstand these forces.

When making repairs to a flood-damaged home or considering retrofitting structures to minimize the impact of future flooding events, there are certain practical steps that can be taken at the same time to reduce the chance of damage from other hazards. Earthquake protection steps can be divided into two categories: steps that deal with the building structure itself, and steps that can be taken with other parts of the building and its contents.

> If no local code is in force, the designer should refer to the ASCE 7-98, *Minimum Design Loads for Buildings and Other Structures*.

Seismic Design Process

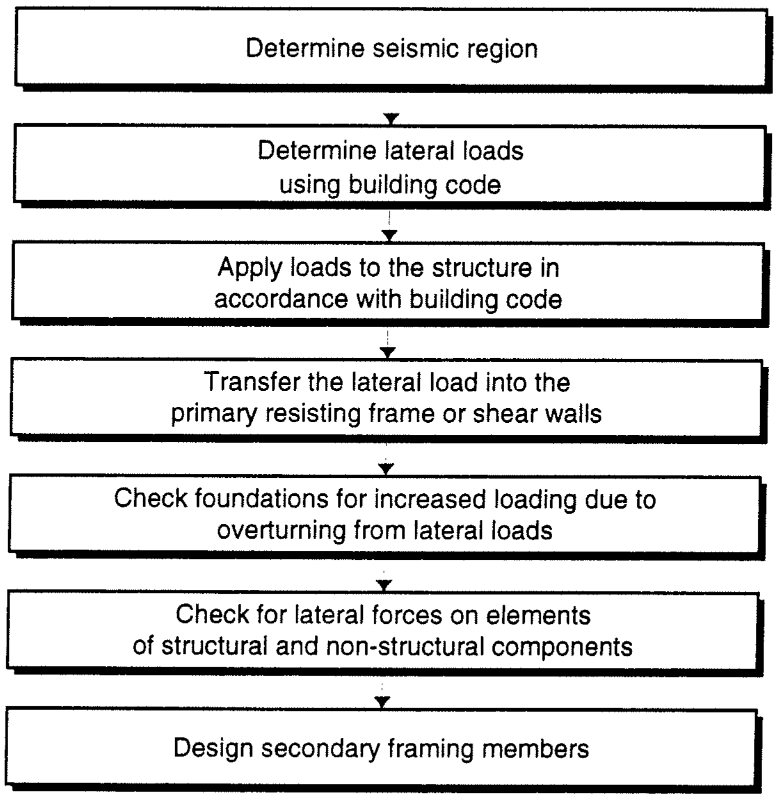


Figure IV-28: Seismic Design Process

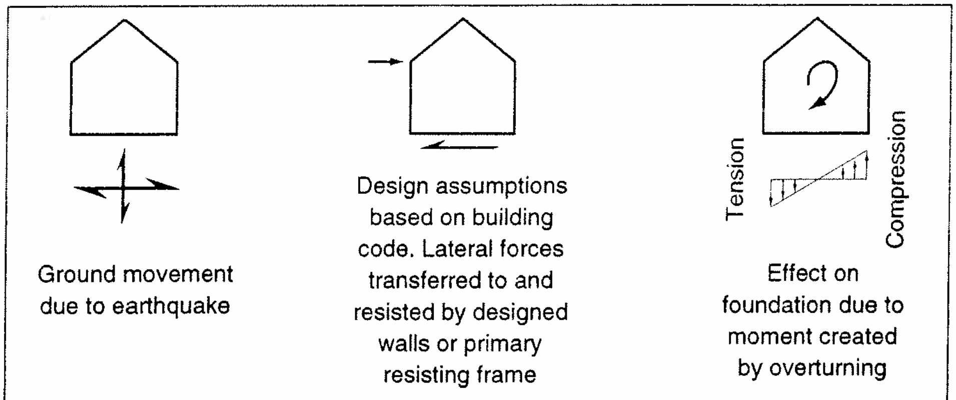


Figure IV-29: Seismic Design Process Illustration

Refer to pages VI-34 through VI-38 for a detailed discussion of load combination scenarios and design methods.

Combining Forces

Once the flood-related and non-flood-related forces are obtained, these forces need to be combined to determine the load combinations, which govern the design of the building components and connections. Analysis of loading combinations is covered in detail in Chapter VI: General Design Practices and ASCE 7-98.

Additional information concerning the determination of flood-related forces is available in the Flood Design Load Criteria incorporated in Section 5 of ASCE 7-98, *Minimum Design Loads for Buildings and Other Structures*, and ASCE 24-98, *Flood Resistant Design and Construction*.

Protection of the Structure

For the building structure, the most important step is making sure the home is properly bolted onto its foundation so that it will not slide off in an earthquake. Another important step, if raising the foundation to place the house above flood levels, is to make sure the foundation can withstand an earthquake.

Key portions of masonry block foundations usually require strengthening by installing reinforcing bars in the blocks and then filling them with concrete grout. FEMA has developed a sample plan for strengthening a masonry block foundation wall. This type of work can be complicated and normally requires the expertise of a professional engineer, architect, or contractor.

FEMA's *Technical Information on Elevating Substantially Damaged Residential Buildings in the Midwest* (August 24, 1993) provides procedures for determining seismic forces and recommendations for seismic retrofitting of a wood-frame structure. For more information on protecting a structure from seismic hazards, contact the appropriate FEMA Regional Office's Mitigation Division.

> The additional cost for seismic strengthening was estimated by FEMA (during the Midwest Flood of 1993) to range from 17-23% of the base repair cost for elevating a 1,000-SF wood-frame structure on masonry foundation walls. FEMA has prepared some simple one-page descriptions (details) and costs associated with these steps that are available in a publication entitled *Protecting Your Home from Earthquake Damage (1993)*.

Protection of Non-Structural Building Components and Building Contents

For non-structural building components and contents, earthquake protection usually involves simpler activities that homeowners can undertake themselves. These include anchoring and bracing of fixtures, appliances, chimneys, tanks, cabinets, and shelves.

LAND SUBSIDENCE

Subsidence of the land surface affects flooding and flood damages. It occurs in at least 38 states. Although there are no national figures for increased flood damage due to subsidence, it can increase flood damage to entire communities that are subject to coastal flooding, and it threatens larger or smaller areas elsewhere. Because the causes of subsidence vary, selected mitigation techniques are required in different situations.

> More information on land subsidence hazards can be obtained from the *CRS Commentary Supplement for Special Hazards Credit*, dated August 1992. This document is available through Flood Publications, *NFIP/CRS*, P.O. Box 501016, Indianapolis, Indiana 46250-1016. Telephone (317) 845-2898.

Subsidence may result in sudden, catastrophic collapses of the land surface or in a slow lowering of the land surface. In either case, it can cause increased hazards to structures and infrastructure. In some cases, the causes of subsidence can be controlled.

Subsidence is typically a function of withdrawal of fluids or gases, the existence of organic soils, or other geotechnical factors; it requires an extensive engineering/geotechnical analysis. While NFIP regulations do not specifically address land subsidence, communities that develop mapping and regulatory standards addressing these hazards may receive flood insurance premium credits through the NFIP Community Rating System. The designer should determine if a local community has mapped or enacted an ordinance covering this special hazard.

GEOTECHNICAL CONSIDERATIONS

> Information on land subsidence, which is sometimes caused by flooding conditions, can be found in the Analysis of Non-Flood-Related Hazards Section of Chapter IV.

Soil properties during conditions of flooding are important factors in the design of any surface intended to resist flood loads. These properties include:

- saturated soil pressures (covered previously in Chapter IV under Hydrostatic Forces);
- allowable bearing capacity;
- potential for scour;
- frost zone location;
- permeability; and
- shrink-swell potential.

The computation of lateral soil forces and determination of soil bearing capacity are critical in the design of foundations. These forces plus the frost zone location and potential scour play an important role in determining the type of foundation to use. Likewise, the permeability and compactibility of soils are key factors in selecting borrow materials for backfill or levee construction.

If unsure of local soil conditions, obtain a copy of the U. S. Department of Agriculture, Natural Resource Conservation Service *Soil Survey* of the general area. This survey provides valuable information needed to conduct a preliminary evaluation of the soil properties, including:

- type, location, and description of soil types;
- use and management of the soil types; and

> The physical properties of soil are critical to the design, suitability, and overall stability of floodproofing measures. Therefore, the designer should consult a geotechnical engineer if the soil properties at a site do not support the use of the chosen retrofitting method. A geotechnical engineer should also be consulted for any information that cannot be obtained from the *Soil Survey* or the local office of the Natural Resources Conservation Service.

- engineering and physical properties including plasticity indexes, permeability, shrink/swell potential, erosion factors, potential for frost action, and other information.

This information can be compiled using Figure IV-30 (Geotechnical Considerations Decision Matrix) to enable the designer to determine the suitability of the specific soil type to support the various retrofitting methods. It is important to note that while the soil properties may not be optimum for specific retrofitting methods, facilities can often be designed to overcome soil deficiencies.

The following sections begin a discussion of the various soil properties, providing the information necessary to fill out the decision matrix (Figure IV-30) and to understand the relationship between these soil properties and retrofitting measures. See Figure IV-31 for an example of a filled-out matrix.

Geotechnical Considerations Decision Matrix

Owner Name: ______________________________ Prepared By: ____________
Address: ______________________________ Date: ____________
Property Location: ______________________________

Soil Properties	Retrofitting Measures	Elevation on Foundation Walls	Elevation on Fill	Elevation on Piers	Elevation on Posts and Columns	Elevation on Piles	Relocation	Dry Flood-proofing	Wet Flood-proofing	Floodwalls and Levees
Lateral Soil Pressure	High									
	Moderate									
	Low									
Bearing Capacity	High									
	Moderate									
	Low									
Potential for Scour	High									
	Moderate									
	Low									
Shrink/Swell Potential	High									
	Moderate									
	Low									
Potential Frost Action	High									
	Moderate									
	Low									
Permeability	High									
	Moderate									
	Low									

Instructions:
This matrix is designed to help the designer identify situations where soil conditions are unsuitable when applied to certain retrofitting measures, therefore eliminating infeasible measures. It is not intended to select the most suitable alternative. Instructions for use of this matrix follow:

1. Circle the appropriate description for each of the soil properties.
2. Use the NRCS survey, information from this and other reference books, and engineering judgment to determine which methods are Suitable (S)/Not Suitable (NS) for each soil property. Enter S or NS in each box.
3. Review the completed matrix and eliminate any retrofitting measures that are clearly unsuitable for the existing soil conditions.

Figure IV-30: Geotechnical Considerations Decision Matrix

Geotechnical Considerations Decision Matrix

Owner Name: J. Q. PUBLIC
Address: 1 MAIN STREET, SPRINGFIELD, IL
Property Location: SOFT CLAY SOILS (CH)

Prepared By: M. SMITH
Date: ____________

Soil Properties		Elevation on Foundation Walls	Elevation on Fill	Elevation on Piers	Elevation on Posts and Columns	Elevation on Piles	Relocation	Dry Floodproofing	Wet Floodproofing	Floodwalls and Levees
Lateral Soil Pressure	High							NS		NS
	Moderate									
	Low									
Bearing Capacity	High									
	Moderate									
	Low	NS		NS	NS					NS
Potential for Scour	High									
	Moderate									
	Low		S					S		S
Shrink/Swell Potential	High		NS					NS		NS
	Moderate									
	Low									
Potential Frost Action	High									
	Moderate									
	Low		S					S		S
Permeability	High									
	Moderate									
	Low		S					S		S

Instructions:

This matrix is designed to help the designer identify situations where soil conditions are unsuitable when applied to certain retrofitting measures, therefore eliminating infeasible measures. It is not intended to select the most suitable alternative. Instructions for use of this matrix follow:

1. Circle the appropriate description for each of the soil properties.
2. Use the NRCS survey, information from this and other reference books, and engineering judgment to determine which methods are Suitable (S)/Not Suitable (NS) for each soil property. Enter S or NS in each box.
3. Review the completed matrix and eliminate any retrofitting measures that are clearly unsuitable for the existing soil conditions.

Figure IV-31: Example Geotechnical Considerations Decision Matrix

> An approach developed by FEMA during the elevation of substantially damaged homes in Florida and the Midwest is to reuse the existing footings, if allowed by code. Refer to FEMA 347: *Above the Flood: Elevating Your Floodprone Home* for details on elevation of structures.

ALLOWABLE BEARING CAPACITY

Another important consideration is the allowable bearing capacity of the soil. The weight of the structure, along with the weight of backfilled soil (if present), creates a vertical pressure under the footing that must be resisted by the soil. The term "allowable bearing pressure" refers to the maximum unit load that can be placed on a soil deposit without causing excessive deformation, shear failure, or consolidation of the underlying soil. The allowable bearing capacity is the ultimate bearing capacity divided by an appropriate factor of safety, typically 2 to 3.

FORMULA

$$Q_{BC} = Q_u/FS = \_\_\_\_ \text{ lbs/SF}$$

where: Q_{BC} is the allowable bearing capacity (in pounds per square foot);

Q_u is the ultimate bearing capacity (in pounds per square foot);

FS is a factor of safety, typically 2 or 3 (as prescribed by code.)

Formula IV-23: Allowable Bearing Capacity

Table IV-7 presents estimated allowable bearing capacities for various soil types to be used for preliminary sizing of footings only. The actual allowable soil bearing capacity should be determined by a soils engineer. Most local building codes specify an allowable bearing capacity to be utilized in design if the soil properties have not been specifically determined.

Once the allowable bearing capacity is determined by the soils engineer or a conservative estimate prescribed by code is made, the designer can determine the capacity of the existing foundation to support the expected loads. Depending on the outcome of that evaluation, the designer may need to supplement the existing footing to support the expected loading condition (i.e., keep the actual bearing

Table IV-7
Typical Allowable Bearing Capacity by Soil Type (from Table IV-3)

Soil Type (Symbol)	Allowable Bearing Capacity (lb./sf.)
Clay, Soft (CL, CH)	600 to 1,200
Clay, Firm (CL, CH)	1,500 to 2,500
Clay, Stiff (CL, CH)	3,000 to 4,500
Loose Sand, Wet (SP, SW, SM)	800 to 1,600
Firm Sand, Wet (SP, SW, SM, SC)	1,600 to 3,500
Gravel (GW, GP, GM, GC)	2,700 to 3,000

Certain types of soil–loose sands and soft clays (SP, SW, SM, SC, CL, CH)–exhibit very poor bearing capacities when saturated; therefore, foundation, levee, and floodwall applications in those conditions would not be feasible without special treatment.

pressure below the allowable bearing pressure of the soil) as a result of the retrofitting project.

The ability of soils to bear loads, usually expressed as shearing resistance, is a function of many complex factors, including some that are site-specific. A very significant factor affecting shearing resistance is the presence and movement of water within the soil. Under conditions of submergence, some shearing resistance may decrease due to the buoyancy effect of the interstitial water or, in the case of cohesive soils, to physical or chemical changes brought about in clay minerals.

While there are many possible site-specific effects of saturation on soil types, some classes of soil can be identified that have generally low shearing resistances under most conditions of saturation. These include:

- fine silty sands of low density, which in some localities may suddenly compact when loaded or shaken, resulting in a phenomenon known as liquefaction;

- sand or fine gravel, in which the hydraulic pressure of upward-moving water within the soil equals the weight of the soil, causing the soil to lose its shear strength and become "quicksand," which will not support loads at the surface; and

- soils below the water table, which have lower bearing capacity than the same soils above the water table.

Other types of saturated soils may also have low shearing resistances under loads, depending on numerous site-specific factors such as slope, hydraulic head, gradient stratigraphic relationships, internal structures, and density. Generally, the soils noted above should not be considered suitable for structural support or backfill for retrofitting, and when they are known to be present, a soils engineer should be consulted for site-specific solutions.

Mechanical properties of all soils are complex. Attempts to construct water- or saturated soil-retaining/resisting structures without a thorough understanding of soil mechanics and analysis of on-site soils can result in expensive mistakes and project failure.

SCOUR POTENTIAL

Erosion of fill embankments, levees, or berms depends on the velocity, flow direction, and duration of exposure. Scour is localized erosion caused by the entrainment of soil or sediment around flow obstructions, often resulting from flow acceleration and changing flow patterns due to flow constriction. Where flow impinging on a structure is affected by diversion and constriction due to nearby structures or other obstructions, flow conditions estimated for the calculation of depths of scour should be evaluated by a qualified engineer.

Scour under building foundations and around supporting walls and posts and the erosion of elevating fill can render structural retrofitting and resistive designs ineffective, possibly resulting in failure. Figures IV-32 and IV-33 illustrate scour at open foundation systems and ground level buildings.

Maximum potential scour is critical in designing an elevated foundation system to ensure that failure during and after flooding does not occur due to any loss in bearing capacity or anchoring resistance around the posts, piles, or piers.

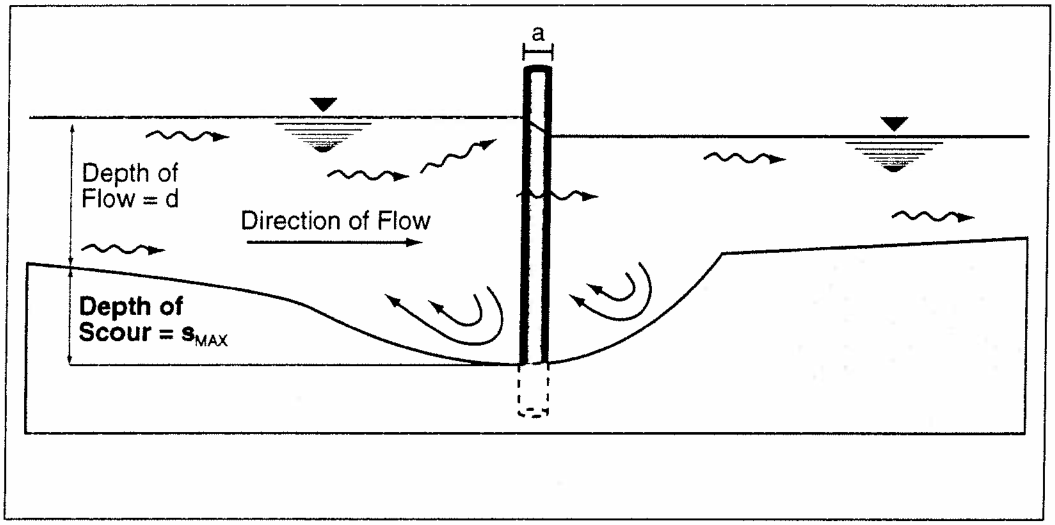


Figure IV-32: Local Scour at Piers, Piles, and Posts

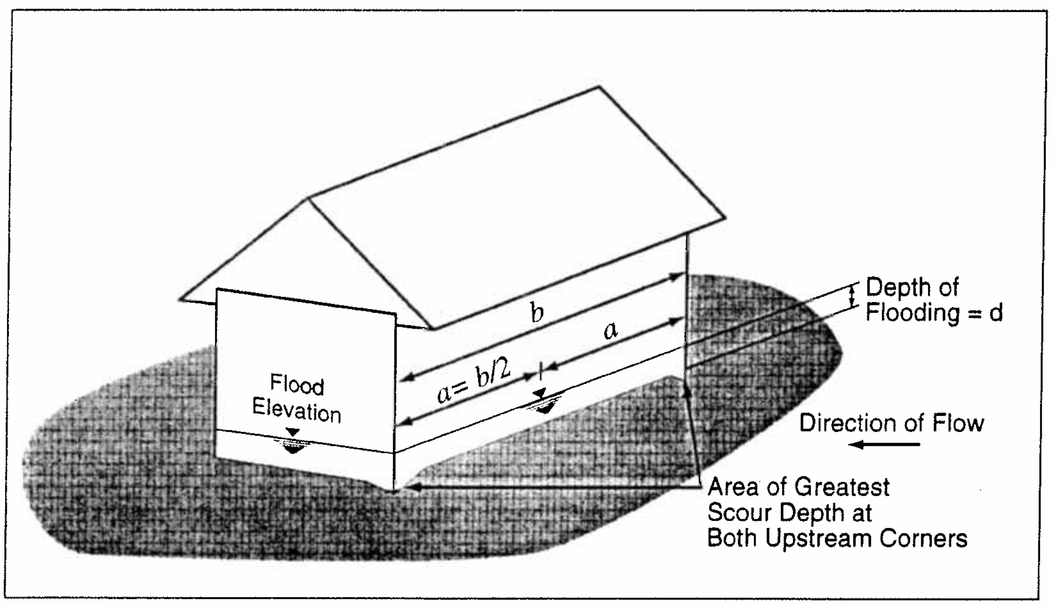


Figure IV-33: Scour Action on a Ground-Level Building

Resistance to scouring increases with clay content and/or the introduction of bonding agents, which help bond the internal particles of a soil together.

The potential for foundation scour is a complex problem. Granular and other consolidated soils in which the individual particles are not cemented to one another are subject to scour erosion and transport by the force of moving water. The greater the velocity or turbulence of the moving water, the greater the scour potential. Soils that contain sufficient proportions of clay to be described as compact are more resistant to scour than the same grain sizes without clay as an intergranular bond. Likewise, soils with angular particle shapes tend to lock in place and resist scour forces.

Shallow foundations in areas subject to flood velocity flow may be subject to scour, and appropriate safeguards should be undertaken. These safeguards may include the use of different, more erosion-resistant soils, deeper foundations, surface armoring of the foundation and adjacent areas, and the use of piles.

The calculation for estimating maximum potential scour depth at an elevated or ground-level foundation member (Formula IV-24) is based upon the foundation (or foundation member) shape and width, as well as the water velocity and depth, and type of soil.

The factor "**a**" in Formula IV-24 is the diameter of an open foundation member or half of the width of the solid foundation perpendicular to flood flow.

Where elevation on fill is the primary retrofitting measure, embankments must be protected against scour and erosion. Scour at the embankment toe may be calculated as shown in Formula IV-24.

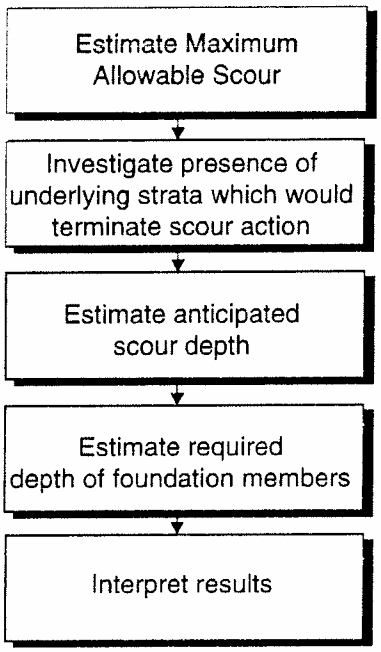


Figure IV-34: Process for Estimating Potential Scour Depth

The scour information presented is the best available; however, there is not a general consensus within the scientific community that these scour formulas are valid. Research continues into this area.

FORMULA

$$s_{max} = d[1.1(a/d)^{0.4}\,(V/(gd)^{0.5})^{0.33}] = \_\_\_\_\_ \text{ feet}$$

Where: s_{max} is the maximum potential depth of scour hole (in feet);
d is the depth of flow upstream of structure (in feet);
a is the diameter of post, pier, or pile or half the frontal length of the blockage (in feet);
V is the velocity of flow approachi ing the structure (in feet per second); and
g is the acceleration of gravity (32.2 feet per second.)

Formula IV-24: Maximum Potential Scour at Embankment Toe

The maximum potential scour depth predicted by the following equation represents a maximum depth that could be achieved if the soil material were of a nature that could be displaced by the water's action. However, in many cases, a stronger underlying strata will terminate the scour at a more shallow elevation. Figure IV-34 illustrates the process of determining the potential scour depth affecting a foundation system.

Step 1: Compute Maximum Allowable Scour. The scour depth at square and circular pier, post, and pile foundation members can be calculated as follows:

$$S_{max} = 2.2a$$

where: S_{max} is the maximum localized scour depth in feet;

a is the diameter of a round foundation element, or the maximum diagonal cross-section dimension for a rectangular element.

Formula IV-25: Localized Scour Around Vertical Pile

Formula IV-25 can also be used to approximate local scour beneath grade beams--set "a" equal to the depth (vertical thickness) of the grade beam.

Localized scour around vertical walls and enclosed areas (e.g., typical A-zone construction) can be greater than that around vertical piles, and should be calculated with Formula IV-26.

Scour depths estimated with Formula IV-26 can be unrealistically high for coastal areas and should be capped at 10 feet of localized scour.

$$S_{max} = d_s\ [2.2(a/d_s)^{0.65}[V/(gd_s)^{0.50}]^{0.43}\}K$$

where: S_{max} is the maximum localized scour depth in feet;

d_s is the design stillwater flood depth in feet (upstream of the structure)

a is the diameter of a round foundation element, or the maximum diagonal cross-section dimension for a rectangular element.

V is the average velocity of water in ft/sec

g is the gravitational constant (32.2 ft/sec^2)

K is the factor applied for Flow Angle of Attack (see Figure IV-35).

Formula IV-26: Localized Scour Around Vertical Enclosure

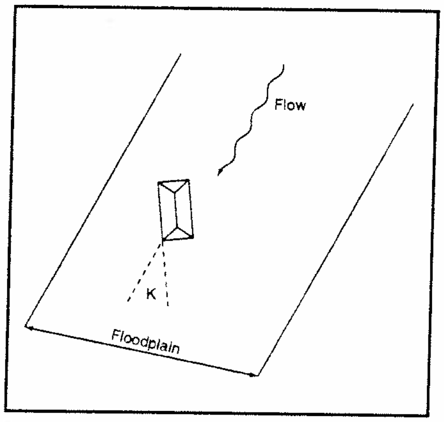


Figure IV-35: Flow Angle of Attack

The above scour equation applies to average soil conditions (2,000 - 3,000 psf bearing capacity). Average soil conditions would include gravels (GW, GP, GM and GC), sands (SW, SP, SM, and SC), and silts and clays (ML, CL, MH, CH). For loose sand and hard clay, the maximum scour values may be increased and decreased, respectively, to reflect their lower and hi gher bearing capacities. However, the assistance of a soils engineer should always be sought when making this adjustment, computing scour depths, and/or designing foundations subject to scour effects.

If a wall or foundation member is oriented at an angle to the direction of flow, a multiplying factor, **K**, can be applied to the scour depth to account for the resulting increase in scour as presented in the following table.

Table IV-8

Scour Factor for Flow Angle of Attack, K

Angle of Attack	Length to Width Ratio of Structural Member in Flow			
	4	8	12	16
0	1	1	1	1
15	1.15	2	2.5	3
30	2	2.5	3.5	4.5
45	2.5	3.5	4.5	5
60	2.5	3.5	4.5	6

Numerous scour equations can be utilized to estimate scour depths. The U.S. Department of Transportation recommends a factor of safety of 1.5 for predicting building scour depth.

Step 2: Investigate Underlying Soil Strata. Once the maximum potential scour depth has been established, the designer should investigate the underlying soil strata at the site to determine if the underlying soil is of sufficient strength to terminate scour activities. Information from the NRCS *Soil Survey* may be used to make this assessment.

Figure IV-36 illustrates a scour terminating strata. If an underlying terminating strata does not exist at the site, the maximum potential scour estimate will become the anticipated scour depth. However, if an underlying terminating strata exists, the maximum potential scour depth will be modified to reflect this condition, as shown in Step 3.

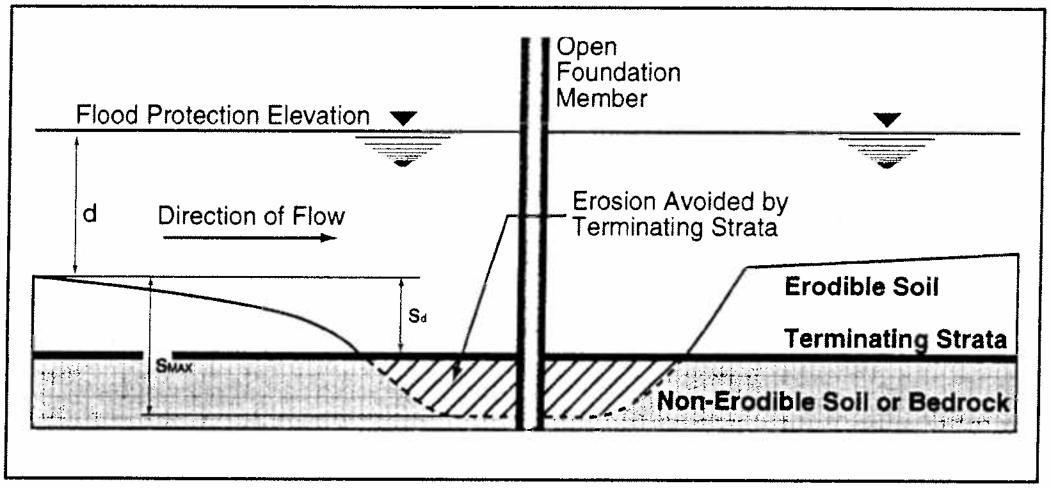


Figure IV-36: Terminating Strata

Step 3: Determine the Anticipated Scour Depth. Based on the results of Step 2, the designer will determine the anticipated scour depth to be used in determining the depth to which the foundation element must be placed to resist scour effects. If a terminating strata exists, the expected scour would stop at the depth at which this strata starts, and the distance from this point to the surface is considered to be the potential scour depth, (s_d), Figure IV-36. If no terminating strata exists, the maximum potential scour (s_{max}) computed earlier becomes the potential scour depth (s_d).

Step 4: Determine Required Depth of Foundation Members. Scour will increase the height above grade of the vertical member, since the grade level would be lowered due to scour and erosion (see Figure IV-37). As this occurs, the depth of burial (**D_b**) of the vertical foundation member also decreases an identical distance. This can result in a foundation failure because the loss of supporting soils would change the assumed conditions under which the elevated foundation system was designed. To account for this, the vertical foundation member depth used for the purpose of determining an acceptable design must be increased by the amount of potential scour depth, (**s_d**).

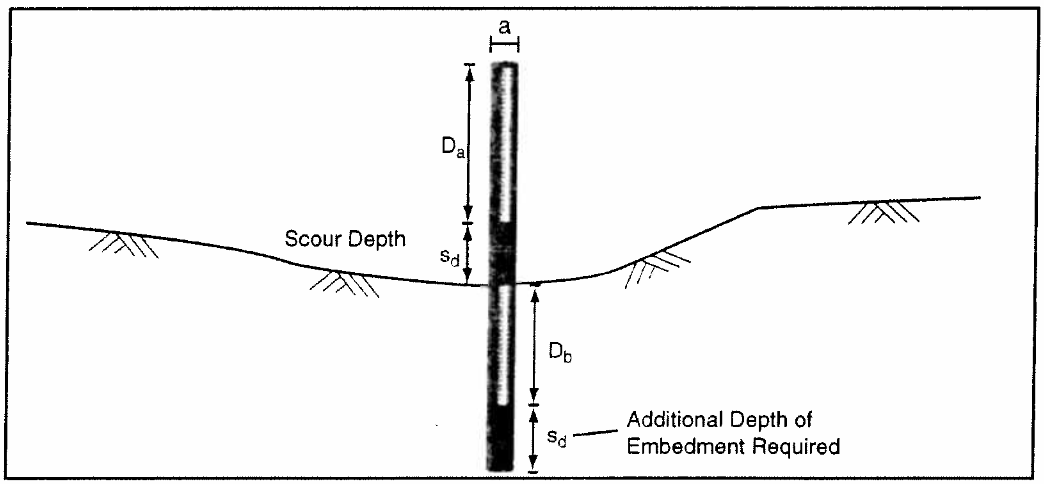


Figure IV-37: Additional Embedment

Step 5: Interpret Results. Foundations, footings, and any supporting members should be protected at least to the anticipated scour depth. If the structural member cannot be buried deeper than the anticipated scour depth, the member should be protected from scour by placing rip-rap (or other erosion-resistant material) around the member, or by diverting flow around the foundation member with grading modification or construction of an independent barrier (floodwall or levee). For situations in which the anticipated scour depth is minimal, the designer should use engineering judgment to determine the required protective measures. Whenever the designer is unsure of the appropriate action, a qualified geotechnical engineer should be consulted.

FROST ZONE CONSIDERATIONS

Because certain soils under specific conditions expand upon freezing, the retrofitting designer must consider the frost heave impact in the design of shallow foundations. When frost-susceptible soils are in contact with moisture and subjected to freezing temperatures, they can imbibe water and undergo very large expansions (both horizontally and vertically). Such heave or expansion exerts forces strong enough to move and/or crack adjacent structures (foundations, footings, etc.). The thawing of frozen soil usually proceeds from the top downward. The melted water cannot drain into the frozen subsoil, and thus becomes trapped, possibly weakening the soil. Normally, footing movements caused by frost action can be avoided by placing part of a foundation below the zone of maximum frost penetration.

Local building codes generally specify the depth of the zone of maximum frost penetration. In the absence of guidance in the local building code, refer to the National Weather Service or the NRCS *Soil Survey*.

PERMEABILITY

Of principal concern for the construction of retrofitting measures such as levees and floodwalls are the properties of the proposed fill material and/or underlying soils. These properties will have an impact on stability and will determine the need for seepage and other drainage control measures.

While impervious cutoffs such as compacted impervious core, sheet pile metal curtains, or cementitious grout curtains can be designed to reduce or eliminate seepage, their costs are beyond the financial capabilities of most homeowners. However, several lower-cost measures to control seepage include pervious trenches, pressure relief wells, drainage blankets, and drainage toes.

It is very important that the designer keep the units in this formula consistent. The results of Formula IV-27 depend on the homogeneity of the foundation and the accuracy of the coefficient of permeability. The results should be considered as an indication only of the order of magnitude of seepage through a foundation.

Since most retrofitting projects are constructed using locally available materials, it is possible that homogenous and impermeable materials will not be available to construct embankments and/or backfill floodwalls and foundations. Therefore, it is essential that the designer determine the physical properties of the underlying and borrowed soils.

Where compacted soils are highly permeable (i.e., sandy soils), significant seepage through an embankment and under a floodwall foundation can occur. Various soil types and their permeabilities are provided in Table IV-9.

The coefficient of permeability provides an estimate of ability of a specific soil to transmit seepage. It can be used (Formula IV-27) to make a rough approximation of the amount of foundation underseepage. Formula IV-27 may be used in lieu of Formula IV-17 for large levee/floodwall applications when the coefficient of permeability for the specific site soil is known.

FORMULA

$$\mathbf{Q = k\, i_{hg}\, A}$$

where: **Q** is the discharge in a given unit of time;

k is the coefficient of permeability for the soil foundation (in feet per unit of time);

$\mathbf{i_{hg}}$ is the hydraulic gradient (h/L) which is the difference in head between two points divided by the length of path between two points (dimensionless); and

A is the gross area of the foundation through which flow takes place (in square feet).

Formula IV-27: Volume of Seepage

SHRINK-SWELL POTENTIAL

> Soils that exhibit severe shrinking-swell characteristics include clays and clay mixtures such as soil types CH, CL, ML-CL, SC, and MH.

As mentioned earlier in this chapter, due to the continual shrink and swell of expansive soil backfills and the variation of their water content, the stability and elevation of these soils and overlaying soil layers may vary considerably. These characteristics make the use of these soils in engineering/construction applications imprudent. The NRCS *Soil Survey* for a specific area offers guidance on the shrink-swell potential of each soil group in the area as well as guidance on the suitability of their use in a variety of applications including engineering, construction, and water retention activities. If the designer is unsure of the type or nature of soil at the specific site, a qualified soils engineer should be contacted for assistance.

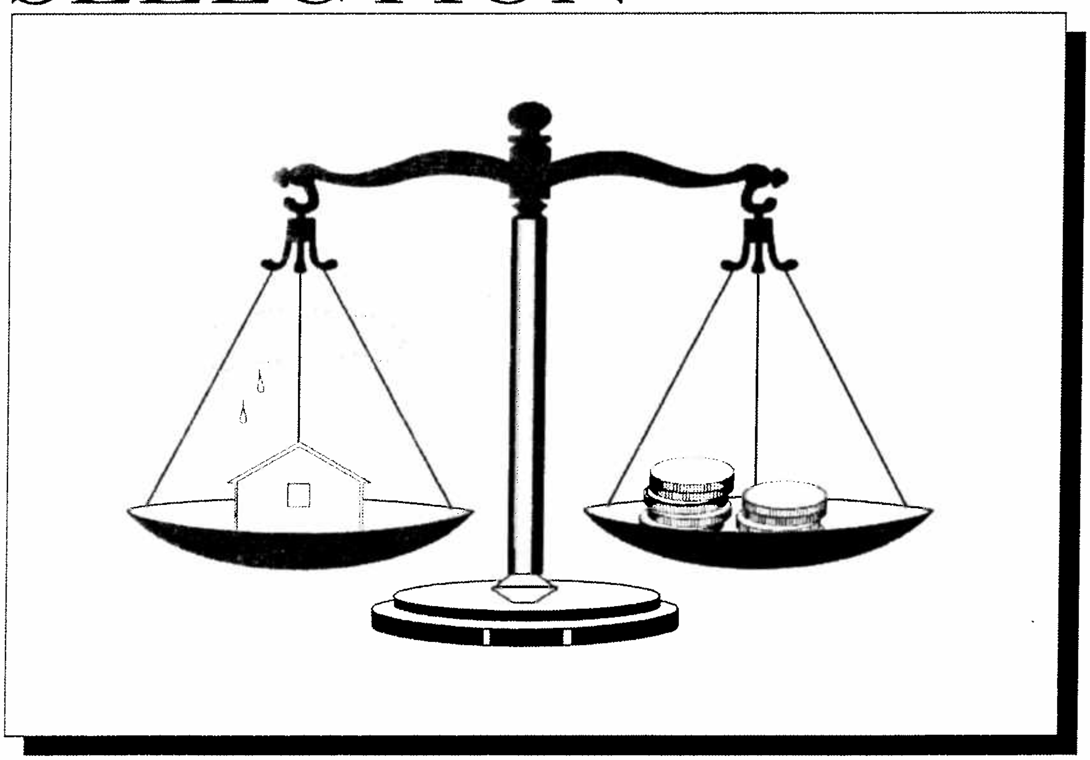
The physical soil parameters at the retrofitting and potential borrow sites are an important design consideration. Homeowners and designers should clearly understand that the advice of a professional soils engineer is vital when planning retrofitting measures that are not ideal for the physical soil parameters at a given site.

Table IV-9 **Typical Values of Coefficient of Permeability *k* for Soils**		
Soil Type and Description	**Symbol**	**Typical Coefficient of Permeability, Ft/Day**
Well-graded clean gravels, gravel-sand mixtures	GW	75
Poorly graded clean gravels, gravel-sand-silt	GP	180
Silty gravels, poorly graded gravel-sand-silt	GM	1.5×10^{-3}
Clayey gravels, poorly graded gravel-sand-clay	GC	1.5×10^{-4}
Well-graded clean sands, gravelly sands	SW	4.0
Poorly graded clean sands, sand-gravel mix	SP	4.0
Silty sands, poorly graded sand-silt mix	SM	2×10^{-2}
Sand-silt clay mix with slightly plastic fines	SM-SC	3.0×10^{-3}
Clayey sands, poorly graded sand-clay mix	SC	7.5×10^{-4}
Inorganic silts and clayey silts	ML	1.5×10^{-3}
Mixture of inorganic silt and clay	ML-CL	3.0×10^{-4}
Inorganic clays of low to medium plasticity	CL	1.5×10^{-4}
Organic silts and silt-clays, low plasticity	OL	Quite variable
Inorganic clayey silts, elastic silts	MH	1.5×10^{-4}
Inorganic clays of high plasticity	CH	1.5×10^{-2}
Organic clays and silty clays	OH	Quite variable

1 cm/sec=2,840 ft/day=
2 ft/min
1 ft/year =1 x 10^{-6} cm/sec

CHAPTER V

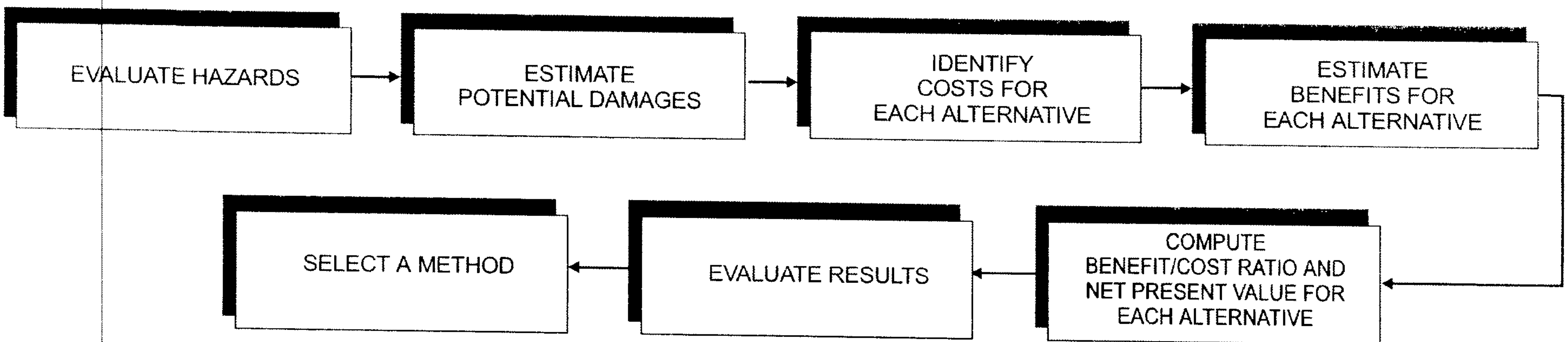
BENEFIT/COST ANALYSIS AND ALTERNATIVE SELECTION



Featuring:

Evaluate Hazards
Estimate Potential Damages (No Action Alternative)
Identify Costs Associated with Alternatives
Estimate Benefits
Compute Benefit/Cost Ratio and Net Present Value
Select a Method

BENEFIT/COST ANALYSIS AND ALTERNATIVE SELECTION



Chapter V: Benefit/Cost Analysis and Alternative Selection

Table of Contents

BENEFIT/COST ANALYSIS AND ALTERNATIVE SELECTION

Benefit/cost analysis is a powerful tool to help determine whether the benefits of a prospective hazard mitigation project are sufficient to justify the costs of the project. This analysis can also be used to assist in ranking different retrofitting alternatives.

A new CD that includes the software "Benefit/Cost Analysis for Flood," and the accompanying user's guide, is scheduled for release by fall 2001. The benefits calculated by the software are expected future benefits estimated over the useful lifetime of a retrofit project to protect against riverine or coastal flooding. To account for the time value of money, a net present value is calculated automatically by the model.

The benefit/cost software has been updated several times since it was first released. It is recommended that the user request the latest version from the appropriate FEMA Region.

THE BENEFIT/COST ANALYSIS PROCESS

Benefit/Cost vs. Cost-Effective Analysis. Benefit/cost analysis differs from cost-effectiveness analysis in one major way—it considers a project's merits (or benefits). Analysis of cost-effectiveness simply identifies the least expensive way to achieve an objective. Benefit/cost analysis also takes into account the usually different benefits of various retrofitting measures.

Benefit/cost analysis provides estimates of the benefits and costs of a proposed project. The term "benefit/cost analysis" is used to denote economic analyses that apply either the net benefit criterion or the benefit/cost ratio criterion to evaluate prospective actions. Both costs and benefits are discounted to their present values. The net benefit criterion subtracts costs from benefits to determine if benefits exceed costs. Benefit/cost ratios provide an alternative evaluation of economically feasible projects: prospective actions in which benefits exceed costs have benefit/cost ratios above 1.0.

> The "benefits" considered in a retrofitting measure are the **future damages** and losses that are expected to be avoided as a result of the measure.

The benefits of retrofitting projects are avoided future damages. Benefits are not the damages incurred in an event already experienced, even if such damages would have been avoided by the retrofit project. Rather, benefits are the present value of the sum of expected avoided future damages for all levels of intensity of future floods.

To estimate future damages (and the benefits of avoiding them), the probabilities of future events must be considered. The probabilities of future events profoundly affect whether a proposed retrofitting measure is cost effective. The benefits of avoiding flood damage for a building in the 10-year floodplain will be enormously greater than the benefits of avoiding flood damage for an identical building situated at the 1,000-year flood level.

Each proposed retrofitting project must be evaluated on its own merits, comparing the benefits and costs of a specific project and/or alternatives. In particular, the benefits of a project may vary markedly depending on the vulnerability of the existing home to damages and losses, the probabilities of future damages, and the effectiveness of the mitigation measure in avoiding future damages.

Figure V-1 presents the basic steps in performing any benefit/cost analysis. These steps are summarized below.

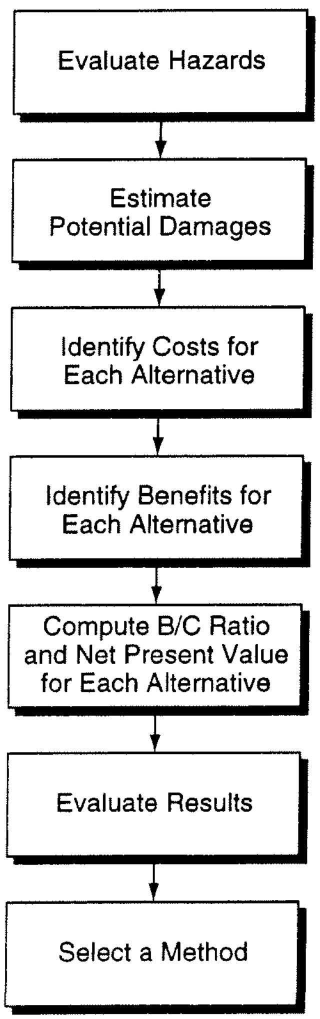


Figure V-1: Benefit/Cost Analysis Process

EVALUATE HAZARDS

Conducting a benefit/cost analysis of flood hazard mitigation projects requires estimating the expected frequency and severity of flooding in the area under consideration. Detailed flood information is given in Flood Insurance Studies (FISs) where such studies are available. State, local, and privately prepared studies may exist as well.

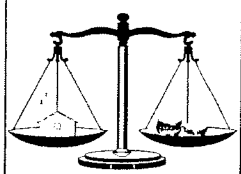
Chapter IV—Determination of Hazards—provides guidance on the development of the flood hazard information required for conducting a benefit/cost analysis.

ESTIMATE THE POTENTIAL DAMAGES (NO ACTION ALTERNATIVE)

Estimating the benefits of prospective flood hazard mitigation projects requires site-specific data to establish expected damages as a function of flood depth (and other flood hazards such as high velocity, ice/debris flows, or soil failure, where appropriate). The expected flood hazard relationships developed in the previous step are used in conjunction with actuarial flood damage data developed from FIA flood insurance claim data and compiled in tables and graphs of damage versus depth of flooding. FEMA's new software, Benefit/Cost Analysis for Flood, considers property damage and certin other economic losses.

COSTS ASSOCIATED WITH ALTERNATIVES

The costs of a flood hazard mitigation project vary according to the retrofitting measure and generally include direct construction costs, engineering or architectural design fees, permit fees, contractor's fees, the cost of temporary living quarters, and loss of income due to design/construction activities. Guidance on estimating these costs is provided in Chapter III.



ESTIMATE BENEFITS

The benefits of a flood hazard mitigation project are the avoided future damages. Benefits cannot be determined exactly because the times and severity of future flooding events are not known exactly. Rather, benefits are estimated by probability, based on experienced or hypothetical floods of various severity.

COMPUTE BENEFIT/COST RATIO AND NET PRESENT VALUE

FEMA's new software, Benefit/Cost Analysis for Flood (fall 2001 release), can be used to evaluate the benefit/cost ratio of the flood hazard mitigation measures presented in this manual. The software operates in a runtime version of Microsoft Access® included with the program.

The computation of benefit/cost values involves discounting projected benefits and their associated costs to their present values and computing either a benefit/cost ratio or a maximum present value. Benefit/cost ratios of 1.0 or greater and positive net present values indicate a cost-beneficial project.

EVALUATE RESULTS

The results of a benefit/cost analysis include the present value of damages and losses avoided, costs of the specific retrofitting measure, and calculation of either the net present value or benefit/cost ratio. As previously stated, alternatives with a positive net present value or a benefit/cost ratio greater than 1.0 indicate a cost-beneficial project.

Where more than one alternative is being considered, ranking of the alternatives from the highest to lowest net present value indicates desirability (from a benefit/cost standpoint) of each alternative with respect to other alternatives.

SELECT A METHOD

The existence of a favorable benefit/cost ratio is not the sole factor for the selection of a retrofitting measure. Other economic, technical, and subjective factors can influence the homeowner's selection of a retrofitting measure.

Conducting a benefit/cost analysis for a flood hazard mitigation project requires various data and judgments to estimate the expected frequencies and intensities of damage-producing flood events. Further estimates are made of both the benefits and costs associated with the different retrofitting measures. The calculations involved with establishing these estimates can be fairly complicated. FEMA's software automates the necessary computations.

For guidance on performing benefit/cost analysis using manual methods, please refer to "How to Evaluate Your Options" prepared by the U.S. Army Corps of Engineers National Flood Proofing Committee. A complete reference for this document is provided in Appendix C.

EVALUATE HAZARDS

To perform a benefit/cost analysis, the flood hazard to the structure in question must be determined in terms of the frequency and intensity of expected floods. The hazard analysis must include the expected frequency of flood hazards (e.g., a 50-year flood), depth of flooding, and in the case of riverine flooding, the corresponding intensity or severity of the flood [e.g., discharge of 1,500 cubic feet per second (cfs)].

Determine Flood Frequency, Discharge and Elevation

↓

Compile Discharge vs. Exceedence Probability Curve

Figure V-2: Critical Steps in Evaluating Flood Hazards

To perform an economic analysis in riverine flooding situations, the relationship between discharge and water-surface elevation (often referred to as the rating curve, depicted in Figure V-3) and the relationship between discharge and exceedence probability must be known. This section describes how to develop this data (the process is illustrated in Figure V-2). In coastal A Zones, FISs provide a table of the flood frequency versus flood elevation.

A Flood Insurance Study (FIS) consists of an FIS report, Flood Insurance Rate Map (FIRM), and (in non-coastal floodplains) a Flood Boundary and Floodway Map (FBFM). The FIS report describes how the flood hazard information was developed for the community. The FIRM shows areas inundated during a 100-year flood event. The FBFM delineates the regulatory floodway adopted within the community.

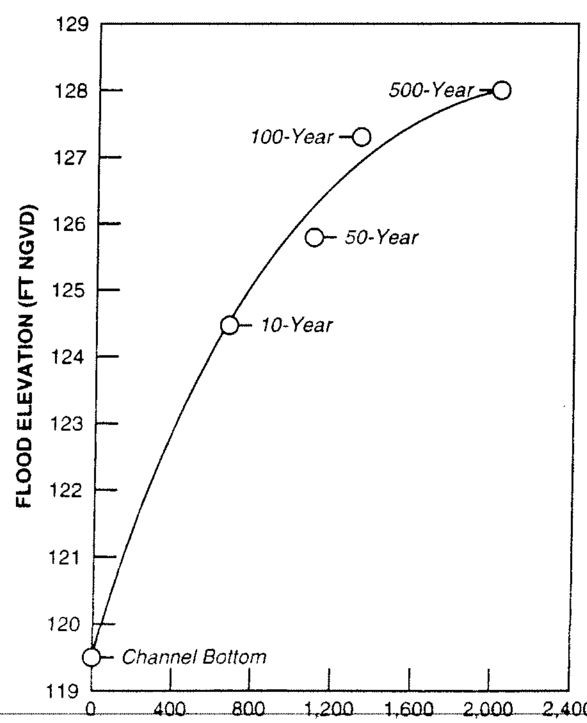


Figure V-3: Discharge Versus Elevation (Rating Curve)

DETERMINE FLOOD FREQUENCY, DISCHARGE, AND ELEVATION

To obtain a copy of the FIS for the community in question, contact FEMA's Map Service Center (MSC) at 1-800-358-9616. You can obtain information about the MSC at www.fema.gov/msc

Several tools exist that can be utilized to obtain information on the flood hazards affecting the structure in question. A Flood Insurance Study (FIS) is available for most flood-prone communities throughout the United States.

In some cases, an FIS may not be available for a community, or it may have insufficient data for the flooding source affecting the building. In these cases, the designer can turn to the U.S. Army Corps of Engineers (USACE) and Natural Resources Conservation Service (NRCS), which provide flood hazard information reports for many flooding sources. The U.S. Geological Survey (USGS) and the Tennessee Valley Authority (TVA) also publish stream gage data and have flood information reports for various flooding sources.

The various agencies that maintain flood information are listed in Appendix C.

State or local floodplain studies may also be available for the community. For more information concerning available data, contact the floodplain management services office of the USACE or the local offices of the USGS, TVA, NRCS, or your municipal engineer, floodplain administrator, flood control district, or water control boards.

COMPILE DISCHARGE VERSUS EXCEEDENCE PROBABILITY CURVE

For riverine Zone AE scenarios, FEMA's benefit/cost computer program takes the data for flood frequency, discharge, and elevation and automatically compiles the discharge versus elevation and discharge versus exceedence probability curves. This information is critical for the development of the depth-damage relationships presented in the next step.

Coastal Zone A flooding is evaluated using storm surge models or tide gage analyses, which predict flood elevations. The FIS gives flood elevations relative to a benchmark elevation, generally, the National Geodetic Vertical Datum of 1929 (NGVD 29) or the North American Vertical Datum of 1988 (NAVD 88).

Unlike riverine FIS data, flood data given in the FIS for coastal A Zones include a table of exceedence probability (flood frequency) versus flood elevation. FEMA's benefit/cost computer model analyzes these data and creates a relationship between exceedence probability and flood depth. This regression fit gives the annual exceedence probability for all floods in one-foot increments of depth.

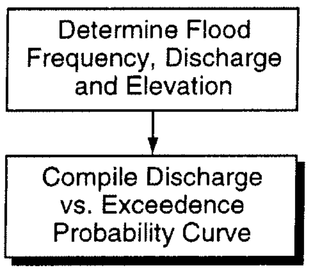


Figure V-4: Critical Steps in Evaluating Flood Hazards

For a given coastal area covered by an FIS and a FIRM, the elevations of the 10-, 50-, 100-, and 500-year floods are constant over the entire area. However, the probability of a given flood depth occurring at a specific site depends very strongly on the elevation of the particular site. Thus, the Zero Flood Depth Elevation of the facility under evaluation has a profound impact on the degree of flood risk experienced at the site.

ESTIMATE POTENTIAL DAMAGES

Estimating the potential damages to a structure for the no-action (before mitigation) alternative is a critical step in the overall development of expected benefits from retrofitting measures. The potential damages (flooding depth and loss of function) from the no-action alternative serve as the baseline from which future avoided damages can be computed for various retrofitting alternatives.

The damage curves derived from FIA data (Figure V-5) are included as default values in FEMA's benefit/cost software. The software also contains default estimates of displacement and loss of function time. Alternatively, the analyst can enter custom curves based on building-specific conditions. The software uses these curves to compute flood-depth vs. damage and probability vs. damage relationships.

The estimated damages and losses for the existing building at each flood depth depend on the depth-damage functions for items such as building and contents, displacement, and rental losses. The expected damages and losses also depend very strongly on the degree of flood risk at the site under evaluation.

Flood Depth	Building Damage Percent by Building Type (based on building replacement value)					
	1 Story without Basement	2 Story without Basement	Split Level without Basement	1 or 2 Story with Basement	Split Level with Basement	Mobile Home
-2	0	0	0	4	3	0
-1	0	0	0	8	5	0
0	9	5	3	11	6	8
1	14	9	9	15	16	44
2	22	13	13	20	19	63
3	27	18	25	23	22	73
4	29	20	27	28	27	78
5	30	22	28	33	32	80
6	40	24	33	38	35	81
7	43	26	34	44	36	82
8	44	29	41	49	44	82
>8	45	33	43	51	48	82

Flood Depth	Contents Damage Percent by Building Type (based on total value of contents)					
	1 Story without Basement	2 Story without Basement	Split Level without Basement	1 or 2 Story with Basement	Split Level with Basement	Mobile Home
-2	0	0	0	6	5	0
-1	0	0	0	12	8	0
0	14	8	5	17	9	12
1	21	14	14	23	24	66
2	33	20	20	30	29	90
3	41	27	38	35	33	90
4	44	30	41	42	41	90
5	45	33	42	50	48	90
6	60	36	50	57	53	90
7	65	39	51	66	54	90
8	66	44	62	74	66	90
>8	68	50	65	77	72	90

Figure V-5: FIA Depth-Damage Data Table for Structural Damage and Contents

FEMA's benefit/cost ratio model characterizes losses expected both before and after mitigation as follows:

Scenario damages are based on depth of flooding, not on flood hazard risk. Two identical buildings at different locations will have identical scenario damages, given the same depth of flooding.

Scenario Damages: Scenario damages indicate the estimated damages that would result from a single flood of a particular depth at the building under evaluation. For example, the scenario damages for a three-foot flood are the expected damages and losses **each time** a three-foot flood occurs at a particular site. Scenario damages do NOT depend on the probability of floods at that location. The model tabulates scenario damages for each flood depth from -2 to >8 feet for building damages, contents damages, displacement costs, and rental income losses (as well as other categories not applicable to residences). The total damages and losses are shown for each flood depth. This information shows the total vulnerability of the existing building to flood damage, how these damages are distributed among different categories of damages, and how these damages vary with flood depth.

Even for buildings with high expected annual damages, mitigation projects are not necessarily cost-beneficial. Whether or not a project is cost-beneficial depends on the cost of the mitigation project and on the effectiveness of the mitigation project in avoiding damages, as well as on the expected annual damages.

Expected Annual Damages: Expected annual damages take into account the annual probabilities of floods of each depth. Expected annual damages are the average damages per year expected over a long period. "Expected annual" does not mean that these damages will occur every year. For each flood depth, expected annual damages are calculated by multiplying the scenario damages times the expected annual number (probability) of floods of each depth.

The expected annual damages are tabulated in the same way as scenario damages. Expected annual damages will generally be much smaller than scenario damages because they are multiplied by the probabilities of occurrence, which are small numbers.

> Scenario damages and expected annual damages provide different information. Scenario damages describe how bad flood damages will be each time a flood occurs. However, because scenario damages do not consider flood probabilities, they do not provide sufficient information for decision making. Scenario damages for a given flood depth may be high, but if the flood probability is very low, no mitigation action may be warranted. If a five-foot flood causes $50,000 in damages but such a flood is expected to occur only once in 1,000 years, then simply repairing the very infrequent flood damage may be the most sensible strategy.

The scenario damages before mitigation and the expected annual damages before mitigation provide, in combination, a complete picture of the vulnerability of the building to flood damage before undertaking a mitigation project.

Expected annual damages consider flood probabilities. A building with high expected annual damages means that not only are scenario damages high, but also that flood probabilities are relatively high. If expected annual damages are high, then there will be high potential benefits in avoiding such damages.

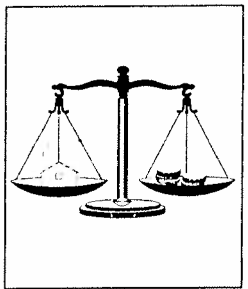
Damages after mitigation depend on the effectiveness of the mitigation measure in avoiding damages. The expected annual damages and losses after mitigation also depend very strongly on the degree of flood risk at the site under evaluation. For some mitigation projects, such as relocation or buyout, the scenario damages and expected annual losses after mitigation will be zero. For other mitigation projects, such as elevation or flood barriers, scenario damages and expected annual losses after mitigation will be lower than before mitigation but not zero. FEMA's benefit/cost model tabulates after-mitigation losses in the same way as before-mitigation losses.

IDENTIFY COSTS ASSOCIATED WITH ALTERNATIVES

Once a detailed review of the flood hazard and associated losses has been performed, the costs associated with each of the technically feasible alternative retrofitting measures must be determined. Developing detailed construction cost estimates is crucial to ensuring that the homeowner can afford to complete the project. In Chapter III, a methodology for developing preliminary estimates of the cost of various retrofitting measures was presented. The methodology for developing detailed construction costs is similar, but requires more detail and definition of project component quantities and unit costs and often occurs after the preliminary economic analysis. Generally, the approach to examining retrofit alternatives and selecting the one that is most appropriate is an iterative cycle that includes the following steps:

- examine technical feasibility of alternatives;
- develop preliminary cost estimates of each alternative being considered;
- model benefit/cost ratios of considered alternatives;
- rank alternatives based on net benefits (benefits minus costs);
- develop more detailed design studies of highly ranked alternatives and detailed cost estimates; and
- refine benefit/cost models if previous step yields cost figures significantly different from previous estimates, and re-rank alternatives as indicated based on new net benefits and homeowner preference.

~~Detailed cost estimating is discussed later in this chapter.~~



ESTIMATE BENEFITS

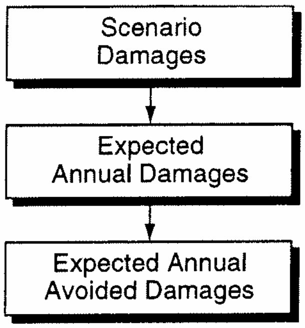


Figure V-6: Types of Benefits Evaluated

The benefits of a flood hazard mitigation project are the reduction in damages that would otherwise be expected. Expected annual benefits are defined as the sum of expected avoided damages. The benefit/cost software automatically computes values for the types of damages illustrated in Figure V-6 and explained below.

- **Scenario Damages**: The expected damages for a flood event reaching a given flood depth at the residence. Scenario damages (SCD) are the sum of building damages (BD), contents damages (CD), displacement costs (DIS), and rental income losses (RENT) caused by the flood depth associated with a particular event.

FORMULA

$$SCD = BD + CD + DIS + RENT$$

where: **SCD** is the total scenario (per event) damages;
BD is scenario building damages in dollars;
CD is scenario contents damages in dollars;
DIS is scenario displacement costs in dollars; and
RENT is scenario rental income losses in dollars.

Formula V-1: Scenario Damages

- **Building Damages:** (BD) are defined as the product of floor area of the building (FA), replacement value of the building per square foot (BRV), and the depth damage function (DDF), which is the expected damage at a given flood depth expressed as a percentage of building replacement value.

FORMULA

$$BD = (FA)\,(BRV)\,(DDF)$$

where: **BD** is the total amount of building damage per scenario in dollars;
FA is the floor area of the building (in square feet);
BRV is the replacement value of the building (dollars per square foot); and
DDF is the expected damage by flood depth, expressed as a percentage of building value.

DDF*
[illegible]
from Fig V-9 (page V-1[illegible])

Formula V-2: Building Damages

- **Contents Damages:** (CD) are estimated as the product of the expected contents damage (ECD) and the total building contents replacement value (CRV) for each flood depth. Building and contents damages can also be taken from the depth-damage curves developed by FIA.

FORMULA

$$CD = (ECD)\,(CRV)$$

where: **CD** is the total contents damage in dollars;
ECD is the expected contents damage for a given flood depth, expressed as a percentage of contents replacement value; and
CRV is the total building contents replacement value in dollars.

Formula V-3: Contents Damages

- **Displacement Costs:** (DIS) are defined as the product of the number of days that the occupant must be out of the building due to flooding (DD), the total costs of displacement per day per square foot of living area (TDC), and the total area occupied (TA).

FORMULA

$$\mathbf{DIS = (DD)\,(TDC)\,(TA)}$$

where: **DIS** is the displacement cost in dollars;
DD is the estimated number of displacement days for a given flood depth;
TDC is the estimated displacement costs per day per square foot; and
TA is the total area occupied in square feet.

Formula V-4: Displacement Costs

- **Rental Income:** Losses are also included if all or part of the residence is rented. Rental income losses (RENT) are the product of displacement days (DD) and the daily rental rate (DRR).

FORMULA

$$\mathbf{RENT = (DD)\,(DRR)}$$

where: **RENT** is the total rental income lost in dollars;
DD is the number of displacement days; and
DRR is the daily rental rate in dollars.

Formula V-5: Rental Income Losses

- **Expected Annual Damages**: Expected annual damages (AD) are the product of scenario damages (SCD) and the expected annual number of floods of a given depth (EAE):

FORMULA

$$AD = (SCD)\,(EAE)$$

where: **AD** is the expected annual damages in dollars;
SCD is the scenario damages (as defined previously) in dollars; and
EAE is the expected annual number of floods of a given depth.

Formula V-6: Expected Annual Damages

- **Expected Avoided Damages**: Expected avoided damages (AVD) are the product of expected annual damages (AD) and the effectiveness of the mitigation measure (EFF):

FORMULA

$$AVD = (AD)\,(EFF)$$

where: **AVD** is the expected avoided damages in dollars;
AD are expected annual damages in dollars;
EFF is the effectiveness of the mitigation measure in reducing expected damages from a flood of a given depth (percent of expected damages expressed as a decimal equivalent).

Formula V-7: Expected Avoided Damages

- **Expected Annual Benefits**: The expected annual benefits (AB) of a hazard mitigation project are the sum of expected avoided damages (AVD) over the range of flood depths considered. FEMA's benefit/cost model includes a range of from -2 feet to >8 feet.

FORMULA

$$AB = \sum_{RF=min}^{max} AVD$$

where:

AB	is the expected annual benefits in dollars;
RF	is the flood depth considered above the zero flood depth elevation (in feet);
min	is the minimum damaging flood considered above the zero flood depth elevation (in feet);
max	is the maximum flood depth considered above the zero flood depth elevation (in feet); and
AVD	is the expected annual avoided damages from each flood depth above the zero flood depth elevation considered (in dollars).

Formula V-8: Expected Annual Benefits

COMPUTE BENEFIT/COST RATIO AND NET PRESENT VALUE

One important aspect of benefit/cost analysis is accounting for the time value of money. The value of money changes over time due to economic, political, and other factors. Interest rate changes may impact the estimation of costs and benefits expected to occur in the future.

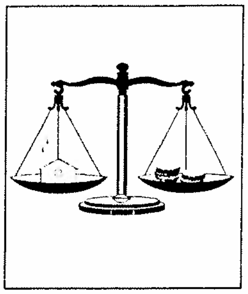
For that reason, benefit/cost analysis requires a common basis for comparing estimates of project costs and benefits. This is usually accomplished by converting present, future, and annual project costs and benefits to a common basis such as present value, future value, or average annual values.

The assumed interest rate, or discount rate, is the factor that controls the conversion of future values to present values.

Increasing the discount rate lowers the present value of future benefits/costs and, conversely, lowering the discount rate raises the present value of future benefits/costs.

Formulas provided here are automated in FEMA's benefit/cost software.

As previously mentioned, either the benefit/cost ratio or net benefit criterion can be used to evaluate each prospective retrofitting action. Earlier sections of this chapter have built the foundation for completion of the analyses discussed below.



CONVERT ESTIMATED ANNUAL BENEFITS TO A PRESENT VALUE

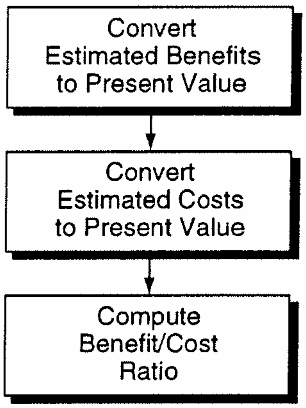


Figure V-7: Critical Steps in Benefit/Cost Ratio Analysis

After determining the average annual damage to be prevented by the retrofitting measure, the present worth of damages prevented over the expected life of the structure can be determined. To make this determination, one must first assume the building's life expectancy; this will normally be the useful life of the structure. However, analysts can use the period the homeowner plans to occupy the home, or the length of the mortgage. Secondly, an interest rate for borrowing money to retrofit must be assumed. This rate may be obtained from any bank. The analyst can then use the following formula to compute a present worth factor for the assumed life of the structure and the assumed interest rate:

FORMULA

$$PWF = \frac{(1+i)^n - 1}{(1+i)^n}$$

where: **PWF** is the present worth factor;
n is the assumed life of the structure (years); and
i is the assumed interest rate for borrowing money (decimal equivalent of percent per year).

Formula V-9: Present Worth Factor

Multiply the average annual damage prevented by retrofitting by the present worth factor to determine the present-day value of these expected flood damages avoided.

FORMULA

$$EAB_{PV} = (PWF)\ (AB)$$

where: EAB_{PV} is the present value of estimated annual benefits in dollars;
PWF is the present worth factor; and
AB is the expected annual benefits of a mitigation project in dollars.

Formula V-10: Present Value of Estimated Annual Benefits

CONVERT ESTIMATED COSTS OF RETROFITTING TO A PRESENT VALUE

The primary cost of a retrofitting measure will be the engineering and construction costs, which already represent present-day values. Should the retrofitting measure require annual operation and maintenance costs (including replacements), these estimated periodic costs should be converted to a present-day value, using the same methodology previously employed to convert annual benefits to a present value worth.

FORMULA

$$EAC_{PV} = (PWF)\ (AC) + ECC_{PV}$$

where: EAC_{PV} is the present value of estimated annual costs in dollars;
PWF is the present worth factor;
AC is the expected annual cost (in dollars) for operation and maintenance of a specific retrofitting measure; and
ECC_{PV} is the present value of the engineering and construction costs associated with a specific retrofitting measure, in dollars.

Formula V-11: Present Value of Estimated Annual Costs

COMPUTE THE BENEFIT/COST RATIO AND/OR NET BENEFIT

Once the present value of the benefits and costs associated with a retrofitting measure is computed, dividing the present value of the benefits by the present value of the costs will enable the designer to evaluate retrofitting alternatives.

$$BCR = EAB_{PV} / EAC_{PV}$$

where: **BCR** is the benefit/cost ratio;
EAC_{PV} is the present value of estimated annual costs in dollars; and
EAB_{PV} is the present value of estimated annual benefit in dollars.

Formula V-12: Benefit/Cost Ratio

An alternative evaluation measure is to subtract the present value of the costs from the present value of the benefits.

$$NPV = EAB_{PV} - EAC_{PV}$$

where: **NPV** is the net present value or benefit of the mitigation measure;
EAC_{PV} is the present value of estimated annual costs in dollars; and
EAB_{PV} is the present value of estimated annual benefits in dollars.

Formula V-13: Net Present Value

A benefit/cost ratio of 1.0 or greater indicates that the benefits of the retrofitting alternative exceed the costs. The alternative with the highest benefit/cost ratio or net benefit would be the preferred alternative from an economic perspective, if the same level of protection (design flood) is being evaluated.

It should be pointed out that the entire procedure of generating a benefit/cost ratio is not an exact science but instead a subjective process. The creation of a benefit/cost ratio is intended to give an idea of the cost effectiveness of a specific retrofitting technique in comparison to the other options available. As long as the same procedures are utilized in all scenarios, the ratio should provide the designer with an idea of the relative cost effectiveness of all options.

Benefit/cost models can be used to optimize the selection of a retrofitting measure by analyzing incremental improvements to a selected alternative. This is accomplished by maximizing (avoided damages) benefits while minimizing project costs. It is an iterative process whereby an original retrofitting solution is modified by adding or deleting design features and/or designated protection levels. Each modification will have an impact on the project benefits and costs and subsequently the benefit/cost ratio. This technique will assess the relationship between increased (decreased) cost and increased (decreased) effectiveness for the range of modifications with a particular retrofitting measure analyzed.

The following example illustrates this optimization technique.

Benefit/Cost Analysis Optimization Example

Given: A one-story, 2,500 SF slab-on-grade building with a first floor elevation of 6.0 NGVD is subject to coastal A Zone flooding (1-yr = 2.0', 10-yr = 5.0', 50-yr = 7.0', 100-yr = 9.0', and 500-yr = 10.0').

Building replacement is estimated at $50/SF; contents replacement at $8/SF, and rental cost (displacement) at $1/SF.

Alternative 1: Construct a 3-foot-high floodwall (9.0' NGVD) around the building. The floodwall has a 30-year useful life and project costs are estimated at $10,000 with an annual maintenance cost of $250.

Floodwalls are considered effective to one foot below their flood protection elevation. In this case, seepage and leakage concerns reduce the project effectiveness to 90% for floods reaching 6.0' NGVD; 85% at 7.0,' NGVD; 80% NGVD, and 0% at 9.0' NGVD and above (since the water elevation is the same both inside and outside the floodwall due to overtopping).

Alternative 1 Results: Benefit/cost ratio of 1.03 indicates this project is beneficial to pursue.

However, the homeowner is concerned that seepage and leakage will damage flooring and building contents (and result in a potentially expensive temporary relocation) and is therefore considering adding an interior drainage system (periphery drainpipe and sump pump system) to Alternative 1. Economic optimization can be used to indicate whether this design change would be cost-beneficial.

Alternative 2: Construct an interior drainage system with the 3' floodwall proposed in Alternative 1. New project costs are estimated at $15,000 with annual maintenance of $350. The drainage system improves project effectiveness to 100% at all flood depths up to and including 8.0' NVD.

Alternative 2 Results: Benefit/cost ratio of 0.81 indicates the addition of an interior drainage system would not be a beneficial modification to Alternative 1.

This results from the fact that the increased benefits (damages avoided) are not sufficient to support the additional construction cost and annual maintenance expenditures.

SELECT A METHOD

While benefit/cost analysis provides an indication as to whether a retrofitting alternative is cost-beneficial, it is not the sole parameter upon which retrofitting measures are selected. Occasionally, there will be more than one favorable alternative, or the designer will customize the retrofitting measure, either by combining several methods or varying the level of protection.

Owner preference can also have an impact on sound economic analysis and make a less cost-beneficial alternative a more preferable choice. The cost of the retrofitting measure may be the pivotal factor in a homeowner-financed retrofitting project. Conversely, local code requirements may limit the use of a method preferred by the homeowner. In the final analysis, it is the owner who must be satisfied with the retrofitting alternative. Each of these factors (aesthetics, local code requirements, and hazards such as wind, earthquake, erosion, impact, and other forces) may affect the applicability of a specific retrofitting measure. The designer is advised to consider these factors along with the economic analysis of the various alternatives (see Figure V-8).

Factors Weighing on Alternative Selection

- Total Project Cost
- Net Benefits
- Technical Feasibility
- Aesthetics
- Need for Human Intervention
- Need for Annual Maintenance

Figure V-8: Factors Weighing on Alternative Selection

- **Total Project Cost**: This represents costs required to construct the retrofitting alternative. The designer should review this value in terms of how the project suits the homeowner's budget.

- **Net Benefits**: As discussed previously, this value indicates whether an alternative is cost-beneficial. The higher the value, the more cost-beneficial the alternative. The designer should review the net benefits for the retrofitting alternatives being considered.

- **Technical Feasibility**: The designer must judge the technical solution(s) that best address the project objectives.

- **Aesthetics**: This value reflects the owner's view on the way the retrofitting alternative fits in with the appearance of his/her house.

- **Human Intervention Requirements**: This reflects the need for human intervention to operate the retrofit measure and the warning time required to conduct the required activity.

- **Annual Maintenance**: This reflects the intensity of annual maintenance required by each retrofitting alternative.

A preference scale or order of preference ranking can be utilized with the table presented in Figure V-9 to arrive at a subjective decision on the retrofitting method to be selected. The preference scale assigns numbers 0 to 10 to each alternative by factor, with 0 indicating undesirable and 10 meaning highly desirable. The values assigned to the various factors for each alternative are totalled, and the alternatives with the highest total should be the optimal choices.

The preference scale process can also be modified by weighting the decision factors to reflect the increased importance of any specific factor. For example, if total project cost were the predominant factor, the value (0-10) could be multiplied by a factor, for example, 2, which would double its contribution to the overall score, thereby reflecting its importance.

Owner Name: ______________________ Prepared By: ______________________
Address: ______________________ Date: ______________________
Property Location: ____

Alternative	Decision Factors: Cost	Net Benefits	Technical Difficulty	Aesthetics	Human Intervention	Annual Maintenance	Other ______ ______	Total Score
1. *Elevation*								
Preference	5	6	10	4	10	10		45
Importance	1.0	1.0	1.0	1.5	0.5	0.5		
Weighted Score	5	6	10	6	5	5		37
1. ______								
Preference								
Importance								
Weighted Score								
2. ______								
Preference								
Importance								
Weighted Score								
3. ______								
Preference								
Importance								
Weighted Score								
4. ______								
Preference								
Importance								
Weighted Score								
5. ______								
Preference								
Importance								
Weighted Score								
6. ______								
Preference								
Importance								
Weighted Score								

Instructions:

This matrix may be filled out by the designer in consultation with the homeowner. The objective of this matrix is to select an alternative for design from competing alternatives which had previously passed screening for technical feasibility and homeowner preference.

For each alternative, enter the alternative name (i.e. 1A, 1B, 1C) and unweighted preference score (0-10) on the first row. A score of 0 indicates the measure is the least preferred in terms of the decision factor, while a score of 10 indicates the measure is the most preferred. A blank column is provided for any additional decision factor(s) which are being considered by the designer or homeowner.

Based upon the relative importance of each decision factor to the designer and homeowner, develop and enter an importance factor (weighting amount) for each decision factor on the second row. Multiply the unweighted preference score by the importance factor (weighting amount) and enter the result on the third line. Total the first and third lines on the right hand column (Total Score). The preferred alternative is the one with the highest weighted score.

Figure V-9: Preference Ranking Worksheet

DETAILED COST ESTIMATING

Previously, in Chapter III, we were able to utilize a unit cost (per square foot) for a specific retrofitting measure, such as elevating a wood-frame building on an open foundation and adding ancillary items for fill and landscaping, to arrive at a preliminary construction cost estimate. When and if the cost estimate is refined after the retrofit measure alternatives are further defined from a design standpoint, the costs of each may be found to differ from earlier estimates that were used to rank the retrofit alternatives. If this difference in estimated cost is significant for a given alternative, the benefit/cost ratio for that alternative could be affected. Therefore, the designer/analyst may re-run the benefit/cost model for any alternatives affected in this way, which could result in a different ranking of potential retrofit alternatives.

When the retrofitting measure is designed (as discussed in Chapter VI), the cost estimate can be refined by identifying and pricing all of the components of the retrofitting measure. For example, site preparation, building preparation, permitting, excavation and earthwork, foundation, concrete, reinforcing, framing, elevation, utility extension, connections, code upgrades, backfill, site stabilization, access/egress, landscaping, and interest costs can be estimated and then aggregated.

Cost estimate accuracy can be directly related to the level of detail in a quantity breakdown. Quantities or components not identified usually do not get estimated and may not be covered by any allowed-for contingency, resulting in less accurate estimates. Figure V-10, the Floodproofing Measure Component Takeoff Guide, was developed to identify cost items typically found in the various retrofitting measures. However, every retrofitting application is unique and may include more of or fewer than the components listed.

Floodproofing Measure Component Takeoff Guide

Elevation Techniques
- Site Preparation
- Building Preparation
- Elevation of Structure
- Foundation Construction
- Connection of Structure to New Foundation
- Extension of Utility Systems
- Required Code Upgrades
- Exterior Finish Work
- Interior Finish Work
- Access and Egress
- Site Grading and Stabilization
- Landscaping

Relocation Techniques
- Preparation of Existing Site
- Preparation of Existing Building
- Preparation of the Route
- Elevation of Structure
- Transfer of Building to Transportable Frame
- Moving Building
- Preparation of New Site (Including Utilities)
- New Foundation Construction
- Transfer of Building to New Foundation
- Connection of Utility Systems
- Exterior Finish Work
- Interior Finish Work
- Access and Egress
- Site Grading and Stabilization
- Landscaping
- Demolition of Old Foundation
- Grading and Stabilization of Old Site
- Route Modification Reversals

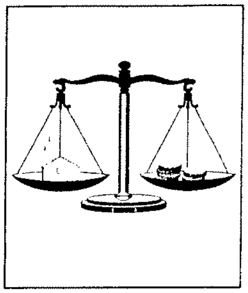
Floodwalls
- Site Preparation
- Excavation
- Construction of Floodwall
- Closure Installation
- Access and Egress
- Drainage System Installation
- Site Grading and Stabilization
- Interior Area Finishing
- Utility System Adjustment
- Landscaping

Levees
- Site and Borrow Area Preparation
- Earthwork
- Drainage System Installation
- Access and Egress
- Site Grading and Stabilization

Shields
- Building Preparation
- Shield Installation
- Interior Drainage System
- Utility System Modification

Sealants
- Building Excavation and Preparation
- Sealant Application
- Interior Drainage System
- Utility System Modification

Figure V-10: Floodproofing Measure Component Takeoff Guide



SOURCES FOR UNIT COSTS

Once a detailed quantity takeoff has been completed, unit-cost information can be obtained for individual items from a variety of sources. These sources include:

- local construction industry data collected from published indexes or solicited from several construction companies;

- average nationwide construction cost data, available from various publications, that contain factors for adjusting the average nationwide costs to specific locations and present-day values; and

- data collected by the FEMA Mitigation Directorate for areas of the United States that have recently experienced major flood damage. These unit costs may have to be adjusted to a specific geographical area by multiplying the FEMA unit cost by a factor of the Bureau of Labor Wholesale Price Index (or other published cost index) for the subject community and the community for which FEMA has data.

FEMA has observed post-disaster inflation due to material and labor shortages that has significantly impacted the costs of restoring flood-damaged houses. For example, the cost of materials and labor was 10% higher after the 1993 Midwest flooding than before the storm. In an extreme case (catastrophic disaster) such as Dade County, Florida, after Hurricane Andrew, the increase was 25%.

Unit costs are adjusted for local conditions with the following computation:

FORMULA

$$UC_{local} = UC_{FEMA}\,(WPI_{local}/WPI_{FEMA})\,(I_{PD})$$

where: UC_{local}	is the unit cost of a specific retrofitting measure component at the location in question;
UC_{FEMA}	is the FEMA unit cost for a specific retrofitting measure at a specific location;
WPI_{FEMA}	is the wholesale price index or other published cost index for the locality at which FEMA has unit price data;
WPI_{local}	is the wholesale price index or other published cost index at the locality for which a unit cost is needed; and
I_{PD}	is post-disaster inflation due to a shortage of skilled labor and limited availability of materials. It ranges from 100 percent to 125 perent, but is normally 110 percent.

Formula V-14: Adjusting Unit Costs for Local Communities

Once appropriate unit-cost information has been collected, the Floodproofing Measure Component Takeoff Guide (Figure V-10) and the Detailed Cost Estimating Worksheet (Figure V-11) can be used to develop the detailed cost estimate. It is important to include the contractor's profit and a contingency item to cover unexpected costs.

At the completion of this chapter, the designer has determined flood and non-flood-related hazards; developed and evaluated retrofitting alternatives; and, in concert with the homeowner, selected a retrofitting measure that addresses the flooding problem. The next step, covered in Chapter VI, is to develop a detailed design of the selected retrofitting measure and produce construction documents.

Owner Name: ______________________ Prepared By: ______________________
Address: ______________________ Date: ______________________
Property Location: ____

Floodproofing Measure: (Describe Project Specifics) ______________________
____
____

Estimating Item	Quantity	Unit	Unit Cost	Item Cost
Subtotal				
Design Fee				
Contractor's Profit				
Subtotal				
Contingency				
Total				

Figure V-11: Detailed Cost Estimating Worksheet

Dry Floodproofing

Table of Contents

DRY FLOODPROOFING

Dry floodproofing measures can be described as a combination of adjustments and/or additions of features to buildings that eliminate or reduce the potential for flood damage by keeping floodwaters out of the structure. Examples of these adjustments and additions include:

- installation of watertight shields for doors and windows;
- reinforcement of walls to withstand floodwater pressures and impact forces generated by floating debris;
- use of membranes and other sealants to reduce seepage of floodwater through walls and wall penetrations;
- installation of drainage collection systems and sump pumps to control interior water levels, collect seepage, and reduce hydrostatic pressures on the slab and walls;
- installation of check valves to prevent the entrance of floodwater or sewage flows through utilities; and
- anchoring of the building to resist flotation, collapse, and lateral movement.

Dry floodproofing is not allowed by FEMA for new or substantially improved or damaged residential structures located in the floodplain.

Buildings that are dry floodproofed may be subject to extensive hydrostatic and other forces against the foundation and other exterior walls and surfaces. As was illustrated in Chapter IV, hydrostatic and soil pressures increase with the depth of flooding. For that reason, foundation walls have severe limitations with regard to the use of dry floodproofing measures. A critical design consideration is the comparison of the ability of the existing foundation walls to withstand the expected flood-related and non-flood-related forces with and without additional strengthening measures.

Dry Floodproofing

In this section (see Figure VI-D1) the process of selection and design of sealants, shields, drainage collection systems, sump pumps, and backflow valves and the provision of emergency power to operate necessary drainage systems are discussed. It is important that the designer understand that dry floodproofing measures are typically needed as part of most retrofitting measures. Each link in the retrofitting system must be designed to work in concert with the others to provide the level of protection desired.

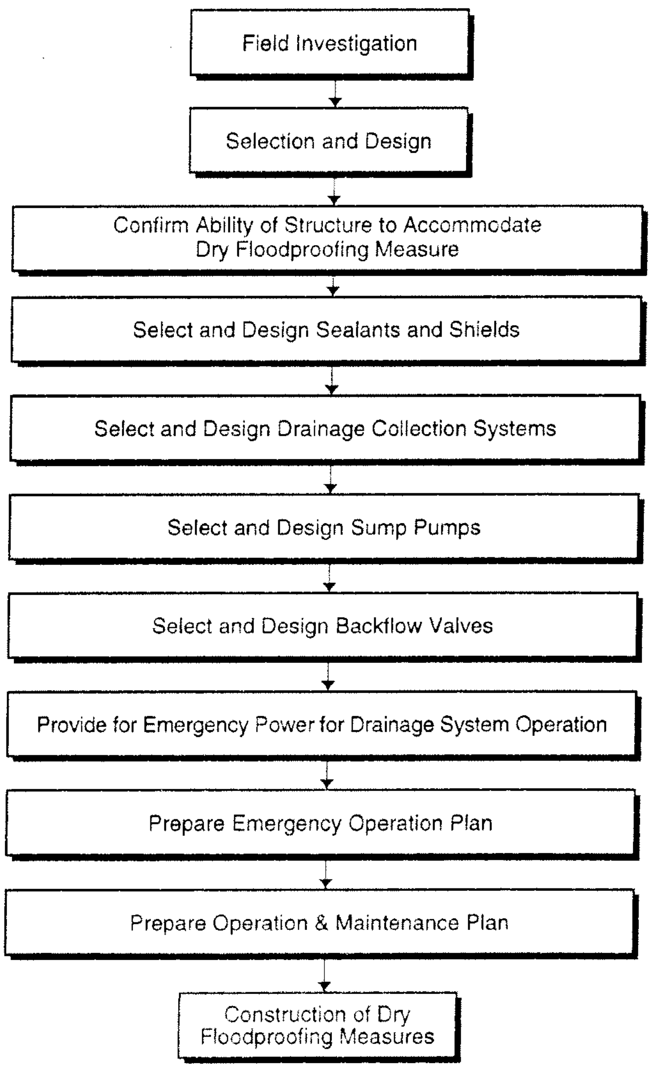


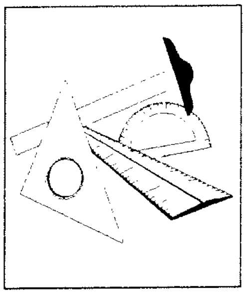
Figure VI-D1: Process of Selection and Design for Dry Floodproofing

> For additional information on dry floodproofing, refer to FEMA Technical Bulletin 3-93, *Non-Residential Floodproofing—Requirements and Certification for Buildings Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program.*

One critical aspect of a successful design of a dry floodproofing measure is the development of Emergency Operation and Inspection and Maintenance Plans. Some of the important elements of these plans are presented below.

EMERGENCY OPERATIONS PLAN

A plan for notifying homeowners (community flood warning system) of the need to install dry floodproofing components and the chain of command/resources (human intervention) to carry out the installation of dry floodproofing measures are two critical aspects of an effective emergency operations plan. In addition, a suitable evacuation plan and periodic training in the installation of dry floodproofing measures are important elements in ensuring their effectiveness.



INSPECTION AND MAINTENANCE PLAN

Every dry floodproofing system requires some degree of periodic maintenance and inspection to ensure that all components will operate properly under flood conditions. Components that should be inspected as part of an annual maintenance and inspection program include:

- All mechanical equipment such as sump pumps and generators;
- Flood shields, to ensure that they fit properly and that the gaskets and seals are in good working order, properly labeled, and stored where accessible; and
- Sealed walls and wall penetrations, for cracks and potential leaks.

SEALANTS AND SHIELDS

Floodwalls and floodwall closures are discussed in Section F of this chapter.

Sealants and shields are methods that can be used to protect a structure from low-level flooding. Mini-floodwalls (low level) can be used as an alternative to shields for protection of windows, window wells, or basement doors. These systems are easily installed and can be inexpensive in relation to other measures such as elevation or relocation. However, by sealing (closing) a structure against flood inundation, the owner must realize that, in most cases, the typical building will not be capable of resisting the loads generated by more than a few feet of water. There will be a point beyond which the sealants and shields may do more harm than good and the owner must allow the building to flood to prevent structural failure from unequalized forces.

The U.S. Army Corps of Engineers, National Flood Proofing Committee, has investigated the effect of various depths of water on brick veneer-over-wood and masonry walls. The results of their work show that, as a general rule, no more than three feet of water should be allowed on a brick veneer wall or on a non-reinforced concrete block wall that has not previously been designed and constructed to withstand flood loads. While no definitive research on floodproofing wood-frame walls without brick veneer facing has been undertaken, it is generally accepted that wood-frame houses will fail at a lower water depth than a masonry or brick veneer home. Therefore, application of sealants and shields should involve a determination of the structural soundness of a building and its corresponding ability to resist flood and flood-related loads.

Sealants include compounds that are applied directly to the surface of the structure to seal exterior walls and floors, or a wrap that is anchored to the exterior wall or foundation at or below the ground and attached to the wall above grade during flooding. The owner may wish to add to the structural strength of the existing building to aid in resisting flood-induced loads (for example adding a brick veneer).

Any dry floodproofing system can be expected to allow some water infiltration, and the owner should have a dewatering system capable of removing the water. Due to this infiltration through exterior walls and floors and percolation of the water around ground anchored wraps, these systems are not recommended for situations where floodwater is in contact with the building for more than 12-24 hours. Underlying soils often dictate the allowable period of inundation before water starts to percolate through the sealant system.

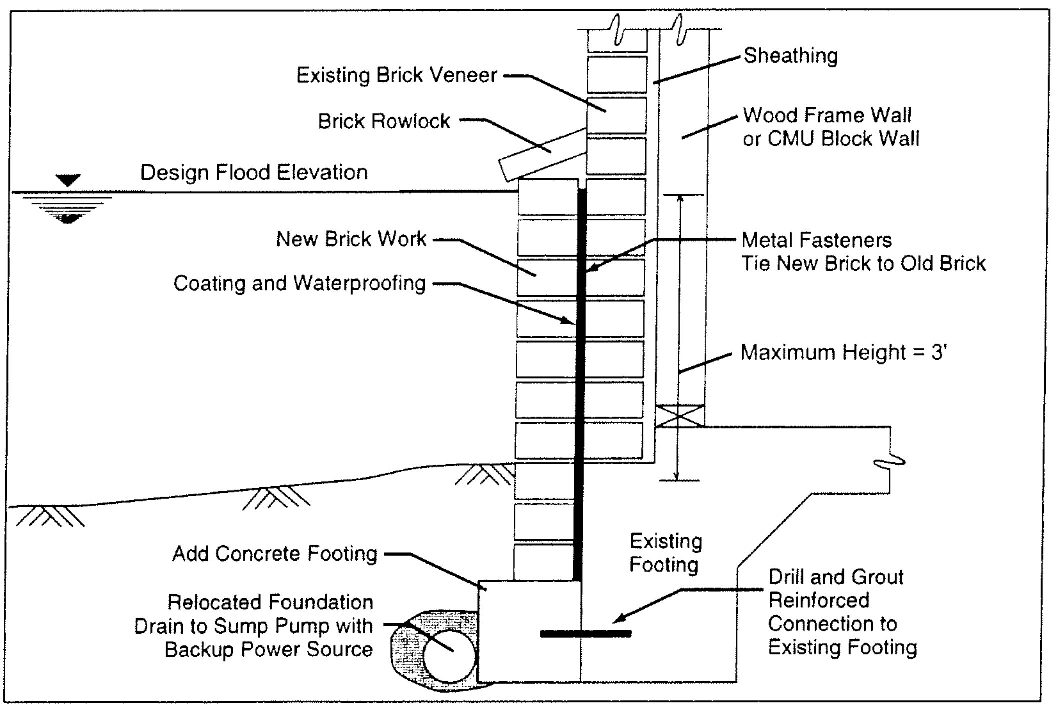


Figure VI-D2: The best way to seal an existing brick-faced wall is to add an additional layer of brick with a seal in between. Just sealing the existing brick is also an option.

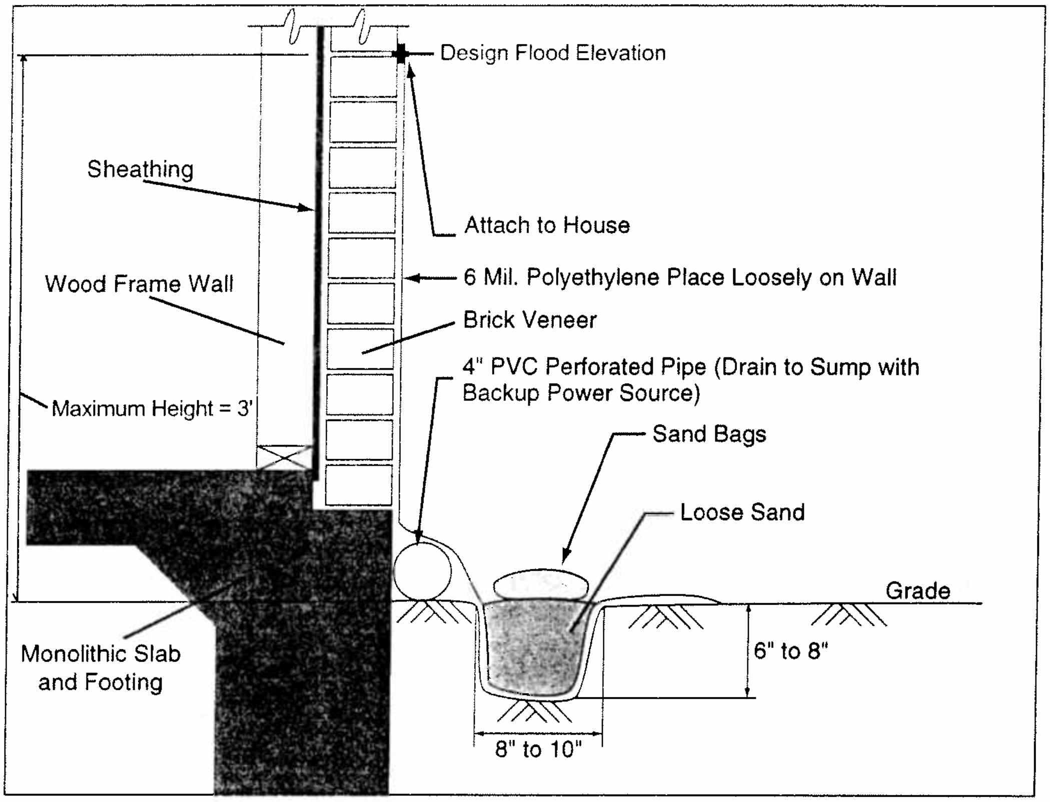


Figure VI-D3: A wrapped house sealing system can be used to protect against low level flooding.

Shields are watertight structural systems that bridge the openings in a structure's exterior walls. They work in tandem with the sealants to resist water penetration. Steel, aluminum, and plywood are some of the materials that can be used to fabricate shields. These features are temporary in most cases, but may be permanent when in the form of a hinged plate or a mini-floodwall at a subgrade opening. Shields transfer flood-induced forces into the adjacent structure components and, like sealants, can overstress the structural capabilities of the building.

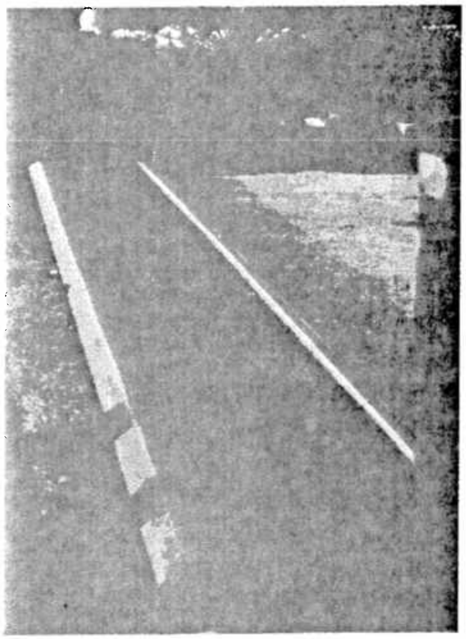


Figure VI-D4: A shield hinged at its bottom could prevent low-level flooding from entering a garage or driveway

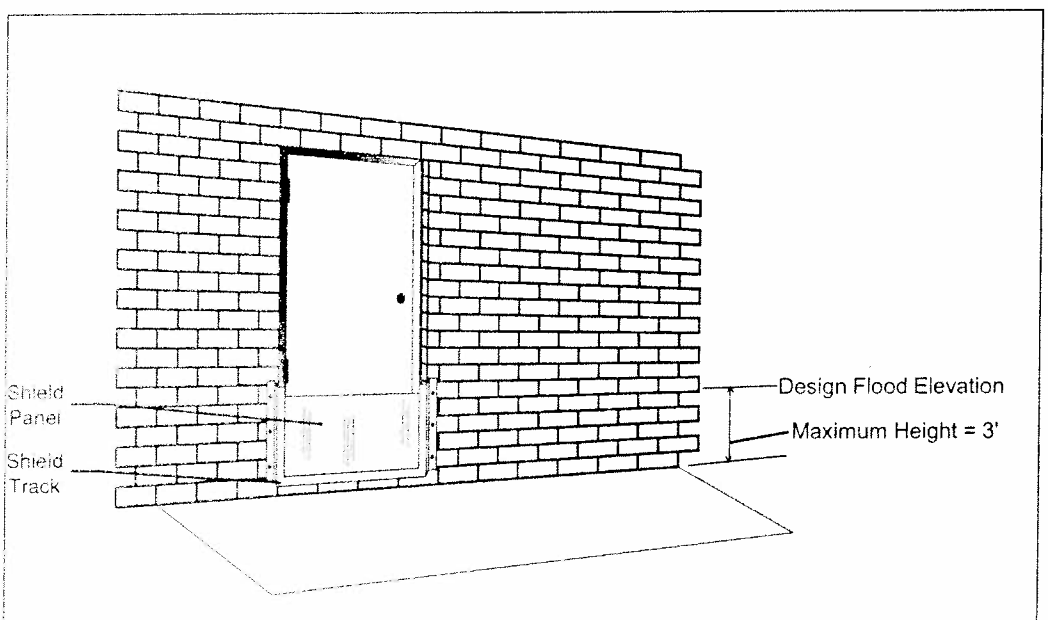


Figure VI-D5. A door opening may be closed using a variety of materials for shields

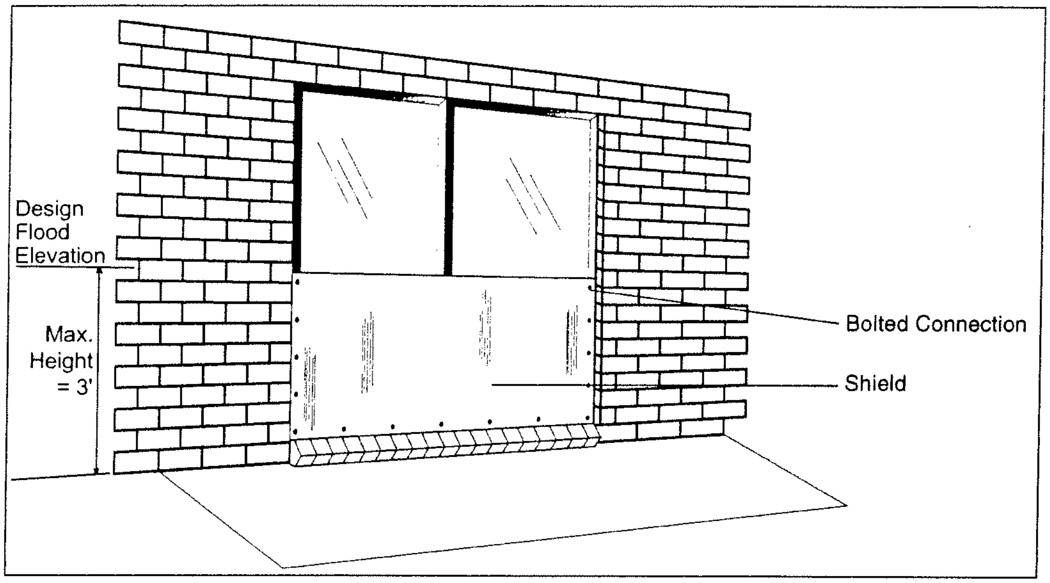


Figure VI-D6: A shield can help prevent low-level flooding from entering through a doorway

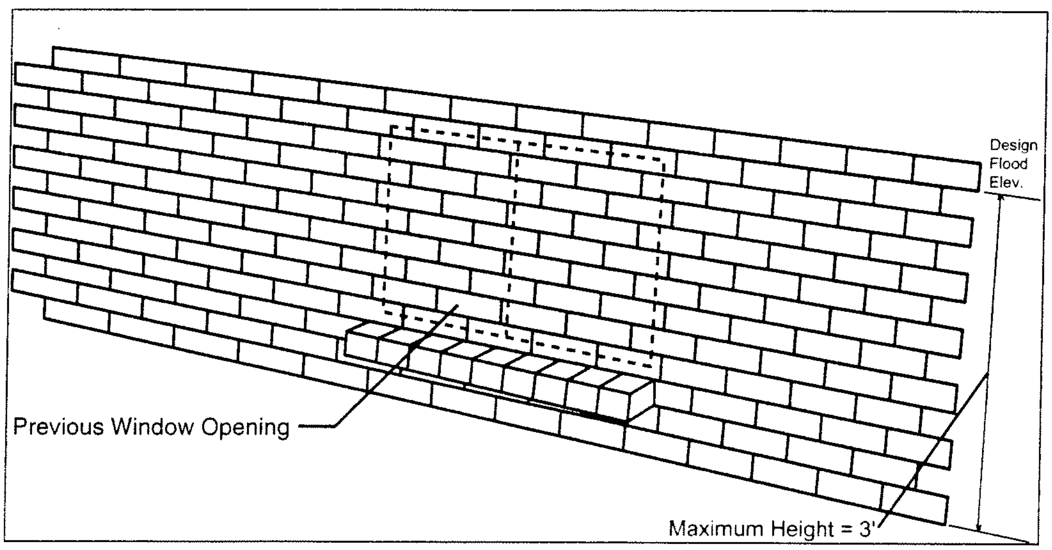


Figure VI-D7: Where a window is exposed to a flood, bricking up the opening could eliminate the hazard

The use of sealants and shields requires that the house have a well-developed interior drain system to collect the inevitable leaks and seepage that will develop. This means establishing drains around footings and slabs to direct seepage to a central collection point where it can be removed by a sump pump.

Additionally, a building employing sealants and shields will usually need backflow devices and other measures designed to eliminate flooding through utility system components. Additional information on this topic is presented later in this section.

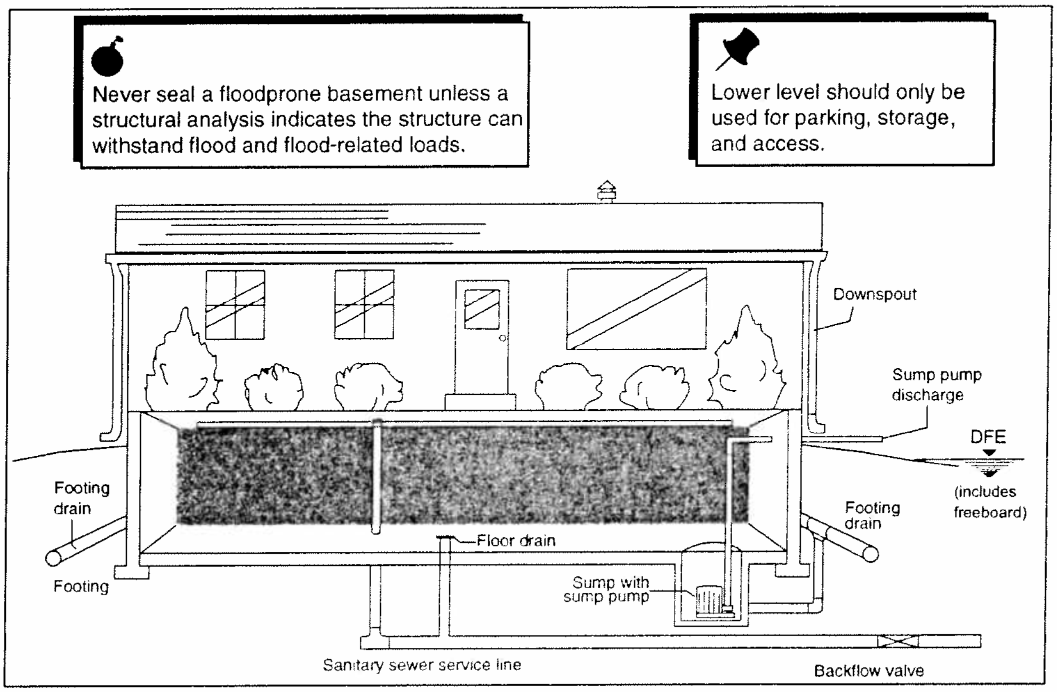


Figure VI-D8: Dry floodproofed homes should have an effective drainage system around footings and slabs to reduce water pressure on foundation walls and basements.

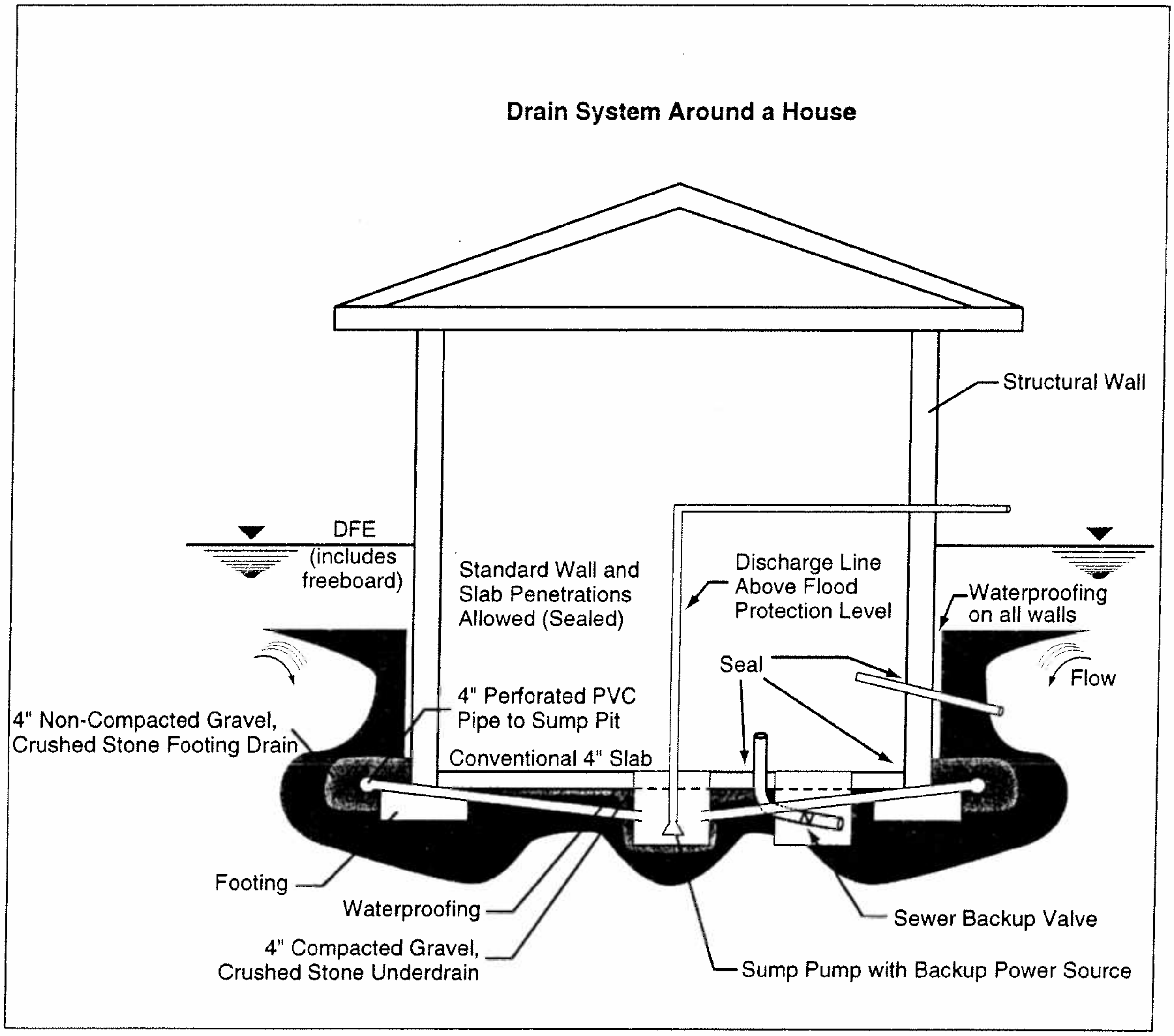


Figure VI-D9: Drain System Around a Slab-on-Grade House

FIELD INVESTIGATION

In addition to, or during consideration of, the field investigation information compiled on the existing building/ building systems data sheet (Figures VI-3 and VI-4), the designer should concentrate on collecting or verifying the following items:

- condition of existing framing, foundation, and footing;
- determination of existing materials used in the house to calculate dead weight;
- determination of type of soil, lateral earth pressures, permeability, and seepage potential;
- building's lateral stability system and adequacy of structural load transfer connections;
- foundation wall, footing, and slab information (thicknesses, reinforcement, condition spans, etc.);
- number, size, and location of openings below the DFE;
- expected flood warning time;
- evidence of previous, and potential for continued, settlement, which could cause cracking after sealant is applied;
- estimates of leakage through the exterior walls and floor;
- manufacturer's data to determine applicability of sealant materials in terms of above- and below-grade applications, and duration of water resistance;
- potential anchorage to secure wrapped systems;

- preliminary selection of shield material to be used based upon the length and height of the openings and duration of flooding; and

- preliminary selection of type of shield anchorage (hinged, slotted track, bolted, etc.), to be utilized by considering accessibility, ease of installation, and amount of time available for installation.

Using this information, a designer should be able to determine if a system of sealants and shields is an option. Of course, further calculations or conditions may dictate otherwise, or that modifications should be made to accommodate the system. The designer can take the information gathered in the field and begin to develop type, size, and location alternatives.

Sealant alternatives include:

- cement- and asphalt-based coatings, epoxies and polyurethane-based caulks/sealants;

- membrane wraps such as polyurethane sheeting; and

- brick veneers over a waterproof coating on the existing foundation.

For additional information concerning the performance of various sealant systems, refer to the U.S. Army Corps of Engineers research study entitled *Flood Proofing Tests*, August 1988, and product evaluation reports prepared by model code groups or NES.

Shield alternatives include:

- a permanent low wall to protect doors and window wells against low-level flooding;

- bricking in a nonessential opening with an impermeable membrane;

- drop-in, bolted, and hinged shields that cover an opening in the existing structure.

Shield alternatives that require human intervention should be considered only if the flooding situation provides sufficient warning time to properly install the shields. The need for both sufficient warning time and "human intervention" is critical, since shield systems usually require personnel to install them and make certain they are properly connected.

DESIGN

CONFIRM ABILITY OF STRUCTURE TO ACCOMMODATE DRY FLOODPROOFING MEASURES

A critical step in the development of initial type, size, and location of the sealant and shield systems is to determine the ability of the existing framing and foundation to resist the expected flood- and non-flood-related forces. This process is illustrated in Figure VI - D10: Existing Building Structural Evaluations.

Step 1: Calculate flood and flood-related forces.

The calculation of flood and flood-related forces (hydrostatic, hydrodynamic, debris impact, soil, and buoyancy forces) as well as determination of seepage and interior drainage rates) was presented in Chapter IV. The designer should account for any non-flood-related forces (i.e., wind, seismic, etc.) by incorporating those forces into Steps 2-6. The determination of non-flood related forces was presented in Chapter IV.

Existing Building Structural Evaluations

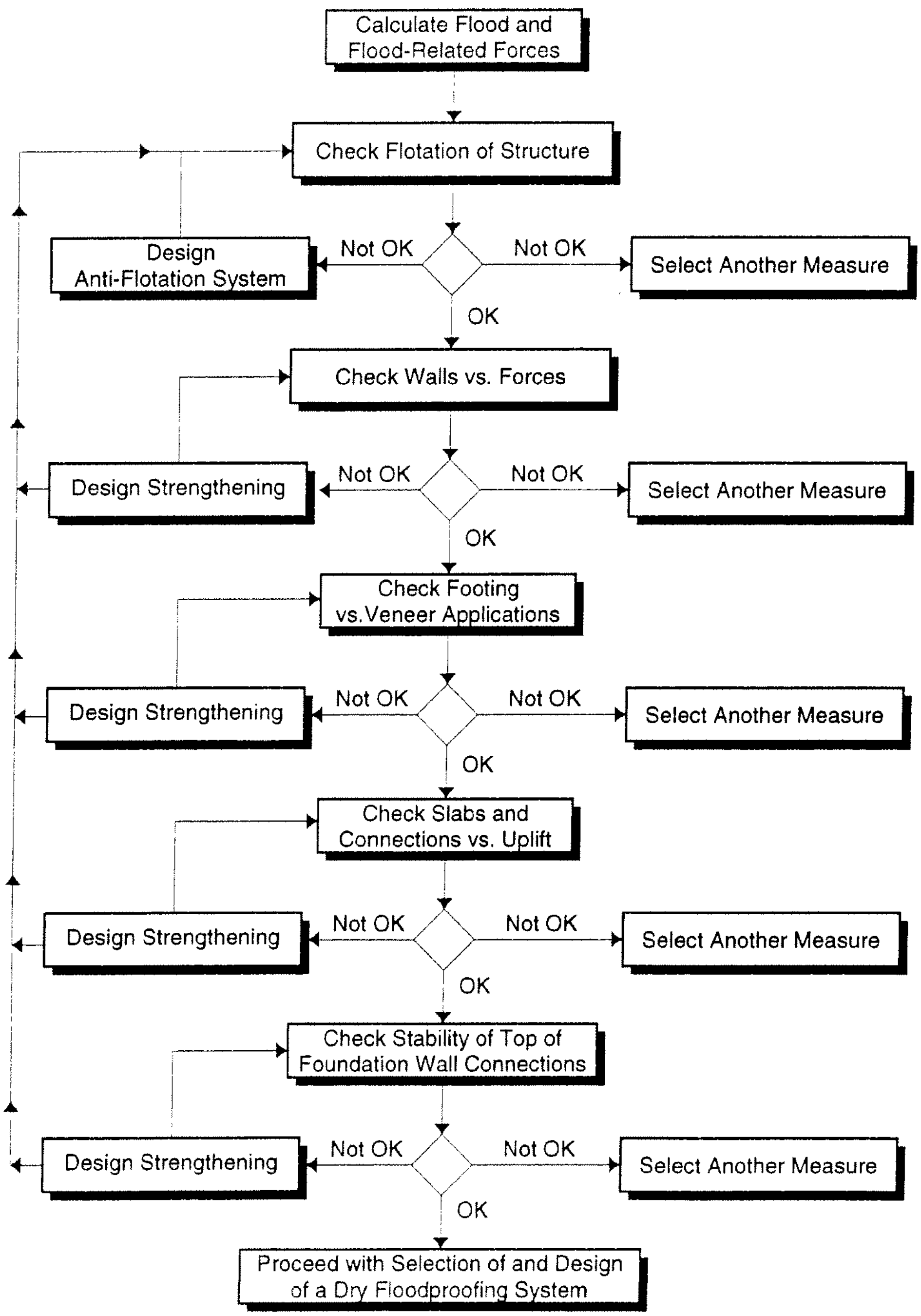


Figure VI-D10: Existing Building Structural Evaluations

Step 2: Check flotation of the wood-frame superstructure.

Residential structures that are determined to be watertight should be checked to ensure that the entire sub- and superstructure will not float. However, it is reasonable to assume that most residential construction will fail prior to flotation of the structure. This failure will most likely occur through the slab-on-grade breaking (heaving/cracking), a window or door failing inward, or extensive leakage through wall penetrations. Should the designer wish to check the failure assumption, guidance is provided in Step 5. If floodwaters come into contact with a wood floor diaphragm (elevated floor or crawlspace home) the floor system/building superstructure should be checked for flotation.

Check the sum of the vertical hydrostatic (buoyancy) forces acting upward against the gravity forces (deadload) acting downward on the structure. The gravity forces acting downward should be greater than the buoyancy forces acting upward. If this is not the case, the designer should consider choosing another floodproofing method or designing an anti-flotation system. The homeowner should make this decision based upon technical and cost information supplied by the designer.

> The typical failure mode for a shield installation is the "kick-in" of the bottom connection where hydrostatic forces are the greatest.

Step 3: Check ability of walls to withstand expected forces.

Frames and connections for closures transfer the retained forces into the adjacent walls. Typically a vertical strip on each side of the opening must transfer the load up to a floor diaphragm and down to the floor or foundation. This "design strip," shown in Figure VI-D11, must be capable of sustaining loads imposed on itself and from the openings. The designer should consider all forces acting on the design strip, as well as the following additional considerations:

a. Check design strip based on simple span, propped cantilever, cantilever, and other end conditions. Consider the moment forces into the foundation.

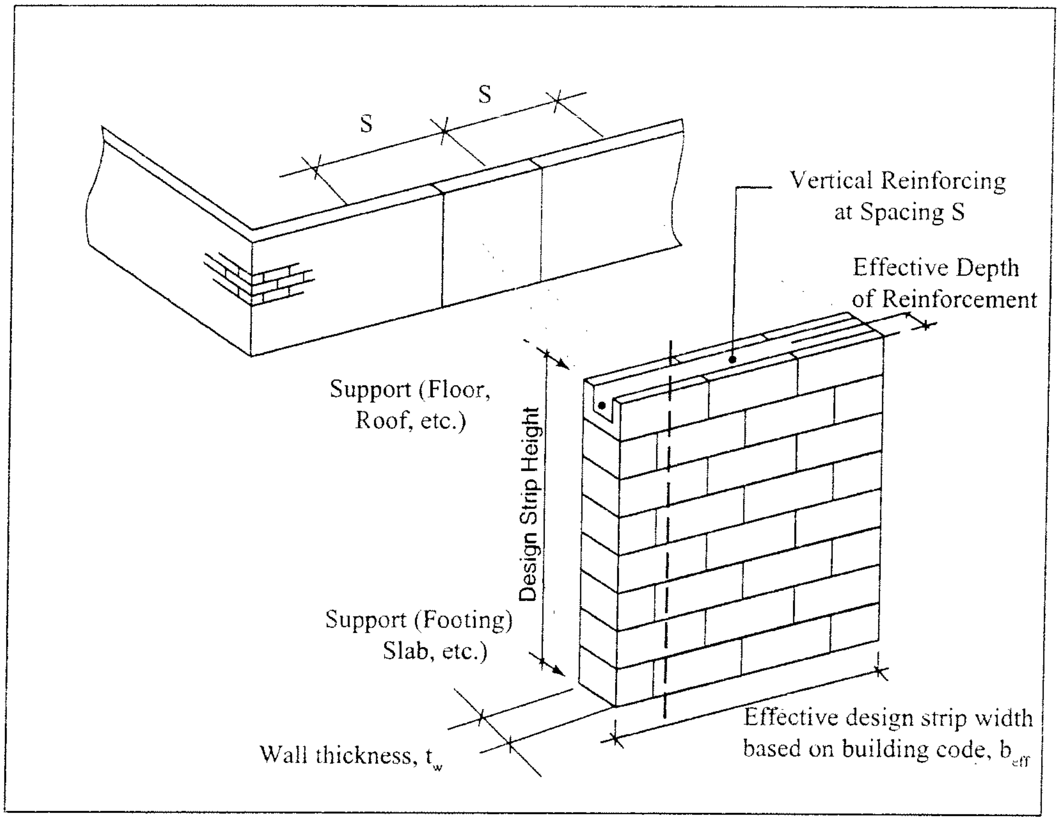


Figure VI-D11: Typical Design Strip for Reinforced Masonry

b. Check design strip for bending and shear based on concrete, wood, masonry, or other wall construction.

Refer to ACI 530 for design of reinforced masonry. Typically, b_{eff} equals the minimum of:
1. center-to-center spacing, S (inches)
2. six times t_w (inches)
3. 72 inches

c. Consider the path of forces from shield into the design strip through the various connection alternatives including hinges, drop-in slots, frames, and others.

d. The designer may want to refer to the American Institute of Steel Construction (AISC) Steel Manual, American Concrete Institute (ACI) documents for concrete and masonry construction, National Design Specifications of Wood Construction (NDS)/ American Institute of Timber Construction (AITC) documents for timber construction, APA documents for plywood, and other applicable codes and standards for more information on the ability of these materials to withstand expected flood and flood-related forces.

Step 4: Check ability of footing to support veneer applications.

The application of veneer to the exterior of an existing wall must be supported at the footing level. The designer should consider all forces acting on the existing footing, as well as the following additional considerations:

a. Supporting the masonry veneer on an existing footing can add an eccentric load onto the footing and can create soil pressure problems. The designer should analyze the footing with the additional load considering all load combinations including the flooded condition.

b. The actual pressure on the footing should not overload the bearing capacity of the existing soils. Consult a geotechnical engineer, if necessary.

c. The designer may want to refer to the ACI Manual for Concrete Construction, various soils manuals/textbooks for detailed footing design, and applicable codes and standards.

Step 5: Check slab and connections against uplift forces.

As floodwaters rise around a structure, a vertical hydrostatic (buoyancy) force builds up beneath floor slabs. For floating slabs, this buoyancy force is resisted by the structure dead load and saturated soil above the footing; for keyed-in slabs, this buoyancy force is resisted by the structure dead load, and the flexural strength of the slab. These slabs must be capable of spanning from support to support with the load being applied beneath the slab (see Figure VI-D12). The designer should consider all forces acting on the existing slab and connections, as well as the following additional considerations:

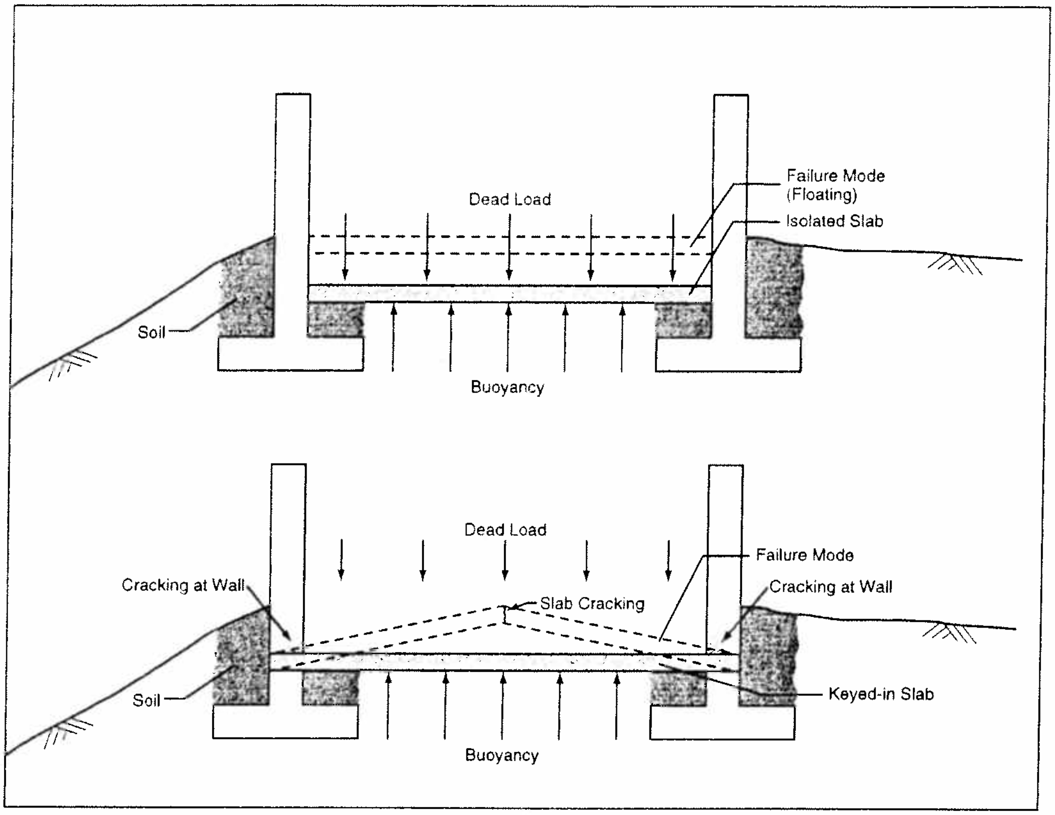


Figure VI-D12: Typical Slab Uplift Failure

a. Verify the existing slab conditions including thickness, reinforcement, joint locations, existence of continuous slab beneath interior walls, existence of ductwork in slab, and edge conditions. If reinforcement and thickness are not easily determinable, make an assumption (conservative) based on consultation with the local building official or contractors.

b. Confirm the slab design by checking reinforcement for bending and edge connection for shear load.

Step 6: Check stability of top of foundation wall connections.

Foundation walls may retain water in some situations. These walls must transfer the additional hydrostatic load down to the footing or slab and up to the floor diaphragm. The designer should consider all forces acting on the top of the existing foundation wall connections, as well as the following additional considerations:

a. Verify existing wall conditions including construction material, reinforcement, design conditions (simple span, propped cantilever, cantilever, and other end conditions), and connections.

b. Connections between the wall and floor are of major importance in consideration of the wall stability. The designer should check the following:

   1. masonry/concrete for shear from bolt;

   2. anchor bolt for shear;

   3. sill for bending from bolt loads; and

   4. transfer of load from sill into joists into plywood diaphragm.

   5. loads have a pathway out of the structure. Additional bracing and/or connectors may be required to support a load pathway out of the structure. Analyze framing and be cognizant that all sides may be loaded.

c. The designer may want to refer to the ACI Manual for Concrete Construction, NDS/AITC for timber construction, AISC for anchor bolts, product literature for wood connectors, and applicable codes and standards.

Step 7: Design foundation supplementation system, as required.

If the checks in Steps 2-6 determined that any structural members were unable to withstand expected flood and flood-related loads (wind, seismic, and other forces can be evaluated as presented in Chapter IV), the designer can either select another retrofitting measure or design foundation supplementation measures. These foundation supplementation measures could range from increasing the size of the footing to adding shoring to the foundation walls, or simply modifying the type, size, number and location of connections. The homeowner should make this decision based upon technical and cost information supplied by the designer.

Footing Reinforcing: in some cases, the footings for walls must be modified to accommodate expected increased loadings. The following considerations should be taken into account during the design of this modification:

a. The wall footing must be checked for the increased soil pressure and sliding. Moment and vertical loads from the wall above should be added.

b. The footing may need more width and reinforcement to distribute these forces to the soil.

c. For some extreme cases (poor soils, high flood depths, flood-related wind and/or earthquake loads), a geotechnical engineer may be required to accurately determine specific soil loads and response.

d. The designer should consider multiple loading situations taking into account building dead and live loads that are transferred into the footing, utilizing whatever load combinations are necessary to design the footing safely and meet local building code requirements. Consider the framing of the structure and how the entire house load is transferred into the foundation.

e. The designer may want to refer to the ACI Manual for Footing Design, recent texts for wall and footing design, and applicable codes and standards.

Step 8: Repeat process in Steps 1-7 incorporating exterior wall foundation supplementation system.

Once the designer has determined that the existing framing and foundation are suitable for the application of sealants or shields, or that reinforcement can be added to make the existing framing and foundation suitable for the application of sealants or closures, the selection/design of a specific system can begin.

Dry floodproofing measures are only as good as their weakest link (i.e., the connection to the existing structure). The designer should ensure that all appropriate details for making the connection watertight as well as allowing for the transfer of loads are developed.

SELECTION AND DESIGN OF SEALANT SYSTEMS

Actual test results of sealant product performance, if available, should be used to supplement the manufacturer's literature. Sources of test results include model building code product evaluation reports, a USACE publication entitled *Flood Proofing Tests,* August 1988, and local building code officials.

Once the determination is made that a foundation system can withstand the expected flood and flood-related forces, the selection of a sealant system is relatively straightforward and centers on the ability of the manufacturer's product to be compatible with the length and depth of flooding expected and the type of construction materials used in the structure.

COATINGS

The selection of a coating follows the flow chart presented in Figure VI-D13, Selection of Sealants/Coatings. If additional structural reinforcing is required, it should be performed in accordance with the guidance presented in the preceding section entitled "Confirm Ability of Structure to Accommodate Dry Floodproofing Measures."

Selection of Sealants/Coatings

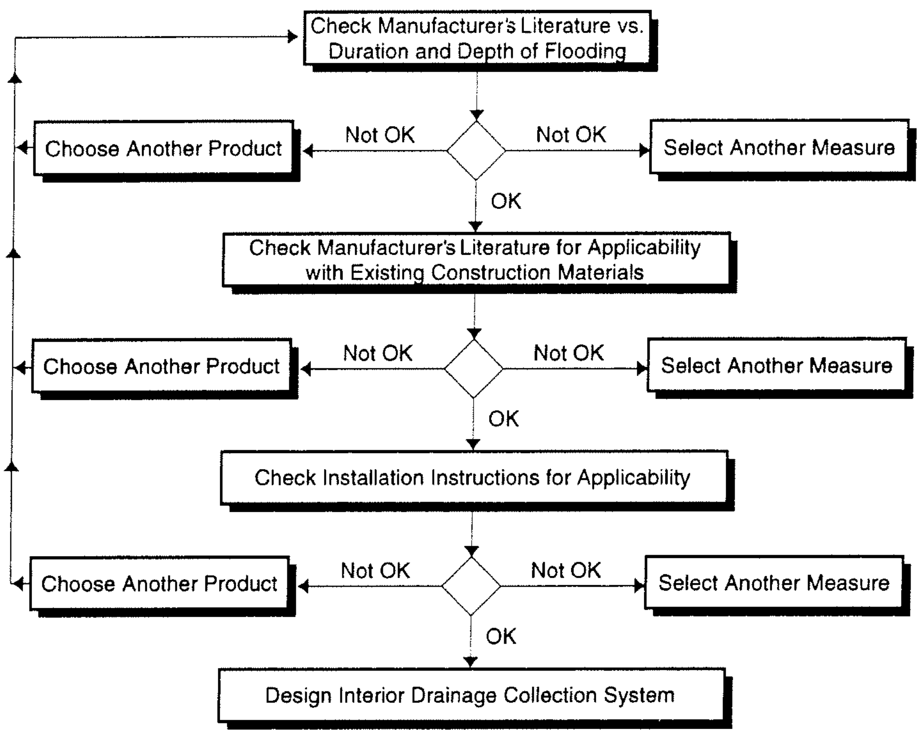


Figure VI-D13: Selection of Sealants/Coatings

WRAPPED SYSTEMS

The selection and design of a wrapped system follows Figure VI-D14, Selection and Design of a Wrapped Sealant System. If additional structural reinforcing is required, it should be performed in accordance with the guidance presented in the preceding section.

Selection and Design of Wrapped Sealant System

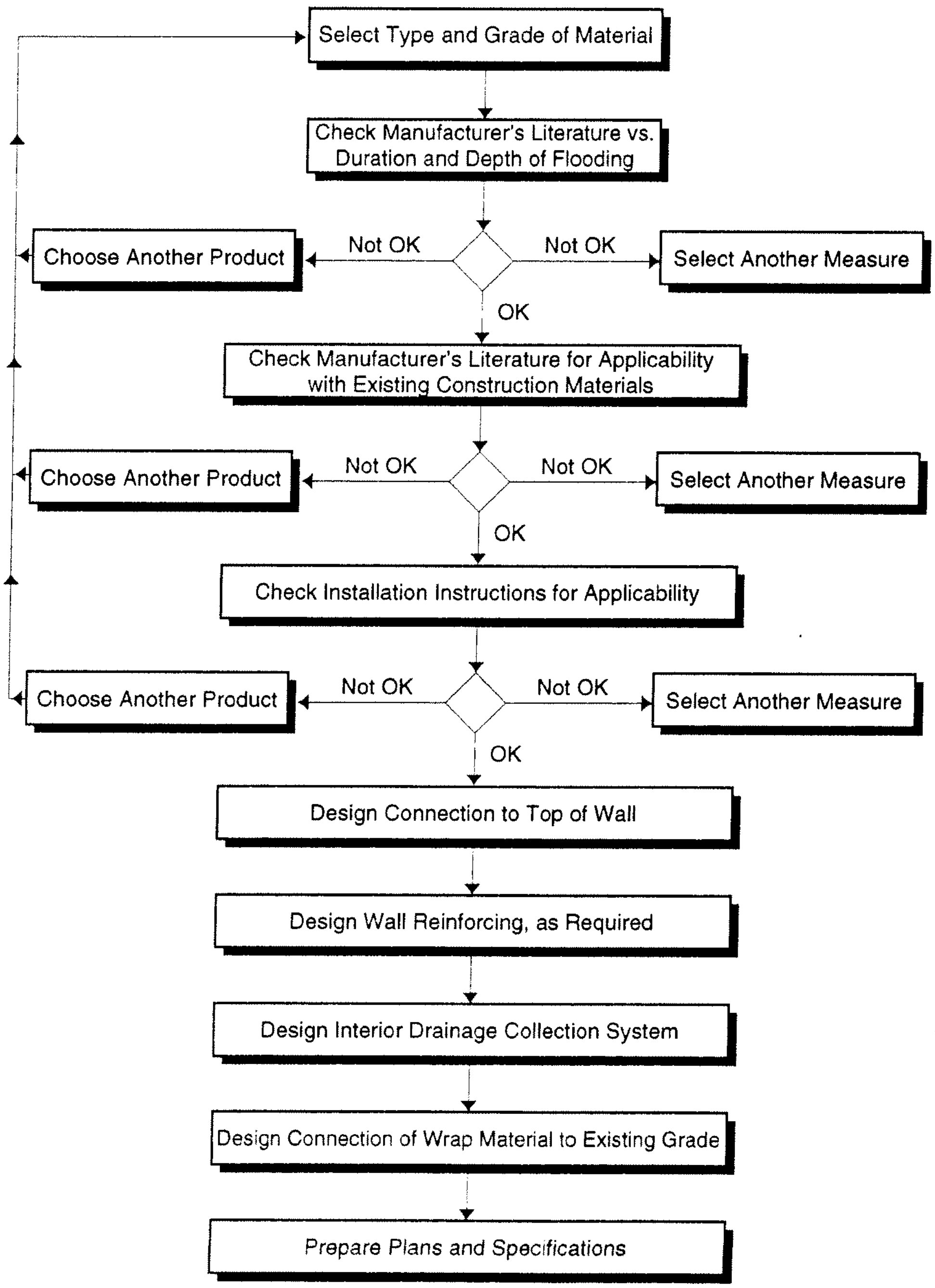


Figure VI-D14: Selection and Design of Wrapped Sealant Systems

Step 1: Select type and grade of material.

Step 2: Check manufacturer's literature against duration and depth of flooding.

> For additional information concerning the performance of various sealant systems, refer to the U.S. Army Corps of Engineers research study entitled *Flood Proofing Tests,* August 1988, and product evaluation reports prepared by model code groups.

If flooding application is satisfactory, proceed with design; if not satisfactory, select another product or another method.

Step 3: Check manufacturer's literature for applicability to building materials. Rely on actual test results, if available.

If building materials application is satisfactory, proceed with design; if not satisfactory, select another product or another method. Manufacturer performance claims can be misleading. The designer should utilize actual test results rather than rely entirely on a manufacturer's performance claim.

Step 4: Check installation instructions for applicability.

If installation procedure is satisfactory, proceed with design; if not satisfactory, select another product or another method.

Step 5: Design connection to top of wall.

Adding a wrap system onto an existing structure will require secure connections at both the top and bottom of the wrap. It is difficult to determine the actual loads imposed vertically on the wrap as this can vary based upon the quality of the installation. Voids left from poor construction may force the wrap to carry the weight of the water and should be avoided. See Figure VI-D15. The following considerations should be followed during selection and design of a top-of-wall connection system:

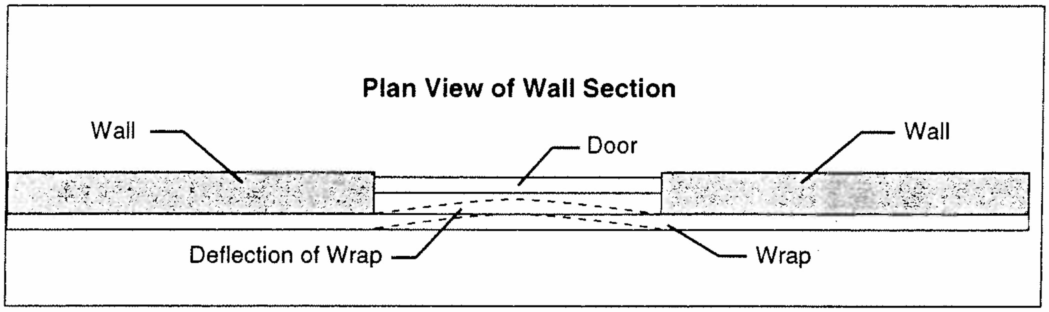


Figure VI-D15: Plan View of Wall Section

a. Use a clamping system that uniformly supports the wrap. A small spacing on the connections and a member with some rigidity on the outside of the wrap can provide this needed support.

b. The existing wall construction is an important consideration for these connections and can vary widely. Part of the connection may need to be a permanent part of the wall.

See Figure VI-D3 for details on wrapped system configuration.

c. The designer may want to refer to the product literature for wrap material, NDS/AITC for connections into wood, and applicable codes and standards.

Step 6: Design foundation reinforcing.

Refer to Chapter VI - Dry Floodproofing Section entitled "Confirm Ability of Structure to Accommodate Dry Floodproofing Measures."

Step 7: Design drainage collection system.

Refer to Chapter VI - Dry Floodproofing Section entitled "Drainage Collection Systems."

Wrap systems may be affected by freeze-thaw cycles. Careful installation in accordance with manufacturer instructions and evaluation of performance in frozen climates is advisable.

Step 8: Specify connection of wrapping to existing structure and existing grade.

Anchoring a wrap into the grade at the base of a wall will be the most important link in the wrap system. The following considerations should be followed during selection and design of a wrap to existing grade connection system:

a. A drain line between the wrap and the house is required to remove any water that leaks through the wrap or that seeps through the soil beneath the anchor.

b. As with the top-of-wall connection, wrap forces are difficult to determine. It is best to follow details that have worked in the past and are compatible to the specific structure.

c. It is recommended that the end of the wrap be buried at least below the layer of topsoil. Additional ballast may be needed (sandbags, stone, etc.,) to prevent wrap movement in a saturated and/or frozen soil condition.

d. The designer may want to refer to the product literature for wrap material and applicable codes and standards.

BRICK VENEER SYSTEMS

See Figure VI-D2 for details on brick veneer system configuration.

The selection and design of a brick veneer sealant system follows Figure VI-D16, Selection/Design of a Brick Veneer Sealant System, and has many components that are similar to the design of other sealant systems. A typical brick veneer sealant system is shown in Figure VI-D2. If additional structural reinforcing is required, it should be performed in accordance with the guidance presented in the preceding section.

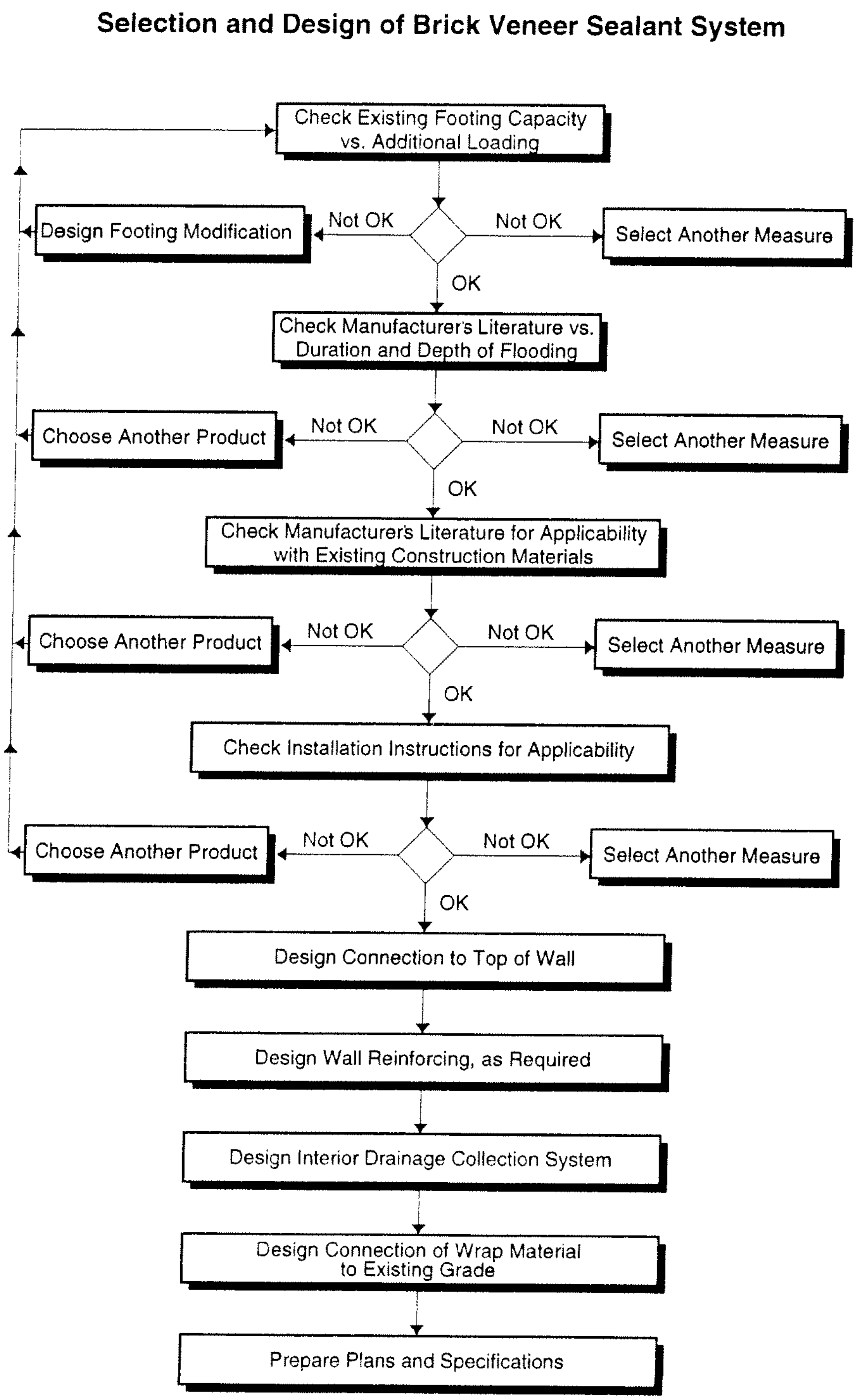


Figure VI-D16: Selection/Design of a Brick Veneer Sealant System

Step 1: Check the capacity of the existing footing.

Calculate the weight of the structure and proposed brick veneer system on a square foot basis and compare it to the allowable bearing capacity for the specific site soils. If the bearing pressure from gravity loads is less than the allowable bearing pressure, the existing footing can withstand the increased loading. If the bearing pressure from gravity loads is greater than the allowable soil bearing pressure, the existing footing is unable to withstand the increased loading and the footing must be modified, or the designer should select another floodproofing measure.

Step 1A: Supplement the footing, as required.

If it is found that the existing footing cannot support the loads expected from a veneer system or that the configuration of the footing is unacceptable, the footing can be widened to accommodate this load. This can be a costly and detailed modification. The homeowner should be informed of the complexity and cost of such a measure. The following considerations should be followed during design of a footing supplement:

a. If additional width is added to the footing, the designer must analyze how the footing will work as a unit. Reinforcing must be attached to both the old and new footing. This will probably involve drilling and epoxy grouting reinforcement into the existing footing. The quality and condition of the existing concrete and reinforcement should be considered in the design.

b. Exercise care when making excavations beside existing footings. Take care not to undermine the footings, which could create major structural problems or failure.

c. Design the footing for the eccentric load from the brick weight. Add any flood-related loads and consider all possible load combinations.

d. For extreme soil conditions, consult a geotechnical engineer to determine soil type and potential response.

e. The designer may want to refer to the ACI Manual for Concrete Design, a soils manual/textbook for detailed footing design, and to applicable codes and standards.

Step 1B: Design foundation reinforcing (as required).

Concrete footings can come in a wide variety of configurations. Design of footings, especially those involved with retaining of materials, can become quite complex. There are many books that deal with the design of special foundations, and once the stresses are determined the ACI can provide guidelines for concrete reinforcement design.

Steps 2-9 are similar to the design of wrapped sealant systems. Refer to the previous section for details on these steps.

SELECTION AND DESIGN OF SHIELD SYSTEMS

Once the determination is made that a foundation system can withstand the expected flood and flood-related forces, the selection of a shield system is relatively straightforward and centers on the ability of the selected material to structurally secure the opening, be compatible with the existing construction materials, and be responsive to the duration and depth of flooding expected.

> Industry has developed manufactured closure systems that may be applicable to specific situations. For additional information on the companies that manufacture these products, contact your local floodplain management or engineering office.

PLATE SHIELDS

The selection and design of a plate shield follows Figure VI-D17, Selection/Design of Plate Shields. If additional existing structural reinforcing is required, it should be performed in accordance with the guidance presented in the preceding section.

Step 1: Select the plate shield material.

Plate shield material selection may be driven by the size of the opening or the duration of flooding. For example, plywood shields would not hold up during long-term flooding.

a. Consider flood duration and select steel or aluminum materials for long duration flooding and consider marine grade plywood materials for short duration flooding.

b. Consider opening size and select steel and aluminum materials with stiffeners for larger openings and shored plywood with appropriate bracing for small openings.

c. Installation of all shields should be quick and easy. Lighter materials such as plywood and aluminum are most suitable for homeowner installation.

Selection/Design of Plate Shields

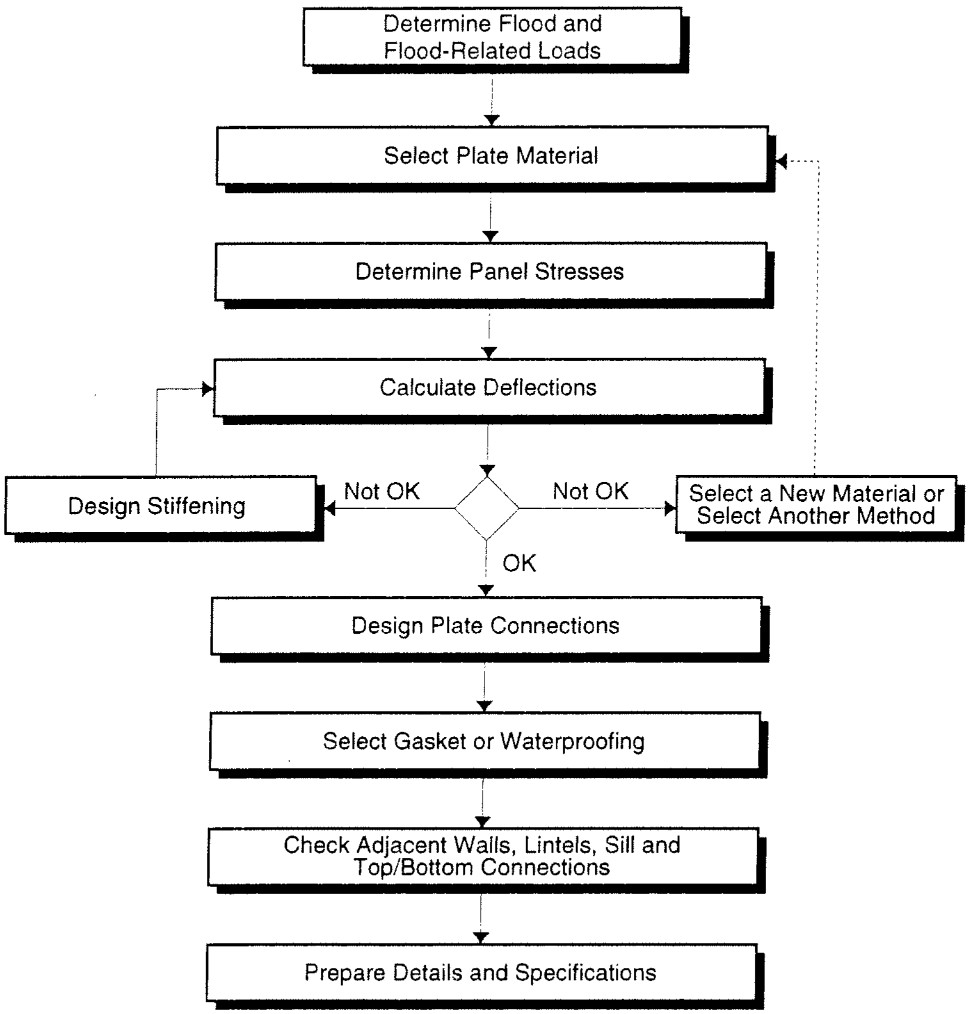


Figure VI-D17: Selection/Design of Plate Shields

Step 2: Determine panel stresses.

> The use of plywood shields in long-term exposure situations may induce possible swelling and deterioration of the laminating glue.

The designer should check the shield panel either as a plate or a horizontal/vertical span across the opening.

a. Using end conditions and attachments to determine how the panel will work, calculate stresses based on bending of the plate. In larger plate applications, also compute the end shear.

b. Compare these stresses to the allowable stresses from the appropriate source.

c. Some shields may have a free end at the top or other unusual configuration. These will need to be addressed on a case-by-case basis.

d. Adjust the plate thickness to select the most economical section. If the plate does not work for larger thicknesses, add stiffeners.

e. The designer may want to refer to the AISC manual for steel plate design, an aluminum design manual, APA for plywood design, and applicable codes and standards.

Step 3: Check deflections.

A plate shield that is acceptable for stresses may not be acceptable for deflection.

a. Calculate deflections for the panel and evaluate on the basis of connections and sealants.

b. If the deflection is unacceptable, add stiffeners.

c. Deflection may be controlled by alternative plate materials.

d. The designer may want to refer to the AISC manual for steel plate design, an aluminum design manual, APA for plywood design, and applicable codes and standards.

Step 3B: Stiffen as required.

Plate overstress or deflection may be solved through the use of stiffeners.

a. Select the section to be used as a stiffener. Angles may be used for steel or aluminum and wood stock for plywood.

b. Calculate the stresses and deflection based on the composite section of stiffener and plate.

c. Calculate the horizontal shear between the two sections and design the connections to carry this load.

d. Keep plate connections and frame in mind when detailing stiffeners.

e. The designer may want to refer to the AISC manual for steel plate design, an aluminum design manual, APA for plywood design, Mechanics of Materials tests, and applicable codes and standards.

Step 4: Design the connections.

Plate connections must be easy to install and able to handle the loads from the plate into the frame and surrounding wall.

a. Determine the type of connection (hinged, free top, bolted, latching dogs, or other).

b. Consider ease of installation and aesthetics.

c. Connection must operate in conjunction with gasket or sealant to prevent leakage.

d. Connection must be capable of resisting some forces in the direction opposite of surges.

e. The designer may want to refer to the AISC manual for bolted connections, ACI manual for connections into concrete and masonry, and applicable codes and standards.

Step 5: Select the gasket or waterproofing.

Gaskets or waterproofing materials, which form the interface between shields and the existing structure, are vital elements of the dry floodproofing system. They should be flexible, durable, and applicable to the specific situation.

a. Determine the type of gasket or waterproofing required.

b. Consider ease of installation and ability to work with plate/connections as a single unit.

c. Gasket/waterproofing must be able to withstand expected forces.

d. Gasket/waterproofing must be able to function during climatic extremes.

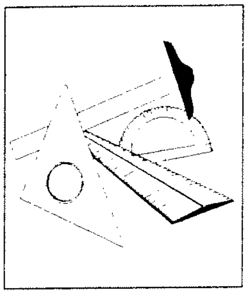
e. The designer should refer to manufacturer's literature and check against duration/depth of flooding and applicability to selected building materials.

Step 6: Check adjacent walls, lintels, sills, and top/bottom connections.

Structural components adjacent to the shield panel, such as adjacent walls, lintels, sills, and top/bottom connections, should be checked against maximum loading conditions. Different methods of attachment may load the adjacent wall differently.

Walls adjacent to the shield should be anchored into the footing to resist base shear. Lintels/sills should be checked for biaxial bending resulting from lateral loading. Top connections should be evaluated for shear resistance and ability to transfer loads to the joists.

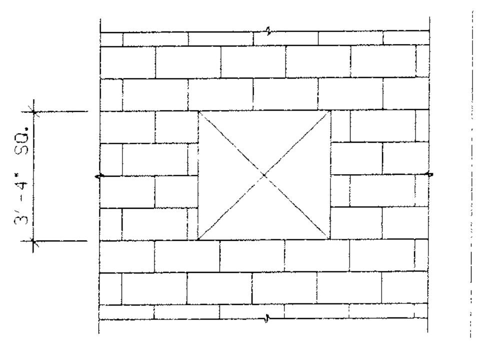
The following design example illustrates the process of selection and design of a window opening shield.



Sample Calculation for Shield Design

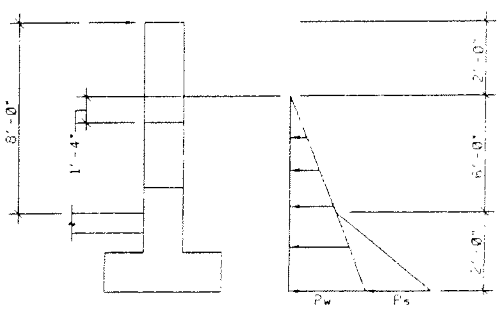
GIVEN:

Shield in 12-inch Concrete Masonry Unit wall subject to hydrostatic (freestanding water) flood loading only.



This example will check only the surrounding wall, design lintel, shield frame, and shield panel. See previous guidelines under the elevation sample calculation for remainder of house. Previous investigation has determined that flotation will not occur for the water depths shown.

Step 1: For an opening of this size, it is unlikely that plywood would work without stiffening (due to its potential for deflection). Therefore, try using a flat plate of steel.



Pressure due to hydrostatic forces

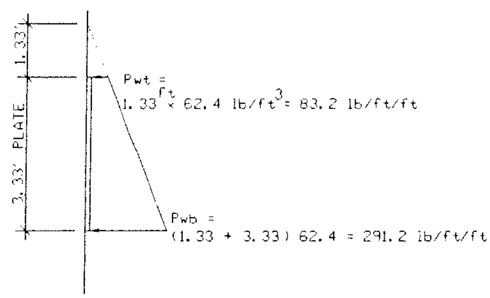
Sample Calculation for Shield Design

Calculations:

Determine water pressure across panel

Unit weight of water = 62.4 lbs/ft^3

Options:



Note: Design guidance for this example was taken from *Roark's Formulas for Stress and Strain*, 6th Edition, Warren C. Young. Use of this reference is not an endorsement. The book is a standard reference (among others) for structural engineers.

Panel can be designed as a plate to distribute the load about the perimeter of the opening.

Panel can be designed to span horizontally or vertically (may require stiffeners).

Whichever option is chosen, the frame and anchors must be capable of transferring the load into the wall and/or lintel. The wall must be capable of transferring the load into the slab or diaphragms.

Sample Calculation for Shield Design

Step 2: For this example, design the plate assuming supported on all sides with A36 steel.

Reference: *Roark's Formulas for Stress & Strain*, 6th edition, Warren C. Young

Assume simple (hinged) connections for panel.

For the calculations it is convenient to divide the loading into two load cases:
the uniform load of 83.2 lbs/ft^2 and
the uniformly increasing load of 291.2 - 83.2 = 208 lbs/ft^2.

Per *Roark*, <u>Case 1a</u>, (page 458), **<u>Uniform Load</u>** in the following calculations,

α	=	tabulated coefficient
β	=	tabulated coefficient
γ	=	tabulated coefficient
a	=	plate dimension horizontal
b	=	plate dimension vertical
q	=	maximum pressure on plate
t	=	plate thickness
E	=	modulus of elasticity for plate
a	=	b = 40" thus,

a/b = 1.0 $\therefore$ β = 0.2874, α = 0.0444, γ = 0.420. The maximum stress at the center of the plate is given by,

for this example,

$$\sigma_u = \frac{\beta q b^2}{t^2}$$

$$\sigma_u = \{0.2874\,(83.2\ \text{lb/ft}^2/144)(40\ \text{in})^2\}/(3/8\ \text{in})^2 = 1889\ \text{lbs/ft}^2$$

Sample Calculation for Shield Design

the **maximum deflection** is given by,

$$y_u = \frac{(-\alpha)qb^4}{Et^3}$$

for this example,

$y_u = \{-0.0444(83.2/144)(40)^4\}/\{29x10^6(3/8\ in)^3\} =$ -0.04 in.

the **maximum reaction (at center of sides)** is given by,

$R_u = \gamma bq$

Per *Roark*, <u>Case 1d</u>, (page 459), <u>Uniform Increasing Load</u>,

a = b = 40" thus,

a/b = 1.0 $\therefore \beta = 0.16,\ \alpha = 0.022$

for this example,

$\sigma_T =$ $\{0.16\ (208\ lb/ft^2/144)(40\ in)^2\ \}/(3/8\ in)^2$ = 2630 psi

$y_T =$ $\{-0.022(208/144)(40)^4\}/\{29x10^6(3/8\ in)^3$ = -0.05 in.

Assume that these stresses and deflections occur at the same locations and are therefore additive (conservative), therefore,

$\sigma_{max} =$ 1889+2630 = 4519 psi

Reference: AISC 9th Edition Section F2.1

Allowable bending stress in plate,

$F_b = 0.75F_y$

where:

F_b = allowable bending stress

F_y = yield stress of steel

for this example,

F_b = 0.75(36,000) = 27,000 psi >>> 4519 psi O.K.

4 of 11

Sample Calculation for Shield Design

Step 3: maximum deflection,

$$y_{max} = 0.04 + 0.05 = 0.09 \text{ in.}$$

maximum allowable deflection is recommended to be,

$$y_{allow} = L/240$$

where:

L = span of member under consideration

for this example,

$$y_{allow} = 40 \text{ in} / 240 = 0.17 \text{ in.} > 0.09 \text{ in. O.K.}$$

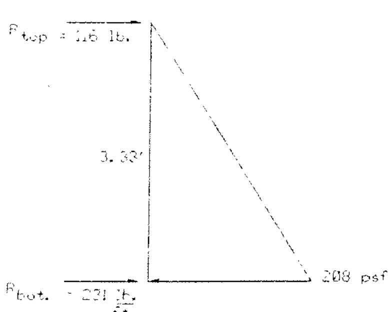
Note: If deflection is a problem, stiffeners can be added to the plate.

Step 4: Check Connection to Wall

The reaction from the uniform load can be determined from the previous equations.

$$Ru = \gamma bq = 0.42(40 \text{ in})(83.2/144) = 9.7 \text{ lb/in}$$

To determine the reactions from the sloped loading, assume the plate spans from the top to bottom.



Sample Calculation for Shield Design

Maximum Conservative Reaction is,
$[(9.7)(12)] + 231 = 347$ lbs

Check Anchor & Masonry

½" ϕ A307 Anchor Bolt

Allowable Shear per AISC 9th Edition, (Connection Section)

$$\frac{(\frac{1}{2})^2 \pi}{4}(10{,}000) = 1963 \text{ lbs} > 346 \text{ lbs O.K}$$

Check horizontal shear in masonry. Locate bolt in middle of 12" CMU.

Reference: ACI 530 Section 5.14.2.2

Area of bolt,
$A_b = 0.2\ in^2$

edge distance,
$l_{be} = 12/2 - ½/2 = 5.75" < 12\ d_b = 6"$

embedment,
$l_b = 6"$ (chosen)

allowable load in shear,

$$B_v = \left|(350)^4 \sqrt{(f'_m \cdot A_b)}\right| 0.12(A_b)(f_y)$$

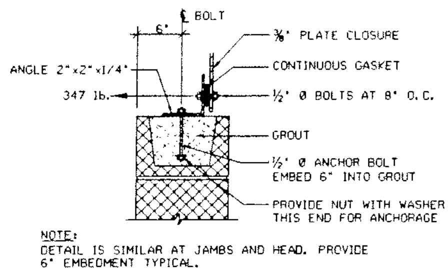
where:

A_b = Area of Anchor bolt
f_m = Compressive Strength of Masonry
f_y = Yield Strength of Anchor Bolt

Sample Calculation for Shield Design

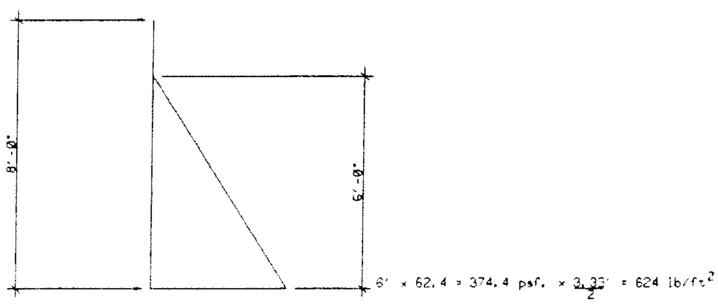
for this anchor bolt pattern,

$$B_v = \min\left(350\left((1500)(0.2)\right)^{1/4}, \left((0.12)(0.2)(20\text{ksi})\right)\right)$$
$$= 1450{,}480$$
$$= 480 \text{ lbs} > 347 \text{ lbs}$$



CLOSURE PANEL CONNECTION DETAIL
N. T. S.

Check Walls Adjacent to Opening for Additional Loads



Check a 1'-0" wide strip of masonry.

Determine the maximum moment and shear (as simple span),

at 5'-0" from top, M_{max} = 1872 lb ft

at bottom, V_{max} = 1404 lbs

Sample Calculation for Shield Design

Note: for Seismic Zone 2, masonry cores on each side of opening are to be reinforced.

Check Masonry

Reference: ACI 530, Working Stress Design

Assume:

d	=	6" (middle)
b	=	12" (1½ cores)
E_s	=	29×10^6 psi
f'_m	=	1500 psi
E_m	=	1.6×10^6 psi Table 5.5.1.3 of ACI 530
n	=	$E_s/E_m = 18$ (modular ratio)

F_b = 1/3(1500 psi) = 500 psi (allowable compressive stress)
F_s = 24,000 psi (allowable tensile stress)

Try using 1-#5 rebar each side of opening, $A_s = 0.31$ in^2, full height,
ρ = steel ratio = $A_s/bd = 0.31/((6)(12)) = 0.0043$

$f_s = M/A_s jd$

where:

M	=	applied moment
A_s	=	area of steel
j	=	ratio of distance between centroid of flexural compressive forces and tensile forces = 1-(K/3)
K	=	$-\rho n + (2\rho n + (\rho n)^2)^{1/2}$
n	=	modular ratio
ρ	=	steel ratio

Sample Calculation for Shield Design

$$K = -0.0043(18)+(2(0.0043)(18)+[(18)(0.0043)]^2)^{1/2}$$
$$= 0.3238$$
$$j = 1 - 0.3238/3 = 0.8921$$
$$f_s = [(1872 \text{ lb ft})(12 \text{ in/ft})]/[(0.31)(0.8921)(6)]$$
$$= 13{,}540 \text{ psi} < 24{,}000 \text{ psi O.K.}$$

for this example,

$$f_b = M/(1/2bjkd)$$

where:

- M = applied moment
- b = width of section
- j = ratio of distance between centroid of flexural compressive forces and tensile forces
  - = $1-(K/3)$
- K = $-\rho n + (2\rho n + (\rho n)^2)^{1/2}$
- d = distance to centroid of tensile stresses from the maxim um compressive stress
- n = modular ratio
- ρ = steel ratio

$$f_b = [(1872 \text{ lb ft})(12 \text{ in/ft})]/$$
$$[\tfrac{1}{2}(12)(0.8921)(0.3238)(6)^2]$$
$$= 360 \text{ psi} < 500 \text{ psi O.K.}$$

Walls adjacent to closure should have 1-#5 (middle) full height with matching dowel into footing, as a minimum.

Sample Calculation for Shield Design

Check Shear in Masonry at Base

$V_{max} = 1404$ lb

Calculate shear stress,

fv = V/(bjd)

where:

V	=	shear at point under construction
b	=	width of section
j	=	ratio of distance between centroid of flexural compressive forces and tensile forces
	=	1-(K/3)
K	=	$-\rho n + (2\rho n + (\rho n)^2)^{1/2}$
d	=	distance to centroid of tensile stresses from the maximum compressive stress
n	=	modular ratio
ρ	=	steel ratio

for this example,

$f_v = 1404 / [(12)(0.8921)(6)] = 21.9$ psi

allowable shear stress, **per ACI 530**

$F_v = (f'_m) = (1500)^{1/2} = 38.7$ psi > 21.0 psi O.K.

Sample Calculation for Shield Design

Additional Considerations

- If water level rises above the top of the opening, the closure may laterally load the lintel. In this case the lintel should be checked for biaxial bending.
- Provide any additional code-required reinforcement around openings for the specific seismic zone.
- Different methods of attachment may load the adjacent wall differently.
- Confirm that gasket is suitable for depth/duration of flooding and selected construction materials.

(CONT'D)

Dry Floodproofing

Table of Contents

CONSTRUCTION CONSIDERATIONS FOR SEALANTS AND SHIELDS

The use of sealants and shields may require careful attention to critical installation activities. When using shields and sealants, it is vital that

- the sealant be applied in accordance with the manufacturer's instructions;
- wrapped systems are anchored properly and the surrounding soil recompacted;
- shields are tightly installed with associated caulking or gaskets, utilizing the proper grade of materials and paying close attention to the anchoring details; and
- multiple closures are accurately labeled and stored in an easily accessible space.

DRAINAGE COLLECTION SYSTEMS

The development of drainage collection systems is a critical component in the design of many dry floodproofing measures and may be utilized in concert with elevation, floodwall, and levee measures. These systems collect drainage and seepage from areas along, adjacent to, or inside the retrofitting measure and the sump pump installation, which transmits the collected drainage and seepage away from the building's foundation. Determination of the amount of surface water inflow and infiltration was presented in Chapter IV. This section presents the parameters that govern the design of these systems.

Typical homes with basements are constructed on concrete footings upon which concrete or cinder block foundation walls are constructed. In some instances, the foundation walls are parged and covered with a waterproof coating, and/or perforated pipe underdrains are installed to carry water away from the exterior foundation walls (see Figure VI-D18: Typical Residential Masonry Block Wall Construction). Then the excavations are backfilled and compacted.

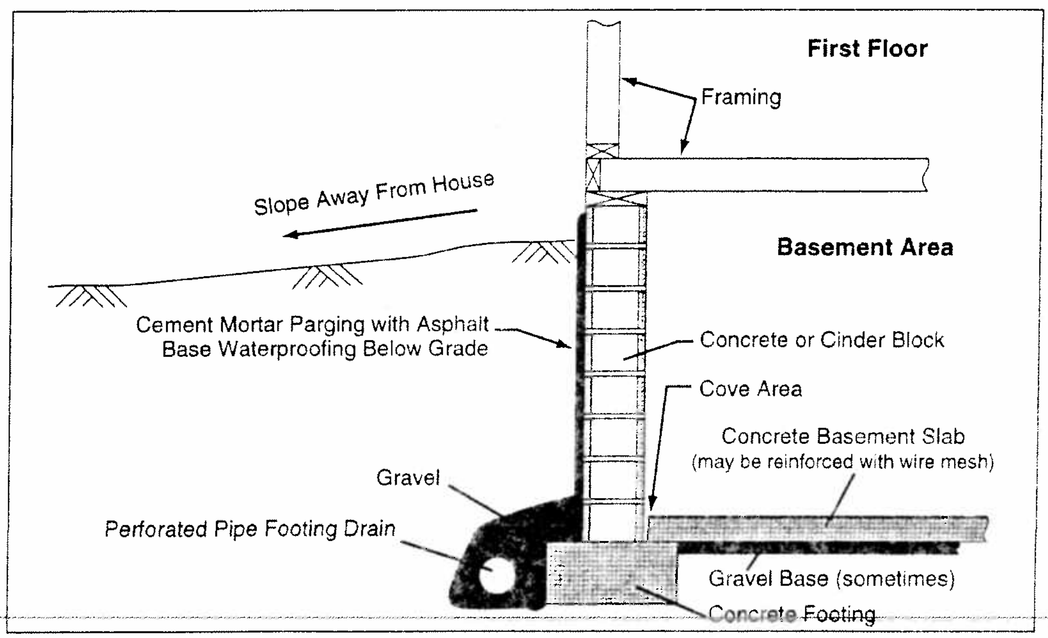


Figure VI-D18: Typical Residential Masonry Block Wall Construction

Check building codes to see if any maximum height of unbalanced fill requirements apply for the given construction.

However, in practice, this fill material is not and often cannot be compacted to a density equal to that of the undisturbed soils around the house. Because of the density difference, the fill material is capable of conducting and holding more water than the soil around it and frequently provides a storage area for the soil water. As flood levels rise around the structure, the combined water and soil pressure in the areas adjacent to the foundation increases to the point of cracking foundation walls and/or entering the basement through existing cracks to relieve the pressure. (See Figure VI-D19: Common Faults Contributing to Seepage into Basements.)

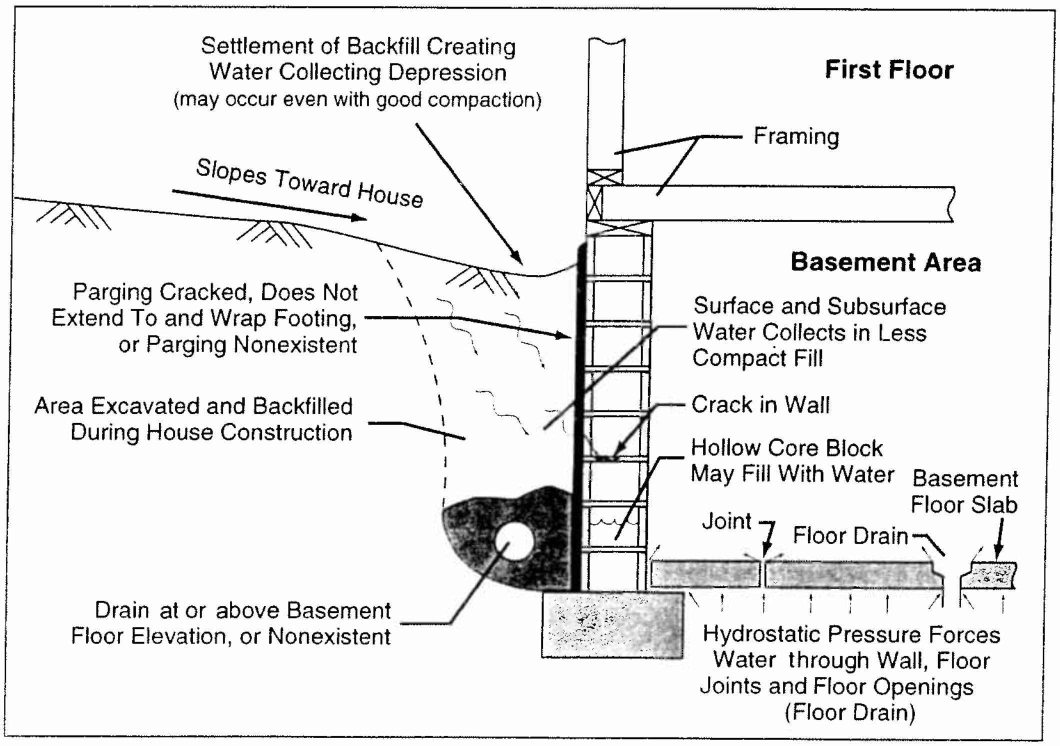


Figure VI-D19: Common Faults Contributing to Seepage into Basements

Depending upon site-specific soil conditions, high water tables, and local drainage characteristics, slab-on-grade homes may experience similar seepage problems. In addition, elevating and/or dry floodproofing a slab-on-grade home may also necessitate the installation of drainage collection systems to counteract buoyancy and lateral hydrostatic forces.

Drainage collection systems consisting of perforated pipe drains are designed to collect this water and discharge it away from the structure, thereby relieving the pressure buildup against the foundation walls. Several types of drainage collection systems exist including french drains, exterior underdrains, and interior drains.

FRENCH DRAINS

> French drains are generally not suitable for areas subject to frequent inundation due to the lack of a gravity discharge point during a flood. However, they can be effective in keeping localized drainage away from the foundation (providing there is no occurrence of a significant flood).

French drains are used to help dewater saturated soil adjacent to a foundation. They are simply trenches filled with gravel, filter fabric, and sometimes plastic pipe. A typical french drain section is shown in Figure VI-D20. The effectiveness of french drains is closely tied to the existence of a suitable discharge point and the slope/depth of the trench. A suitable discharge for the drain usually means an open stream, swale, ditch, or slope to which the drain can be run. If such a discharge point is not available, a french drain is generally not feasible.

If feasible, the french drain should be dug to a sufficient depth to ensure the capture of soil water that might infiltrate the fill material in the footing area of the basement. The slope of the trench should be such that good flow can be maintained between the gravel stones. This typically means a minimum slope of 1.0% or more.

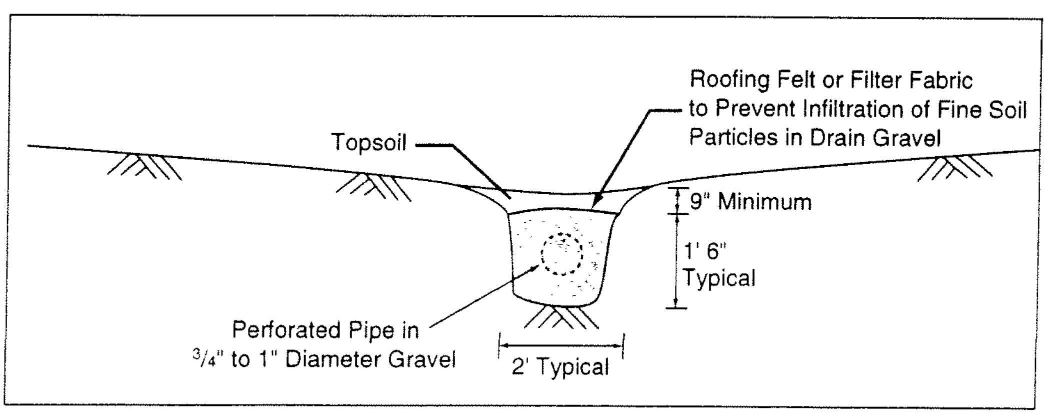


Figure VI-D20: Typical French Drain System

EXTERIOR UNDERDRAIN SYSTEMS

Exterior underdrain systems are generally the most reliable drainage collection system when combined with some type of foundation parging and waterproofing. Their chief advantage is that they will remove water that would otherwise exert pressure against the foundation walls and floors.

Similar to the french drain, an exterior underdrain system with gravity discharge will not work during a flood. Therefore sump pump discharge with a backup energy source is the preferred alternative.

Underdrains are normally constructed of continuous perforated plastic pipe laid on a gravel filter bed, with drain holes facing up. The underdrains are placed along the building foundation just below the footing and carry water that collects to a gravity discharge or sump pump for disposal into a public drainage system, natural drainage course, or ground surface (as permitted by local agencies). (See Figure VI-D21: Typical Exterior Underdrain System with Sump Pump and Figure VI-D22: Details of a Combination Underdrain and Foundation Waterproofing System.)

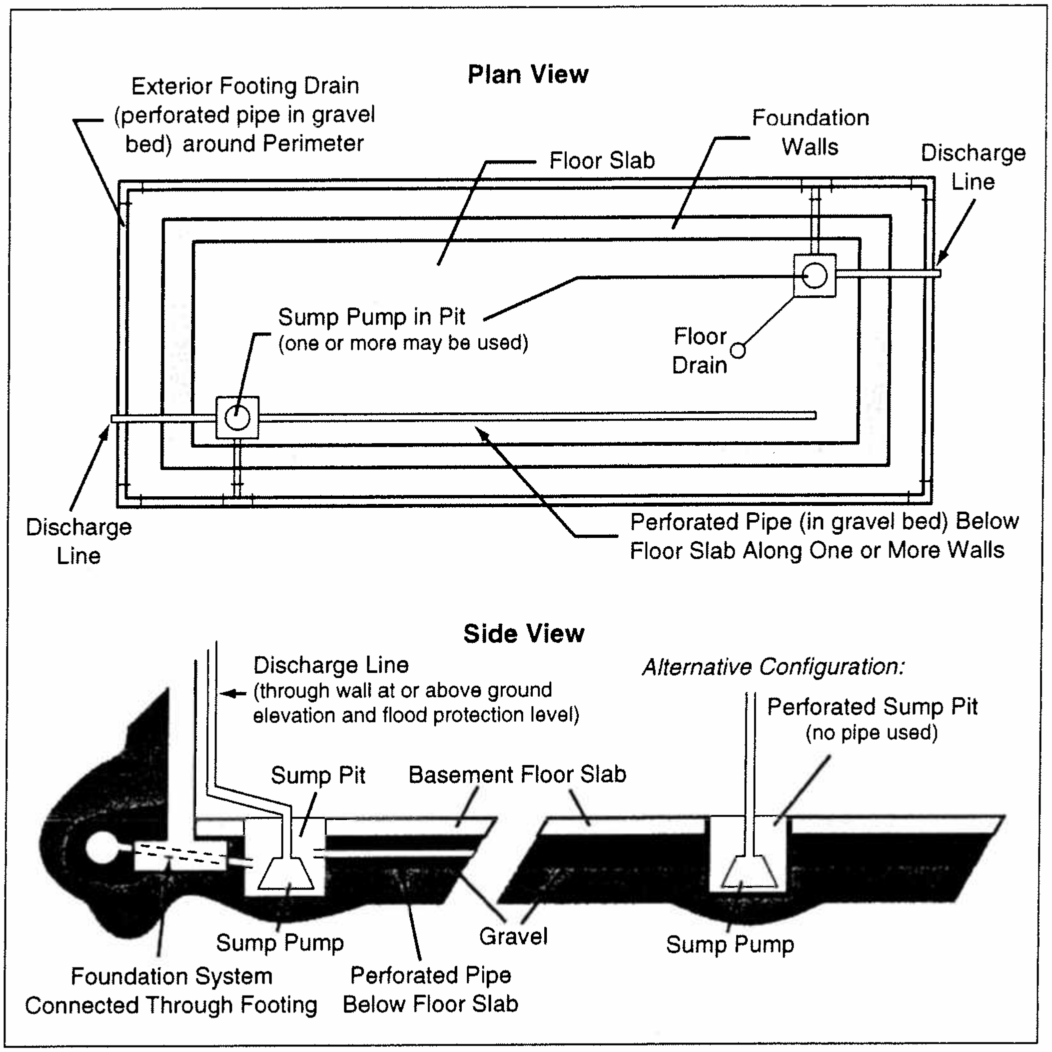


Figure VI-D21: Typical Exterior Underdrain System with Sump Pump Showing Two Alternative Configurations in the Side View

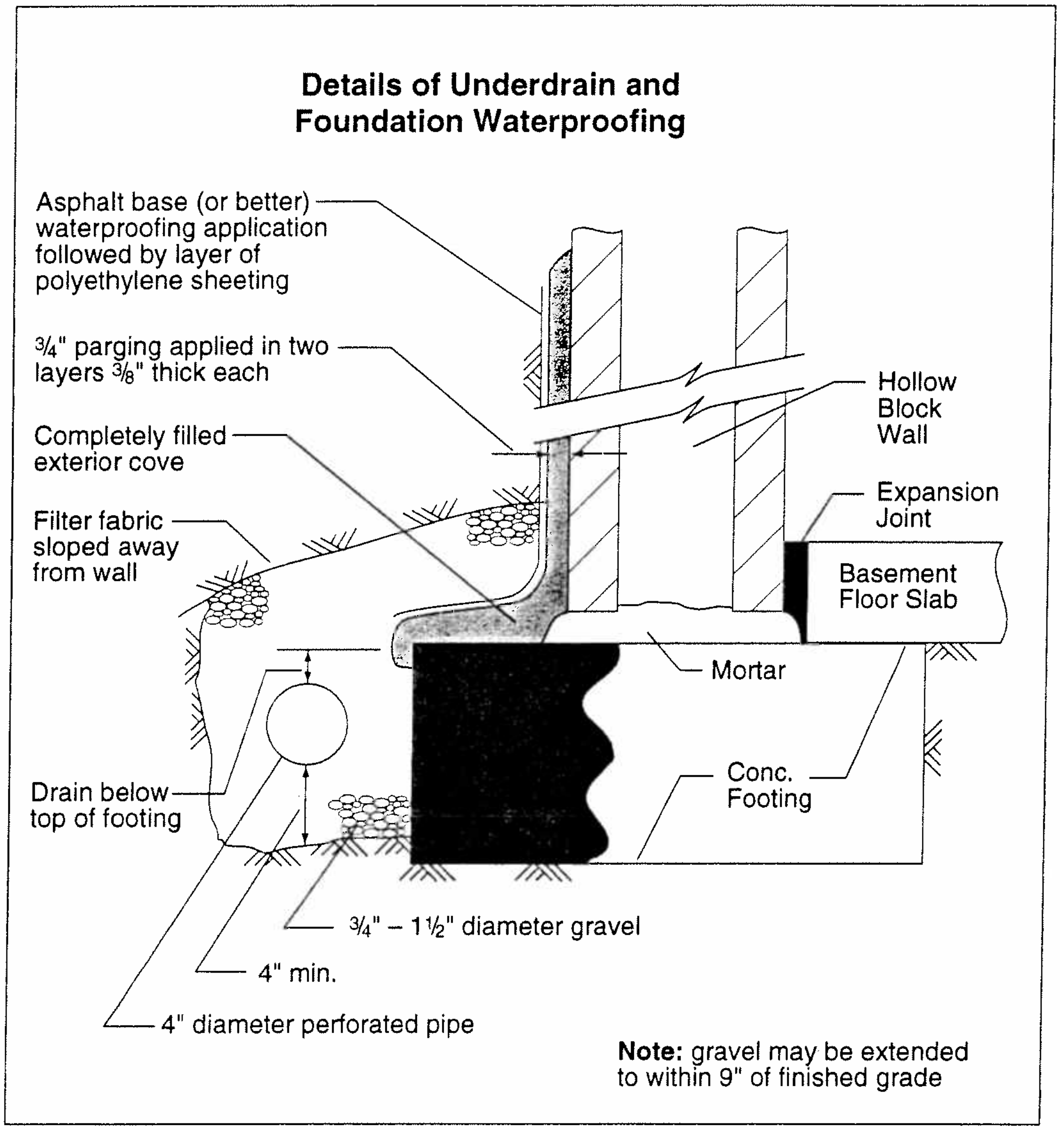


Figure VI-D22: Details of a Combination Underdrain and Foundation Waterproofing System

INTERIOR DRAIN SYSTEM

Interior drain systems are designed to relieve hydrostatic pressure from the exterior basement walls and floors and do not require that the soil be excavated from around the exterior basement walls for installation. Sump pumps are perhaps the most familiar of all methods used to dewater basements. The sump is generally constructed so that its bottom is well below the base of the basement floor slab. Water in the areas adjacent to the basement walls and floor migrate toward the area of least pressure along the lines of least resistance, in this case toward and into the sump. It may be necessary to provide a more readily accessible path of least resistance for water that has collected in the fill material and around the house to follow. To achieve this, pipe segments are inserted and sometimes drilled through the basement wall and into the fill behind. These pipe segments are then connected to larger diameter pipes running along a gravel-filled trench or cove area into the basement floor and into one or more sumps. (See Figure VI-D23: Typical Interior Drain Systems.)

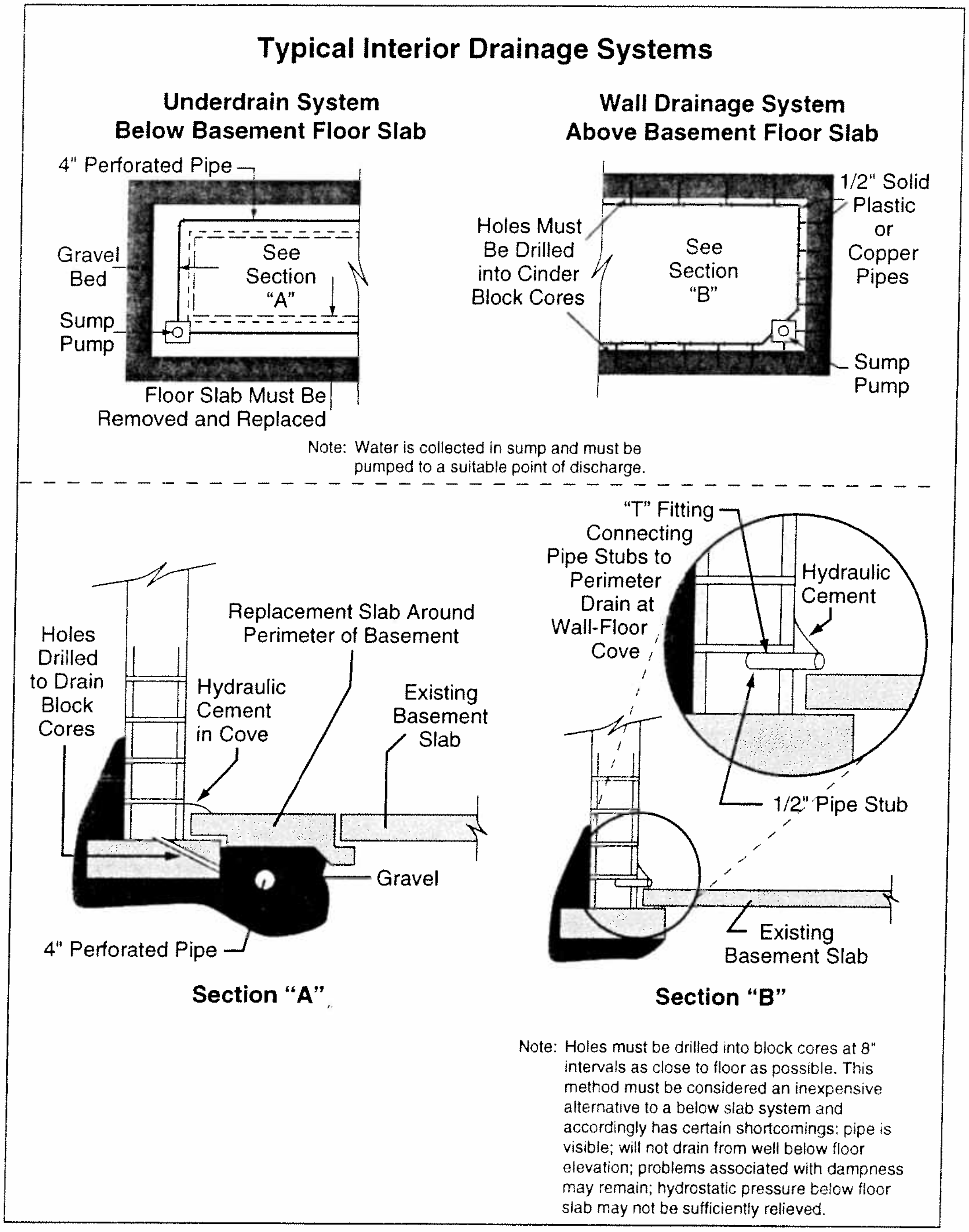


Figure VI-D23: Typical Interior Drain Systems

SUMP PUMPS

TYPES OF SUMP PUMPS

Two types of sump pumps commonly used are the submersible and the pedestal. The submersible type has a watertight motor that is directly connected to the pump casing. It is installed at the bottom of the sump. The pedestal sump pump uses an open motor supported on a pipe column with the pump at its base. A long shaft inside the column connects the motor to the pump impeller. Figure VI-D24 depicts both of these pumps. Submersible pumps are preferred because they will continue to operate if the flood level exceeds the height of the pump.

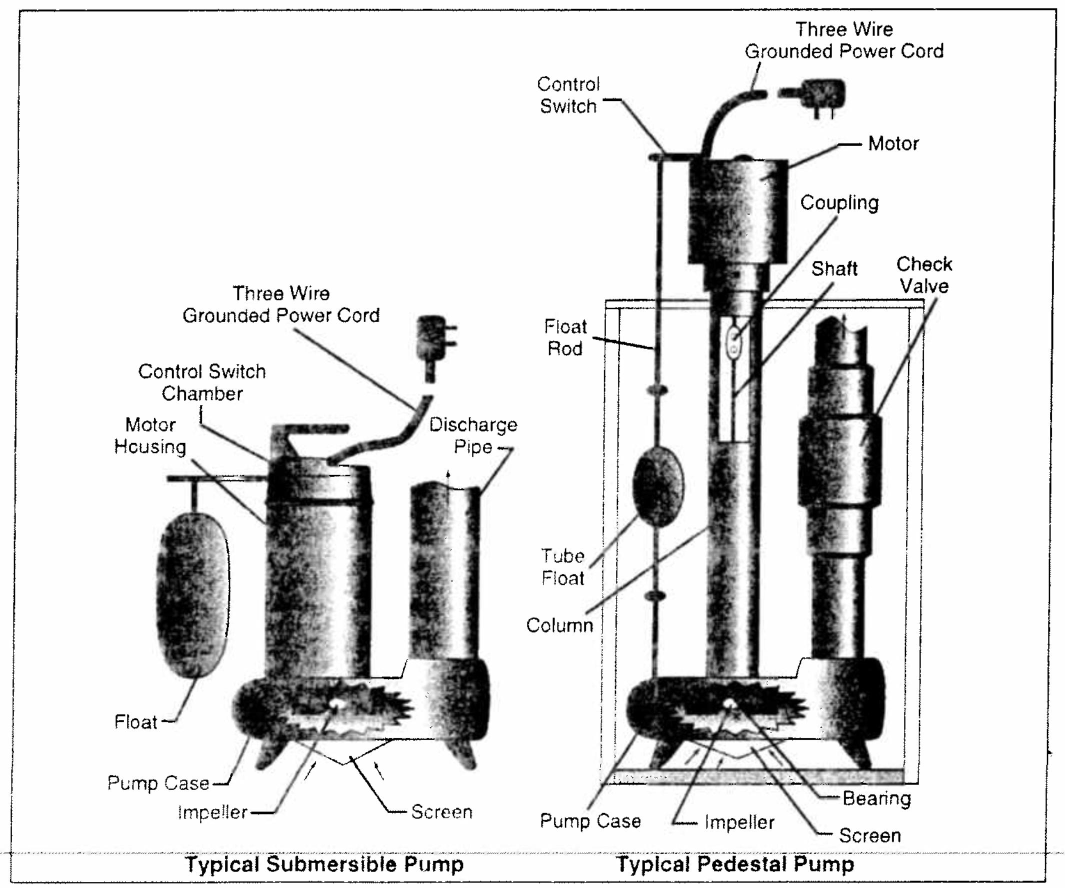


Figure VI-D24: Types of Sump Pumps

Battery powered marine-type bilge pumps are an alternative to sump pumps/electrical generator installations.

In selecting a sump pump for use in residential floodproofing, the designer should consider the advantages of each pump type and make a selection based on requirements determined from investigation of the residence. Considerations include pump capacity (gallons per minute or gallons per hour), pump head (vertical height that the water is lifted), and electrical power required (residential electrical power is usually 120/240 volts AC, single phase). Sump pump motors generally range in size from 1/4 horsepower to 1/2 horsepower designed to operate on either 120 or 240 volts.

Infiltration vs. Inundation

The capacities of sump pumps used in residential applications are limited. In floodproofing, sump pumps are used to prevent accumulations of water within the residence. In conjunction with other floodproofing methods, sump pumps can be used to protect areas around heating equipment, water heaters, or other appliances from floodwaters. Sump pumps are useful to protect against infiltration of floodwaters through cracks and small openings. In the event that there are large openings, or that the structure is totally inundated, the pumping capacity of sump pumps is often exceeded, but they are useful for controlled dewatering after floodwaters slowly recede (if submersible pumps are used).

COORDINATION WITH OTHER FLOODPROOFING METHODS

Design and installation of a sump pump should be coordinated with other floodproofing methods such as sealants and shields, protection of utility systems (furnaces, water heaters, etc.) and emergency power.

FIELD INVESTIGATION

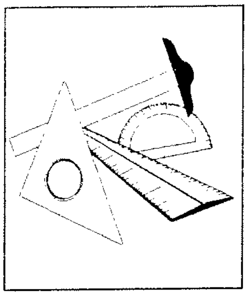
Detailed information must be obtained about the existing structure to make decisions and calculations concerning the feasibility of using a sump pump. Use the Building/Building System Data Sheets (Figures VI-3 and VI-4) as a guide to record information about the residence. Items that the designer may require are covered on the sump pump field investigation worksheet, (Figure VI-D25).

Owner Name: ____________________ **Prepared By:** ________________
Address:____________________ **Date:** ________________
Property Location: ____

Sump Pump Field Investigation Worksheet

- [ ] Document physical location and characteristics of electrical system on sketch plan below.
- [ ] Determine base flood elevation: ______________________________
- [ ] Check with local building official's office for version of National Electrical Code (NEC) NFPA 70, and local Electrical Code requirements: ______________________________
- [ ] Check with local building official's office for established regulations concerning flooded electrical equipment: ______________________________
- [ ] Check with the regulatory agencies to determine which state and local codes and regulations regarding the design and installation of plumbing systems may apply to the installation of a sump pump: ______________________________
- [ ] Determine location and condition of any existing drainage collection systems, including sump pits and pumps.
  - [ ] Does residence have subterranean areas such as a basement? ____Yes ____ No
  - [ ] Is there a sump pump installed presently? ____Yes ____ No: If so:
  - [ ] Record nameplate data from pump: capacity (______GPH or GPM @ ______ FT HEAD), motor horsepower, voltage, and manufacturer's name and model number. ____________
    ____
    ____
  - [ ] Sketch plan of basement indicating location of sump, heating and cooling equipment, water heaters, and floor drains.
  - [ ] How high above floor is receptacle outlet serving cord and plug connected to sump pumps?

Figure VI-D25: Sump Pump Field Investigation Worksheet



Once this data is collected, the designer should answer the questions below to develop a preliminary concept for the installation of a sump pump.

☐ If there is no sump pump and one is needed, note potential location for a sump and tentative location for pump discharge piping on above sketch plan.

- ☐ Is there an electrical outlet nearby? ____ Yes ____ No
- ☐ Does electrical panel have capacity to accommodate additional GFI circuit if necessary? ____Yes ____ No

☐ If other floodproofing measures are to be considered, such as placing a flood barrier around heating equipment or other appliances, is the existing sump pump in an appropriate location? ____ Yes ____ No
Does another sump and sump pump need to be provided?____ Yes ____ No

☐ Select emergency branch circuit routing from sump pump to emergency panel. Note on above sketch plan.

☐ Is sump pump branch circuit located above flood protection elevation and is it a GFI circuit? ____ Yes ____ No

☐ Locate sump pump disconnect or outlet location near sump pump location above FPE.

Once these questions have been answered the designer can confirm sump pump installation applicability through:

☐ Verify constraints because of applicable codes and regulation. ______________________________

____

____

☐ Sump pump needed? ____Yes ____No
☐ Is sump pump required by code? ____Yes ____ No
☐ Code constraints known? ____Yes ____ No
☐ Proceed to design? ____ Yes ____ No
☐ Confirm that wiring can be routed exposed in unfinished areas and concealed in finished areas. ____ Yes ____ No
☐ Confirm that panel has enough power to support sump pump addition. ____ Yes ____ No

Figure VI-D25: Sump Pump Field Investigation Worksheet (continued)

DESIGN

The design of sump pump applications follows the procedure outlined in the flow chart in Figure VI-D26: Sump Pump Design Process.

Sump Pump Design Process

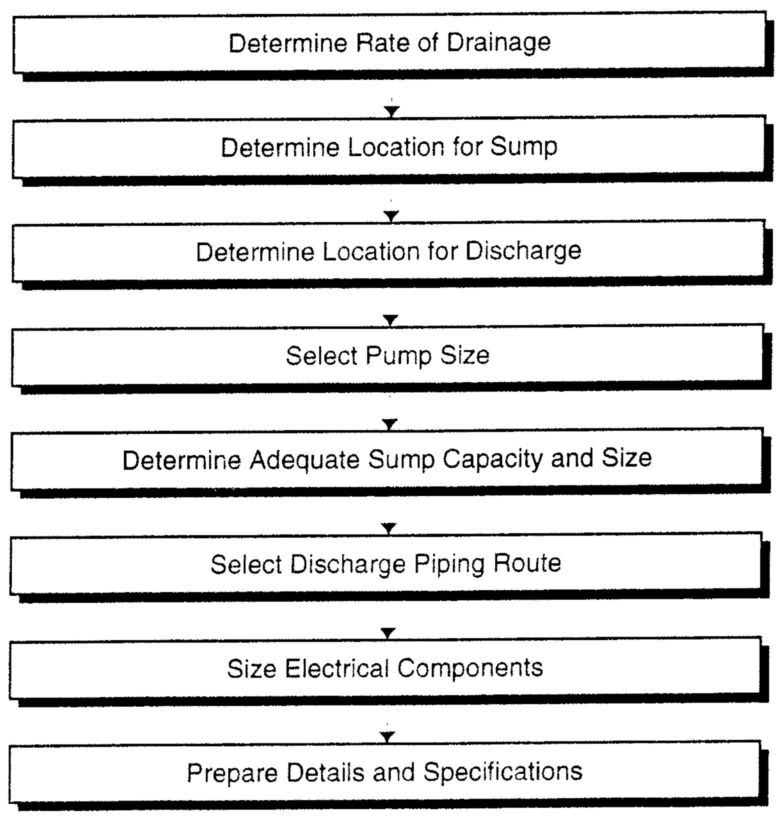


Figure VI-D26: Sump Pump Design Process

Step 1: Determine rate of drainage.

(Covered previously in Chapter IV.)

Step 2: Determine location for sump.

Refer to Figure VI-D27 for typical sump pump installation. Consider the following in locating the sump.

- Is there adequate room for the sump?

- Are there sub-floor conditions (i.e., structural footings) that would interfere with sump installation?

- If penetration of floor is not recommended, consider using a submersible pump design for use on any flat surface.

- Are other floodproofing measures being considered, such as placing a flood barrier around heating equipment or plumbing appliances? If so, locate sump or provide piping to sump to keep protected area dewatered. Make preliminary sketch showing location of sump pump, discharge piping, and location of electrical receptacle for pump.

- Coordinate sump location with design of drainage collection system.

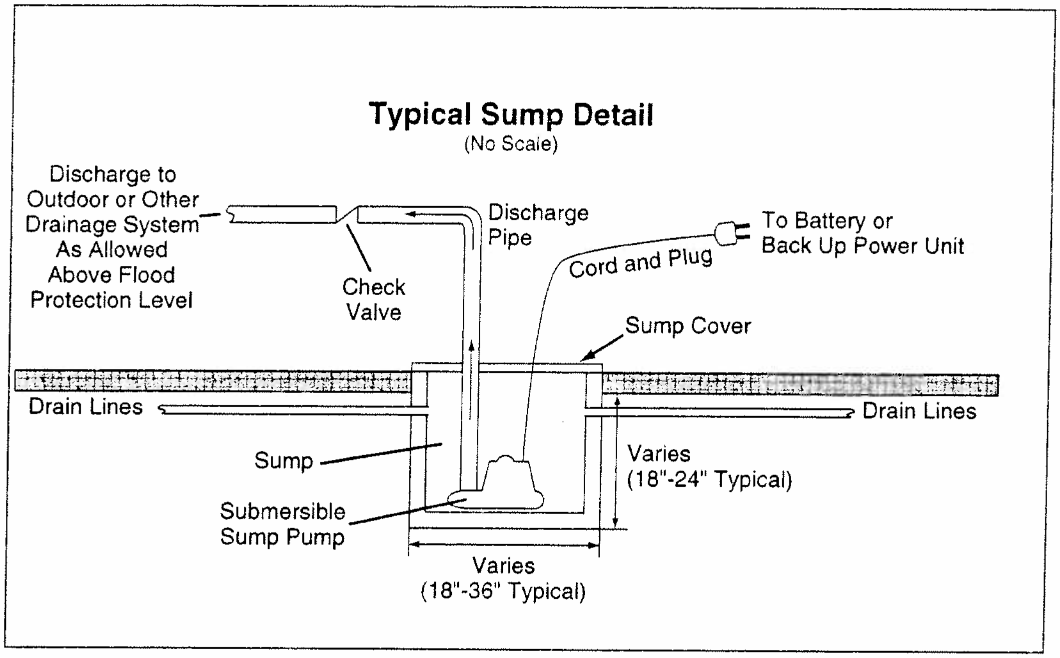


Figure VI-D27: Typical Sump Detail

Step 3: Determine location for discharge.

Check with local authorities having jurisdiction about the discharge of clear water wastes. In most jurisdictions, it is not acceptable to connect to a sanitary drainage system, nor may it be desirable since, in a flood situation, it may back up. If allowable, the desirable location for the discharge is a point above the BFE at some distance away from the residence. The discharge point should be far enough away from the building that water does not infiltrate back into the building. From the information obtained during the field investigation, tentatively lay out the route of the discharge piping and locate the point of discharge.

Step 4: Make selection of pump.

Sump pumps for residential use generally have motors in the range of 1/6 to 3/4 horsepower and pumping capacities from 8 to 60 gallons per minute. In selecting a pump, the designer needs the following information:

- Estimate of the quantity of floodwater that will infiltrate into the space per unit of time (GPM or GPH).

- The total dynamic head for the sump discharge. This equals the vertical distance from the pump to the point of discharge plus the frictional resistance to flow through the piping, the fittings, and the transitions. Use the preliminary sketch and field investigation information developed earlier to determine these parameters. The total discharge head, TH, is computed as follows:

FORMULA

$$TH = Z + h_{f\text{-}pipe} + h_{f\text{-}fittings} + h_{f\text{-}trans}$$

where: **TH** is the total head in feet;
Z is the elevation difference between the bottom of the sump and the point of discharge, in feet;
$h_{f\text{-}pipe}$ is the head loss due to pipe friction, in feet;
$h_{f\text{-}fittings}$ is the head loss through the fittings, in feet; and
$h_{f\text{-}trans}$ is the head loss through the transitions, in feet.

Formula VI-D1: Total Discharge Head

Formula VI-D2 assumes steady pipe flow with no change in pipe size to maintain constant velocity.

The head loss due to pipe friction can be obtained from hydraulic engineering data books and is dependent on the pipe material and pipe length. The head losses due to pipe fittings and transitions are calculated as follows:

FORMULA

$$h_{f\text{-fittings}} + h_{f\text{-trans}} = (K_b + K_e + K_o)(V^2/2g)$$

where:
- $h_{f\text{-fittings}}$ is the head loss through pipe fittings, in feet;
- $h_{f\text{-trans}}$ is the head loss through the transitions, in feet;
- K_b is the loss coefficient of the pipe fitting(s), taken from hydraulic engineering data books;
- K_e is the loss coefficient of the pipe entrance, assumed to be 0.5;
- K_o is the loss coefficient of the pipe exit/outlet, assumed to be 1.0;
- V is the velocity of flow through the pipe, in feet per second, taken from hydraulic engineering data books; and
- g is weight of gravity, 32.2 pounds per second squared.

Formula VI-D2: Head Losses Due to Pipe Fittings and Transitions

The following example illustrates the use of these equations to determine the total head requirements for a sump pump installation.

Sample Calculation for Sump Pump

GIVEN:

Z = 10 feet; flow assumed to be 20 gpm; 1.5 inch steel discharge pipe length of 30 feet includes one elbow, one gate valve and one check valve.

SOLUTION:

From Hydraulic Engineering Data Books, resistance to flow in a 1.5-inch steel pipe is 2.92 feet per 100 feet of pipe;

$$h_{f\text{-pipe}} = 2.92\,(30/100) = 0.876 \text{ feet}$$

resistance coefficients for fittings, entrance and exit, are

K_b (elbow) = 0.63;
K_b (gate valve) = 0.15;
K_b (check valve) = 2.1;
K_e (pipe entrance) = 0.5
K_o (sudden enlargement/outlet) = 1.0

$$(K_b + K_e + K_o) = 0.63 + 0.15 + 2.1 + 0.5 + 1.0 = 4.38$$

velocity converted from gallons per minute to feet per second =

$$V_{fps} = \frac{Q}{450\ A_{pipe}}$$

$$= \left(20\,\frac{gal}{min}\right) \Big/ \left(\frac{450\,\frac{ft^3}{sec}}{\frac{1gal}{min}}\right)\left(3.14\right)\left(\frac{.75\text{ in}}{12\ \frac{in}{ft}}\right)^2$$

$$= 3.62\,\frac{ft}{sec}$$

$$h_{f\text{-fittings}} + h_{f\text{-trans}} = (K_b + K_e + K_o)(V^2/2g) = 4.38\,(3.62)^2/(2)(32.2) = 0.891 \text{ feet}$$

$$TH = Z + h_{f\text{-pipe}} + h_{f\text{-fittings}} = 10 + 0.876 + 0.891 = 11.77 \text{ feet}$$

Therefore select a pump capable of pumping 20 gallons per minute at 11.77 feet of total head.

Step 5: Determine adequate sump capacity and size.

The capacity and size of the sump depends on several factors:

- Physical size of the sump pump
- Recommendations of the sump pump manufacturer regarding pump cycling or other constraints.

The designer should take these considerations into account in locating the sump and configuring the sump pump discharge.

Step 6: Select discharge pipe route.

- Minimize length of pipe between sump and discharge point.
- Avoid utility and structural components along route.
- Attach discharge pipe to structure as required by code.
- Protect discharge point against erosion.

Step 7: Size electrical components.

- Obtain horsepower and full load amperage rating for sump pump.
- Select GFI circuit, as required by code.
- Size minimum circuit ampacity and maximum fuse size
- Size maximum circuit breaker size.
- Obtain recommended fuse size or circuit breaker size from manufacturer and compare to above maximum and minimum NEC sizes.

At this point the designer should prepare a floor plan sketch showing the location of the sump pump, routing of discharge line, location of discharge point, and preliminary specifications for the sump pump, sump, piping, and appurtenances and confirm the preliminary design with the homeowner, covering the following items:

- Verify that proposed location of sump pump is feasible.
- Verify electrical availability for sump pump.
- Verify existing conditions along proposed routing of discharge piping and at location of discharge pipe termination.
- Confirm selection and size of sump pump.
- Confirm size and location of sump.
- Confirm special considerations regarding existing conditions affecting design and installation of sump pump and sump.

Step 8: Details and specifications.

Prepare final plans showing:

- Floor plan with location of sump and backwater valves
- Routing of discharge pipe and location of termination
- Details, notes, and schedules
  - Sump pump detail
  - Wall, floor, and wall penetration details
  - Sump construction details

  - Installation notes
  - Equipment notes (or schedule)
  - Discharge pipe termination

- Prepare specifications (on drawing or as a specifications booklet)
  - Pipe and fittings
  - Insulation
  - Hangers and supports
  - Valves (including backwater valves)
  - Sump pumps

- Coordinate plans with work of others on additional floodproofing measures that may be proposed at the same residence.

BACKWATER VALVES

> Depending upon the hydrostatic pressure in the sewer system, a simple wood plug can be used to close floor drains.

Backwater valves can help prevent backflow through the sanitary sewer and/or drainage systems into the house. They should be considered for sanitary sewer drainage systems that have fixtures below the FPE. In some instances, combined sewers (sanitary and storm) present the greatest need for backwater valves because they can prevent both a health and flooding hazard. Backwater valves are not foolproof: their effectiveness can be reduced because of fouling of the internal mechanism by soil or debris. Periodic maintenance is required.

The backwater valve is similar to a check valve used in domestic water systems (Figure VI-D28). It has an internal hinged plate that opens in the normal direction of flow. If flow is reversed ("backflow"), the hinged plate closes over the inlet to the valve. The valve generally has a cast-iron body with a removable cover for access and corrosion-resistant internal parts. The valves are available in nominal sizes from two to eight inches in diameter.

As an added feature, some manufacturers include a shear gate mechanism that can be manually operated to close the drain line when backwater conditions exist. The valve would remain open during normal use. A second type of backwater valve is a ball float check valve (Figure VI-D29) that can be installed on the bottom of outlet floor drains to prevent water from flowing up through the drain. This type of valve is often built into floor drains or traps in newer construction.

Advanced backwater valve systems have ejector pump attachments that are used to pump sewage around the backflow valve, forcing it into the sewer system during times of flooding. This system is useful in maintaining normal operation of sanitary and drainage system components during a flood.

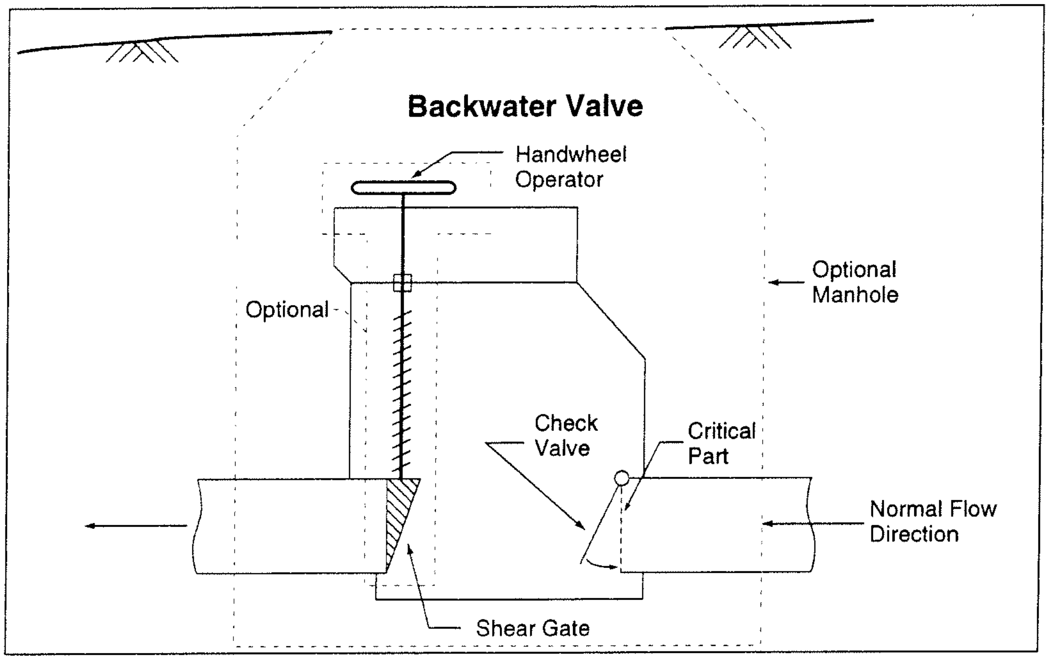


Figure VI-D28: Backwater Valve

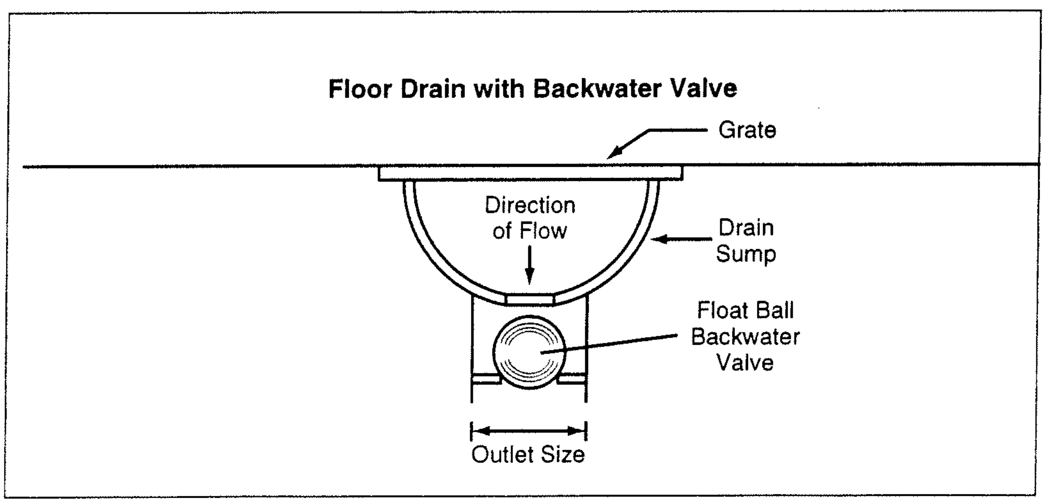
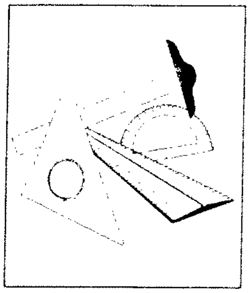


Figure VI-D29: Floor Drain With Ball Float Check Valve



FIELD INVESTIGATION

Alternatives to backwater valves include overhead sewers and standpipes. Their use should be evaluated carefully.

Detailed information must be obtained about the existing structure to make decisions and calculations concerning the feasibility of using a backwater valve. Use the Building/ Building System Data Sheets as a guide to record information about the residence. Once this data is collected, the designer should answer the questions below to develop a preliminary concept for the installation of a backflow valve.

DESIGN

The designer should follow the process illustrated in Figure VI-D30: Backwater Valve Selection, to design, select, and specify the backflow valve.

Backflow Valve Selection

Determine Relationship of Drains to Flood Protection Elevation

↓

Confirm Regulations Concerning Backwater Valves

↓

Determine Layout of Drains that Serve Impacted Fixtures

↓

Determine Pipe Sizes on Impacted Drains

↓

Develop Type, Size and Location for Valves

↓

Prepare Details and Specifications

Figure VI-D30: Backwater Valve Selection

Owner Name: ____________________ **Prepared By:** ________________
Address:____________________ **Date:** ____________________
Property Location: ____

Backwater Valve Field Investigation Worksheet

☐ Does residence have plumbing fixtures or floor drains below FPE: ____ Yes ____ No

☐ Is building drainage system equipped with backwater valves, or do floor drains have backwater device? ____ Yes ____ No: If so, locate on a floor plan sketch of the residence.

☐ If there are no backwater valves and they are needed, consider the following in selecting a location for their installation.

- ☐ Can adequate clearance be maintained to remove access cover and service valve? ____ Yes ____ No
- ☐ Are there any codes that regulate or restrict installation of such valves? ____ Yes ____ No; If yes, explain. ________________________________
- ☐ Tentatively locate on sketch box where backwater valves might be installed.
- ☐ Proceed To Design? _____Yes _____No

Figure VI-D31: Backwater Valve Field Investigation Worksheet

The elements of this process include:

Step 1: Determine relationship of drains to FPE.

If any drain or pipe fixtures are located below the FPE, backwater valves should be installed. If all drains and fixtures are located above the FPE, backwater valves are not necessary.

Step 2: Determine regulations concerning backwater valves.

Based upon information collected during the field investigation, confirm the allowability of and the regulations governing the installation of backflow valves.

Step 3: Determine layout of drains that serve the impacted fixtures.

Make a floor plan sketch showing location of all plumbing fixtures and appliances, floor drains, and drain piping that is below the FPE.

Step 4: Determine pipe sizes on impacted drains.

Obtain from field investigation the size of drainage lines below the FPE.

Step 5: Determine type, size, and location for backwater valves.

Determine type, size, and location of backwater valves required, paying considerable attention to any special conditions related to installation. Factors to be considered include:

- Clearance for access and maintenance

- Cutting and patching of concrete floors

- Indicate on floor plan sketch the tentative location(s) of the backwater valve(s).

If possible, backwater valves should be located outside a structure so as to minimize damage should the pressurized line fail.

At this point the designer should confirm the preliminary design with the homeowner, discussing the following items:

- Verify that proposed locations of backwater valves are feasible.

- Verify existing conditions at location of proposed backwater valve installation.

Confirm the size and location of needed backwater valves.

- Confirm special considerations regarding existing conditions affecting design and installation of backwater valves.

Step 6: Prepare details and specifications.

The final plans and specifications should include the following items:

- Floor plan with location of backwater valves

- Details, notes, and schedules

    - Backwater valve detail

  - Wall, floor, and wall penetration details
  - Installation notes
  - Equipment notes (or schedule)

- Prepare specifications governing the installation of:
  - Pipe and fittings
  - Insulation
  - Hangers and supports
  - Valves

- Coordinate plans with work of others on additional floodproofing measures that may be proposed at the same residence.

EMERGENCY POWER

Emergency power equipment can be applied to residential applications if the proper guidelines are observed. First, it is not feasible to apply emergency power equipment to the operation of a whole house with electric resistance heat, heat pumps, air conditioning equipment, electric water heater, electric cooking equipment, or sump pump(s). These large loads would require very expensive emergency power equipment that would have considerable operating costs. However, small, economical, residential portable generators or battery backup units can be successfully installed to operate selected, critical electrical devices or equipment from the limited power source.

A list of appliances or equipment that a homeowner might choose to operate is shown in Table VI-D1. It is important to note that all of these appliances would most likely not be operated at the same time.

Table VI-D1

Essential Equipment/Appliances to Operate from Emergency Power Source

Critical Items include:	
• Floodwater sump pump	- typically 1/3 to 1/2 hp 120 volt single phase.
• Domestic sewage pump	- typically 3/4 hp to 1 hp 120 volt single phase.
Non-critical items include:	
• Refrigerator	- 350 watts to 615 watts.
• Freezer	- 341 watts to 440 watts.
• Gas or oil furnace	- 1/7 hp burner, 1/3 hp to 1/2 hp blower motor.
• Some lighting or a light circuit	- limit to about 400 watts.
• A receptacle or a receptacle circuit	- limit to about 600 watts.

Several sources of technical information are available to assist in the design of emergency residential generator set installations.

- Some manufacturers provide application manuals and sizing forms to select small gasoline-powered, natural or liquid petroleum gas, or battery sets.
- Other manufacturers even offer software to size the small generator/battery sets.
- Another good source is the supplier of the standby generator/battery set. These have additional application data for sizing the unit to suit the anticipated load.
- The manufacturer of the set will provide a wattage and volt-ampere rating for each size at a particular voltage rating.

Selection of a generator/battery set is a matter of matching the unit capacity to the anticipated maximum load. The chief complication in sizing the generator/battery set is the starting characteristics of the electric motors in the pumps and appliances to be served.

FIELD INVESTIGATION

Detailed information must be obtained about the existing structure to make decisions and calculations concerning the feasibility of using an emergency generator or battery backup unit. Use the Building/Building Systems Data Sheets (Figures VI-3 and VI-4 located in the beginning of Chapter VI) as a guide to record information about the residence. Among the activities the designer may pursue are:

- Examine the routing and condition of the existing building electrical system, noting potential locations for emergency power components (above the FPE and away from combustible materials).
- Determine utility or power company service entrance location and routing.
- Determine utility constraint data.
- Record these items and locations on an electrical site plan/combination floor plan sketches.
- Confirm space for cable routing between main panel, emergency panel, transfer switch, and proposed generator/battery set.
- Examine existing panel branch circuit breakers and select circuits to be relocated to emergency panel.
- Confirm utility regulations on emergency power equipment with local power company.

DESIGN

The design of emergency power provisions is a straightforward process that is illustrated in Figure VI-D32. The steps include:

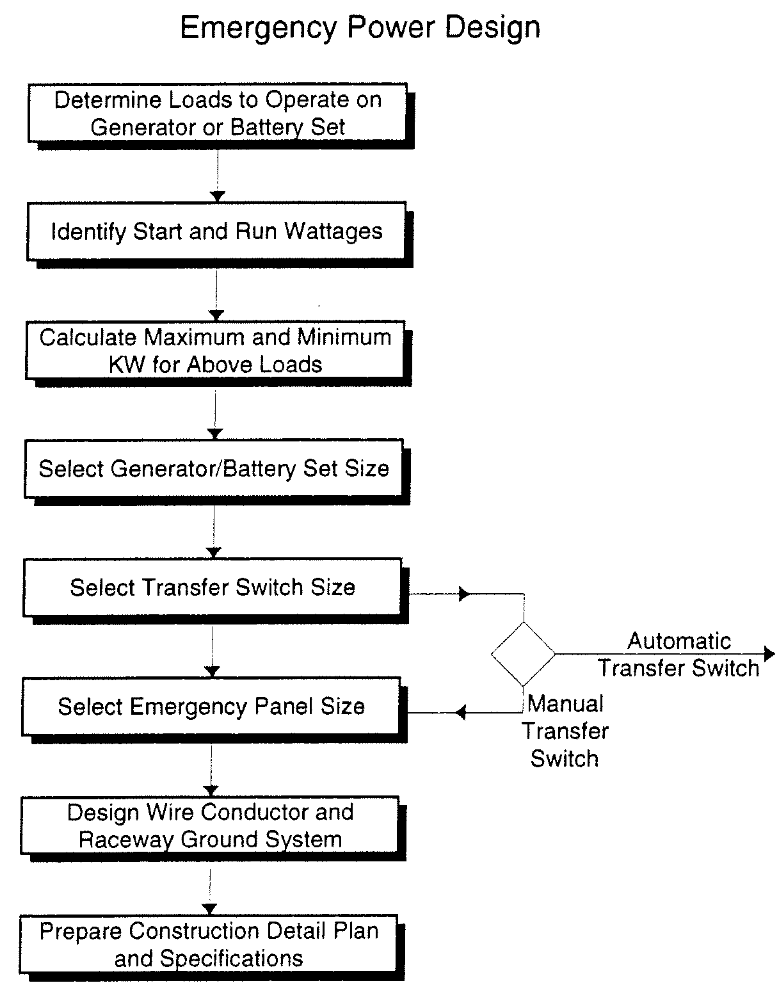


Figure VI-D32: Emergency Power Design Process

Since most power outages are temporary and relatively short lived, a battery backup source for sump pumps (only) may be the simplest solution for a homeowner. However, as the duration of the power outage increases, the suitability of battery backup systems decreases. Generator sets are a more secure source of power in these situations, especially for those residents who need/desire power to operate medical equipment or standard household appliances during power outages. Battery systems used in conjunction with emergency generators can provide service during a limited period if the owner is not home when the power goes out.

Step 1: Determine loads to operate on generator set.

Table VI-D2 presents typical electrical appliance loads for some home equipment. The designer should work with the owner to select only those pumps/appliances that must be run by emergency power and confirm the estimated appliance and motor loads.

Step 2: Identify start and run wattages.

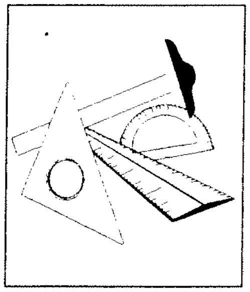
Start and run wattages for the appliance loads selected by the homeowner can be obtained from Table VI-D2, Typical Electric Appliance Loads.

Step 3: Calculate maximum and minimum KW for operating loads.

Based upon the loads determined in Step 1, the designer should develop the range of minimum and maximum wattages for the desired applications. Table VI-D2, Typical Electric Appliance Loads, can be used to estimate these minimum and maximum loads.

Table VI-D2 **Typical Electrical Appliance Loads**

Home Equipment	Typical Wattage	Start Wattage
Critical items:		
Limited lights (safety)	400	400
Sewage pump (3/4 hp to 1 hp)	1000	4000
Sump pump (1/3 hp to 1/2 hp)	333	2300
Water pump	800-2500	800-10000
Non-critical items:		
Refrigerator	400 - 800	1600
Freezer	600 - 1000	2400
Furnace blower	400 - 600	1600
Furnace oil burner	300	1200
Furnace stoker	400	1600
Limited receptacles	600	600



Emergency power equipment should be located above the flood protection level.

Step 4: Select generator/battery unit size:

Size the generator/battery unit set from load information obtained in Step 1. Generator/battery unit set sizing is based upon the approximation that motor starting requirements are three to four times the nameplate wattage rating; thus, generator sets/battery units should be sized to handle four times the running watts of the expected appliance loads.

Small generators/battery unit sets are usually rated in watts. Two ratings are often listed—a continuous rating for normal operation and a higher rating to allow for power surges. Match higher surge ratings with the starting wattage.

Generator sets can be loaded manually with individual loads coming on line in a particular sequence, or the loads can be transferred automatically with all devices trying to start at one time. This is illustrated by the following examples.

Table VI-D3 Example of Maximum Generator Sizing Procedure

	RUNNING LOAD	STARTING LOAD
SEWAGE PUMP	1000	4000
FURNACE	300+400=700	1200 + 1600 = 2800
SUMP PUMP	333	2300
REFRIGERATOR	400	1600
FREEZER	600	2400
RECEPTACLES	600	600
LIGHTS	400	400
TOTALS	4033 WATTS	14100 WATTS

Even though many of the above appliances cycle on and off, common practice is to select a generator with a continuous rating that is at least as large as the total wattage to start all loads at once. 14KW appears to be the minimum size to start all motors at once.

Table VI-D4

Example Step Sequence Manual Start - Minimum Generator Sizing

		Starting Loads		Running Loads
Sewage Pump	Step 1	4000		
Furnace	Step 2	2800	+	1000 = 3800
Sump Pump	Step 3	2300	+	700 + 1000 = 4000
Refrigerator	Step 4	1600	+	333 + 700 + 1000 = 3633
Freezer	Step 5	2400	+	400 + 333 + 700 + 1000 = 4833
Receptacles	Step 6	600	+	600 + 400 + 333 + 700 + 1000 = 3633
Lights	Step 7	400	+	600 + 600 + 400 + 333 + 700 + 1000 = 4033

Largest Load 4,833 Watts; Thus 5KW Generator Set is minimum size.

For each step or appliance load, add the running wattage of items already operating to the starting wattage of the items being started in that step. Select the largest wattage value out of all steps. Compare maximum wattage with continuous wattage rating of the generator.

At this point, the designer has sufficient information to present preliminary equipment recommendations to the homeowner, prior to the design of transfer switches, emergency panels, wiring, and other miscellaneous items. Among the issues the designer should confirm with the homeowner are:

- The essential power loads proposed for the generator/battery set. Discuss any other essential loads pertaining to life or property safety.
- Generator/battery set siting and proposed location. This should be discussed in light of unit weight, portage, storage, and handling methods.
- Provisions for fuel storage and fuel storage safety.

The designer should also:

- Educate the homeowner on battery operating time and/ or generator operating time vs. fuel tank capacity.
- Present initial generator/battery set cost and future operating costs.
- Discuss requirements for having equipment located above FPE.
- Discuss generator heat radiation and exhaust precautions to prevent carbon monoxide poisoning.

Step 5: Selection transfer switch size.

Transfer switches are designed to transfer emergency loads from the main house system to the generator/battery system in the event of a power failure. After power has been restored, the transfer switch is used to transfer power from the generator/battery set to the house system. Transfer switches can be manual or automatic. It is important to check with local code officials regarding requirements for how transfer switches are set up.

Manual Transfer Switches generally have the following characteristics:

- Double pole, double throw, nonfusible, safety switch, general duty with factory installed solid neutral, and ground bus. Double pole, double throw transfer switches are typically required to prevent accidentally feeding power back into the utility lines to workers servicing the line. This switch also protects the generator set from damage when the power is restored.
- Transfer switches are available with NEMA 1 enclosures for indoor mounting and NEMA 3R enclosures for outdoor locations.

- The voltage rating of transfer switches is typically 250 volts.
- Available sizes are 30 amp, 60 amp, 100 amp, and 200 amp.

The designer should consider the following items when selecting a manual transfer switch:

- Coordinate amperage to match emergency panel rating, continuous current rating of branch circuits, genset overcurrent protection, and panel branch feeder circuit breaker size.
- Fusible manual transfer switches are required as service entrance equipment and are required if the panel circuit breaker size does not correspond to the emergency panel size and generator/battery set circuit breaker size.
- Several manufacturer models are not load break rated and require load shedding before transfer operation. These switches must be used for isolation only. They do not have quick make-quick break operation.
- Some transfer switches are padlockable in the "off" position.
- Switches should have door interlocks to prevent the door from opening with the handle in the "on" position.
- Avoid locating the transfer switch at a meter or service entrance outdoor location. Switches are not service entrance rated unless they are fusible, and with this scenario the total house load is transferred to the genset. This method requires a much larger switch and cannot be taken out of service without de-energizing the entire dwelling.

Automatic transfer switches are much more expensive than manual transfer switches and require an electrical start option for the generator/battery set. These switches are usually not cost effective for homeowner generator/battery set installations but may, in certain applications involving life safety issues, warrant the added expense.

Automatic transfer switches automatically start the generator/battery set upon loss of regular power and transfer the emergency load to the generator/battery source. After power has been restored for some time, the transfer switch automatically transfers back to normal power source. The generator set continues to run for some time unloaded until the set has cooled down, then it shuts off. The designer should contact the manufacturers for specific applications data for these automatic transfer switch devices.

Step 6: Select emergency panel size.

Equipment and appliances that need to be powered by a generator/battery set are typically wired in an emergency panel box. The design of the emergency panel box should be conducted with the following considerations in mind:

- Select branch circuit loads for emergency operation.
- Size branch circuit over current devices in emergency panel to protect equipment and conductor feeding equipment. Appliance circuits and motor loads should be sized in accordance with NEC requirements.
- Size panel bus based upon NEC requirements and on continuous rating at 125% calculated load for items that could operate over three hours.
- Verify panel box size vs. number and size of circuit breakers.

- See Tables VI-D5 and VI-D6 for minimum panel bus sizes and emergency panel specification criteria.

Table VI-D5 **Minimum Panel Bus Sizes**

AMPACITY	POLE SPACES
30	2
70	2
100	6-8
125	12-24

Table VI-D6 **Emergency Panel Specification Criteria**

- Load center type residential panel
- Main lug
- Indoor NEMA 1 enclosure above flood protection level with isolated neutral for sub panel application
- Same short circuit current rating as main panel with ground bar kit
- Pole spaces as required for appliance and motor circuit breakers

At this point, the designer should confirm several items with the homeowner including:

- emergency panel location above flood protection level
- transfer switch location above flood protection level
- no load transfer switch operation

Step 7: Design wire conductor and raceway ground system.

Select route for wiring between panel, transfer switch, and generator set and specific wiring materials in accordance with local electric codes or NEC.

Operation and Maintenance Issues: The following instructions should be provided to the homeowner with generator equipment.

For manual start generators, operating procedures include:

1. Turn off or disconnect all electrical equipment including essential equipment in emergency panel. CAUTION: Make sure solid state appliances remain off while standby power is operating.

2. Connect generator to receptacle.

3. Place transfer switch in generator position.

4. Start generator and bring it up to proper speed (1800 rpm or 3600 rpm). Check generator volt meter; it should read 115-125 volts; the frequency meter should read 60 Hz plus or minus three hertz.

5. Start the motors and equipment individually, letting the genset return to normal engine speed after each load has been applied. The load should be applied in the sequence used to determine the genset size and generally with the largest motor load applied first. If the generator cuts out, turn off all the electrical equipment and restart.

6. *Check the volt meter frequently. If it falls below 200* volts for 240-volt equipment or 100 volt for 120-volt equipment, reduce the load by turning off some equipment.

> CAUTION: if problems occur, turn off existing panel circuit breaker feeding the transfer switch before investigating problems with faulty connections or wiring.

7. When normal power has been restored, turn off all the electrical equipment slowly, one load at a time. Turn off all emergency load, place transfer switch in normal load position, and turn electrical equipment back on.

8. Turn off genset circuit breaker. However, allow genset approximately five minutes to run for cool-down. Then turn off generator engine. Return generator to storage location.

For manual start generators, maintenance procedures include:

1. Operate generator at about 50% load monthly or bi-monthly to ensure reliability.

2. Check for fuel leaks.

3. Change engine oil per manufacturer's requirements.

4. Replace or use the fuel supply about every 30 to 45 days to prevent moisture condensation in the tank and fuel breakdown. Gasoline additives can keep gasoline-powered generator fuel from breaking down.

5. Keep tank full.

6. Replace air filter element per manufacturer's requirements.

CONSTRUCTION

All wiring shall be installed by licensed electricians to meet NEC requirements, local electrical regulations, and requirements of the local power company. Bond ground from generator emergency panel through transfer switch back to main service panel.

Elevation

Table of Contents

ELEVATION

One of the most common of all retrofitting techniques is to raise an entire existing superstructure above the desired flood protection elevation. When properly done, the elevation of a house places the living area above all but the most severe floods.

In general, the steps required for elevating a building are essentially the same in all cases. A cradle of steel beams is inserted under (or through) the structure; jacks are used to raise both the beams and structure to the desired height; a new, elevated foundation for the house is constructed; utility systems are extended and modified; and the structure is lowered back onto the new foundation and reconnected.

While the same basic elevation techniques are used in all situations, the final siting and appearance of the house will depend on the final elevation and type of foundation used. However, the actual elevation process is only a small part of the whole operation in terms of planning, time, and expense. The most critical steps involve the preparation of the house for elevation and the construction of a new, adequately elevated foundation. The elevation process becomes even more complex with added weight, height, or complex design or shape of the house. Brick or stucco veneers may require removal prior to elevation. Building additions may need to be elevated independently from the main structure.

TYPES OF RESIDENTIAL STRUCTURES THAT CAN BE ELEVATED

Figures VI-E1 through VI-E5 illustrate the elevation of a home on extended solid foundation walls. Subsequent figures for various elevation techniques will include only those illustrations unique to that technique.

The elevation of houses over a crawlspace; houses with basements; houses on piles, piers, or columns; and houses on a slab-on-grade are examined here. In each of these situations, the designer must account for multiple (non-flood-related) hazards, such as wind and seismic forces. The various methods utilized to elevate different home types are illustrated in the pages that follow, providing the designer with an introduction to the design of these measures.

Information on the design of foundation wall openings and adjustment of existing utility systems can be found in the Wet Floodproofing section of Chapter VI.

HOUSES OVER A CRAWLSPACE

These are generally the easiest and least expensive houses to elevate. They are usually one- or two-story houses built on a masonry crawlspace wall. This allows for access in placing the steel beams under the house for lifting. The added benefit is that since most crawlspaces have low clearance, most utilities (heat pumps, water heaters, air conditioners, etc.) are not placed under the home; thus the need to relocate utilities may be limited. Houses over a crawlspace can be:

- elevated on extended solid foundation walls (see Figures VI-E1 through VI-E5); or

- elevated on an open foundation such as masonry piers (see Figures VI-E6 through VI-E8).

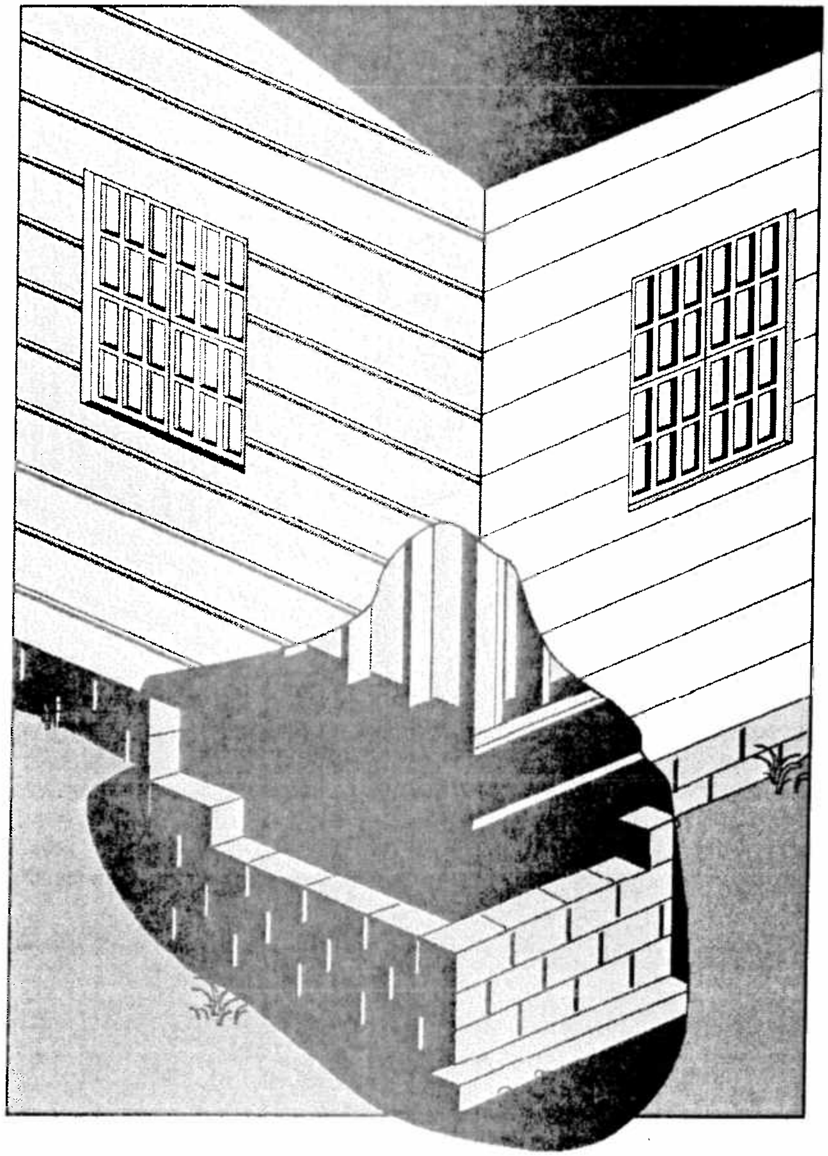


Figure VI-E1: Existing Wood-Frame Residence with Crawlspace

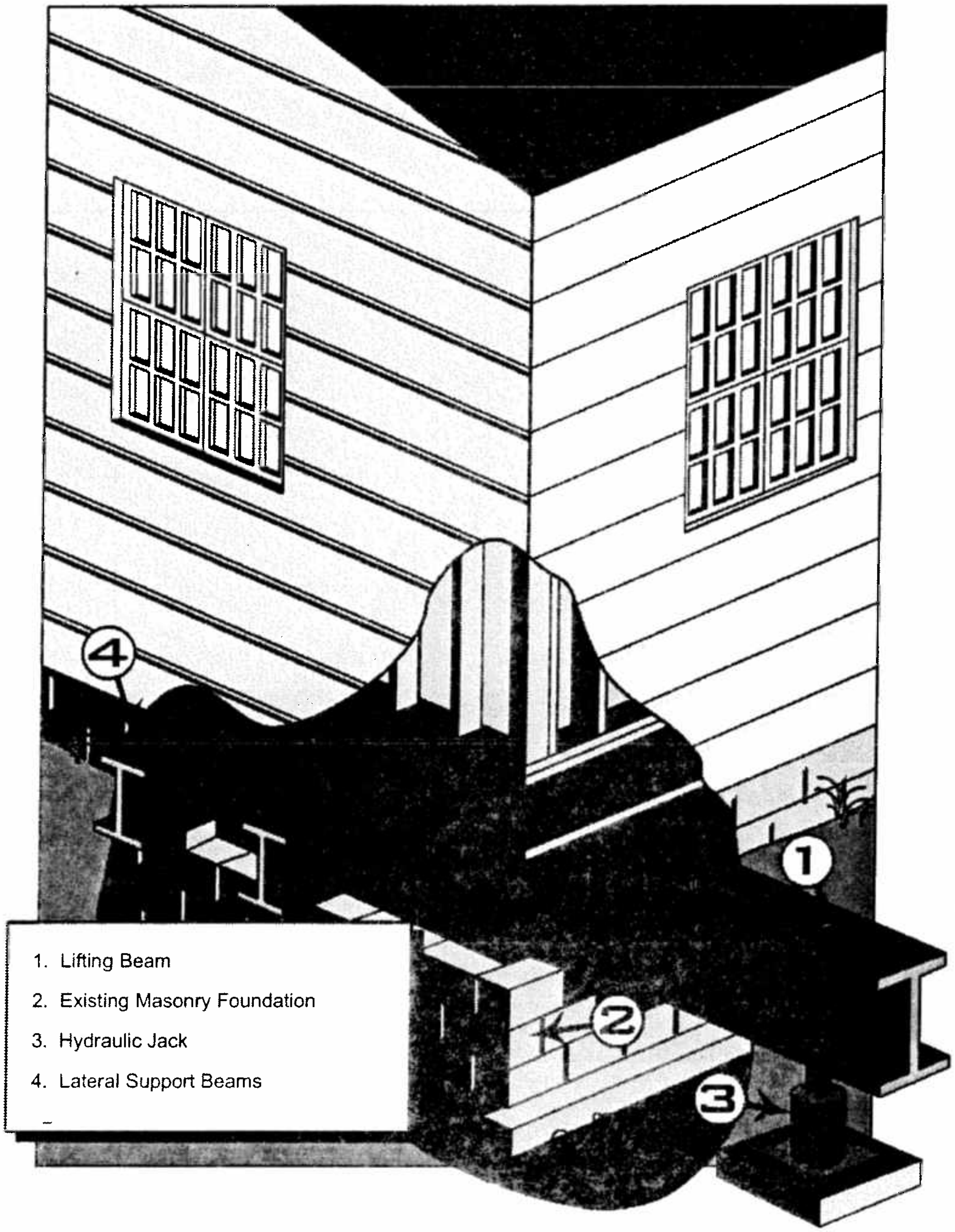


Figure VI-E2: Install Network of Steel "I" Beams

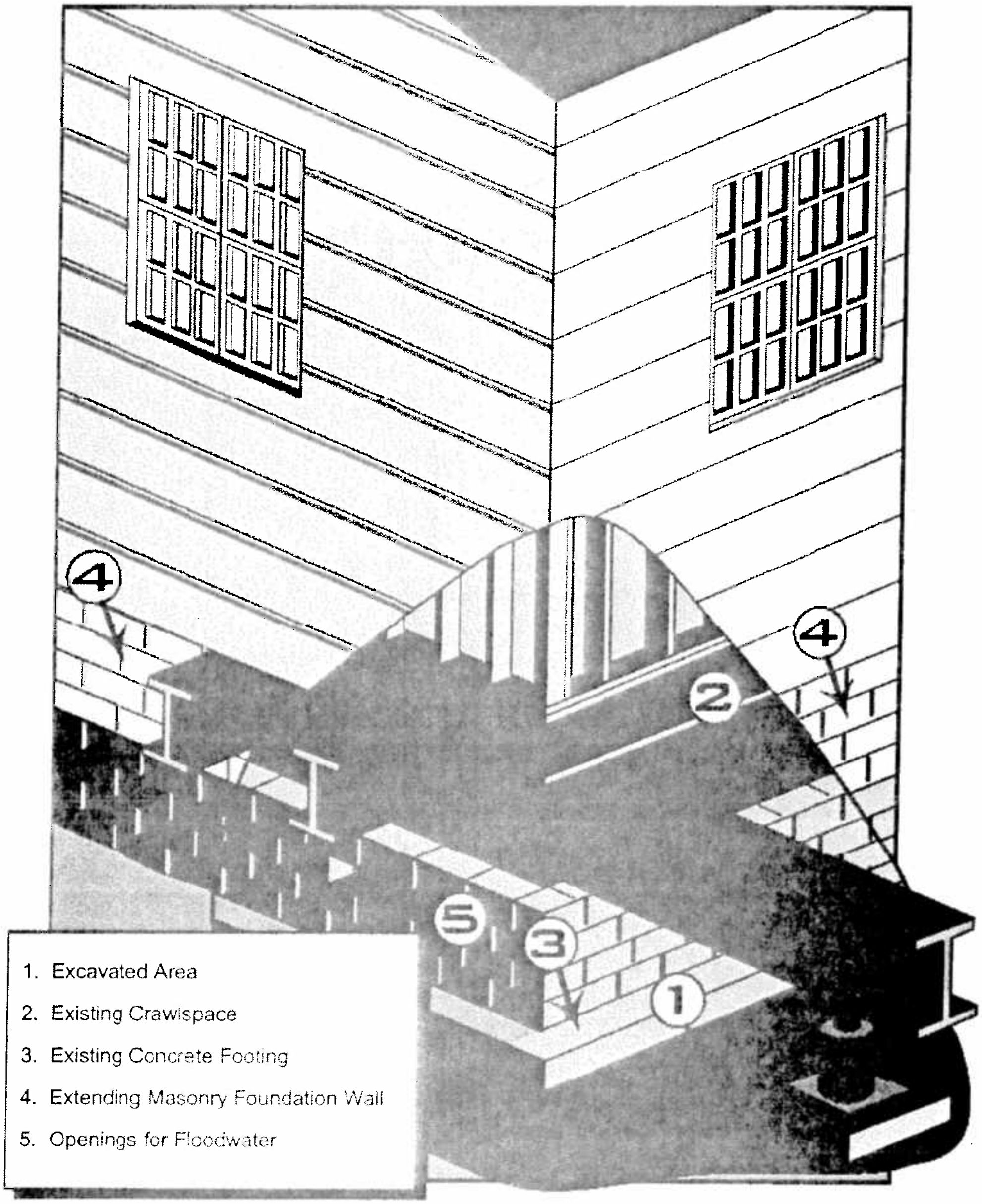


Figure VI-E3: Lift Residence and Extend Foundation Walls; Relocate Utility and Mechanical Equipment Above Flood Level

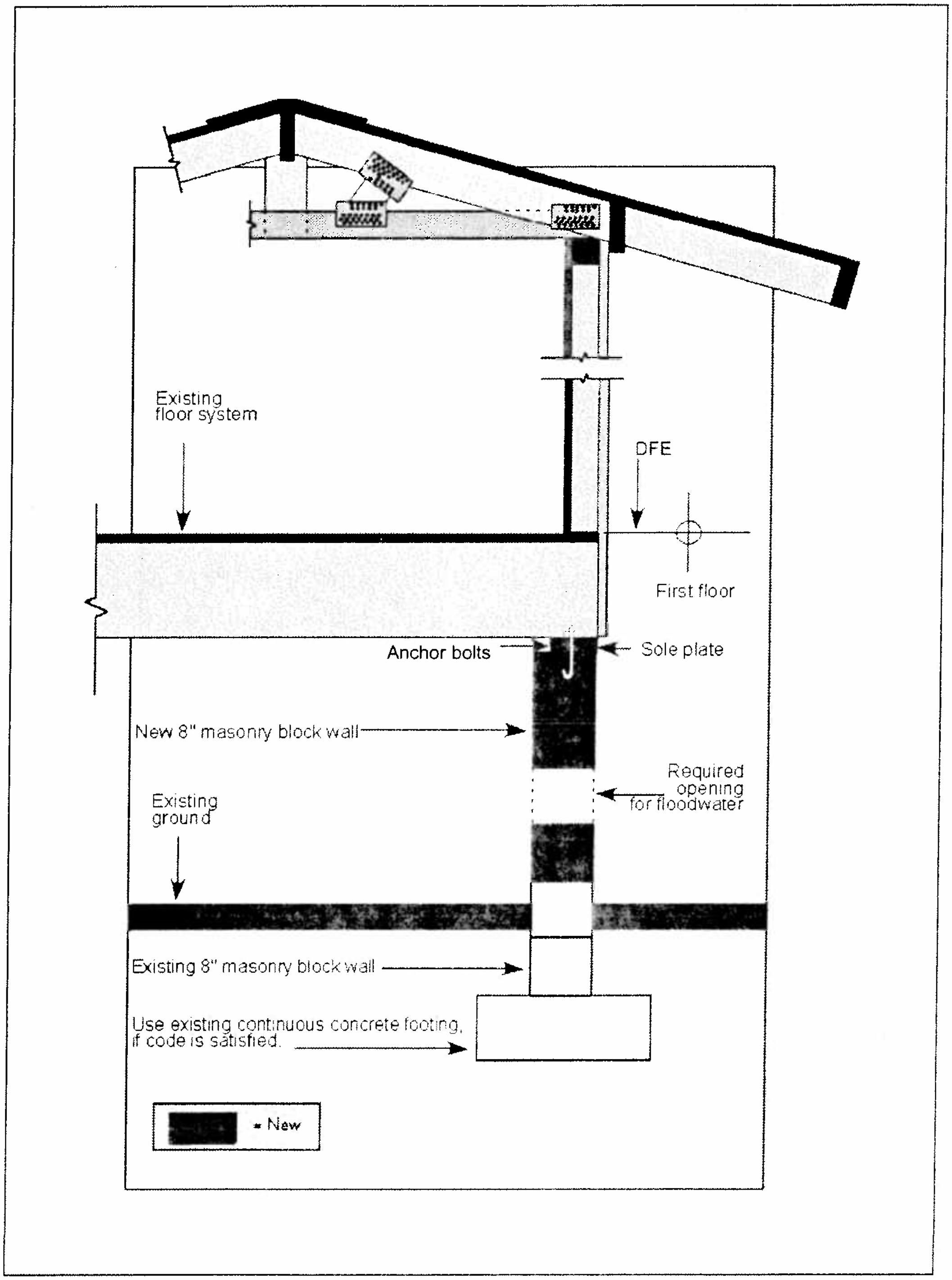


Figure VI-E4: Raising a Wood-Frame-Over-Crawlspace Structure

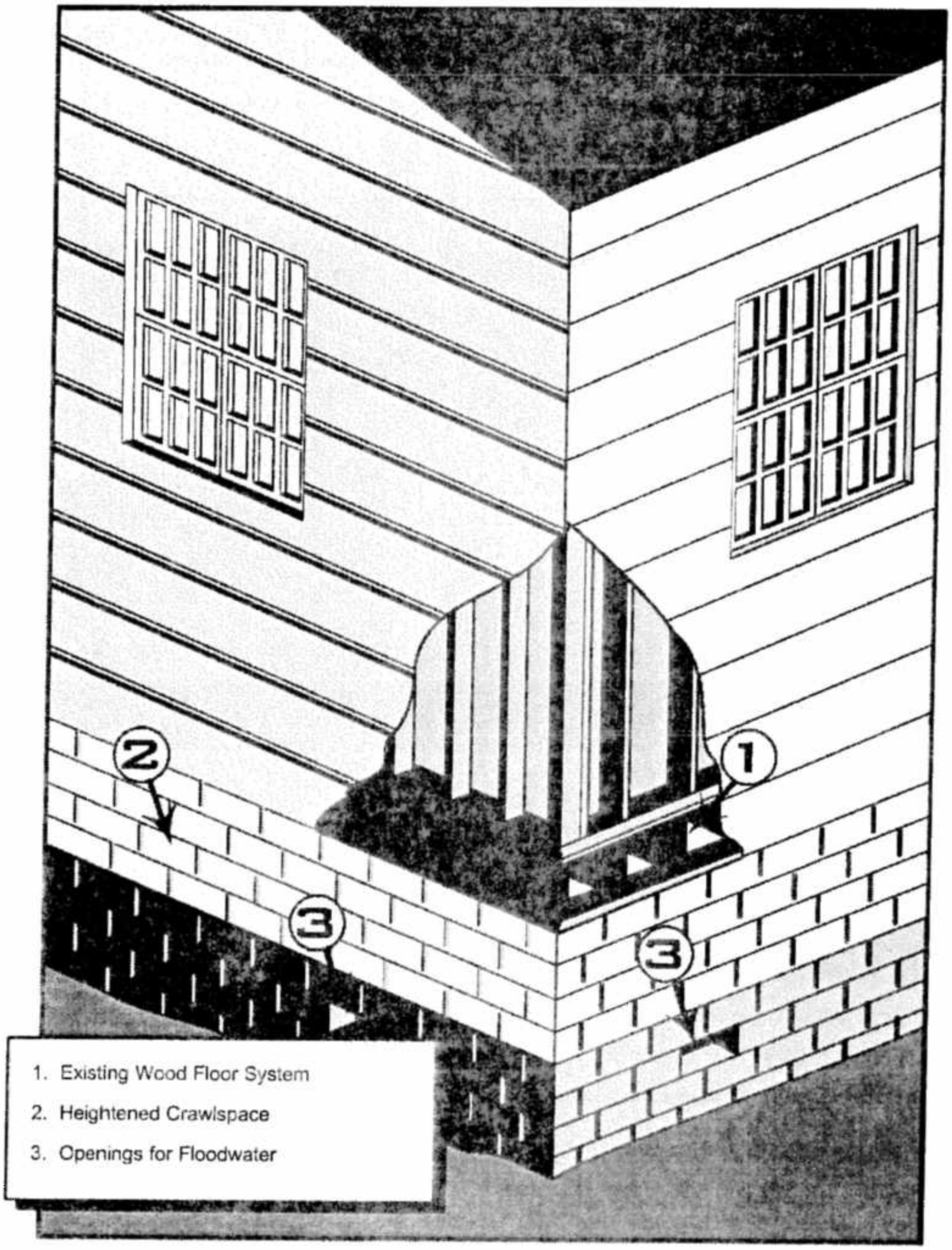


Figure VI-E5: Set Residence on Extended Foundation and Remove "I" Beams

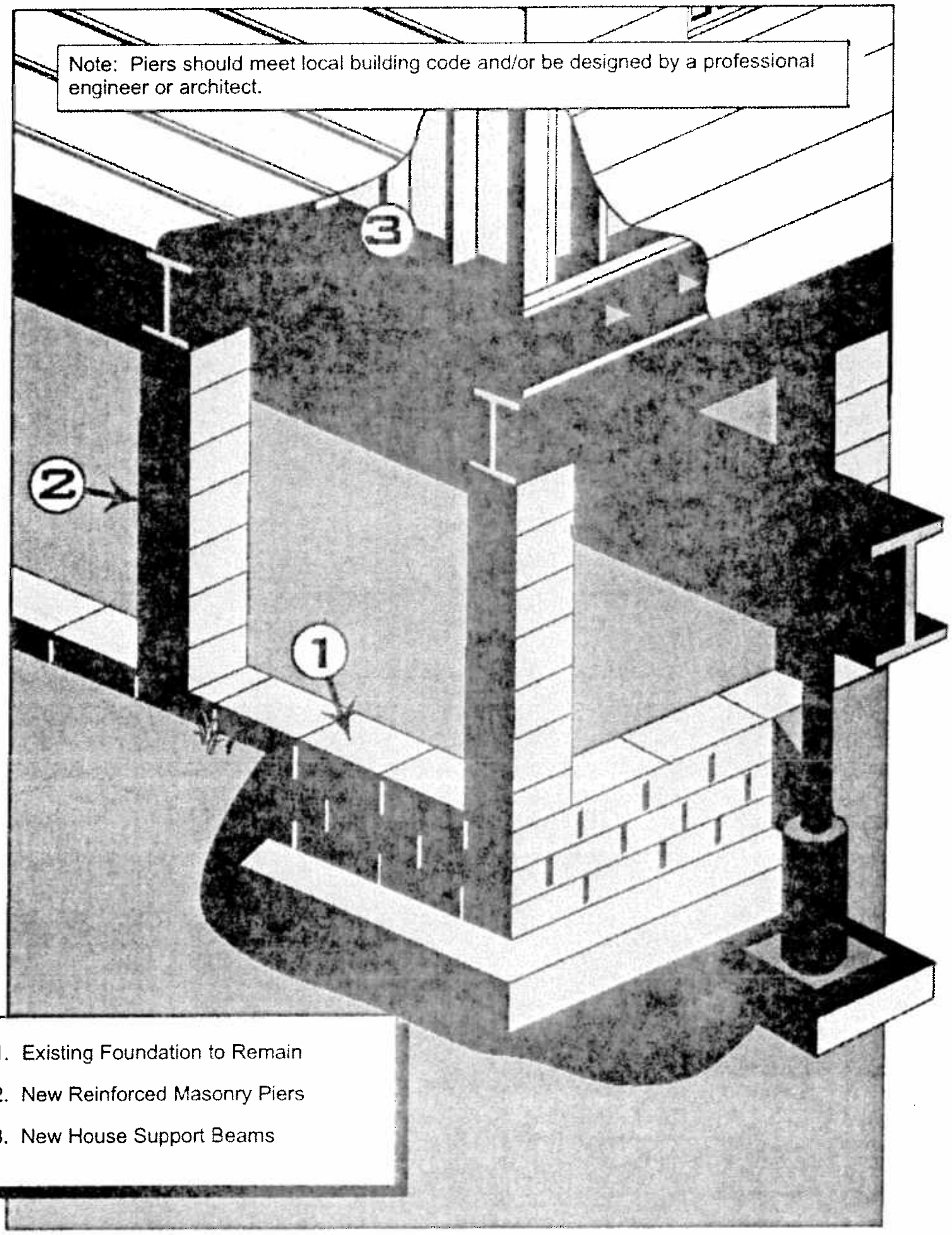


Figure VI-E6: Install Network of Steel "I" Beams

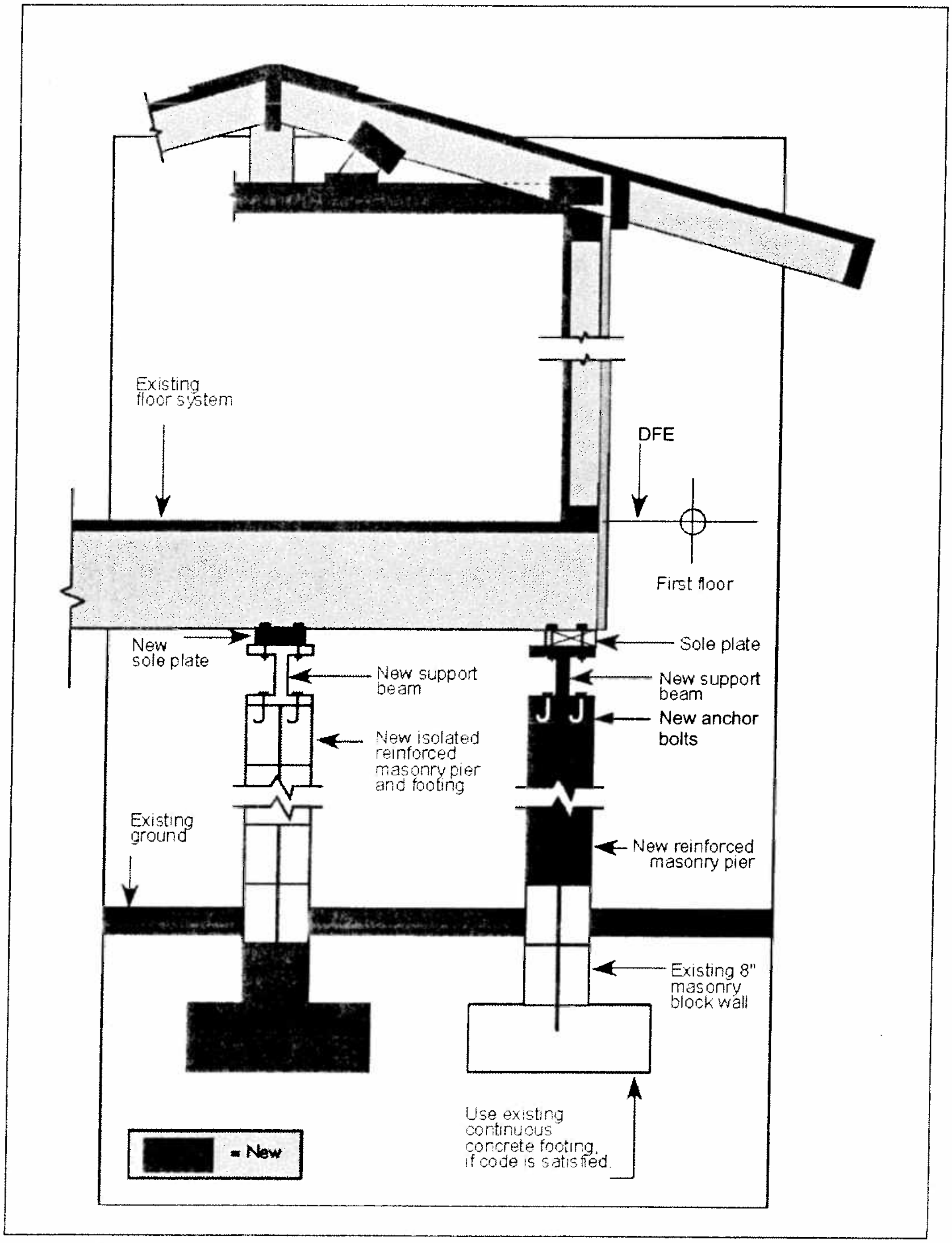


Figure VI-E7: Raising a Wood-Frame-Over-Crawlspace Structure on Piers

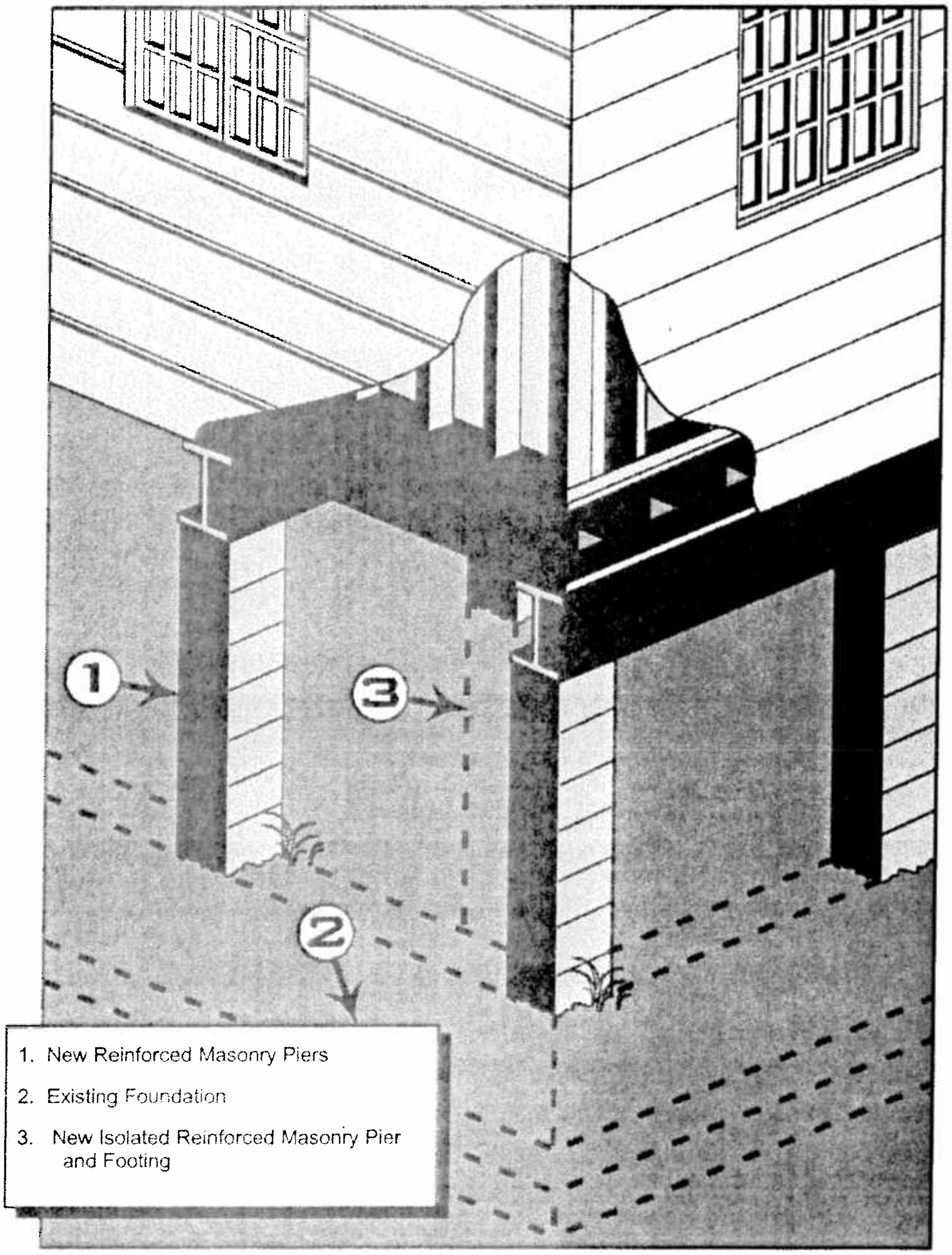


Figure VI-E8: Set Residence on Reinforced Piers

HOUSES OVER BASEMENTS

> FEMA's post- and pre-FIRM requirements do not allow basements below the Base Flood Elevation (BFE) for substantially damaged/improved and post-FIRM applications. For more information on what retrofitting measures are allowable under FEMA guidelines, refer to Chapter II, Regulatory Framework.

These houses are slightly more difficult to elevate because their utilities are usually in the basement. In addition, basement walls may have been extended to the point where they cannot structurally withstand flood forces. Houses over basements can be:

- elevated on solid foundation walls by creating a new masonry-enclosed area on top of an abandoned and filled-in basement (see Figures VI-E9 through VI-E10); or
- elevated on an open foundation, such as masonry piers, by filling in the old basement (see Figures VI-E11 and VI-E12).

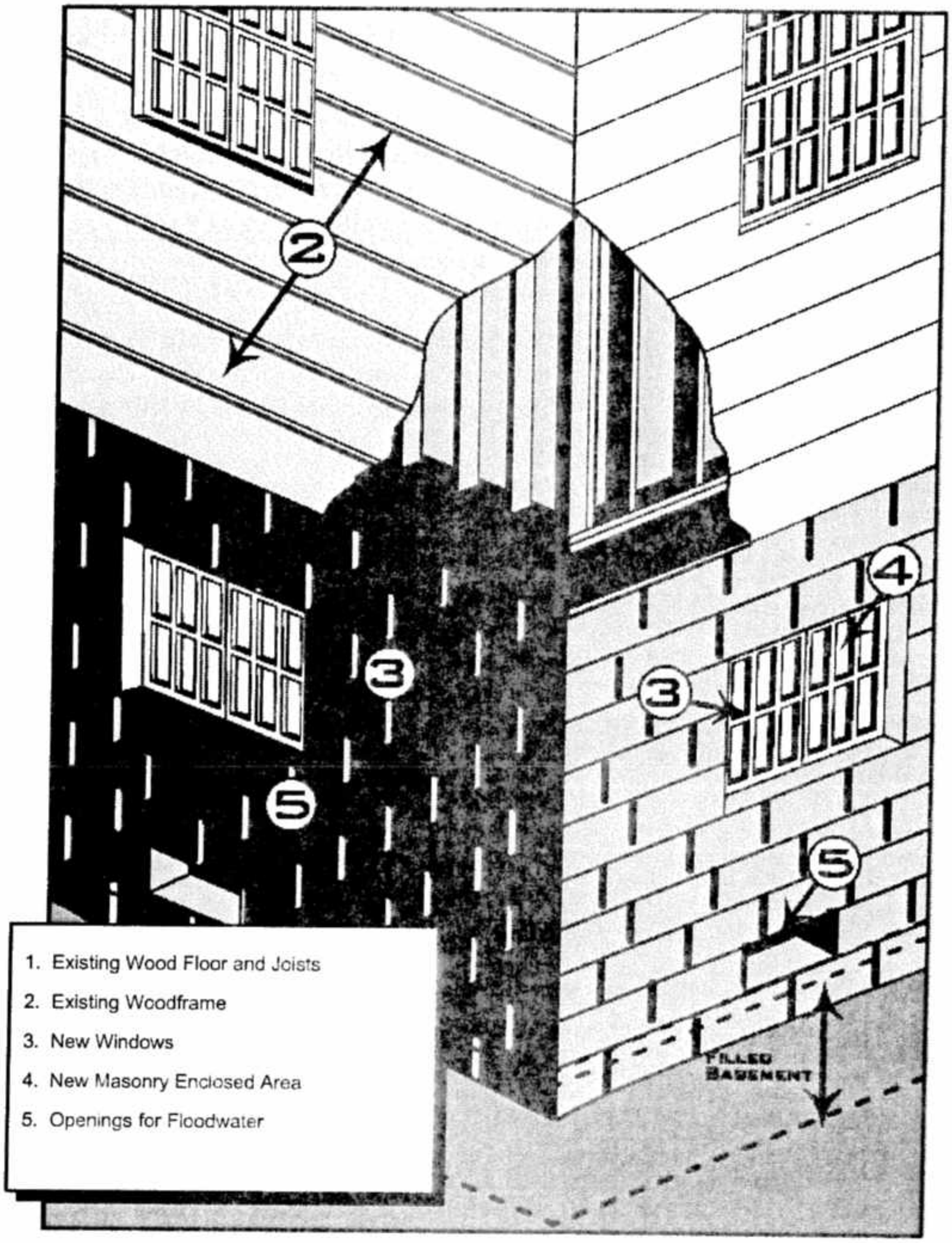


Figure VI-E9: Relocate Utility and Mechanical Equipment Above Flood Level

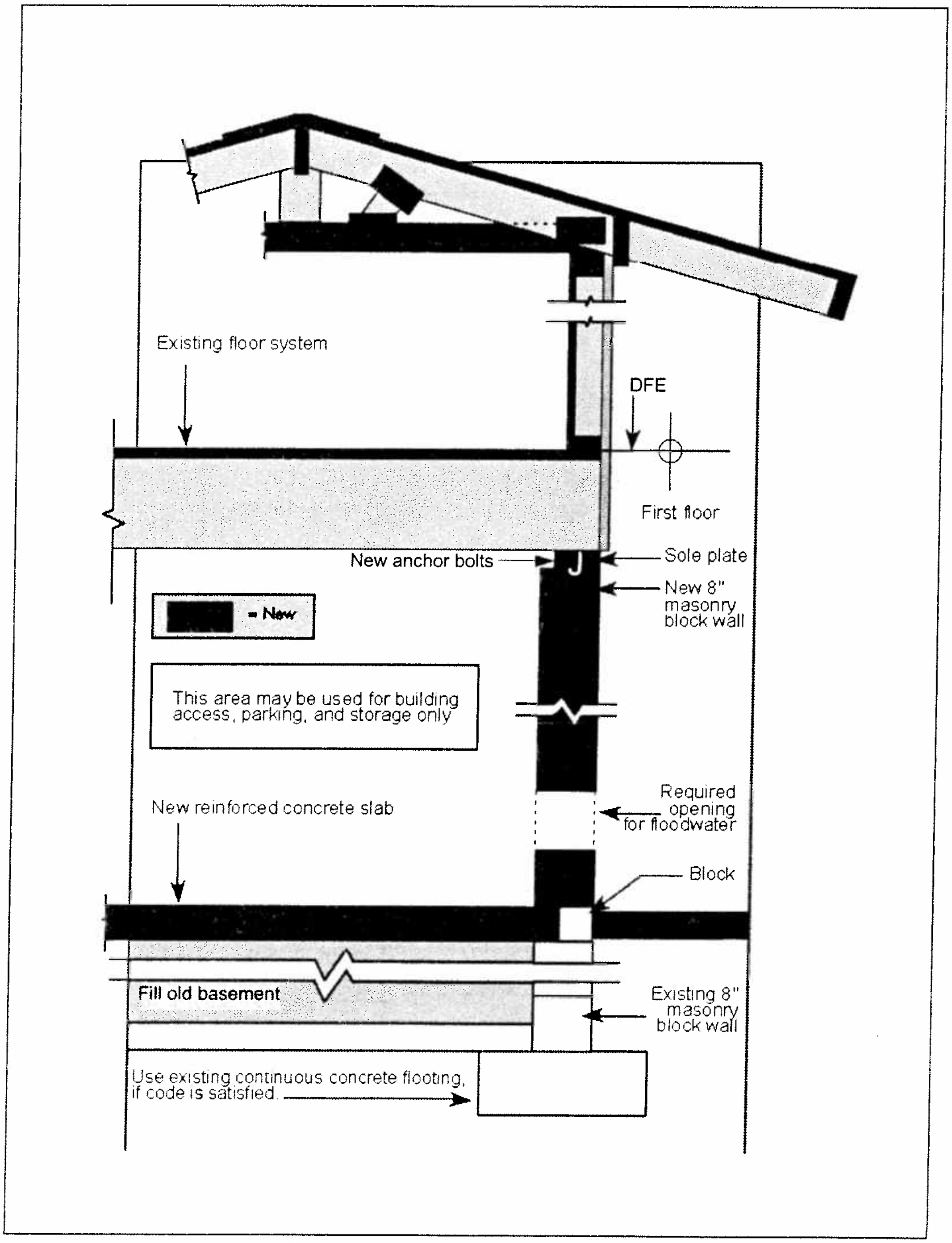


Figure VI-E10: Creation of a New Masonry Enclosed Area on Top of an Abandoned Basement

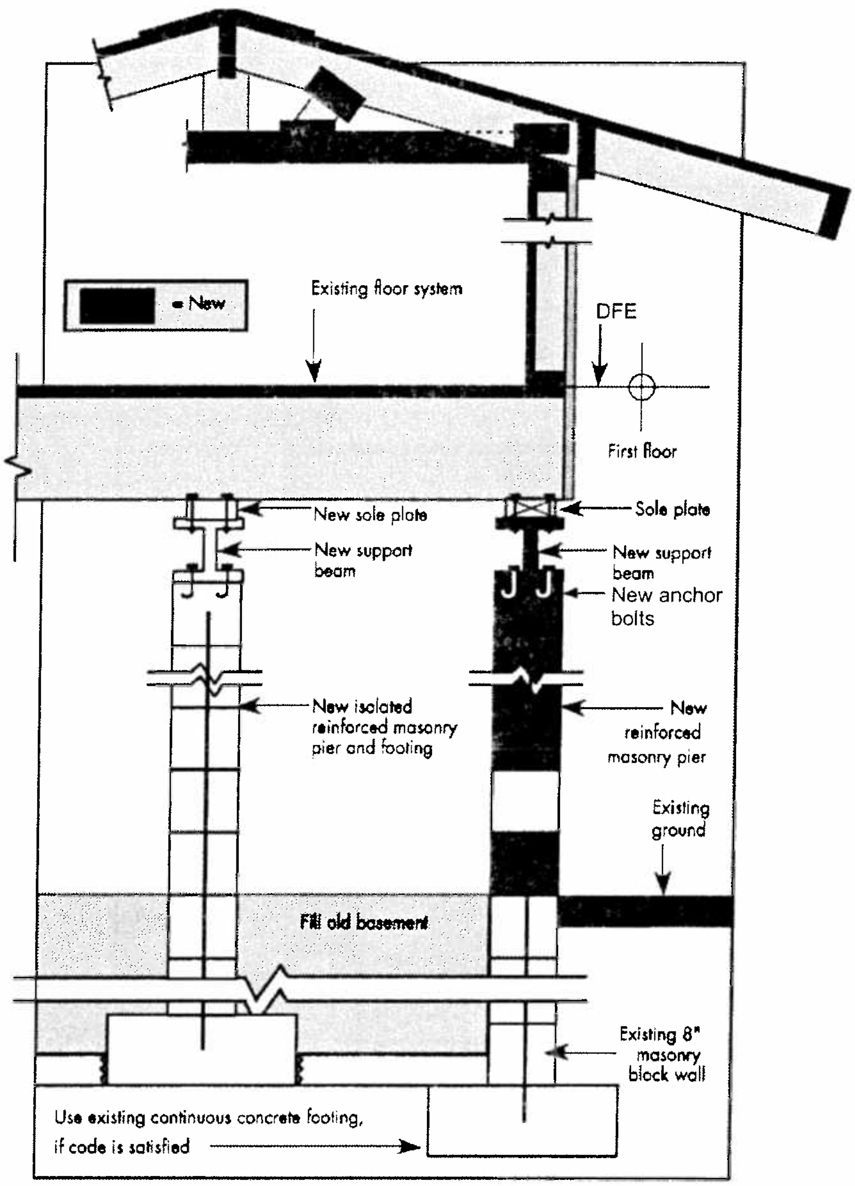


Figure VI-E11: Creation of a New Masonry Enclosed Area on Top of an Abandoned Basement (Piers)

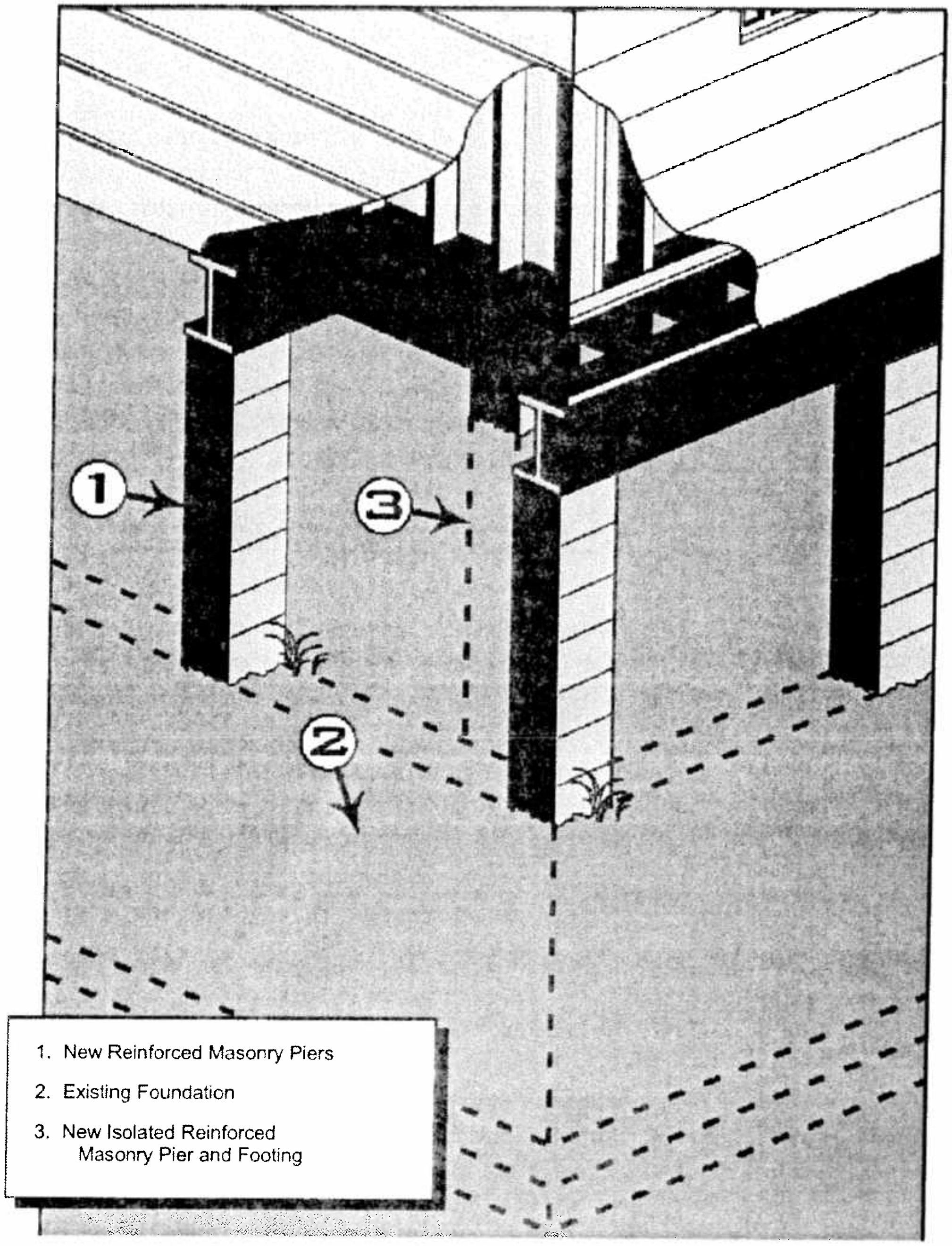


Figure VI-E12: Set Residence on Reinforced Piers

HOUSES ON PILES, PIERS, OR COLUMNS

The process of elevating a house on piles, piers, or columns is slightly more complex in that temporary relocation of the house may be part of the elevation process. With the use of this type of foundation, the house may need to be lifted off the existing foundation and temporarily relocated on-site. The existing foundation is then removed and/or reconstructed, and the house is reset on the new foundation. However, raising the home above the working area may provide sufficient room to auger pier and column foundations and to jet pile foundations.

SLAB-ON-GRADE HOUSES

These houses are the most difficult to raise in that if the slab is to be raised with the house, a trench must normally be dug under the house to provide a space for inserting lifting beams. However, intrusive techniques that place beams through the structural walls have proved to be successful in elevating slab-on-grade homes, as well. If the existing slab is to remain in place, then the house must be detached from the slab, the structure raised separately from the slab, and a new floor system built, along with an elevated foundation.

While slab-on-grade houses may be the most difficult to raise, a number of elevation options exist with regard to raising the structure with or without the slab and using a first floor composed of wood or concrete. The various alternatives include:

Many of the techniques that require interior home modifications are applicable only to structures that have suffered extensive interior damage. For additional information, refer to FEMA publications entitled *Technical Information on Elevating Substantially Damaged Residential Structures in the Midwest*, August 24, 1993, and *Technical Information on Elevating Substantially Damaged Residential Buildings in Dade County, Florida*, January 29, 1993.

Elevating a Slab-on-Grade Wood-Frame House

- Elevating a slab-on-grade wood-frame house without the slab, using a new first floor constructed of wood trusses (see Figures VI-E13 through VI-E17);

- Elevating a slab-on-grade wood frame house without the slab, using a new first floor constructed of a concrete slab on top of fill (see Figures VI-E18 through VI-E20);

- Elevating a slab-on-grade wood frame house with the slab intact (see Figures VI-E21 through VI-E23);

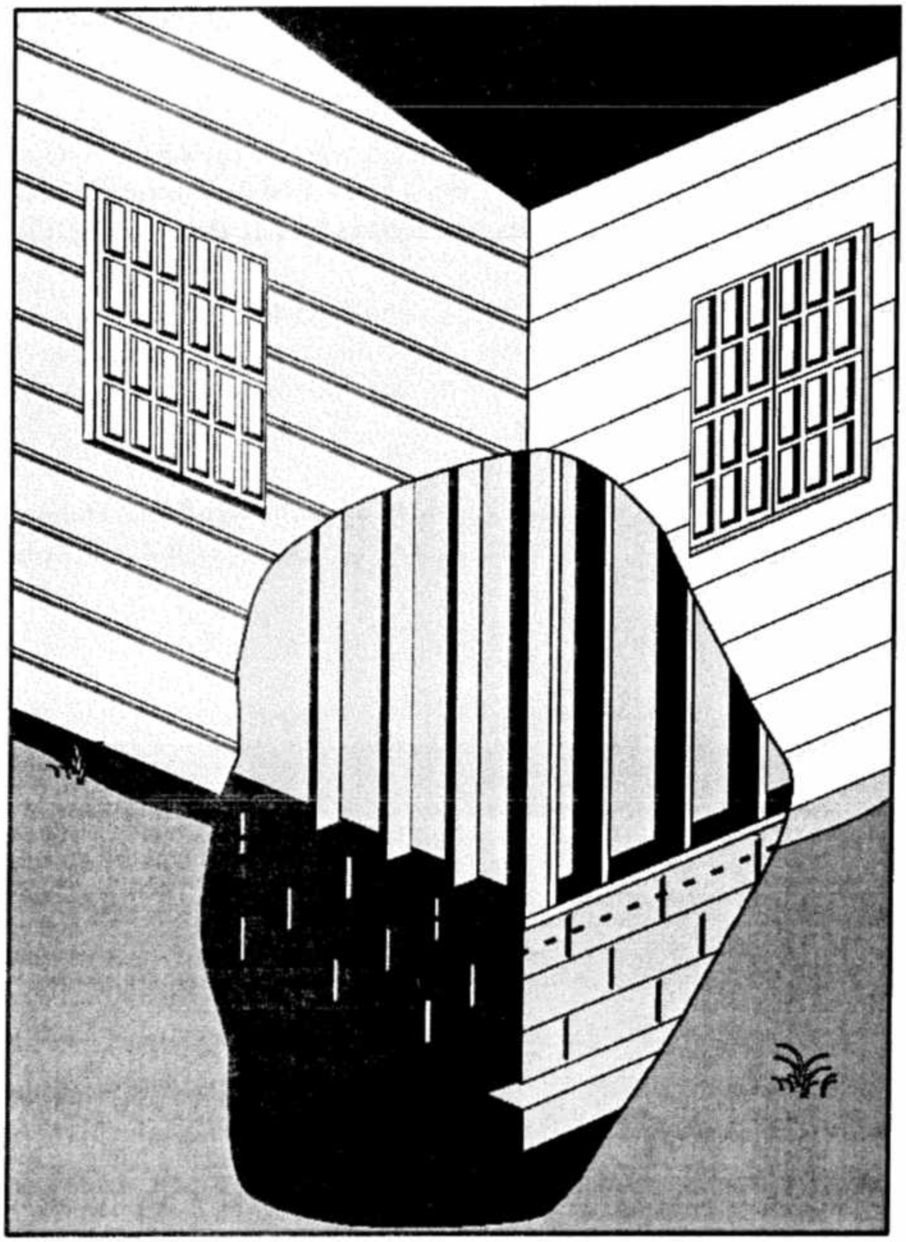


Figure VI-E13: Existing Slab-on-Grade Wood-Frame Residence

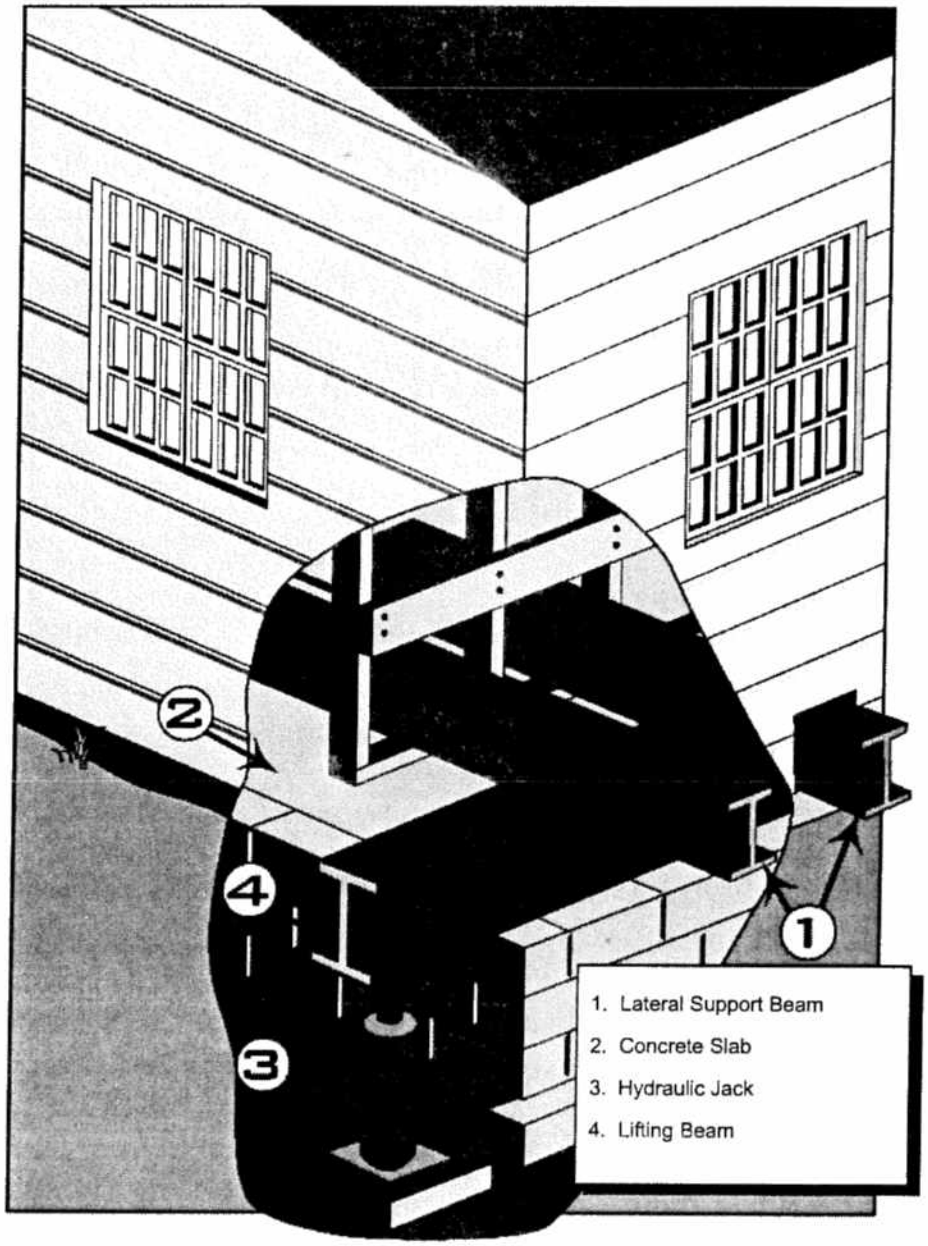


Figure VI-E14: Install Steel "I" Beam Network and Prepare to Lift Walls

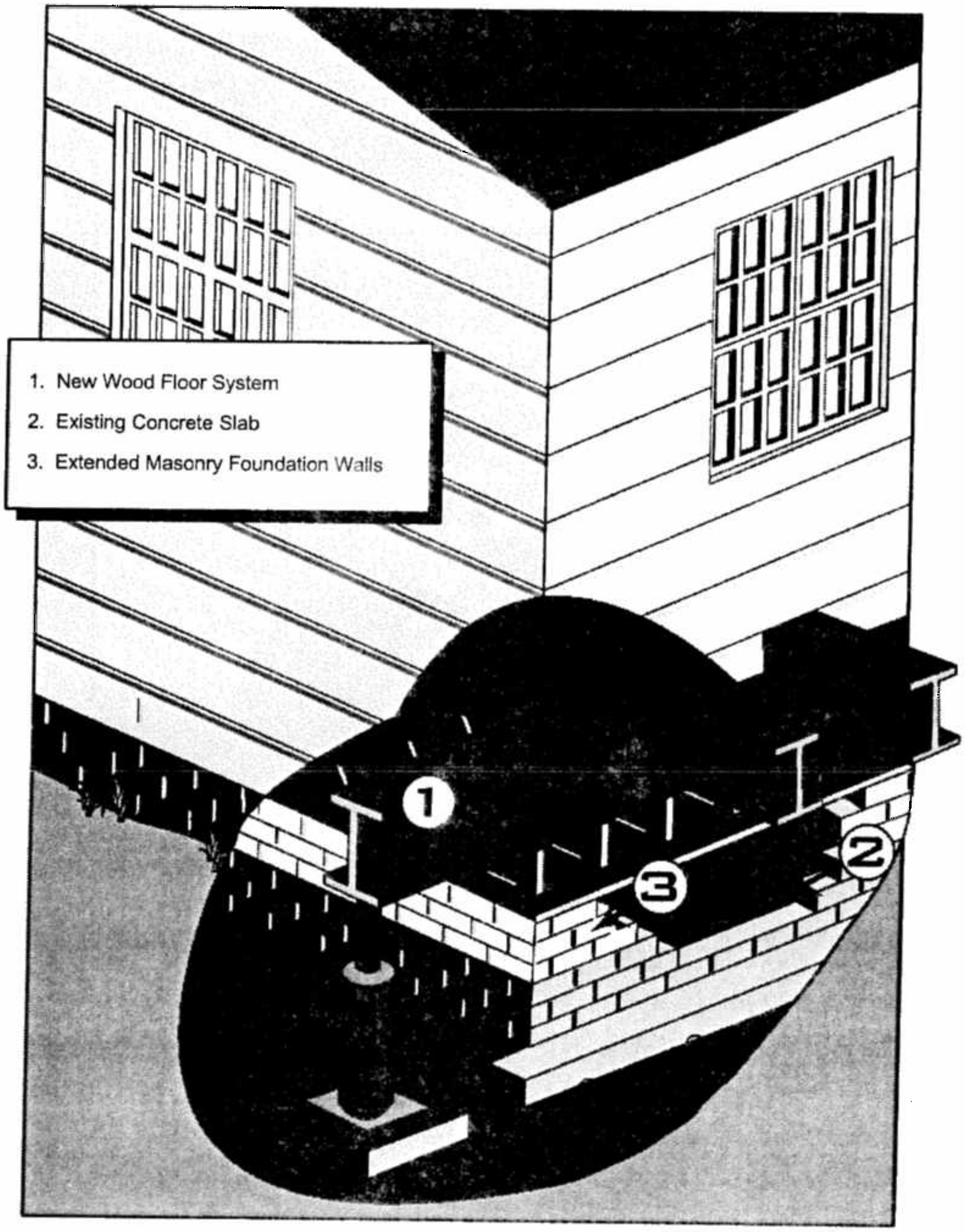


Figure VI-E15: Lift Residence and Extend Masonry Foundation Wall; Relocate Utility and Mechanical Equipment above Flood Level

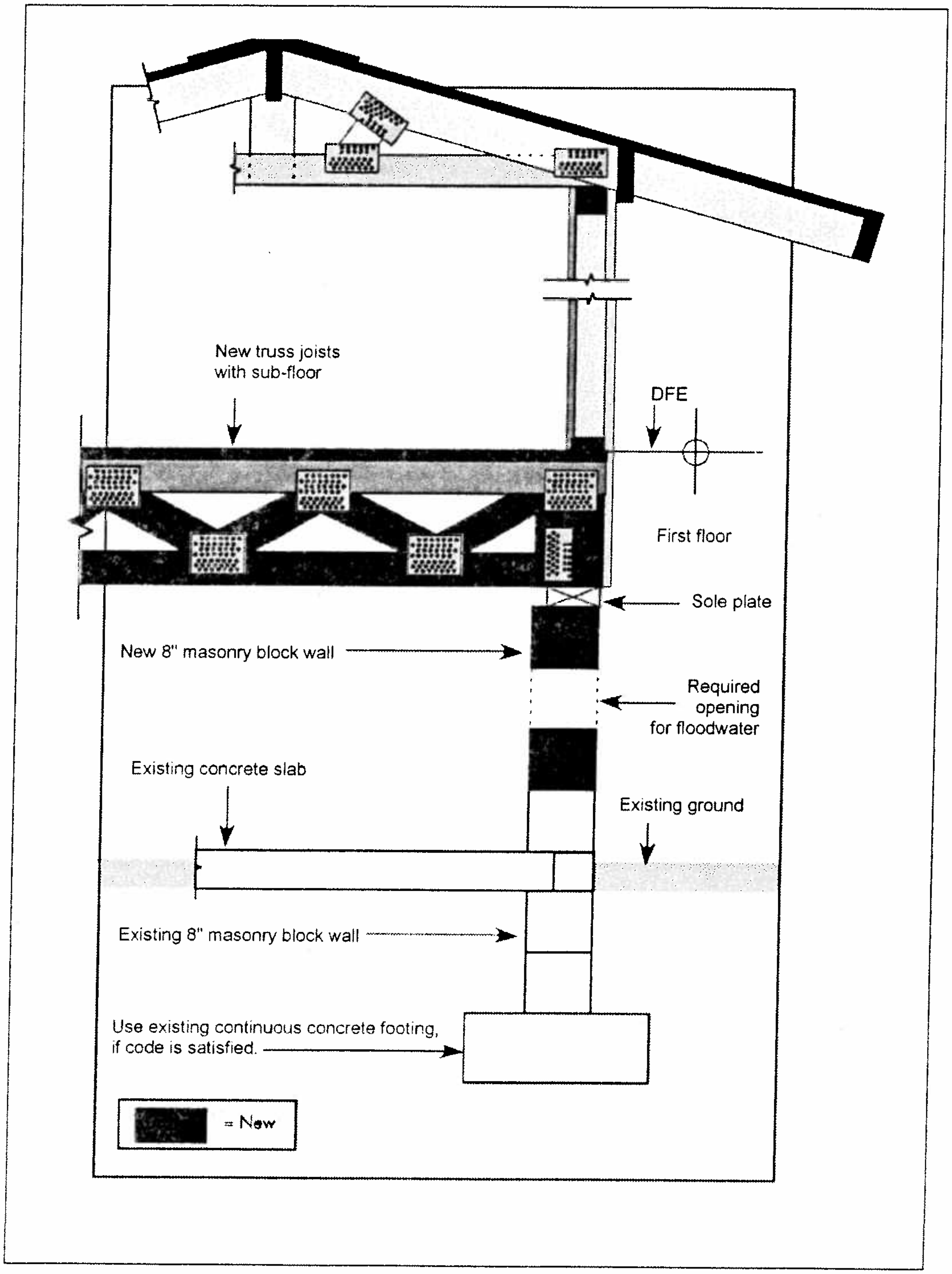


Figure VI-E16: Raising a Slab-on-Grade Wood-Frame Structure Without the Slab

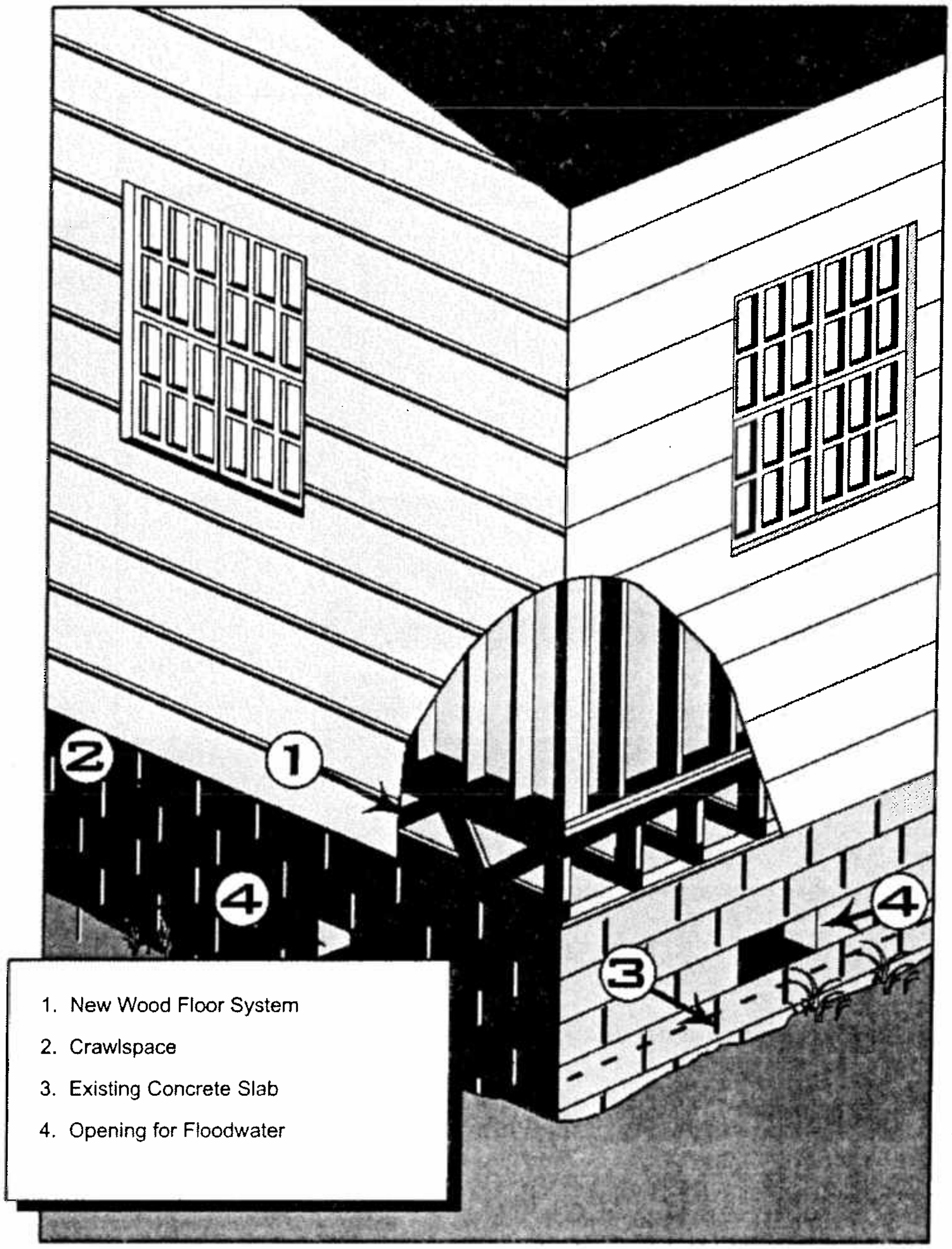


Figure VI-E17: Set Residence on New Foundation and Remove "I" Beams

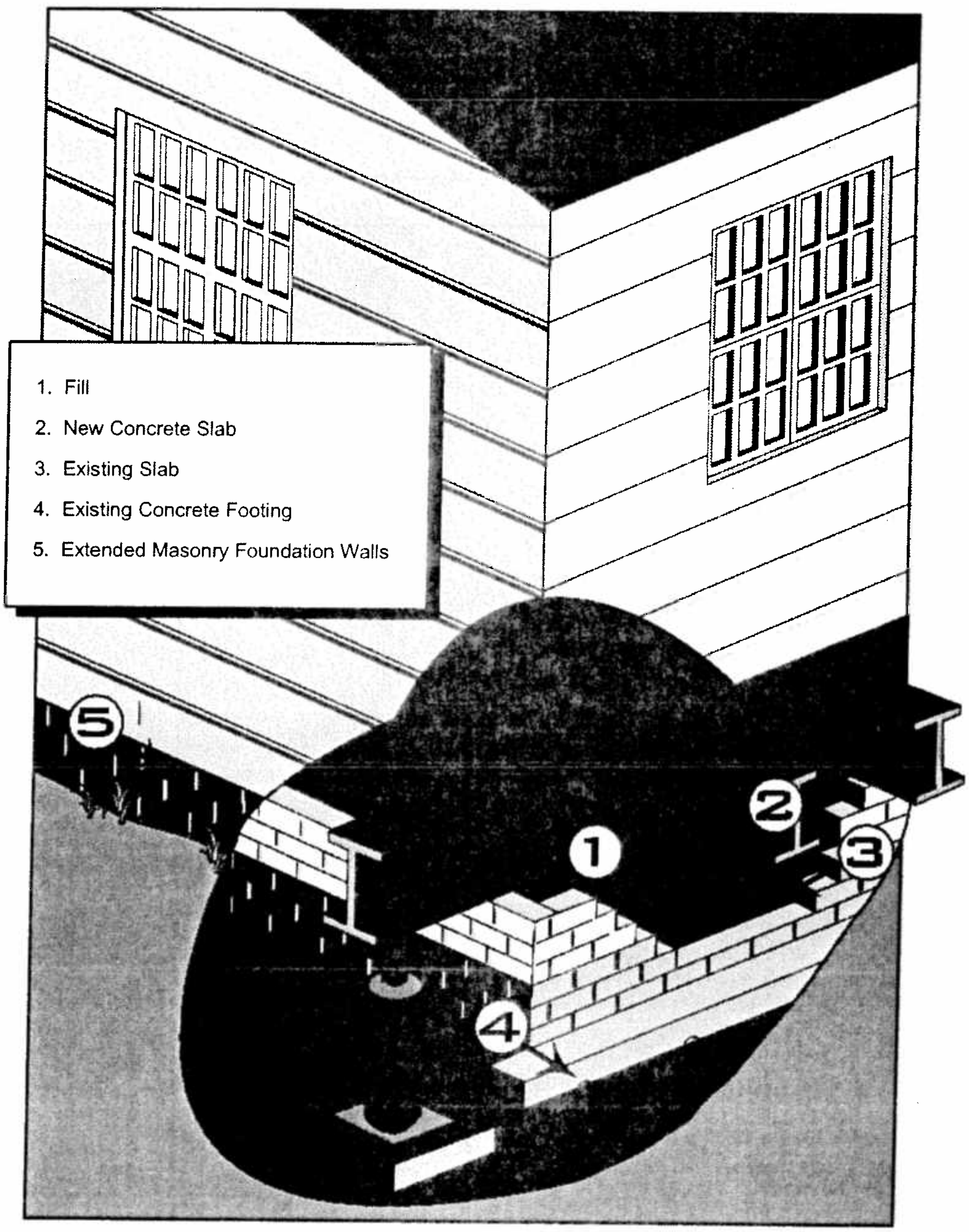


Figure VI-E18: Lift Residence and Extend Masonry Foundation Wall; Relocate Utility and Mechanical Equipment Above Flood Level

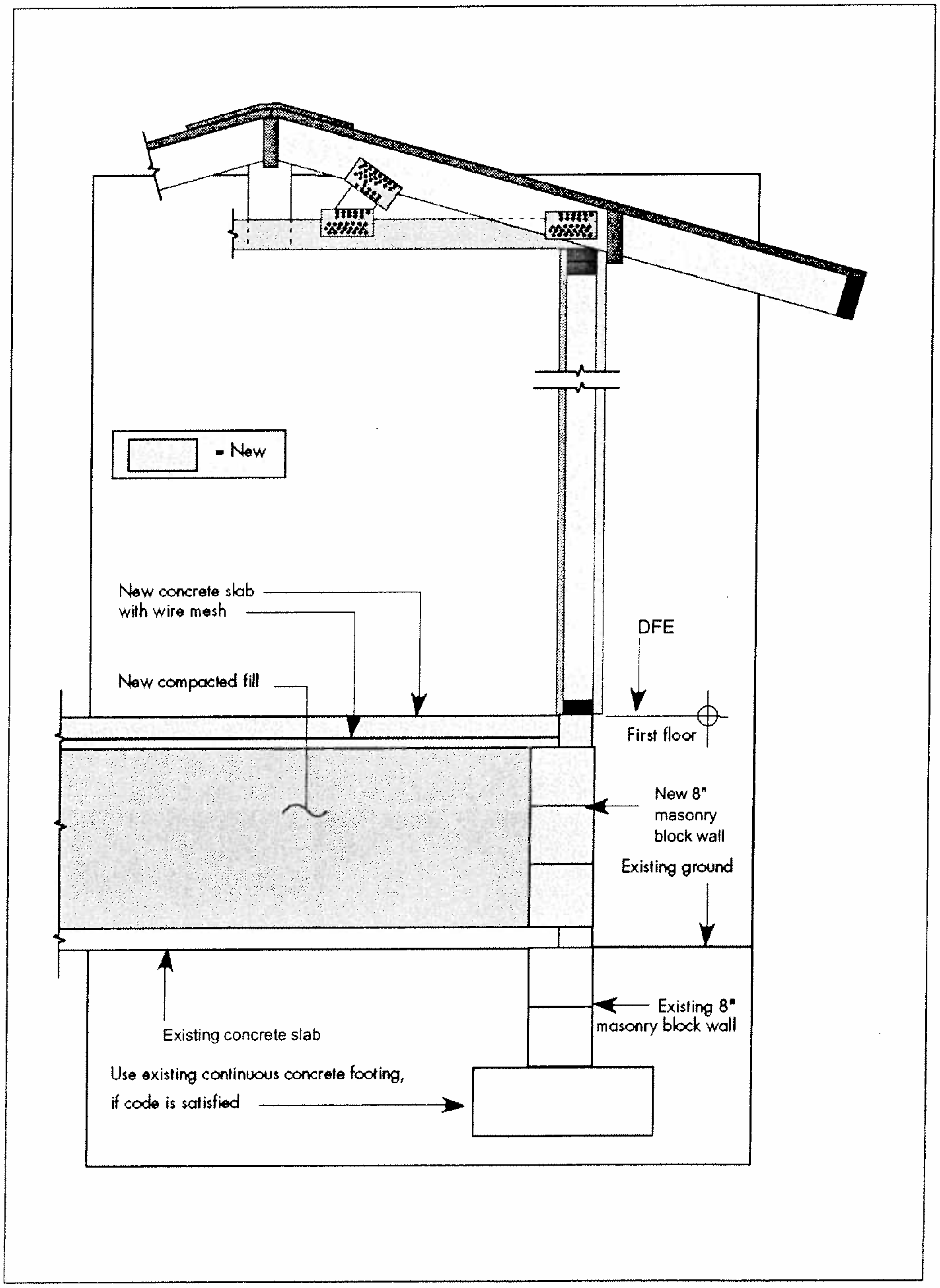


Figure VI-E19: Raising a Slab-on-Grade Wood-Frame Structure Without the Slab Intact

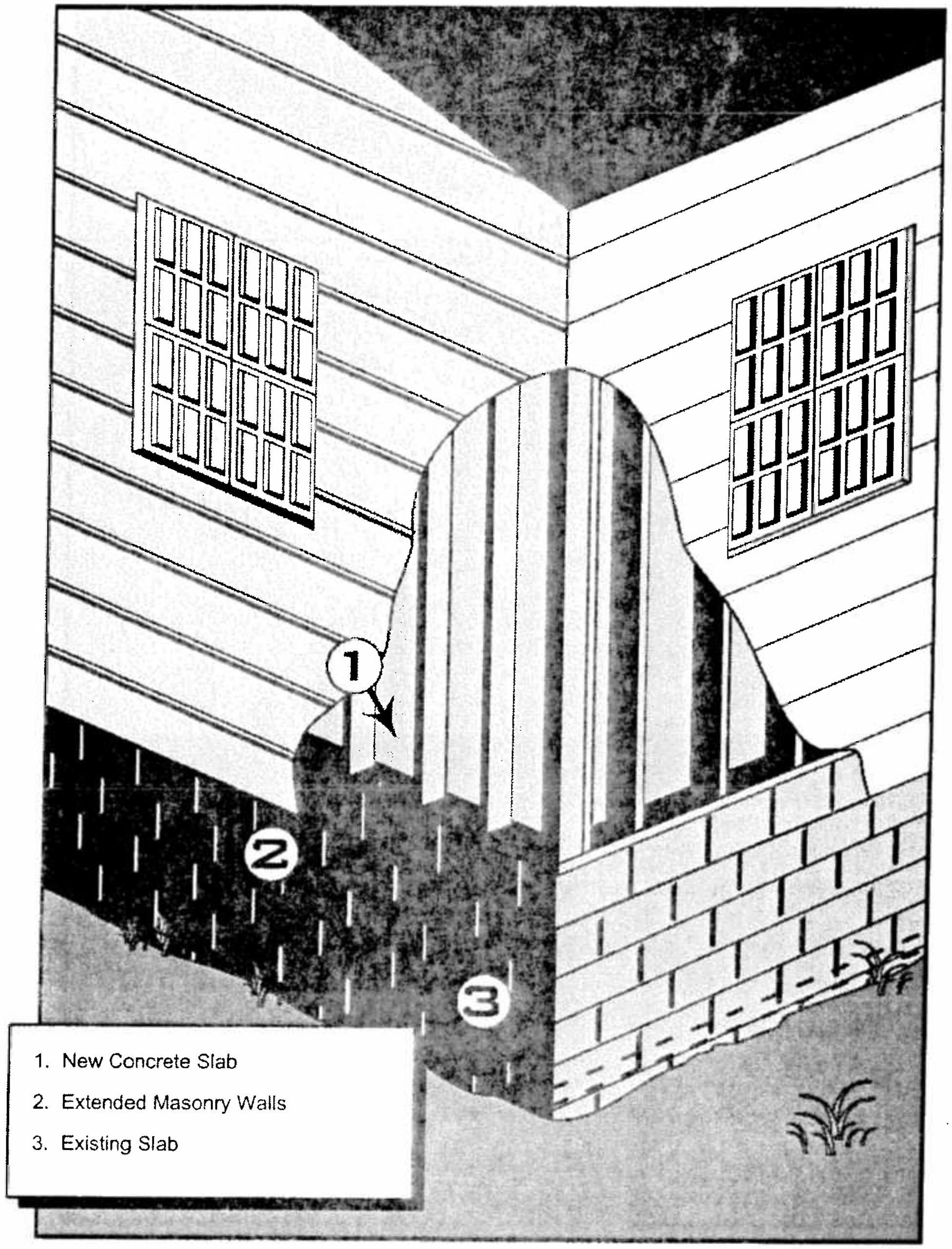


Figure VI-E20: Set Residence on New Foundation and Remove "I" Beams

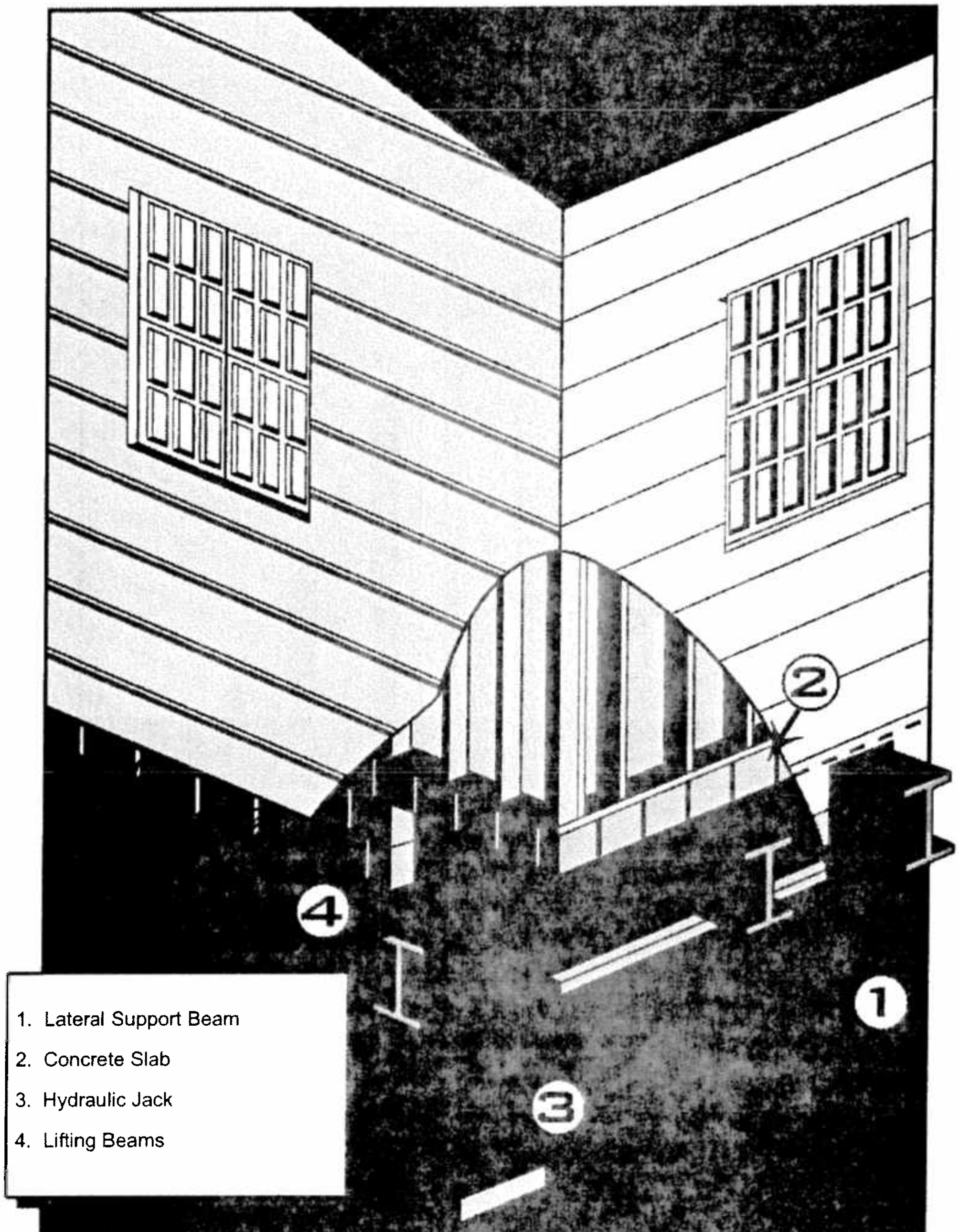


Figure VI-E21: Excavate Under Existing Slab and Install Network of Steel "I" Beams

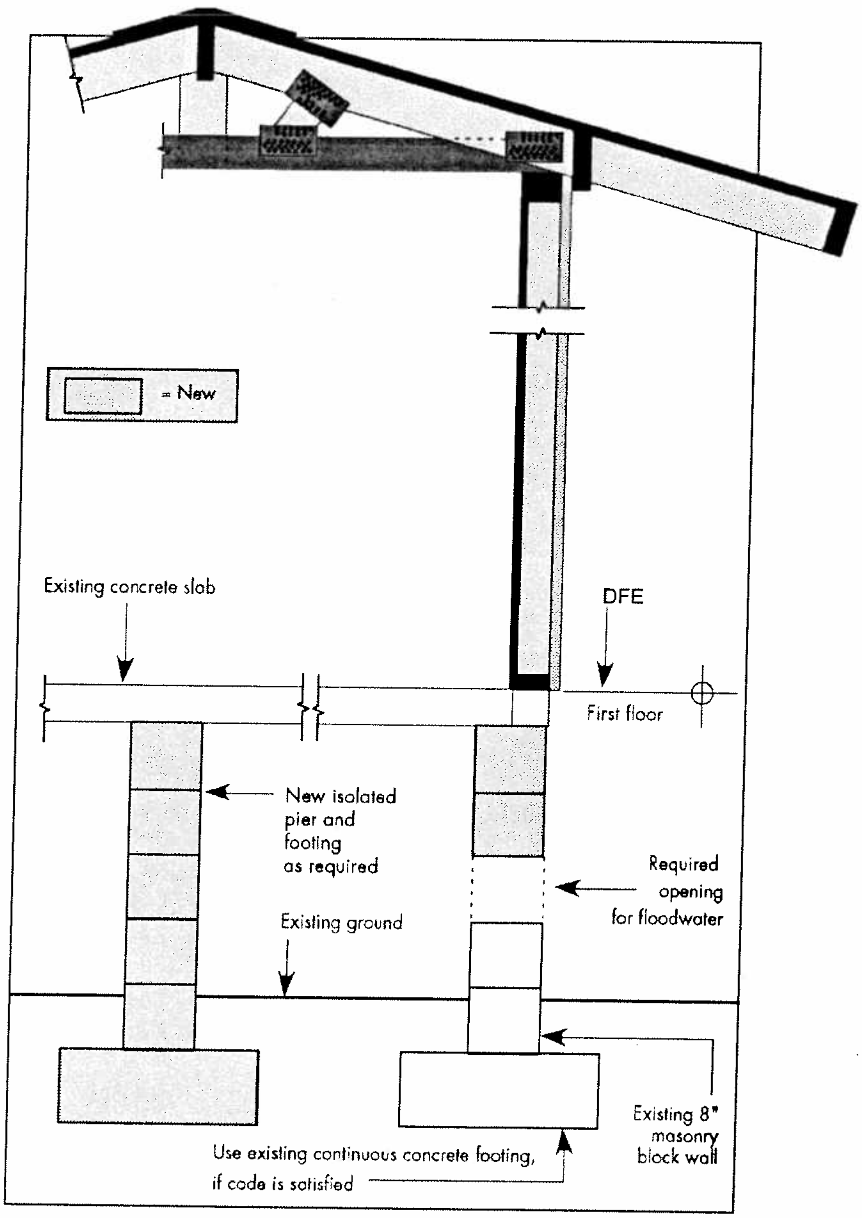


Figure VI-E22: Raising a Slab-on-Grade Wood-Frame Structure With the Slab

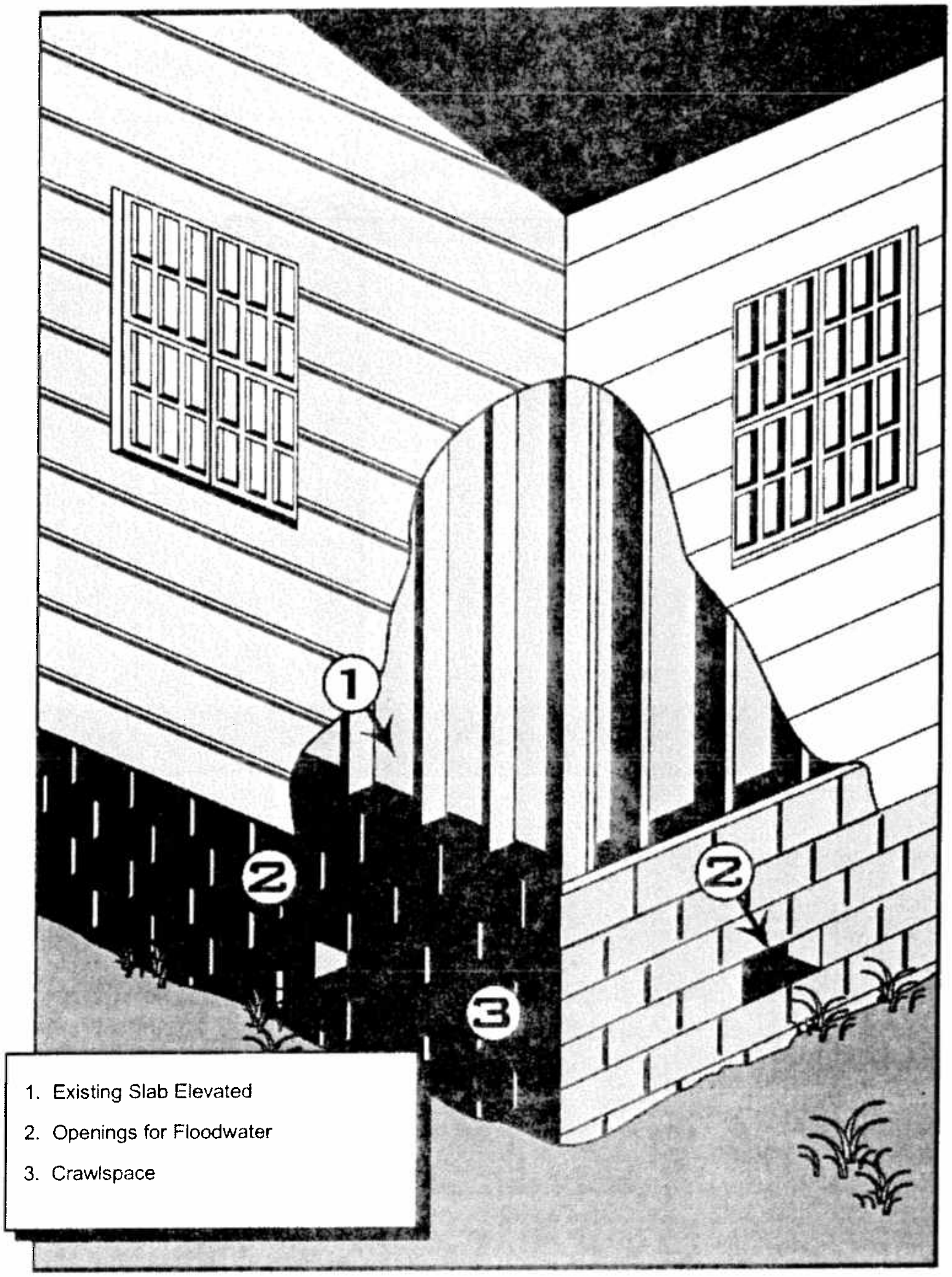


Figure VI-E23: Set Residence on New Foundation and Remove "I" Beams

Elevating a Slab-on-Grade Masonry Structure

- Elevating a slab-on-grade masonry structure with the slab intact;
- Elevating a slab-on-grade masonry structure without the slab using a first floor constructed of a concrete slab on top of fill;
- Elevating a slab-on-grade masonry structure without the slab using a first floor constructed of wood framing;
- Installation of an elevated concrete slab within an existing masonry structure;
- Installation of an elevated wood-frame floor system within an existing masonry structure;
- Creation of a new masonry livable area on top of an existing one-story masonry structure; and
- Creation of a new wood-frame livable area on top of an existing one-story masonry structure.

HEAVY BUILDING MATERIALS/ COMPLEX DESIGN

The elevation process becomes even more complex with added weight, height, or complex design of the house. Brick or stucco veneers may require removal prior to elevation. Combination foundations (i.e., slab-on-grade and basement) should be evaluated jointly and separately and the worst case scenario utilized for design purposes. Building additions may need to be elevated independently from the main structure. Due to the extreme variability of structural conditions, a structural engineer should evaluate the suitability of lifting this type of house.

The entire elevation design process is discussed here and then illustrated with a detailed example of the design for a crawlspace home (Figure VI-E24).

Elevation Design Process

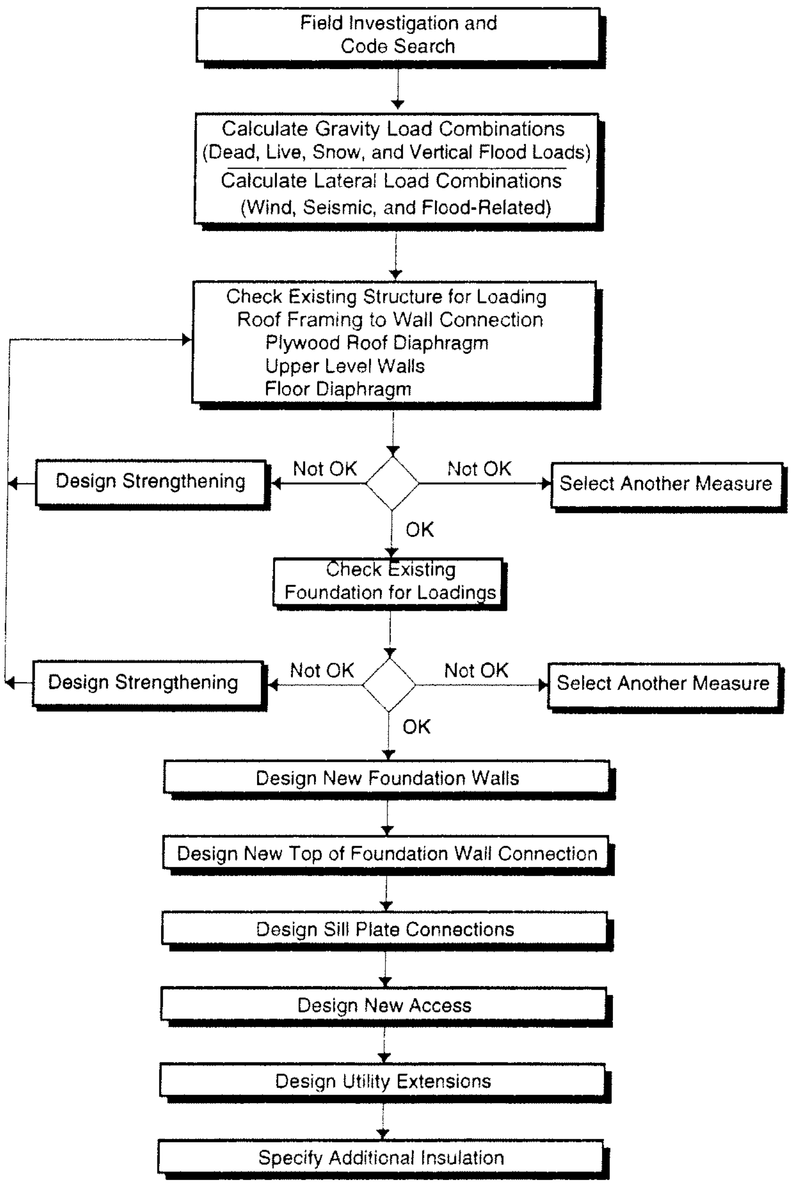


Figure VI-E24: Design Process for an Elevated Structure

FIELD INVESTIGATION CONCERNS

PROPERTY INSPECTION AND EXISTING DATA REVIEW

During the field investigation, the designer should inspect the property and review existing data to confirm the applicability of the selected alternative and to confirm specific design guidance such as the height of elevation and type of foundation to be utilized. The designer should utilize the guidance presented in the beginning of this chapter where detailed information and checklists for the collection of information on the Structural, Mechanical, Plumbing, and Electrical Systems was presented. Much of the data has been discussed previously in Chapters III and IV. At a minimum, the designer should collect information on the following checklist (Figure VI-E25).

CODE SEARCH

During the field investigation the designer should also conduct a search of local floodplain ordinances, building codes, restrictions to deeds, restrictions in subdivisions, zoning regulations, and state building codes. Included with this search, a visit with the local building official should be planned to determine any special requirements for the locality. During the code search, the following should be determined:

- floodplain ordinance;
- building code in effect;
- design wind speed;
- design seismic zone;

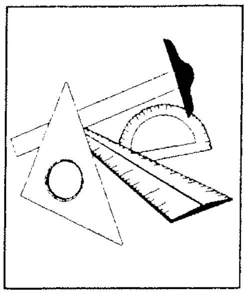
- ground snow loads;
- frost depths;
- restrictions on height (overall building, portions of building relative to materials in use, allowable height/ thickness ratios); and
- restrictions on foundations.

Owner Name: ______________________ Prepared By: ______________________
Address: ______________________ Date: ______________________
Property Location: ______________________

Elevation Field Investigation Worksheet

- ❑ Does site topography data cover required area ❑ Yes ❑ No
  Additional data required ______________________
- ❑ Any construction access issues? ______________________
- ❑ Site and building utilities identified? ❑ Yes ❑ No
  Potential utility conflicts identified? ❑ Yes ❑ No
  Describe conflicts: ______________________
- ❑ Review homeowner preferences:
  Can aesthetics reconcile with site and building constraints? ❑ Yes ❑ No
  How? ______________________
- ❑ Confirm type and condition of existing framing:
  ❑ member sizes ______________ ❑ spans ______________
  ❑ connections ______________ ❑ supports ______________
- ❑ Confirm type and condition of foundation:
  ❑ type ______________ ❑ depth ______________
  ❑ size ______________
- ❑ Confirm types and condition of existing construction materials:
  ❑ roof ______________ ❑ floor ______________
  ❑ walls ______________ ❑ foundation ______________
- ❑ Confirm soil information:
  ❑ type ______________ ❑ depth of rock ______________
  ❑ bearing capacity ______________ ❑ susceptibility to scour and erosion ______________
- ❑ Confirm characteristics of flood-related hazards:
  ❑ BFE ______________ ❑ velocity ______________
  ❑ frequency ______________ ❑ duration ______________
  ❑ potential for debris flow ______________
- ❑ Confirm characteristics of non-flood-related hazards:
  ❑ wind ______________ ❑ seismic ______________
  ❑ snow ______________ ❑ other: ______________
- ❑ Review accessibility considerations:
  ❑ access/egress ______________
  ❑ special resources for elderly, disabled, children ______________
  Architectural constraints noted: ______________
- ❑ Is clearance available to install lifting beams and jacking equipment? ❑ Yes ❑ No
- ❑ Check local codes/covenants for height or appearance restrictions:
  ❑ deed/subdivision rules ______________ ❑ local building codes ______________
  Restrictions: ______________

Figure VI-E.25: Elevation Field Investigation Worksheet



DESIGN

> To illustrate the design process, a worked example is shown following the instructions for Steps 1-7. Information on Step 9 is presented in the Chapter VI section on Wet Floodproofing. The designer should refer to local codes for guidance on Steps 8 and 10.

The design process for an elevated structure shown in Figure VI-E24 consists of the following steps:

Step 1: Calculate gravity loads.

The computation of gravity (vertical) loads such as building dead and live loads and buoyancy forces was presented in Chapter IV.

Snow Loads: There are no "typical" formulas for houses, since the calculation of snow loads depends upon the building code in use, the geographic area in which the house is located, and the size and shape of the house and roof. The governing building code will clearly spell out the correct procedure to follow. Most procedures are simple and straightforward. Some houses will be more complex due to their shape or quantity of snow that must be allowed for. However, the general procedure is as follows:

- To determine the ground snow load, consult snow maps within the building code, and/or local requirements with the local building official.
- Determine importance factors.
- To determine the exposure factors, analyze the surrounding terrain, trends in snow patterns, and slope of roof.
- Determine the snow load.
- Determine considerations for drifting snow by examining any adjacent house or structure, a mountain above the house, or higher roofs.

> If building and flood guidance is not covered by the IBC/IRC or local building code, refer to ASCE 7-98.

- Determine considerations for sliding snow by examining steep slope on roof or higher roofs.

> If the local building code does not cover wind, snow, or seismic issues, refer to ASCE 7-98.

Step 2: Calculation of lateral loads.

The calculation of building lateral loads includes wind, seismic, and flood-related loads. One objective of the wind and seismic analysis is to determine which loading condition controls the design of specific structural components.

Wind Analysis: There are no "typical" formulas for houses, since the calculation of wind loads depends upon the building code in use and the size and shape of the house. The governing building code will clearly spell out the correct procedure to follow. Most procedures are simple and straightforward. Some houses will be more complex due to their shape. However, the general procedure, as illustrated in Chapter IV, is presented below.

- Determine wind speed and pressure by consulting wind maps within the building code, and checking local requirements with the local building official.

- Determine the importance factors and the exposure category.

- Determine wind gust and exposure factors and analyze the building height and shape, whether the wind is parallel or perpendicular to the roof ridge, and whether it is windward or leeward of roofs/walls.

- Determine the wind load.

- Distribute the load to resisting elements based upon the stiffness of shear walls, bracing, and frames.

Seismic Analysis: There are no "typical" formulas for houses since the calculation of seismic loads depends upon the building code in use and the size and shape of the house. The governing building code will clearly spell out the correct procedure to follow. Some houses will be more complex due to their shape. However, the general procedure, as illustrated in Chapter IV, is presented below.

- Calculate dead loads by floor. These include permanent dead loads (roof, floor, walls, and building materials) and permanent fixtures (cabinets, mechanical/electrical fixtures, stairs, new locations for utilities, etc.).

- Determine if the snow load must be included in the dead load analysis. Most building codes require the snow load to be included for heavy snow regions. The building code will list these requirements.

- Determine the seismic zone and importance factors.

- Determine the fundamental period of vibration (height of structure materials used in building).

- Determine total seismic lateral force by analyzing site considerations, building weights, and the type of resisting system.

- Distribute the loads vertically per the building code, keeping in mind additional force at the top of the building.

- Distribute the loads horizontally according to the building code and the stiffness of resisting elements. The code-prescribed minimum torsion of the building (center of mass vs. center of rigidity), shear walls, bracing, and frames must be considered.

Flood-Related Forces: The computation of flood-related forces was presented in Chapter IV, and includes the following:

- Determine Flood Protection Elevation (FPE).
- Determine type of force (hydrostatic or hydrodynamic).
- Determine the susceptibility to impacts from debris (ice, rocks, trees, etc.).
- Determine susceptibility to scour.
- Determine applicability of and susceptibility to alluvial fans.
- Determine design forces.
- Distribute forces to resisting elements based upon stiffness.

Step 3: Check ability of existing structure to withstand additional loading.

Chapter IV presented general information on determining the ability of the existing structure to withstand the additional loadings imposed by retrofitting methods. The process detailed below is similar for each of the building types we expect to encounter. First, the expected loadings are tabulated and compared against allowable amounts determined from soil conditions, local code standards, or building material standards. The following list of existing building components and connections should be checked.

Elevation

Table of Contents

Roofs: The plywood roof diaphragm, trusses, connections, and uplift on roof sheathing should be capable of resisting the increased wind and seismic loads. The Engineered Wood Association has published several references that are useful in this calculation. These include:

- *Roof Sheathing Fastening Schedules for Wind Uplift;*
- *Diaphragms;* and
- *Residential and Commercial.*

These reference materials or the local building codes will give the designer the necessary plywood thicknesses and connection specifications to resist the expected loadings, and/or will provide loading ratings for specific material types and sizes.

If the roof diaphragm and sheathing are not sufficient to resist the increased loading, the design can strengthen these components through the following:

- increase the thickness of the materials, and/or
- strengthen the connections with additional plates and additional fasteners.

Roof Framing to Wall Connections: The roof framing connections to walls should be checked to ensure that they will resist the increased wind loads. Of critical importance are the gable ends, where many wind failures occur. The Engineered Wood Association has published several references that are useful in this calculation. These include:

- *Panel Handbook and Grade Glossary,* and
- *Residential and Commercial.*

These reference materials or the local building codes will give the designer the necessary truss size, configuration, and connection specifications to resist the expected loadings, and/or will provide loading ratings for specific truss and connection types and sizes.

For additional information on the performance of various building system products, refer to product evaluation reports prepared by the model code groups or the National Evaluation Service (NES).

If the roof trusses and wall connections are not sufficient to resist the increased loading, the design can strengthen these components through the following:

- increase the amount of bracing between the trusses; and/or

- strengthen the connections with additional plates and additional fasteners.

Upper Level Walls: The upper level walls are subject to increased wind pressure and increased shear due to increased roof loads. Both the short and long walls should be checked against the shear, torsion, tension, and deflection, utilizing the governing loading condition (wind or seismic).

The Engineered Wood Association has published several references that are useful in this calculation. These include:

- *Panel Handbook and Grade Glossary;*

- *Residential and Commercial;* and

- *Diaphragms.*

These reference materials or the local building codes will give the designer the necessary wall size and configuration and connection specifications to resist the expected loadings, and/or will provide loading ratings for specific wall types, sizes, and connection schemes.

If the upper-level walls are determined to be unable to withstand the increased loadings, the designer is faced with the difficult task of strengthening what amounts to the entire house. In some situations this may be cost prohibitive, and the homeowner should look for another retrofitting method, such as relocation. Measures the designer could utilize to strengthen the upper-level walls include:

- adding steel strapping (cross bracing) to interior or exterior wall faces;

- adding a new wall adjacent to the exterior or interior of the existing wall;

- bolstering the interior walls in a similar fashion; and/or

- increasing the number and sizes of connections.

Floor Diaphragm: The floor diaphragm and connections are subject to increased loading due to flood, wind, and seismic forces. The existing floor diaphragm and connections should be checked to ensure that they can withstand the increased forces that might result from the elevation.

The Engineered Wood Association has published several references that are useful in this calculation. These include:

- *Residential and Commercial,* and

- *Diaphragms.*

These reference materials or the local building codes will give the designer the necessary floor size and configuration and connection specifications to resist the expected loadings, and/or will provide loading ratings for specific floor types, sizes, and connection schemes.

If the floor diaphragm or connections are determined to be unable to withstand the increased loadings, the designer could strengthen these components by:

- adding a new plywood layer on the bottom of the existing floor diaphragm;
- increasing the number and size of bracing within the floor diaphragm; and
- increasing the number and size of connections.

Step 4: Analyze existing foundation.

The existing foundation should be checked to determine its ability to withstand the increased gravity loads from the elevation, the increased lateral loads due to soil pressures from potential backfilling, and the increased overturning pressures due to seismic and wind loadings. The designer should tabulate all of the gravity loads (dead and live loads) plus the weight of the new foundation walls to determine a bearing pressure, which is then compared with the allowable bearing pressure of the soil at the site. Not including expected buoyancy forces in this computation will yield a conservative answer.

If the existing footing is insufficient to withstand the additional loadings created by the elevated structure, the design of foundation supplementation should be undertaken. The foundation supplementation may be as straightforward as increasing the size of the footing and/or more substantial reinforcement. The designer may refer to the ACI manual for footing design, recent texts for walls and footing design, and applicable codes and standards.

> For wet floodproofing applications, where openings in foundation walls are necessary, refer to the section on Wet Floodproofing in this chapter.

Step 5: Design the new foundation walls.

The design of a new foundation, whether it be a solid or open foundation, is usually governed by the local building codes. These codes will have minimum requirements for foundation wall sizes and reinforcing schemes, including seismic zone considerations. The designer should consult the appropriate code document tables for minimum requirements for vertical wall or open foundation reinforcement.

For new slab applications where the lower level is allowed to flood and the slab is not subject to buoyancy pressures, the designer can utilize the Portland Cement Association document *Concrete Floors on Ground* as a source of information to select appropriate thicknesses and reinforcing schemes based upon expected loadings. The slab loadings will vary based upon the overall foundation design and the use of the lower floor.

Step 6: Design top-of-wall connections.

Top-of-wall connections are critical to avoid pullout of the sole plate, floor diaphragm, and/or sill plate from the masonry foundation. A preliminary size and spacing of anchor bolts is assumed, and uplift, shear, and tension forces are computed and compared against the allowable loads for the selected bolts. Where necessary, adjustments are made to the size and spacing of the anchor bolts to keep the calculated forces below the allowable forces. Connections should be designed for all appropriate load combinations as discussed in the General Design Practices section of this chapter.

Step 7: Design sill/sole plate connections.

The existing sill/sole plate connections will be subject to increased lateral loads and increased uplift forces due to increased wind and buoyancy loading conditions. The sill/sole plate is designed to span between the anchor bolts and resist bending and horizontal shear forces. The designer should refer to the appropriate wood design manual that provides recommended compression, bending, shear, and elasticity values for various sill/sole plate materials. Using these values, the designer checks the connection against the expected forces to ensure that the actual forces are less than the allowable stresses. If the sill/sole plate connection is insufficient to withstand expected loadings, the size of the sill/sole plate can be increased (or doubled), and/or the spacing of the anchor bolts can be reduced.

Step 8: Design new access.

The selection and design of new access to an elevated structure is done in accordance with local regulations governing these features. Special homeowner requirements—for aesthetics, handicapped accessibility, and/or special requirements for children and the elderly—can be incorporated using references previously discussed in Chapter III.

Connection of the new access to the house should be designed in accordance with the local codes. The foundation for the access measure will either stand alone and be subject to its own lateral stability requirements or it will be an integral part of the new elevated structure. In either case, analysis of the structure to ensure adequate foundation strength and lateral stability should be completed in accordance with local codes.

It should be noted that any access below the BFE should incorporate the use of flood-resistant materials. The designer should refer to FEMA Technical Bulletin 2-93, entitled *Flood Resistant Materials Requirements for Buildings Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program.*

Step 9: Design utilities extensions.

The field investigation will reveal the specific utility systems that will require relocation, extension, or modification. Where possible, utility systems should be relocated above the flood protection level. Local utility companies should be contacted about their specific requirements governing the extension of their utility service. In many instances, the local utility company will construct the extension for the homeowner. Critical issues in this extension process include:

- handling of utilities encased in the existing slab or walls;
- coordination of disconnection and reconnection;
- any local codes that require upgrades to the utility systems as part of new construction or substantial repair or improvement;
- introduction of flexible connections on gas, water, sewer, and oil lines to minimize potential for seismic damage;
- potential for relocation or elevation of electrical system components from existing crawl space and/or basement areas; and
- design of separate GFI-type electrical circuits and use of flood-resistant materials in areas below the BFE.

Step 10: Specify increased insulation requirements.

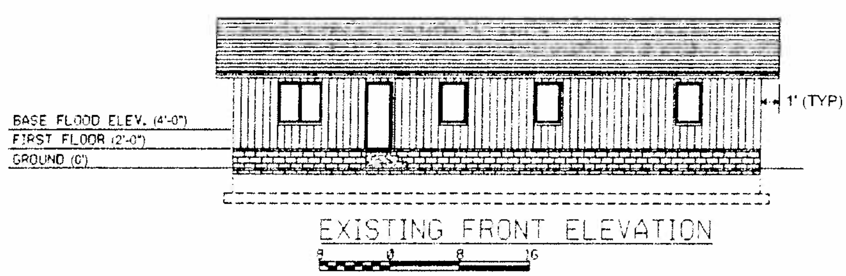
Elevated floors and extended utility system components may increase the potential for heat loss through increased exposure and airflow and necessitate additional insulation. The designer should evaluate the energy efficiency of each aspect of the project, compare existing insulation (R-values) against the local building code, and specify additional insulation (greater R-value) where required.

Elevation Sample Calculation

GIVEN OR OBTAINED FROM THE FIELD INVESTIGATION:

The owner of a single-story crawlspace home intends to elevate the structure to eliminate a repetitive flooding hazard. Her desire is to raise the structure one full story (8 feet) and use the lower level for storage and parking. She contracted with a local engineer to perform the design. The engineer's investigation revealed the following information about the existing structure:

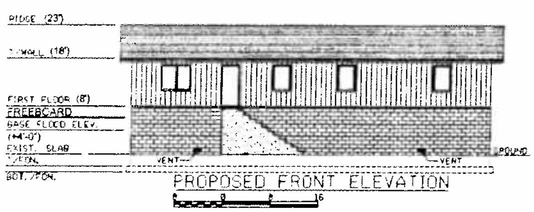
- crawlspace home with four (4) block courses (no reinforcement);
- the first-floor elevation is two (2) feet above the surrounding grade (which is level);
- the property is located in a FEMA-designated floodplain (Zone AE) and is subject to a 100-year flood four (4) feet in depth above ground level;
- floodwater velocities in the area of the house average six (6) feet per second;
- floodwaters flow parallel to front elevation and impact side elevation;
- floodwater debris hazard exists and is characterized as normal;
- the structure is classified as a pre-FIRM structure; and
- local regulations require an additional one (1) foot of freeboard above the 100-year flood elevation.

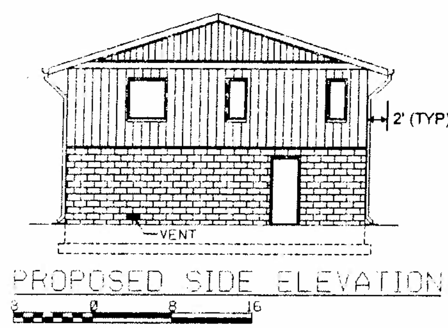


Elevation Sample Calculation

Additional Information on Existing Home

- Wood-Framed House 30 ft. x 60 ft.
- Gable Roof 4:12 slope
- Per ASCE 7-98 and the 2000 International Building Code
  - 90 mph Basic Wind Speed (3-second gust)
  - Seismic Use Group I, Site Class D, $S_2 = 0.25$ ft./S^2
  - Ground Snow Load of 40 psf
- Flat open terrain surrounding house
- Wind Exposure Category C, partially enclosed building



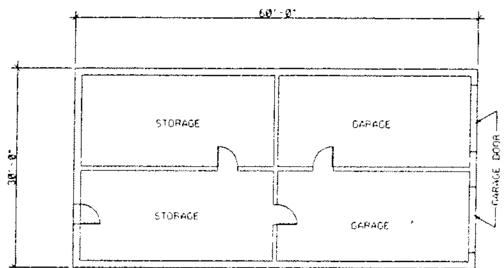


Extended foundation walls are proposed to be constructed of 8-inch-thick concrete masonry units. The existing footing is 2 feet wide by 1 foot thick concrete reinforced with 3-#4 rebars continuous and #4 dowels extending up into masonry 24 inches. Slab on grade will be 3-1/2 to 4 inches thick.

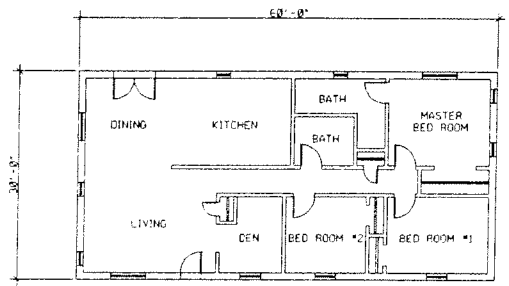
Interior walls of the living area (elevated) are composed of 4-inch studs at 16 inches o.c. with plaster on each side. Exterior walls have 4-inch studs at 16 inches o.c., plaster on the inside, and sheathing and wood siding on the exterior—walls are insulated with fiberglass insulation.

Elevation Sample Calculation

First-floor framing consists of 2x12's at 16 inches on center supported by the exterior long walls and a center support. Floor coverings are hardwood (oak) with a 3/4-inch plywood subfloor. There is 10 inches of insulation between the joists. A gypsum ceiling in the proposed lower area is planned.

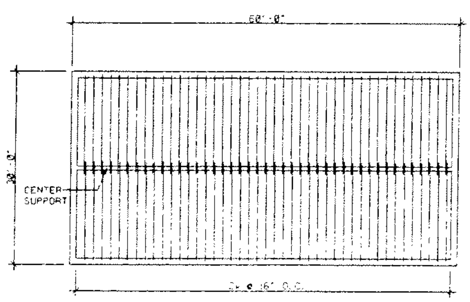


PROPOSED LOWER LEVEL PLAN
OF THE LIVING AREA (ELEVATED)

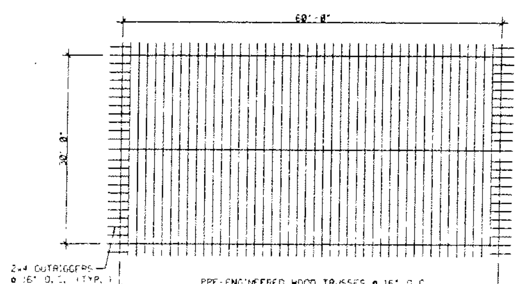


FIRST FLOOR PLAN

Roof framing consists of pre-engineered wood trusses at 16 inches on center. The top chord consists of 2x6's and the web and bottom chord consist of 2x4's. The roof is fiberglass shingles with felt on 1/2-inch plywood. The ceiling is 1/2-inch plaster with 1/2-inch plywood backup. There are 16 inches of fiberglass insulation above the ceiling.



FIRST FLOOR FRAMING PLAN



ROOF FRAMING PLAN

Elevation Sample Calculation

Calculations:

Step 1: Calculate vertical flood loads
Calculate floodproofing design depth (Formulas IV-2 and IV-3)

DFE = FE + f = 4 + 1 = 5 feet
H = DFE - GS = 5 feet - (0)
H = 5 feet

The calculation of buoyancy forces and comparison with structure weight is a critical determination of this problem. While buoyancy of the first floor is not an issue (since it is elevated three feet above the DFE), buoyancy of the entire structure (slab, foundation walls, and superstructure) must be checked if dry floodproofing is being considered for the lower level. If buoyancy forces control, dry floodproofing of the lower level is not applicable.

Calculate Buoyancy Forces (from Formula IV-8)

$F_b = \gamma AH = (62.4\ \text{lbs/ft}^3)(30\ \text{ft} \times 60\ \text{ft})(5\ \text{ft}) = 561{,}600\ \text{lbs} = 561.6\ \text{kips}$
(1 kip = 1,000 lbs.)

Calculate Structure Weight by Level

Tabulate Dead Loads by Floor (based on ASCE 7-98, Table C3-1)

Roof:

Shingles - Asphalt - 1 layer	2.0 psf
Felt	0.7 psf
Plywood - 32/16 - 1/2 inch	1.5 psf
Trusses @ 16 inches o.c.	5.0 psf
2x6 Top Chord	
2x4 Web and Bottom	
Total	9.2 psf (Roof)

Elevation Sample Calculation

First Floor Ceiling:	
Insulation - 16 inch of fiberglass	8.0 psf
1/2 inch plywood	1.5 psf
1/2 inch plaster and lath	10.0 psf
Misc., heating, electrical, cabinets	2.0 psf
Total	21.5 psf (1st Floor Ceiling)
First Floor:	
Oak Floor	4.0 psf
Subfloor - 3/4 inch plywood	3.0 psf
Joists (2x12)	4.0 psf
Insulation - 10 inch fiberglass	5.0 psf
Misc., piping, electrical	3.0 psf
Gypsum ceiling - 1/2 inch	2.5 psf
Total	21.5 psf (1st Floor)
Walls:	
Interior - wood stud, plaster each side	20 psf
Exterior - 2x4 @ 16 inches o.c., plaster insulation, wood siding	18 psf
Lower Level - 8 inch masonry, reinforcement at 48 inches on center	50 psf

Elevation Sample Calculation

Total Weights by Level

Roof:

Surface Area = [15.81 ft. + 2 ft. overhang]x[60 ft + 2 ft. overhang]x[2] = 2208 ft^2
Projected Area = [15 + 2 (15/15.81)]x[60 + 2]x[2] = 2095 ft^2

Shingles:	2208 ft^2(2 psf)	= 4416 lbs
Felt:	2208 ft^2(0.7 psf)	= 1546 lbs
Plywood:	2208 ft^2(1.5 psf)	= 3312 lbs
Truss:	2095 ft^2(5 psf)	= 10,475 lbs

First Floor Ceiling:

Area = 60 x 30 = 1800 ft^2

Insulation:	1800 ft^2(8 psf)	= 14,400 lbs
Plywood	1800 ft^2(1.5 psf)	= 2,700 lbs
Plaster	1800 ft^2(10 psf)	= 18,000 lbs
Misc.	1800 ft^2(2 psf)	= 3,600 lbs
Walls		
180 lf ext.	(4' trib.)(18 psf)	= 12,960 lbs
157 lf int. ±	(4' trib.)(20 psf)	= 12,560 lbs

Subtotal **W2 = 83,970 lbs**

Elevation Sample Calculation

First Floor Including Lower Level:

Area = 60 x 30 = 1800 ft^2

Oak Floor	1800 ft^2(4 psf)	= 7,200 lbs
Subfloor	1800 ft^2(3 psf)	= 5,400 lbs
Joists	1800 ft^2(4 psf)	= 7,200 lbs
Insulation	1800 ft^2(5 psf)	= 9,000 lbs
Misc	1800 ft^2(3 psf)	= 5,400 lbs
Ceiling	1800 ft^2(2.5 psf)	= 4,500 lbs
Walls		
180 lf ext.	(4' trib.)(18 psf)	= 12,960 lbs
157 lf int.	(4' trib.)(20 psf)	= 12,560 lbs
180 lf lower level	(10.67' trib.)(50 psf)	= 96,030 lbs
Footing		
180 lf	(2')(1')(150 psf)	= 54,000 lbs
Slab	1800 ft^2(.33')(150 pcf)	= 89,100 lbs
Subtotal		**W1 = 303,350 lbs**

Total Weight, W = W1 + W2 = 387,320 lbs = 387 kips

Compare Buoyancy Force Against Structure Weight

$DL \geq 1.5\ F_b$

387,000 lbs < 1.5 (562,000)

387 kips lbs < 843 kips lbs   N.G. (No Good)

Therefore, buoyancy forces control and the building (if dry floodproofed) will float during flood events, unless structural measures, such as floor anchors or additional slab mass, or non-structural measures such as allowing the lower level to flood, are utilized to offset/equalize the buoyancy forces.

In our example, since buoyancy controls and the magnitude of the project represents a substantial improvement, the homeowner is required to allow the lower level to flood by incorporating vent openings in the foundation wall. While this action will equalize hydrostatic pressures on the foundation walls, hydrodynamic and impact forces will still apply.

Elevation Sample Calculation

Step 2: Compute lateral loads

Lateral Flood Loads

Compute lateral hydrostatic forces due to five (5) feet of water moving at six (6) feet per second.

From Formula IV-4

$$F_h = 1/2\ \gamma H^2$$
$$= (1/2)\ (62.4\ \text{lbs/ft}^3)\ (5\ \text{ft})^2$$
$$= 780\ \text{lbs/lf acting at } 1.67' \text{ above ground surface}$$

From Formula IV-9

$$b/H = \frac{30\ \text{ft}}{5\ \text{ft}} = 6 < 12$$

$\therefore$ Table IV-4

$C_d = 1.25$

$$dh = \frac{C_d V^2}{2g} = \frac{(1.25)\ (6\ \text{ft}/\text{sec})^2}{2\ (32.2\ \text{ft}/\text{sec}^2)}$$
$$= 0.70\ \text{ft}$$

From Formula IV-10

$$F_{dh} = \gamma\ (dh)H$$
$$= (62.4\ \text{lbs/ft}^3)\ (0.70\ \text{ft})\ (5\ \text{ft})$$
$$= 218.4\ \text{lbs/lf acting at } 1.67' \text{ above ground surface}$$

From Formula IV-11

$$F_H = F_h + F_{dh}$$
$$= 780\ \text{lbs/lf} + 218.4\ \text{lbs/lf}$$
$$= 998.4\ \text{lbs/lf acting at } 1.67' \text{ above ground surface}$$

Because the owner decided to intentionally flood the lower level, the above-calculated lateral hydrostatic flood forces are negated and not considered further in this example computation. However, if dry floodproofing were being considered, these lateral forces may have exceeded the allowable stress on the wall, resulting in a possible wall failure.

Elevation Sample Calculation

Calculate Hydrodynamic Forces on CMU Wall (From Formula IV-12)

$$P_d = C_d \rho \frac{V^2}{2}$$

$$= (1.25)\,(1.94\ \text{slugs/ft}^3)\left[\frac{(6\ \text{ft}/\text{sec})^2}{2}\right]$$

$$= 43.65\ \text{lbs/ft}^2$$

$$b/H = \frac{30\ \text{ft}}{5\ \text{ft}} = 6 < 12$$

$\therefore$ Table IV-4

$C_d = 1.25$

Calculate Total Force on Building Face (upstream) (From Formula IV-13)

$$F_d = P_d A$$

$$= (43.65\ \text{lbs/ft}^2)\,(5')\,(30')$$

$$= 6{,}548\ \text{lbs} = 6.55\ \text{kips}$$

Elevation Sample Calculation

Calculate Normal Impact Forces (From Formula IV-14)

$$F_n = \frac{w_n V}{gt} \quad \text{From Table IV-5: } t = 0.3 \text{ sec for concrete masonry wall construction}$$

$$= \frac{(1{,}000 \text{ lbs})\ (6 \text{ ft/sec})}{(32.2 \text{ ft/sec}^2)\ (0.3 \text{ sec})} = 621 \text{ lbs}$$

Since vents are being used to equalize the hydrostatic pressure, the wall will be subject to a net load equal to the combined hydrodynamic and impact loads. The ability of the new foundation wall to withstand these forces is presented toward the end of Step 5.

WIND

Since the house is being elevated, wind pressures will be increased on the home. Depending upon the amount of elevation, additional bracing of the roof or walls may be necessary.

Reference: ASCE 7-98

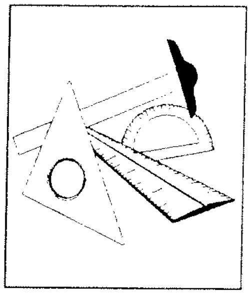
Basic Wind Speed has been determined to be 90 mph. (From Figure 6-1 in ASCE 7-98 and verification with local building official.)

From Equation 6-13, calculate velocity pressure calculated at height z above ground (q_z):

$$q_z = 0.00256\ K_z K_{zt} K_d V^2 I$$

Compute q_z at two different heights:

1. At $z_1 = 15$ feet
2. At mean roof height: $z_2 = h = (18) + (5)/2 = 20.5$ feet



Elevation Sample Calculation

House is given as Exposure Category C

From Table 6-5, compute velocity pressure exposure coefficients (K_z) at heights listed above:

1. For z_1 = 15 feet, K_{z1} = 0.85
2. For z_2 = h = 20.5 feet, K_{z3} = K_h = 0.904 by linear interpolation

Use topographic factor, K_{zt} = 1.0 since house is surrounded by flat, open terrain
From Table 6-6, use directionality factor, K_d = 0.85 for buildings
Given Basic Wind Speed, V = 90 mph
From Table 6-1, Importance Factor, I = 1.00 for residential construction

$$q_{z1} = 0.00256\,(0.85)(1.0)(0.85)(90)^2(1.00) = 15.0 \text{ psf}$$

$$q_{z2} = q_h = 0.00256\,(0.904)(1.0)(0.85)(90)^2(1.00) = 15.9 \text{ psf}$$

From Equation 6-15, calculate Design Wind Pressures on Building Main Wind Force Resisting System, MWFRS (p):

$$p = q\,G\,C_p - q_i\,(GC_{pi})$$

From Equation 6-13, use q = q_{z1} = 15.0 psf, the velocity pressure computed for windward walls calculated at wall height z_1 or z_2 above ground (psf)
From Equation 6-13, use q = q_h = 15.9 psf for all other walls and roof surfaces (psf)
From Section 6.5.8.1, use gust effect factor G = 0.85 for rigid structures
From Figure 6-3, compute external pressure coefficients (C_p) for the following scenarios:

1. Perpendicular to the ridge:
   a. For windward walls, C_p = 0.8
   b. For leeward walls, L/B = (30)/(60) = 0.5, C_p = -0.5
   c. For windward roof, h/L = (20.5)/(30) = 0.683 and $\theta = \tan^{-1}(4/12) = 18.4^0$, C_p = -0.26 by linear interpolation
   d. For leeward roof, h/L = 0.683 and $\theta = 18.4^0$, C_p = -0.58 by linear interpolation

Elevation Sample Calculation

2. Parallel to the ridge:
   a. For windward walls, $C_p = 0.8$
   b. For leeward walls, $L/B = (60)/(30) = 2$, $C_p = -0.3$
   c. For windward roof, $h/L = (20.5)/(60) = 0.34$ and $\theta = \tan^{-1}(4/12) = 18.4^0$,
   $C_p = -0.9$ for 0 to 20', -0.5 for 20' to 40', -0.3 for 40' to 60', by interpolation

From Equation 6-13, use velocity pressure calculated at mean roof height, $q_h = q_i = 15.9$ psf
From Table 6-7, use internal pressure coefficients for partially enclosed buildings,
$GC_{pi} = \pm 0.55$

MWFRS - Wind Perpendicular to Ridge

Walls:

Windward:	$p = (15.0)(0.85)(0.8) - (15.9)(0.55) =$	1.5 psf *(inward)*
	$p = (15.0)(0.85)(0.8) - (15.9)(-0.55) =$	18.9 psf *(inward)*
Leeward:	$p = (15.0)(0.85)(-0.5) - (15.9)(0.55) =$	-15.5 psf *(outward)*

Roof:

Windward:	$p = (15.0)(0.85)(-0.26) - (15.9)(0.55) =$	-12.1 psf *(outward)*
	$p = (15.0)(0.85)(-0.26) - (15.9)(-0.55) =$	5.4 psf *(inward)*
Leeward:	$p = (15.0)(0.85)(-0.58) - (15.9)(0.55) =$	-16.6 psf *(outward)*

MWFRS - Wind Parallel to Ridge

Walls:

Windward:	$p = (15.0)(0.85)(0.8) - (15.9)(0.55) =$	1.5 psf *(inward)*
	$p = (15.0)(0.85)(0.8) - (15.9)(-0.55) =$	18.9 psf *(inward)*
Leeward:	$p = (15.0)(0.85)(-0.3) - (15.9)(0.55) =$	-12.8 psf *(outward)*

Roof:

Windward, for distance from leading edge:

0' to 20'	$p = (15.0)(0.85)(-0.90) - (15.9)(0.55) =$	-20.2 psf *(outward)*
	$p = (15.0)(0.85)(-0.90) - (15.9)(-0.55) =$	-2.7 psf *(outward)*
20' to 40'	$p = (15.0)(0.85)(-0.5) - (15.9)(0.55) =$	-15.1 psf *(outward)*
40' to 60'	$p = (15.0)(0.85)(-0.3) - (15.9)(0.55) =$	-12.6 psf *(outward)*

Elevation Sample Calculation

From Equation 6-18, calculate Design Wind Pressures on Components and Cladding (p):

$$p = q_h\,[(GC_p) - (GC_{pi})]$$

From Equation 6-13, $q_h = 15.9$ psf
From Figures 6-5A and 6-5B, compute external pressure coefficients (GC_p) at the following locations:

1. For windward walls, effective wind area = (18)(16/12) = 24 ft², $GC_p = 0.92$ (Zones 4 & 5)
2. For leeward walls, effective wind area = 24 ft²:
   a. $GC_p = -1.05$ (Zone 4)
   b. $GC_p = -1.25$ (Zone 5)
3. For windward roof, $\theta = \tan^{-1}(4/12) = 18.4°$ and effective wind area = (15.81+2)(16/12) = 23.75 ft²:
   a. $GC_p = -0.85$ (Zone 1)
   b. $GC_p = -2.2$ (Zone 2)
   c. $GC_p = -3.2$ (Zone 3)
4. For leeward roof, $\theta = 18.4°$ and effective wind area = (15.81+2)(16/12) = 23.75 ft²:
   a. $GC_p = -0.85$ (Zone 1)
   b. $GC_p = -2.2$ (Zone 2)
   c. $GC_p = -3.2$ (Zone 3)

From Table 6-7, use internal pressure coefficients for partially enclosed buildings, $GC_{pi} = \pm 0.55$

Components and Cladding

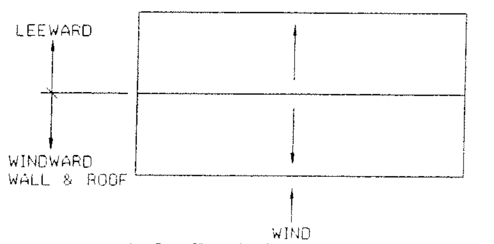
Walls:

Windward:	(Zone 4, 5)	$p = (15.9)[(0.92) - (0.55)] =$	5.9 psf *(inward)*
	(Zone 4, 5)	$p = (15.9)[(0.92) - (-0.55)] =$	23.4 psf *(inward)*
Leeward:	(Zone 4)	$p = (15.9)[(-1.05) - (0.55)] =$	-25.4 psf *(outward)*
	(Zone 5)	$p = (15.9)[(-1.25) - (0.55)] =$	-28.6 psf *(outward)*

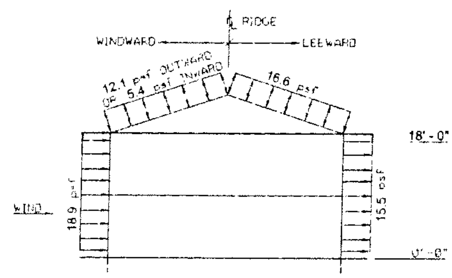
Elevation Sample Calculation

Roof:

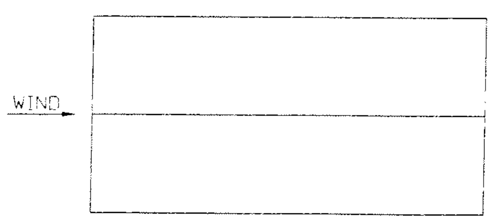
Windward:	(Zone 1)	p = (15.9)[(-0.85) – (0.55)] =	-22.3 psf *(outward)*
	(Zone 1)	p = (15.9)[(-0.85) – (-0.55)] =	-4.8 psf *(outward)*
	(Zone 1)	p = (15.9)[(0.40) – (-0.55)] =	15.1 psf *(inward)*
	(Zone 2)	p = (15.9)[(-2.2) – (0.55)] =	-43.7 psf *(outward)*
	(Zone 2)	p = (15.9)[(-2.2) – (-0.55)] =	-26.2 psf *(outward)*
	(Zone 3)	p = (15.9)[(-3.2) – (0.55)] =	-59.6 psf *(outward)*
	(Zone 3)	p = (15.9)[(-3.2) – (-0.55)] =	-42.1 psf *(outward)*
Leeward:	(Zone 1)	p = (15.9)[(-0.85) – (0.55)] =	-22.3 psf *(outward)*
	(Zone 2)	p = (15.9)[(-2.2) – (0.55)] =	-43.7 psf *(outward)*
	(Zone 3)	p = (15.9)[(-3.2) – (0.55)] =	-59.6 psf *(outward)*



WIND PERPENDICULAR TO RIDGE
N. T. S.



MWFRS -
WIND PERPENDICULAR TO RIDGE
N.T.S.



WIND PARALLEL TO RIDGE
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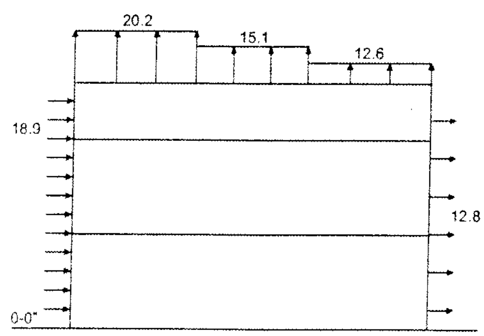
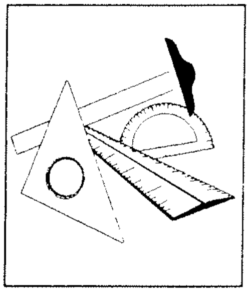


MWFRS -
WIND PARALLEL TO RIDGE
N.T.S.



Elevation Sample Calculation

Seismic

Since the house is being elevated, the potential for seismic loading/overturning design loads will be increased on the home. Depending upon the amount of elevation, additional bracing of the roof or walls may be necessary.

Reference: 2000 International Building Code
(page references are taken from Chapter 16 using the Simplified Analysis Procedure)

From Equation 16-16, calculate the Maximum Considered Earthquake Spectral Response Acceleration for Short Periods (S_{MS}):

$$S_{MS} = F_a S_S$$

Existing house is given as Site Class D
From Table 1615.1.2(1), Site coefficient, $F_a = 1.6$
Given the mapped spectral acceleration for short periods, $S_S = 0.25$ ft/sec^2

$$S_{MS} = (1.6)(0.25) = 0.4 \text{ ft/sec}^2$$

From Equation 16-18, calculate the Design Spectral Response Acceleration (S_{DS}):

$$S_{DS} = 2/3\ S_{MS} \qquad (\text{ft/sec}^2) \qquad (\text{Equation 16-18})$$

From Equation 16-16, $S_{MS} = 0.4$ ft/sec^2

$$S_{DS} = 2/3(0.4) = 0.27 \text{ ft/sec}^2$$

From Equation 16-49, calculate the Seismic Base Shear (V):

$$V = 1.2\ S_{DS}\ W/R$$

From Equation 16-16, $S_{DS} = 0.27$ ft/sec^2

Elevation Sample Calculation

The Effective Seismic Weight of the building, W = 387 kips. Refer to page 7 for calculations.
Basic seismic-force-resisting system consists of ordinary reinforced masonry shear walls
From Table 1617.6, the Response Modification Coefficient of the structural system, R = 2.5

$$V = 1.2\,(0.27)(387)/(2.5) = 50.2 \text{ kips}$$

From Equation 16-50, calculate the Vertical Distribution of Seismic Forces for the first floor ceiling (F_2) and at the first floor level (F_1):

$$F_x = 1.2\,S_{DS}\,w_x/R$$

From Equation 16-16, $S_{DS} = 0.27$ ft/sec²
The Effective Seismic Weight of the building:

1. at the first floor ceiling, $w_x = w_2 = 83.97$ kips
2. at the first floor level, $w_x = w_1 = 303.03$ kips

From Table 1617.6, conservatively assume the Response Modification Coefficient of the structural system remains unchanged, R = 2.5

$$F_2 = 1.2\,(0.27)(83.97)/(2.5) = 10.9 \text{ kips}$$

$$F_1 = 1.2\,(0.27)(303.03)/(2.5) = 39.3 \text{ kips}$$

The seismic load on the house depends upon the effective seismic weight. This weight must be tabulated on a floor-by-floor basis as was presented in Step 1 under Tabulate Dead Loads by Floor.

Check if Snow Load must be included in Seismic calculations:

Reference: ASCE 7-98

Ground Snow = 40 psf
Roof Slope, θ = 18.4°

Elevation Sample Calculation

From Section 7.3, Equation 7-1

$p_f = 0.7C_eC_tIp_g$

where,

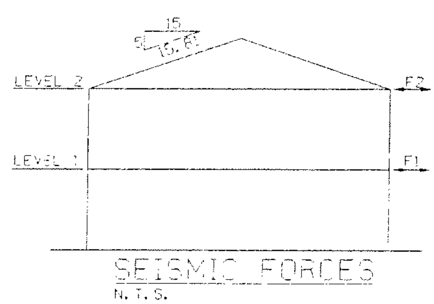
p_f	=	Minimum Roof Snow Load
C_e	=	Exposure Factor
C_t	=	Thermal Factor
I	=	Importance Factor
p_g	=	Basic Ground Snow Load = 40 psf

From Table 7-4, Importance factor, $I = 1.0$
From Table 7-3, Thermal factor, $C_t = 1.0$
From Table 7-2, Exposure factor, $C_e = 0.9$

for this house,

$$p_f = 0.7(0.9)(1.0)(1.0)(40) = 25 \text{ psf} < 30 \text{ psf}$$

thus, by Section 1617.5.1 snow load is <u>not</u> included (it is recommended that the building official be consulted if in doubt) and the total weight of 387 kips as calculated in Step 1 under Total Weights by Level can be used in this seismic analysis.



Elevation Sample Calculation

LATERAL FORCES PERPENDICULAR TO LONG DIRECTION

Seismic

Level	Height (ft) h_x	Level Weight (kips) w_x	$(w_x)(h)_x$	Lateral Force (kips) F_x	Level Shear (kips) ΣF_x
2	18'-0"	83.97	1511	10.9	10.9
1	10'-0"	303.03	3030	39.3	50.2
			4541	50.2	

Wind

Level	Wind Pressure (psf) p_x	Area (ft^2) a_x	Lateral Force (kips) H_x	Level Shear (kips) ΣF_x
2	22*	300*	6.6	6.6
1	34.4	1080	37.2	43.8

* Wind Pressure Calculations (Force = Pressure x Area)

Elevation Sample Calculation

Roof:	(5.4 + 16.6)	(5)(60)	(slope)	
	(pressure)	(area)		
	(16.6 psf)	(15.81 ft)		= 6.6 kips
Wall:	(area)	(pressure)		
	(18 ft)(60 ft)	(18.9 + 15.5 psf)		= 37.2 kips
				43.8 kips

LATERAL FORCES PARALLEL TO LONG DIRECTION

Seismic

Level	Height (ft) h_x	Level Weight (kips) w_x	$(w_x)(h_x)$	Lateral Force (kips) F_x	Level Shear (kips) ΣF_x
2	18'-0"	83.97	1511	10.9	10.9
1	10'-0"	303.03	3030	39.3	50.2
			4541	50.2	

Wind

Level	Wind Pressure (psf) p_x	Area (ft²) a_x	Lateral Force (kips) H_x	Level Shear (kips) ΣF_x
2	31.7*	75*	2.4	2.4
1	31.7*	540*	17.1	19.5

* Wind Pressure Calculations (Force = Pressure x Area)

Elevation Sample Calculation

Roof Gable:	(projected area)	(pressure)	
	1/2(15x5)	(18.9 + 12.8 psf)	= 2.4 kips
Wall:	(area)	(pressure)	
	(30 ft)(18 ft)	(18.9 + 12.8 psf)	= 17.1 kips
			19.5 kips

LOAD COMBINATIONS

Now select the most appropriate load combination for both lateral and vertical loads.
Use ASCE 7-98 Section 2.4.1. For allowable stress design (ASD), the load combinations are:

1. D
2. $D + L + F + H + T + (L_r \text{ or } S \text{ or } R)$
3. $D + W + L + (L_r \text{ or } S \text{ or } R) + .75F_a$
4. $0.6D + W + H + .75F_a$
5. $0.6D + 0.7E + H$

Each possible building failure mode (sliding, overturning, uplift or buoyancy) must be investigated using the most restrictive load combination.

Sliding

Σ Forces in horizontal direction < Sliding resistance provided by soil in order for building to not slide. By inspection, Eq. 4 is most restrictive sliding condition because dead load is reduced and flood and wind loads are included for direction parallel to ridge.

$W + .75F_a < .6D$ (coefficient of soil friction = 0.3); 19.5 kips + (.75)(6.5 kips) = 24.4 kips
(.6D)(0.3) = (.6)(387 kips)(0.3) = 70 kips
24.4 kips < 70 kips OK in parallel to ridge direction.

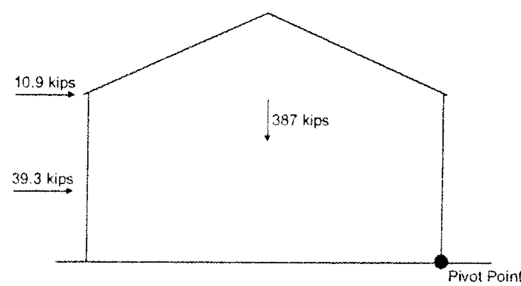
For perpendicular to ridge direction. By inspection, Eq. 4 is most restrictive even though no flood load is involved in this direction.

$W < (.6D)(0.3)$
43.8 kips < 70 kips OK in perpendicular to ridge direction.

Elevation Sample Calculation

Overturning

Most likely overturning direction is at short dimension of the building caused by seismic forces.



$\Sigma M_{PIVOT} = (10.9)(18') + (39.3)(10') - (.6)(387)(15) = -2893.8$ kips-ft. OK Building weight keeps building from overturning.

Uplift/Buoyancy

Σ Forces in vertical direction > buoyancy force in order for building to stay in the ground.

$0.6D + W + H + .75F_a = (0.6)(387 \text{ kips}) + (.75)(562 \text{ kips}) = 189.3$ kips N.G., house will float.

Must develop strategy to keep building in ground - probably accomplish with flood vents and let lower level flood.

Step 3: Check existing structure for new loads

For this example analysis, the existing structural components were assumed to be adequate for the loading conditions. However, the designer should check the existing truss-to-wall-connections, plywood roof diaphragm, upper level walls, and floor diaphragm for their ability to resist increased loadings.

Elevation Sample Calculation

Step 4: Check Existing Foundation

Per ASCE 7-98, Live Load = 40 psf with no concentrated load requirements for a 1-foot-wide strip through the short distance of the house

Snow:	(25 psf) (1') (15'+2' overhang) =	425 plf
First Floor LL:	(40 psf) (1') (15'/2) =	300 plf
Dead Loads:		
Roof:		
shingles:	(15.81' + 2') (2 psf) (1') =	35.6 plf
felt:	(15.81' + 2') (0.7 psf) (1') =	12.5 plf
plywood:	(15.81' + 2') (1.5 psf) (1') =	26.7 plf
truss:	(15' + 2'(15/15.81)) (5 psf) (1') =	84.5 plf
Ceiling:		
insulation:	(15') (1') (8 psf) =	120 plf
plywood:	(15') (1') (1.5 psf) =	22.5 plf
plaster:	(15') (1') (10 psf) =	150 plf
misc:	(15') (1') (2 psf) =	30 plf
wall (ext)	(4') (1') (18 psf) =	72 plf
wall (int)[1]	(15'/2) (1') (20 psf) =	150 plf
First Floor:		
flooring:	(15'/2) (1') (4 psf) =	30 plf
subfloor:	(15'/2) (1') (3 psf) =	22.5 plf
joists:	(15'/2) (1') (4 psf) =	30 plf
insulation:	(15'/2) (1') (5 psf) =	37.5 plf
misc:	(15'/2) (1') (3 psf) =	22.5 plf
ceiling:	(15'/2) (1') (2.5 psf) =	18.8 plf
wall (ext)	(4') (1') (18 psf) =	72 plf
footing	(2') (1') (150 pcf) =	300 plf
wall (int)[1]	(15'/2) (1') (20 psf) =	150 plf
new lower level wall	(10') (1') (50 psf) =	500 plf
Total Dead Load		**1887 plf**

[1] Note that a 20 psf partition load is applied here; this approach is conservative due to the amount of interior walls in this building.

Elevation Sample Calculation

From our field investigation it was determined that an allowable bearing pressure of 2000 psf was acceptable.

Total load on foundation:

Most restrictive load combination is Eq. 3.

$$TL = D + W + L + S + .75\ F_a\ (plf)$$

Horizontal loads created by wind & flood, must be converted to compression on foundation.

Wind: (6.6 kips/60'/30' wide building)(20.5') = 75 plf
(37.2 kips/60'/30')(9') = 186 plf

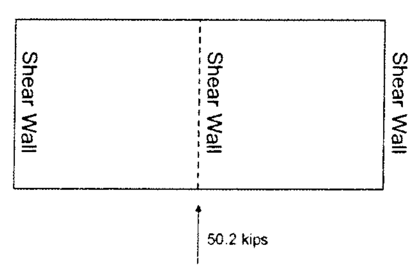
Flood: (43.65 psf)(5'/30')(5')(.75) = 27 plf

$$TL = 1887 + (75 + 186) + 300 + 425 + 27 = 2900\ plf$$

The existing foundation is 2'-0" wide; thus, the bearing pressure for total loads is 2900/2' plf = 1450 psf < 2000 psf Allowable, OK.

Step 5: Design of New Foundation Wall
distribute lateral loads (seismic controls); one intermediate shear wall is assumed.

For connection of foundation wall to footing:



In addition, IBC Section 1617.4.4.4 requires a 5% x building length induced accidental torsion on seismic loads. For this house, the center of mass and the center of rigidity will coincide.

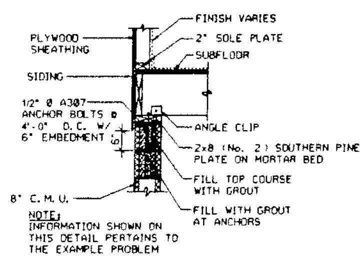
Elevation Sample Calculation

(5%)(60') = 3 feet of induced torsion; therefore, $M_{(acctorsion)}$ = (50.2 kips)(3') = 150.6 kips-ft.

The total seismic shear in the end wall is:

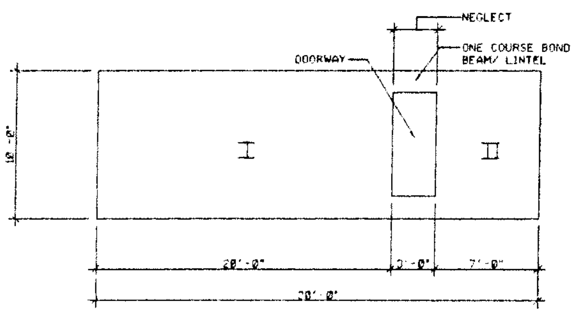
3/8 (50.2 kips) + 3/8 (150.6 kips-ft./60') = 19.8 kips

See typical Hollow-Masonry unit Exterior Foundation Wall Detail.



HOLLOW-MASONRY UNIT
FOUNDATION WALL - WOOD FLOOR

Worst Case, wall with penetrations.



WALL	A, AREA, ft^2	INERTIA, ft^4 I
I	20·0.38 = 7.6	$\frac{1}{12}$ (0.38) (20^3) = 253
II	7·0.38 = 2.7	$\frac{1}{12}$ (0.38) (7^3) = 10.9

SAMPLE ELEVATION

Assume #4 reinforcing bars @ 48" o.c in solid grouted cores.

Elevation Sample Calculation

With #4@48" o.c. the equivalent solid thickness is 4.6 inches = 0.38 ft.

Reference: National Concrete Masonry Association Notes

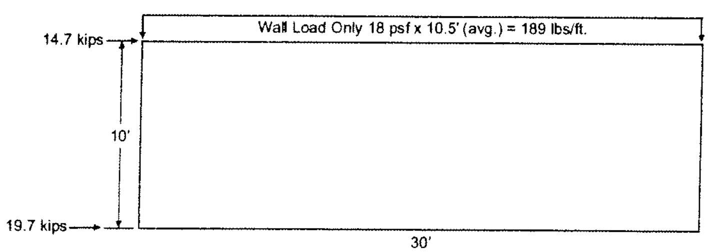
Assume shear modulus, G is = 0.4 E where E = modulus of elasticity per ACI 530-99 Section 1.8.2.2.2.

Assume compressive strength of masonry, f'_m = 2000 psi and type M or S mortar

per ACI 530-99 Section 1.8.2.2 $E_m = 900\ f'_m = 900\ (2000) = 1.8 \times 10^6$ psi

New Wall Design

Minimum wall reinforcement is #4@48" A_s = 0.20/48" Load diagram for end shear wall:



Elevation Sample Calculation

V_m (Shear strength provided by masonry)

$V_m = [4.0 - 1.75\ (M/Vd)]\ A_n\sqrt{f'_m} + 0.25P$

M = (14.7 kips)(10') = 147 kips-ft. = 1,764,000 in-lb

V = 19.7 kips

d = (30')(12) = 360 in

$A_n = 120\ in^2$

$f'_m = 2000$ psi = 2 ksi

P = (189 lb/ft)(30') = 5670 lb = 5.67 kips

$V_m = [4.0 - 1.75\ (1764\ kips/(19.7\ kips)(360))]\ (120)\sqrt{2} + 0.25\ (5.67)$

$= (3.56)(120)(44.72) + 1417 = 20.5$ kips

$V_s = 0.5\ (A_v/s)\ f_y d_v$

$= 0.5\ (0.20/48)(60\ kips)(360)$

$= 45$ kips

M/Vd = (14.7 kips)(10')/(19.7 kips)(30') < 1 (by inspection)

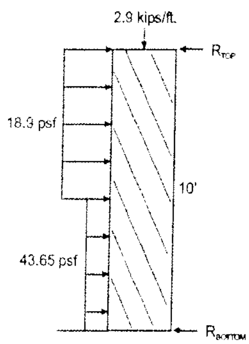
So $F_v = (1/3)(4\text{-}M/Vd)\ \sqrt{f'_m} = (1/3)(4 - .25)\ \sqrt{2000} = 55.9$ psi

Elevation Sample Calculation

Nominal Shear Strength $V_n = V_m + V_s = 20.5$ kips + 45 kips = 65.5 kips

65.4 kips > 50.2 kips OK #4@48" o.c. in-plane shear

Investigate long (60') wall for out-of-plane bending because axial load is also supported by this wall. Neglect impact load in this analysis.



Treat wall as simple T-beam 4' wide

$f_a = (2.9 \text{ kips})(4)/(48")(7.5") = 32.2$ psi

for $h/r \leq 99$ $h/r = 120/2.16 = 55.6 < 99$

where: $h = (10)(12) = 120"$

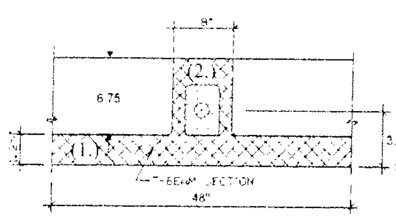
$r = \sqrt{t^2/12} = \sqrt{7.5^2/12} = 2.16$

So $F_a = 1/4\, f'_m\, [1 - (h/140_r)^2] = 1/4\,(2000)\,[1-(120/(140)(2.16))^2] = 421$ psi

$M_{max} = (4)(18.9)(10')^2/8 + [(4)(43.65)(5)[(2)(10) - 5]/(2)(10)]^2/(2)(43.65)$

$= 945$ ft-lbs + 4911 ft-lbs = 5856 ft-lbs

Calculate section modulus for T-beam section of wall.



Elevation Sample Calculation

Area (in.^{2})		δ	$A\delta$	$A\delta^2$	I_i	$(I_x)_i$
(1.) 1.25 x 48 =	60	.625	37.5	23.44	7.81	31.25
(2.) 8" x 6.75" =	54	3.81	205.74	783.87	205.03	988.90
	114		243.24			1020.15 in^4

y = 243.24/114 = 2.13"

$I_{(N_iA_i)} = I_x - Ay^2 = 1020.15 - (114)(2.13)^2 = 502.94$ in^4

$S = I_{(N_iA_i)}/y = 502.94/2.13 = 236.12$ in^3

$f_b = M/S = (5856$ ft-lbs$)(12$ in/ft$)/236.12$ in$^3 = 298$ psi

$F_b = 1/3\ f'_m = 667$ psi

Check combined bending & axial load using interaction formula:

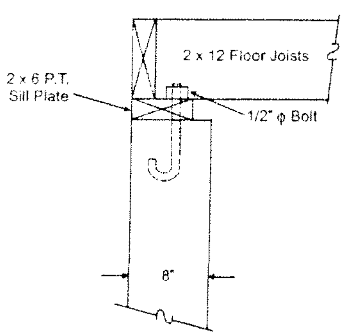
$f_a/F_a + f_b/F_b < 1$ where $32.2/421 + 298/667 < 1$ OK (by inspection)

$R_{BOTTOM} = [43.65)(5)[(2)(10) - 5]/(2)(10) + 18.9(10/2)](4) = 1032.8$ lbs

$R_{TOP} = (4)[(18.9)(10) + (43.65)(5)] - 1032.8 = 596.2$ lbs

f_v (at base of wall) = V/bd = 1032.8 lbs/(7.5)(12) = 11.5 psi

$F_v = 50$ psi OK, $> f_v$



Reaction R_{TOP} must be resisted by attachment of floor diaphragm to wall by bolts and wood sill plate.

Elevation Sample Calculation

Investigate pure bending of wall:

$M = kbd^2$ where M = 5856 ft-lbs or 70272 in-lbs, b = 48", d = 3.81"

$k = 70272 \text{ in-lb}/(48)(3.81)^2 = 100.85$ psi

$k = pf_s j = (A_s/bd)\, f'_s j$, $k = n/n+r$ where $n = E_s/E_m$ & $r = f_s/f_m$

$n = 29 \times 10^6/1.8 \times 10^6 = 16.1$ $r = 24{,}000/2000 = 12$

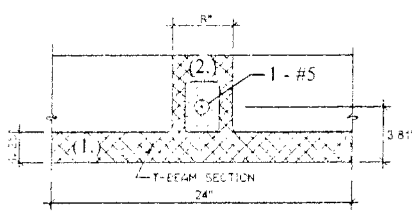
$k = 16.1/(16.1 + 12) = .573$, $j = 1 - k/3 = 1 - .573/3 = .809$

So, reqd. A_s for M = 5856 ft-lb and 48" wide T-beam is:

$A_{sreqd} = (k)(b)(d)/f_s j = (100.85)(48)(3.81)/(24{,}000)(.809) = .95 \text{ in}^2$ N.G., - 1 - #4 BAR $= 0.20 \text{ in}^2$

Try #5 @ 24" o.c. ($A_s = 0.31 \times 3 \text{ in}^2$ to satisfy requirement)

Now T-beam shape is:



$M = (2)(18.9)(10')^2/8 + [(2)(43.65)(5)[(2)(10) - 5]/(2)(10)]^2/(2)(43.65)$

$= 473$ ft-lbs + 1228 ft-lbs = 1701 ft-lbs

Elevation Sample Calculation

Revised section modulus:

	Area (in.^{2})	δ	$A\delta$	$A\delta^2$	I_i	$(I_x)_i$
(1.) 1.25 x 24 =	30	.625	18.75	11.72	3.91	15.63
(2.) =	54	3.81	205.74	783.87	205.03	988.90
	84		224.49			1004.53 in^4

$y = A\delta/A = 224.49/84 = 2.67"$

$I = I_x - Ay^2 = 1004.53 - (84)(2.67)^2 = 405.7$

$S = I/y = 405.7/2.67 = 151.95 \text{ in}^3$

$f_b = M/S = (1701)(12)/151.95 = 134$ psi Still OK, $< F_b = 667$ psi

$R_{BOTTOM} = 516.4$ lbs

$R_{TOP} = 298.1$ lbs

$k = (1701)(12)/(24)(3.81)2 = 58.6$

$A_{sreqd} = (58.6)(24)(3.81)/(24{,}000)(.809) = .276 \text{ in}^2 < .31(1\text{-}\#5)$ OK

Use 1-#5 @ 24" o.c. to resist bending out-of-plane.

Bolt Design

Shear at top of wall = 298.1 lbs/2' or 149 lb/ft

Check shear capacity of 1/2" ϕ A307 A.B. in southern pine sill plate. Use National Design Specification for Wood, 1997 edition, Table 8.2A. Try sill plate 2"x6" so side member = 1-1/2", main member = 3", shear perpendicular to grain = 400 lbs. Modify value with adjustment factors for connections.

So $Z' = ZC_dC_mC_tC_gC_A = (400)(1.6)(1.0)(1.0)(1.0)(1.0) = 640$ lbs

Elevation Sample Calculation

Bolt spacing = 640 lbs/149 lbs/ft = 4.29' or 51.5 in, use 48" max.

Edge distance of bolt 4D = 4(1/2") = 2"

Check bending of sill plate with bolts @ 4' o.c.

M = (149 lb/ft)$(4)^2/8$ = 298 ft-lb

S_{OF} 2x6 = 7.56 in^3 ($bd^2/6 = (1.5)(5.5)^2/6 = 7.56\ in^3$)

f_b = M/S = (298)(12)/7.56 = 473 psi

F_b for southern pine = 1050 psi (No. 1 select structural) $f_b < F_b$ OK

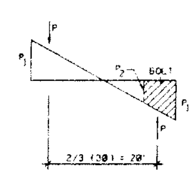
Check pullout of A.B. Uplift Force = 10.9k(10')/(60')(30') = 60.6 lb/ft

For anchors spaced 4' o.c., uplift force = 242 lb/bolt

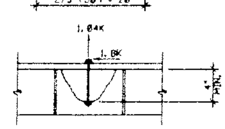
8" CMU wall w/#5 @ 24" o.c. centered on grouted cell - 2,000 psi masonry (f'm) is acceptable.

Step 6: Design top of wall connection. (Checking anchor bolts for pullout from masonry)

Try 1/2"ϕ A307 anchor bolts @ 4'-0" o.c.



uplift on bolt = 242 lb/bolt



try 1/2" ϕ A307 anchor bolt, area of bolt, A_b = 0.2 in^2
edge distance, l_{be} = 75/8 /2 - ½/2 = 3.56"
embedment, l_b = 4" (chosen)

Elevation Sample Calculation

Reference: ACI 530-99 Section 2.1.2

$$Ap = \min(\pi l_b^2 \text{ or } \pi l_{be}^2) = \pi(3.56)^2 = 39.8 \text{ in}^2$$

allowable load in tension,

$$B_a = \min \begin{vmatrix} 0.5(A_b)\sqrt{f'_m} \\ 0.2(A_b)f_y \end{vmatrix}$$

where:

A_b = Area of Anchor Bolt
f'_m = Compressive Strength of Masonry
f_y = Yield Strength of Anchor Bolt

for this anchor bolt pattern,

$B_a = \min(((0.5)(39.8)(2000))^{1/4}),(0.2)(30\text{ksi})) = 890{,}800 = 1200 \text{ lbs} > 242 \text{ lbs}$ OK

allowable load in shear,

$$B_v = \min \begin{vmatrix} 350((f'm)\,A_b)^{1/4} \\ 0.12A_b(f_y) \end{vmatrix}$$

where:

A_b = Area of Anchor Bolt
$f'm$ = Compressive Strength of Masonry
f_y = Yield Strength of Anchor Bolt

for this anchor bolt pattern,

$B_v = \min(350((2000)0.2)^{1/4}),(0.12(0.2)(30\text{ksi})) = 1565,\ 720 = 720 \text{ lbs} > 298.1 \text{ lbs}$ OK

Per ACI 530-99 Section 2.1.2.2.4, the combined ratio is

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} \leq 1.33 \text{ (with 1/3 increase for wind/seismic per Section 2.1.1.1.3)}$$

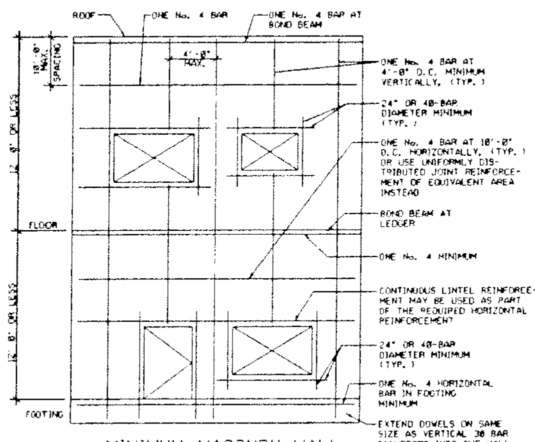
where:

b_a = Actual Bolt Tension
b_a = Actual Bolt Shear
B_v = Allowable Bolt Shear
B_t = Allowable Bolt Tension

Elevation Sample Calculation

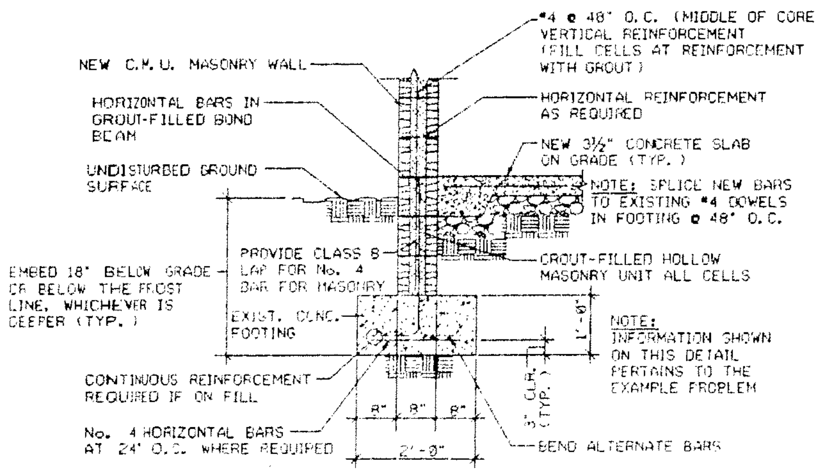
SAMPLE DETAILS

Minimum Reinforcement Required by Code for Seismic Zone 2



MINIMUM MASONRY WALL
REQUIREMENTS IN SEISMIC ZONE No. 2
N.T.S.

Sample Bearing Wall Detail

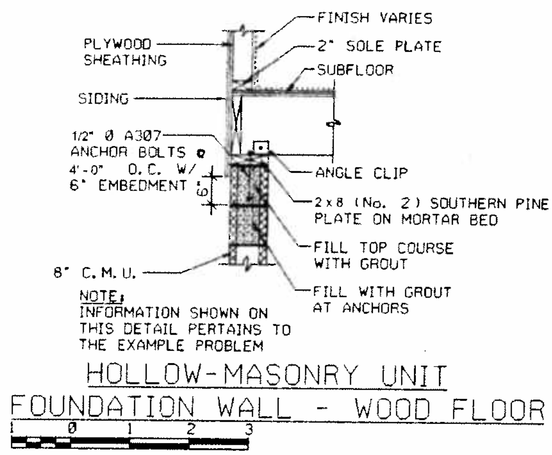


HOLLOW-MASONRY UNIT
EXTERIOR FOUNDATION WALL

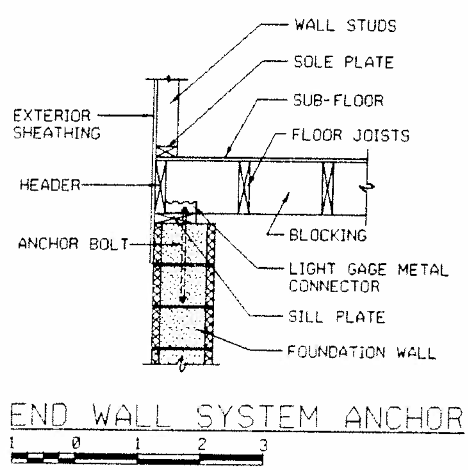
1 0 1 2 3

Elevation Sample Calculation

Sample Foundation Detail

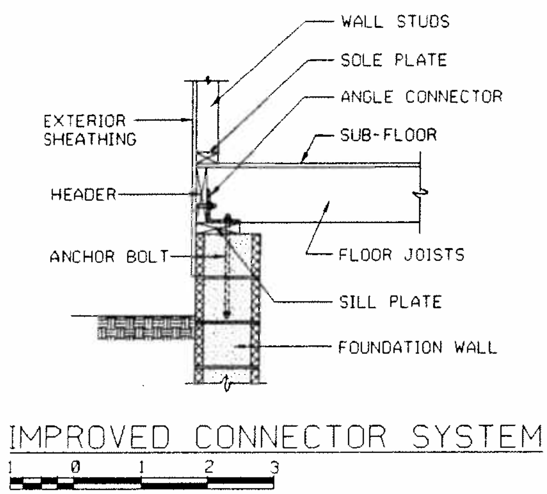


Sample Detail of End Wall



Elevation Sample Calculation

Sample End Wall Detail for Higher Loads



CONSTRUCTION CONSIDERATIONS

Guidance on the selection of an elevation or relocation contractor is provided in Chapter VI-R, Relocation.

PRIOR TO LIFTING ANY HOUSE

- Obtain all permits and approvals required.
- Ensure that all utility hookups are disconnected (plumbing, phone, electrical, cable, and mechanical).
- Estimate the lifting load of the house.
- Identify the best location for the principal lift beams, lateral support beams, and framing lumber, and evaluate their adequacy (generally performed by a structural engineer or the elevation contractor).

SLAB-ON-GRADE HOUSE, NOT RAISING SLAB WITH HOUSE

- Holes are cut for lift beams in the exterior and interior wall.
- Main lifting beams are inserted.
- Holes are cut for the lateral beams.
- Lateral beams are inserted.
- Bracing is installed to transfer the loads across the support walls and lift remaining walls.
- Jacks are moved into place and structure is prepared for lifting.

- Straps and anchors used to attach house to slab-on-grade are released.
- The house is elevated and cribbing installed.
- Slab around edges is removed to allow for new foundation.
- The new foundation is constructed.
- New support headers and floor system are installed.
- Any required wind and seismic retrofit is completed.
- House is attached to new foundation.
- All temporary framing is removed, holes are patched.
- Reconnect all utilities.
- Construct new stairways and access.
- Floodproof all utilities below the FPE.

SLAB-ON-GRADE HOUSE, RAISING SLAB

- Trenches are excavated for placement of all support beams beneath slab.
- Lifting and lateral beams are installed.
- Jacks are moved into place and the structure is prepared for lifting.
- The house is elevated and cribbing installed.

- The new foundation is constructed.
- Any required wind and seismic retrofit is completed.
- House is attached to new foundation.
- Support beams are removed.
- Access holes are patched.
- Reconnect all utilities.
- Construct new stairways and access.
- Floodproof all utilities below the FPE.

HOUSE OVER CRAWLSPACE/ BASEMENT

- Remove masonry necessary to allow for placement of support beams.
- Install main lifting beams.
- Install lateral beams.
- Jacks are moved into place and the structure is prepared for lifting.
- All connections to foundation are removed.
- House is elevated and cribbing installed.
- Existing foundation walls are raised or demolished depending upon whether the existing foundation walls can handle the new loads.

- New footings and foundation walls are constructed if the existing foundation walls/footings cannot withstand the additional loading.
- Backfill basement where appropriate.
- House is attached to new foundation.
- Support beams are removed.
- Access holes are patched.
- Reconnect all utilities.
- Construct new stairways and access.
- Floodproof all utilities below the FPE.

HOUSE ON PILES, COLUMNS, OR PIERS

If the house is to remain in the same location, the house will most likely need to be temporarily relocated to allow for the footing and foundation installation. If the house is being relocated within the same site, the footings should be constructed prior to moving the house.

- Install main support beams.
- Install lateral beams.
- Jacks are moved into place and the structure prepared for lifting.
- House is elevated and cribbing installed.

- If the house is being relocated, see the Chapter VI relocation section.
- House is attached to new foundation.
- Remove support beams.
- Reconnect all utilities.
- Construct new stairways and access.
- Floodproof all utilities below the FPE.

Floodwalls

Table of Contents

FLOODWALLS

A properly designed and constructed floodwall can often be an effective device for repelling floodwaters. Floodwalls are typically used in three roles:

- as a barrier against inundation,
- as a defense for unequalized hydrostatic and hydrodynamic loading situations, and
- to deflect debris and ice away from the structure.

The selection of a floodwall design is primarily dependent on the type of flooding expected at the building's site. High water levels and velocities can exert hydrodynamic and hydrostatic forces and impact loads, which must be accounted for in the floodwall design. The composition of any type of floodwall must address three broad concerns:

- Overall stability of the wall as related to the external loads,
- Sufficient strength as related to the calculated internal stresses, and
- Ability to provide effective enclosures to repel floodwaters.

These internal and external forces pose a significant safety hazard if floodwalls are not properly designed and constructed, or their design level of protection is overtopped. Additionally, a tall floodwall can become very expensive to construct and maintain and can require additional land area for grading and drainage. Therefore, in most instances, residential floodwalls are practical only up to a height of three to four feet above existing grade, although residential floodwalls can be and are engineered for greater heights.

Under NFIP regulations, floodwalls cannot be used to bring non-compliant structures into compliance.

TYPES OF FLOODWALLS

Placement of floodwalls in the floodway and within V Zones is not allowed under NFIP regulations.

Figures VI-F1 and VI-F2 illustrate the use of floodwalls in residential applications. Figures VI-F3 and VI-F4 illustrate several types of floodwalls including gravity, cantilever, buttress, and counterfort. The gravity and cantilever floodwalls are the more commonly used types.

Figure VI-F1: Typical Residential Floodwall

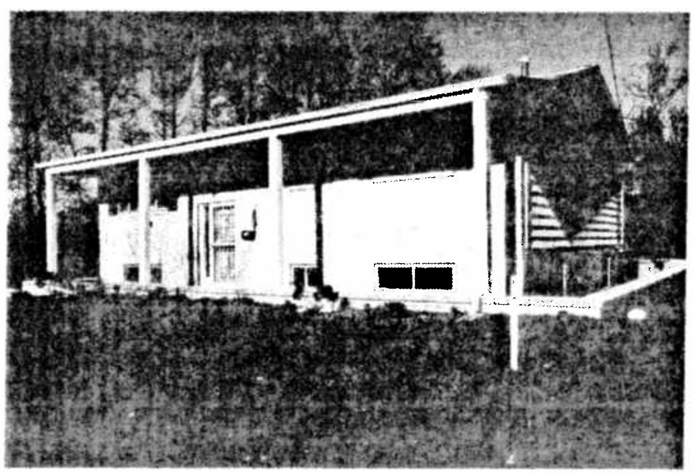


Figure VI-F2: Typical Residential Floodwall

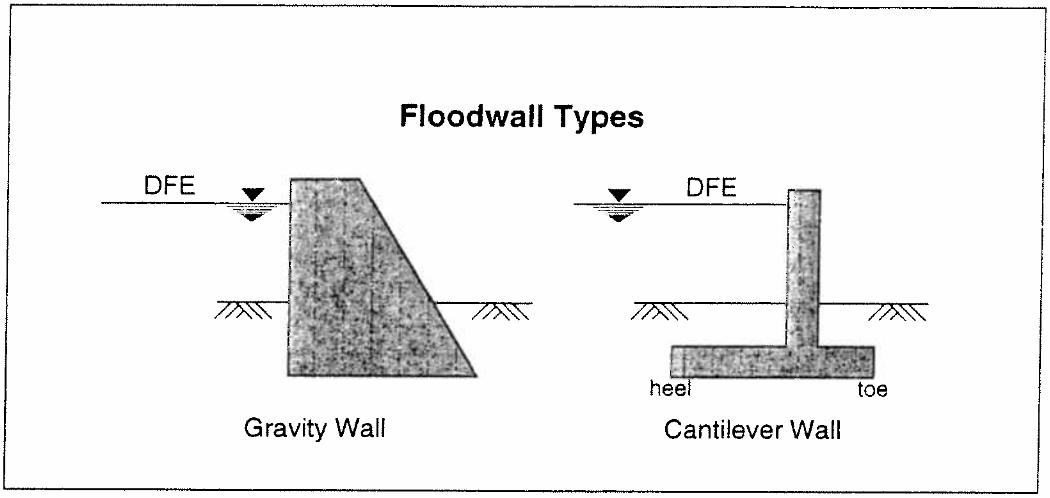


Figure VI-F3: Gravity and Cantilever Floodwalls

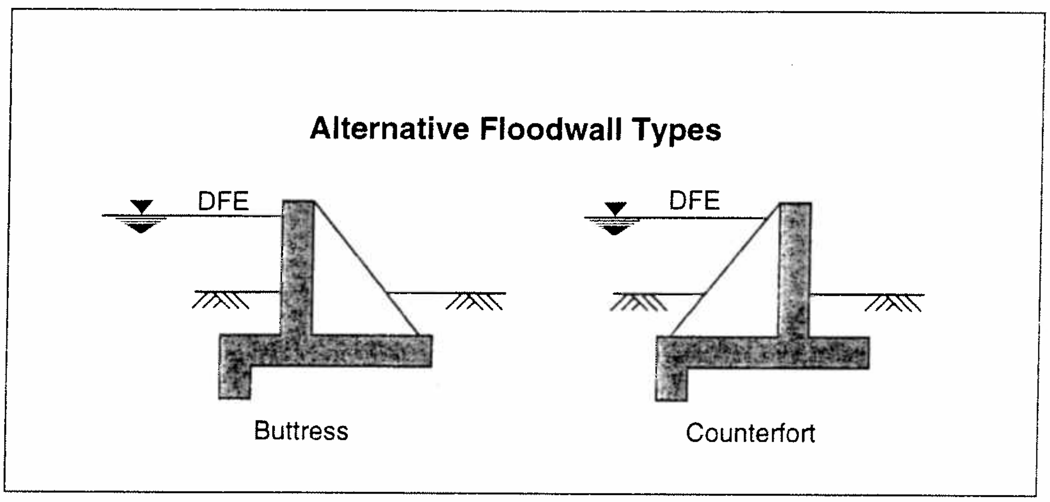


Figure VI-F4: Buttress and Counterfort Floodwalls

GRAVITY FLOODWALL

A gravity floodwall depends upon its weight–as its name implies–for stability. The gravity wall's structural stability is attained by effective positioning of the mass of the wall, rather than the weight of the retained materials. The gravity wall resists overturning primarily by the dead weight of the concrete and masonry construction. It is simply too heavy to be overturned by the lateral flood load.

Frictional forces between the concrete base and the soil foundation generally resist sliding of the gravity wall. Soil foundation stability is achieved by ensuring that the structure neither moves nor fails along possible failure surfaces. Figure VI-F5 illustrates the stability of gravity floodwalls. Gravity walls are appropriate for low walls or lightly loaded walls. They are relatively easy to design and construct. The primary disadvantage of a gravity floodwall is that a large volume of material is required. As the required height of a gravity floodwall increases, it becomes more cost effective to use a cantilever wall.

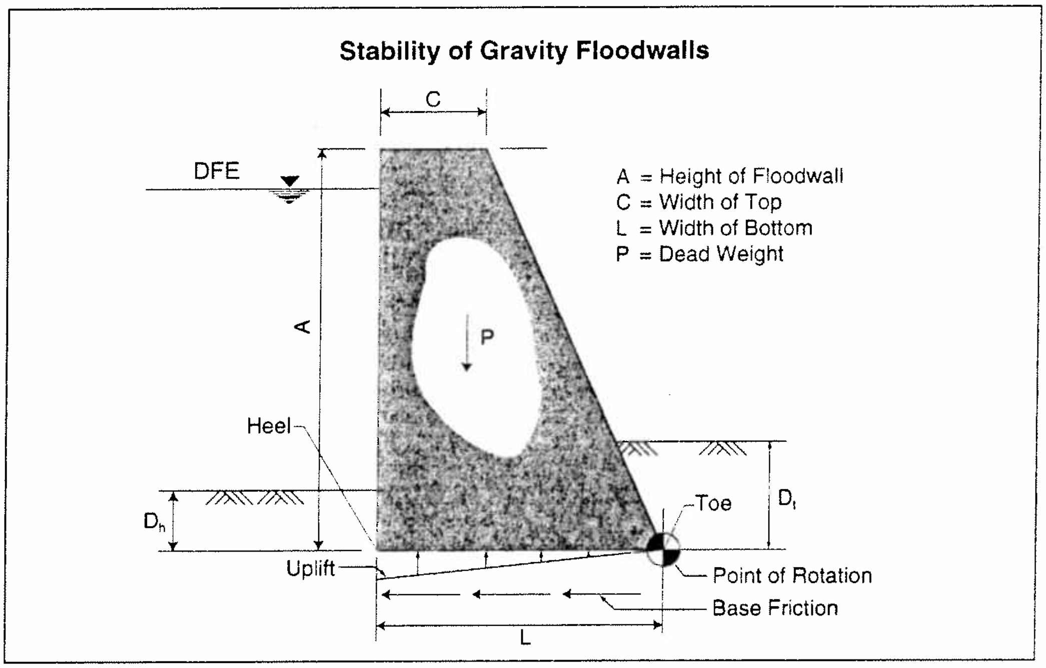


Figure VI- F5: Stability of Gravity Floodwalls

Reinforced concrete provides an excellent barrier in resisting water seepage, since it is monolithic in nature. The reinforcement not only gives the wall its strength, but limits cracking as well.

CANTILEVER FLOODWALL

A cantilever wall is a reinforced-concrete wall (cast-in-place or built with concrete block) that utilizes cantilever action to retain the mass behind the wall. Reinforcement of the wall is attained by steel bars embedded within the concrete or block core of the wall (illustrated by Figure VI-F6). Stability of this type of wall is partially achieved from the weight of the soil on the heel portion of the base, as illustrated in Figure VI-F7.

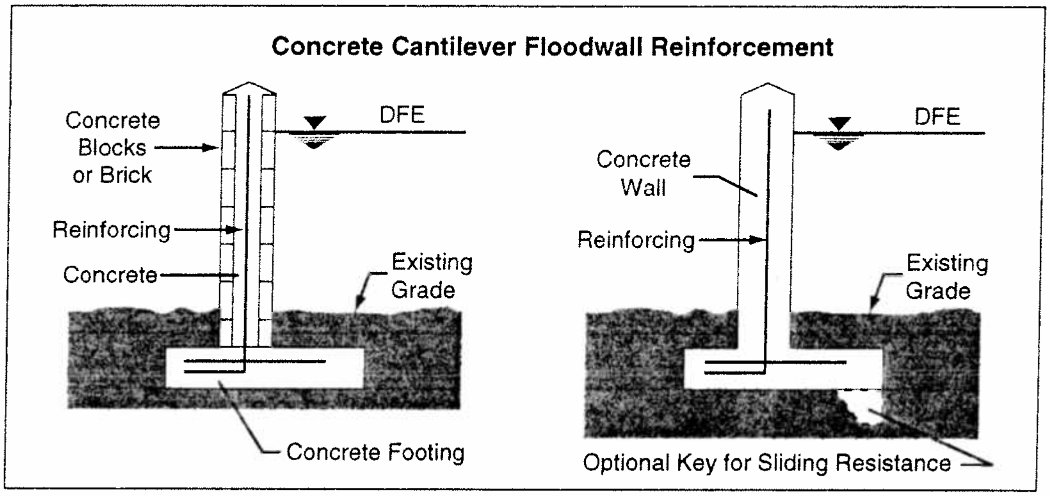


Figure VI-F6: Concrete Cantilever Floodwall Reinforcement

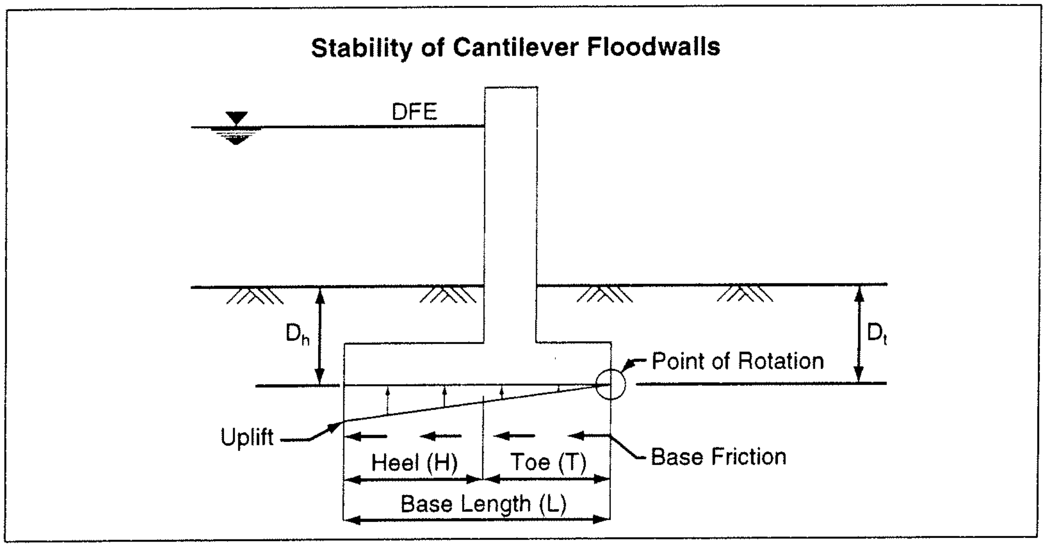


Figure VI-F7: Stability of Cantilever Floodwalls

The floodwall is designed as a cantilever retaining wall, which takes into account buoyancy effects and reduced soil bearing capacity. However, other elements of a floodproofing project (i.e., bracing effects of any slab-on-grade, the crosswalks, and possible concrete stairs) may help in its stability. This results in a slightly conservative design for the floodwall but provides a comfortable safety factor when considering the unpredictability of the flood. Backfill can be placed along the outside face of the wall to keep water away from the wall during flooding conditions.

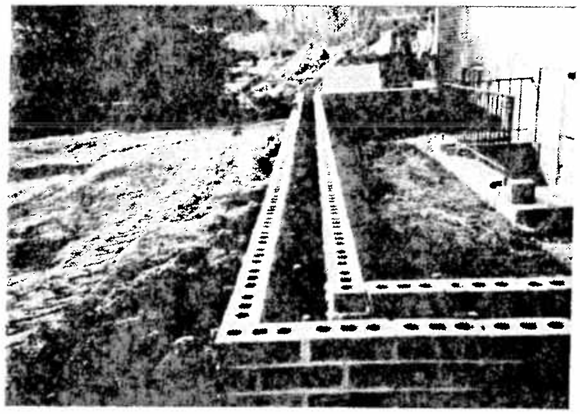


Figure VI-F8: Typical Reinforced Concrete Floodwall

While the double-faced brick floodwall application is used on either side of concrete block with cores reinforced and grouted, experience has indicated it is not as strong or leakproof as monolithic cast-in-place applications.

Information and details for a standard reinforced concrete floodwall are provided in case studies 4, 5, and 6 in Chapter VII.

The concrete floodwall may be aesthetically altered with a double-faced brick application on either side of the monolithic cast-in-place reinforced concrete center (illustrated in Figure VI-F8). This reinforced concrete core is the principal structural element of the wall that resists the lateral hydrostatic pressures and transfers the overturning moment to the footing. The brick-faced wall (illustrated in Figures VI-F9 and VI-F10) is typically used on homes with brick facades. Thus the floodwall becomes an attractive modification to the home. In terms of the structure, the brick is considered in the overall weight and stability of the wall and in the computation of the soil pressure at the base of the footing, but is not considered to add flexural strength to the floodwall.

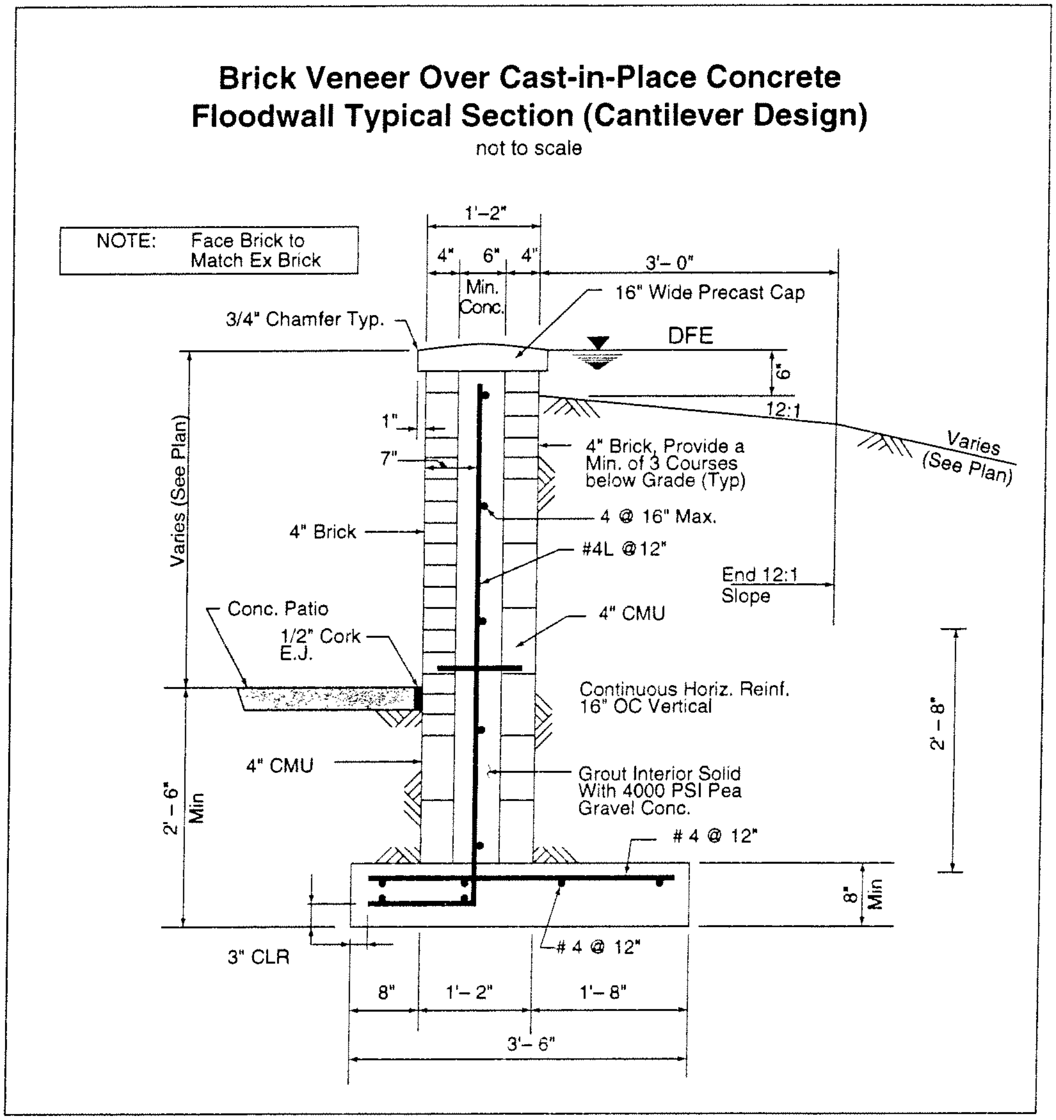


Figure VI-F9: Typical Section of a Brick-Faced Concrete Floodwall

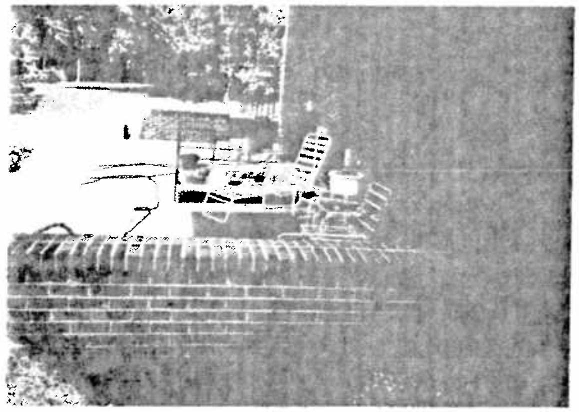


Figure VI-F10: Typical Brick-Faced Concrete Floodwall

When the flood protection elevation requirements of a gravity or cantilever wall become excessive in terms of material and cost, alternative types of floodwalls can be examined. The use of these floodwall alternatives is generally determined by the relative costs of construction and materials and amount of reinforcement required.

COUNTERFORT FLOODWALL

A counterfort wall is similar to a cantilever retaining wall, except that it can be used where the cantilever is long or when very high pressures are exerted behind the wall. Counterforts, or intermediate traverse support bracing, are designed and built at intervals along the wall and reduce the design forces. Generally, counterfort walls are economical for wall heights in excess of 20 feet, but are rarely used in residential applications.

BUTTRESSED FLOODWALL

A buttressed wall is very similar to a counterfort wall. The only difference between the two is that the transverse support walls are located on the side of the stem, opposite the retained materials.

The counterfort wall is more widely used than the buttress because the support stem is hidden beneath the retained material (soil or water), whereas the buttress occupies what may otherwise be usable space in front of the wall.

FIELD INVESTIGATION

Detailed information must be obtained about the site and existing structure to make decisions and calculations concerning the design of a floodwall. The designer should utilize the guidance presented in this chapter where detailed information and checklists for field investigation are presented. Key information to collect includes the low point of elevation survey, topographic and utilities surveys, hazard determinations, local building requirements, and homeowner preferences. Once the designer has developed the above-mentioned low point of entry and site and utility survey information, a conceptual design of the proposed floodwall can be discussed with the homeowner. This discussion should cover the following items:

- Previous floods and which areas were flooded or affected by floods.

- A plan of action as to which opening(s) and walls of the structure can be protected by a floodwall and floodwall closures.

- Evidence of seepage/cracking in foundation walls, which would indicate the need to relieve hydrostatic pressure on the foundation.

- A plan of action to use a floodwall to relieve hydrostatic pressure on the foundation and other exterior walls.

- The various floodwall options and conceptual designs that would provide the necessary flood protection. Obtain consensus on the favored type, size, location, and features of the floodwall(s).

- A plan of action as to which utilities need to be adjusted or floodproofed as a result of the floodwall.

- A plan of action for construction activity and access/ egress to convey to the owner the level of disruption to be expected.

The designer of a floodwall should be aware that the construction of these measures may not reduce the hydrostatic pressures against the below-grade foundation of the structure in question. Seepage beneath the floodwall and the natural capillarity of the soil layer may result in a water level inside the floodwall that is equal to or above grade. This condition is worsened by increased depth of flooding outside the floodwall and the increased flooding duration. Unless this condition is relieved, the effectiveness of the floodwall may be compromised. This condition is illustrated in Figure VI-F11.

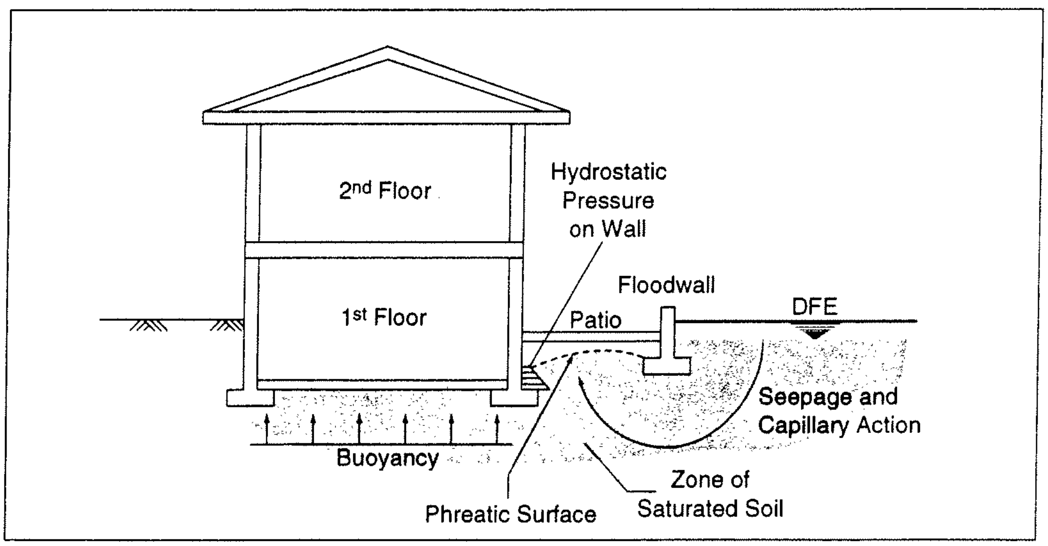


Figure VI-F11: Seepage Underneath a Floodwall

It is important that the designer check the ability of the existing foundation to withstand the saturated soil pressures that would develop under this condition. The computations necessary for this determination are provided in Chapter IV.

Determination of an appropriate distance from the structure for the floodwall is a function of the depth of the foundation. The deeper the lowest level of the structure, the further away the floodwall should be placed.

The condition can be relieved by installation of foundation drainage (drainage tile and sump pump) at the footing level, and/or by extending the distance from the foundation to the floodwall. The seepage pressures can also be decreased by placing backfill against the floodwall to extend the point where floodwaters submerge the soil, but the effectiveness of this measure depends on the relative characteristics of the soils in the foundation and the backfill. The design of foundation drains and sump pumps is presented in the Chapter VI Dry Floodproofing section.

Computation of the spacing required to obviate the problem is a complicated process that should be done by an experienced geotechnical engineer. Figure VI-F12 illustrates the change in phreatic surface as a result of increasing the distance between the foundation and the floodwall and/or the installation of a foundation drain and sump pump system.

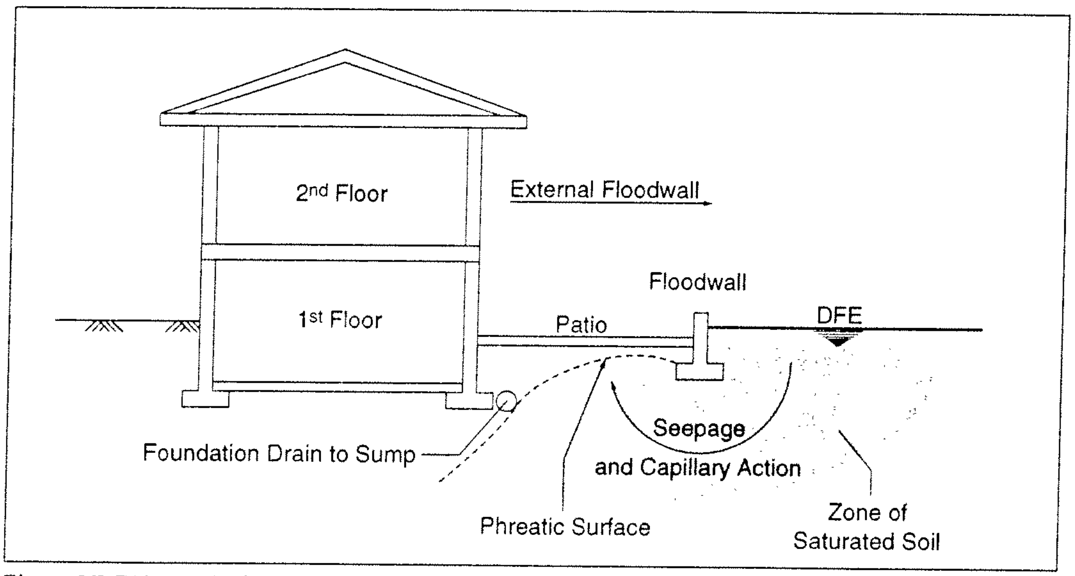
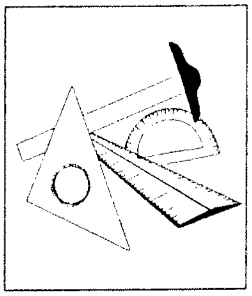


Figure VI-F12: Reducing Phreatic Surface Influence by Increasing Distance from Foundation to Floodwall



DESIGN

The permeability of concrete block may necessitate the use of a monolithic core or the application of sealants to eliminate seepage through the wall.

FLOODWALL DESIGN (SELECTION AND SIZING)

The design of floodwalls consists of the proper selection and sizing of the actual floodwall and the specification of appurtenances such as drainage systems; waterproof materials to stop seepage and leakage; and miscellaneous details to meet site and homeowner preferences for patios, steps, wall facings, and support of other overhead structures (posts and columns).

The structural design of a floodwall to resist anticipated flood and flood-related forces presented in Chapter IV follows the seven-step process outlined in Figure VI-F13.

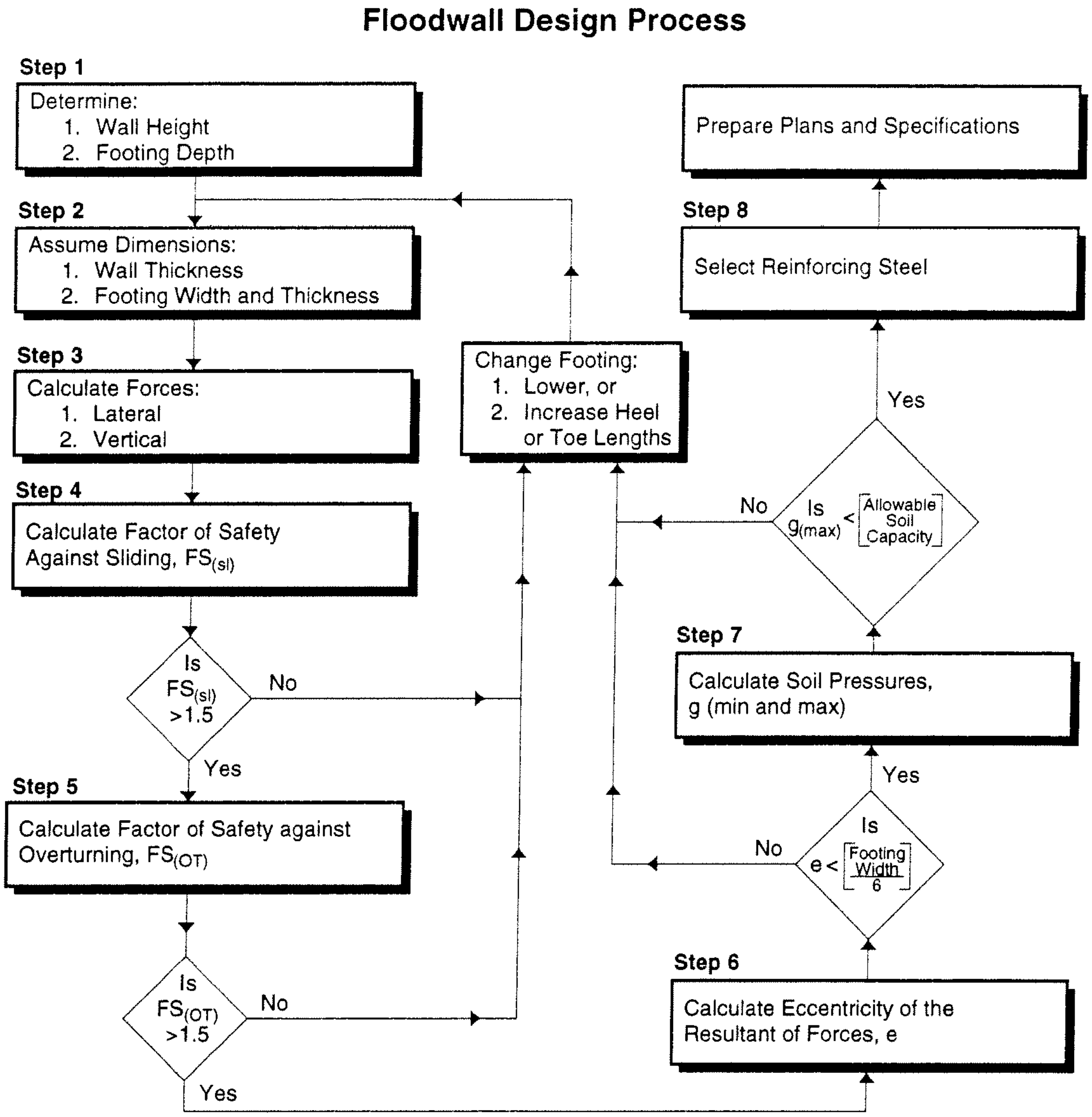
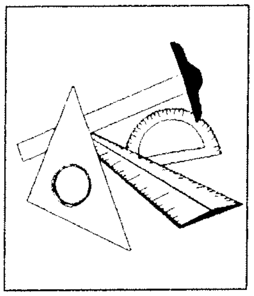


Figure VI-F13: Floodwall Design Process



In general the stability of the floodwall should be investigated for different modes of failure.

Sliding

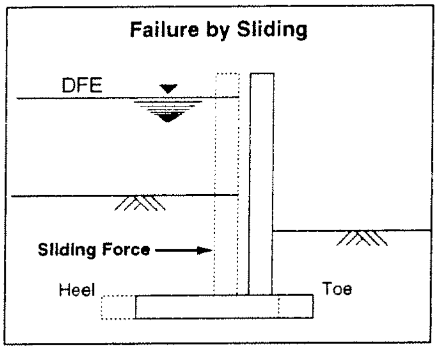


Figure VI-F14: Failure by Sliding

A wall including its footing may fail by sliding if the sum of the lateral forces acting upon it is greater than the total forces resisting the displacement. The resisting forces should always be greater than the sliding forces by a factor of safety. (See Figure VI-F14.)

Overturning

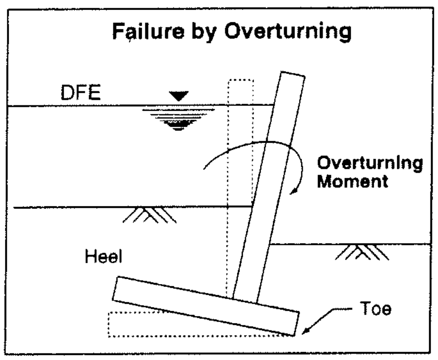


Figure VI-F15: Failure by Overturning

Another mode of failure is overturning about the foundation toe. This type of failure may occur if the sum of the overturning moments is greater than the sum of the resisting moments about the toe. The sum of resisting moments should be greater than the sum of the overturning moments by a factor of safety. (See Figure VI-F15.)

Pressure

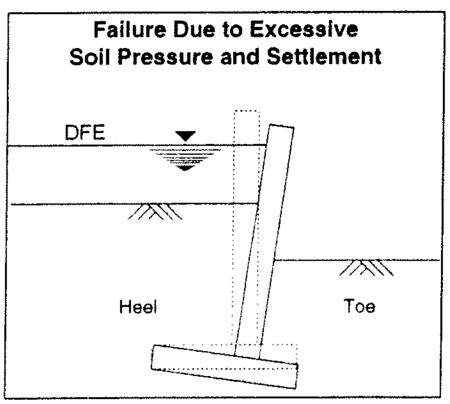


Figure VI-F16: Failure Due to Excessive Pressure

Finally, a wall may fail if the pressure under its footing exceeds the allowable soil bearing capacity. (See Figure VI-F16.)

In the following paragraphs, the step-by-step process for completing the structural design of a floodwall is presented, followed by an example illustrating the use of the formulas. Table VI-F1 provides soil information that is necessary in the computations that follow.

Table VI-F1 **Soil Factors for Floodwall Design**

Soil Type	Allowable Bearing Pressure, S_a, in pounds per square foot	Coefficient of Friction, C_f
Clean, dense sand and gravel, GW, GP, SW and SP	2,000	0.55
Dirty sand and gravel of restricted permeability, GM, GM-GP, SM, and SM-SP	2,000	0.45
Firm to stiff silts, clays, silty fine sands, clayey sands and gravel, CL, ML, CH, SM, SC, and GC	1,500	0.35
Soft clay, silty clay, and silt, CL, ML, and CH	600	0.30

Step 1: Determine wall height and footing depth.

1. Determine wall height based on flood protection elevation, which equals the design flood elevation plus one foot. The extra one foot is the minimum recommended freeboard as a safety measure against future flood levels that exceed the design flood.

2. Determine minimum footing depth based on the frost depth, local code requirements, and the soil condition. The footing should rest on suitable natural soil or on controlled and engineered backfill material.

Step 2: Determine dimensions.

Based on the following guidelines or reference to engineering handbooks, assume dimensions for the wall thickness, footing width, and footing thickness.

1. The choice of wall thickness depends on the wall material, the strength of the material, and the height of the wall. Typical wall thicknesses are 8, 12, and 16 inches for masonry, concrete, or masonry/concrete walls.

2. The footing width depends on the magnitude of the lateral forces, allowable soil bearing capacity, dead load, and the wall height. The typical footing width is the proposed wall height. Typically the footing is located under the wall in such a manner that 1/3 of its width forms the toe and 2/3 of the width forms the heel of the wall as shown in Figure VI-F17. Typical footing thicknesses are based upon strength requirements and include 8, 12, and 16 inches.

Step 3: Determine forces.

There are two types of forces acting on the wall and its footing: lateral and vertical. These forces were discussed in Chapter IV and are illustrated in Figure VI-F17.

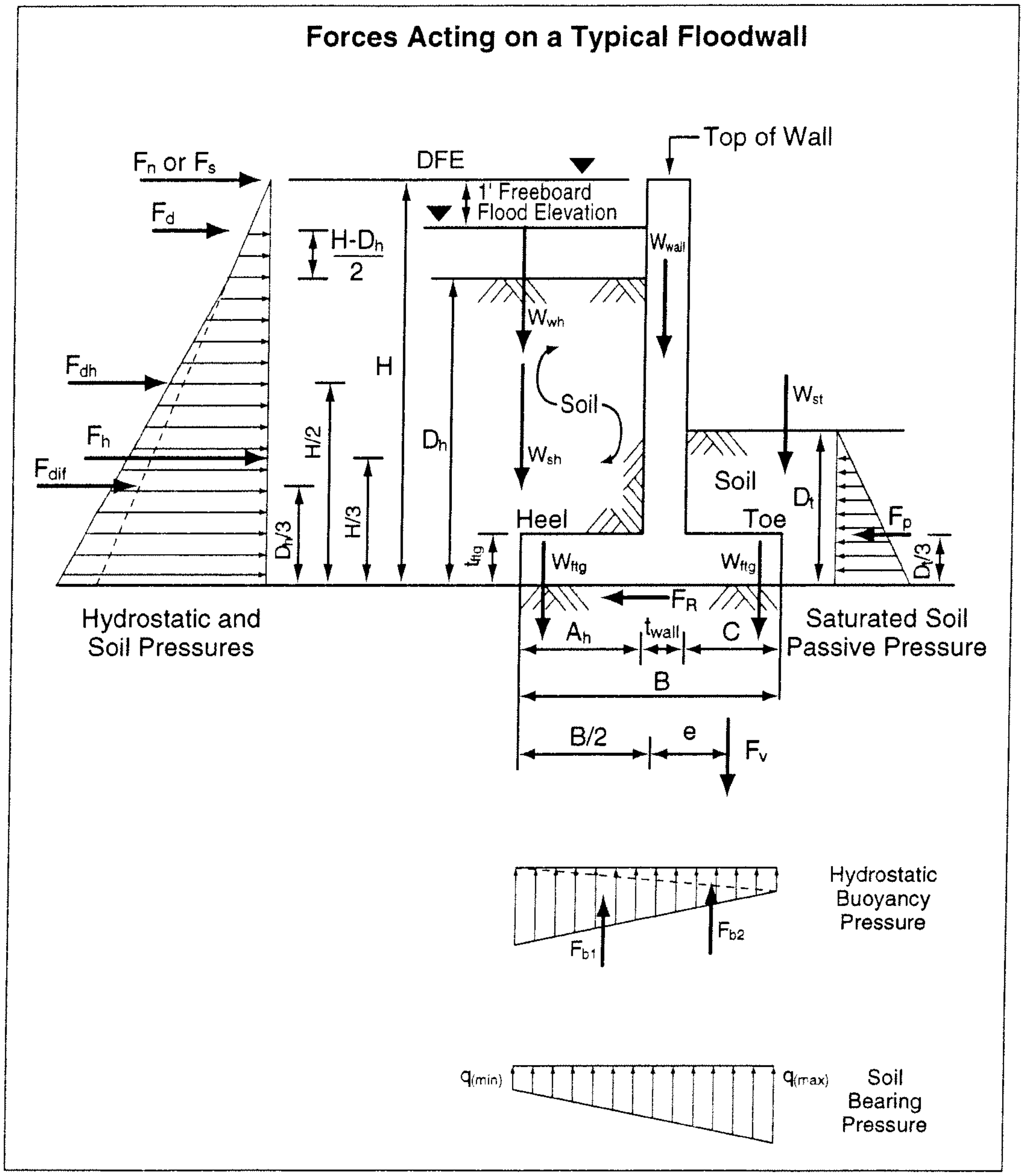


Figure VI-F17: Forces Acting on a Floodwall

1. **Lateral forces:** These forces are mainly the hydrostatic and differential soil/water forces behind the wall, and the saturated soil force in front of the wall. Hydrostatic and soil forces are as described in Chapter IV - Determination of Hazards.

2. **Vertical forces:** The vertical forces are buoyancy and the various weights of the wall, footing, soil, and water acting upward and downward on the floodwall. The buoyancy force, $\mathbf{F_b}$, acting at the bottom of the footing is computed as follows:

FORMULA

$$\mathbf{F_b = F_{b1} + F_{b2} = \_\_\_\_ \ lbs}$$

with $\mathbf{F_{b1}}$ and $\mathbf{F_{b2}}$ computed as follows:

$$\mathbf{F_{b1} = 1/2\ \gamma\ H\ B}$$
(From Formula IV-8)

$$\mathbf{F_{b2} = 1/2\ \gamma\ D_t\ B}$$
(From Formula IV-8)

where: $\mathbf{F_b}$ is the total force due to buoyancy, in pounds;
$\mathbf{F_{b1}}$ is the buoyancy force, in pounds, due to hydrostatic pressure at the floodwall heel acting at a distance of B/3 from the heel;
$\mathbf{F_{b2}}$ is the buoyancy force, in pounds, due to hydrostatic pressure at the floodwall toe, acting at a distance of B/3 from the toe;
γ is the specific weight of water (62.4 pounds per cubic foot);
B is the width of the footing, in feet;
H is the floodproofing design depth, in feet;
$\mathbf{D_t}$ is the depth of soil above the floodwall toe, in feet.

(See Figure VI-F17)

Formula VI-F1: Buoyancy on a Floodwall

The gravity forces acting downward are:

- the unit weight of floodwall ($\mathbf{W_{wall}}$);

$$W_{wall} = (H - t_{ftg})t_{wall}S_g = \_\_\_\_\_ \text{ lbs/LF}$$

where: $\mathbf{W_{wall}}$ is the weight of the wall, in pounds;
$\mathbf{H}$ is the floodproofing design depth in feet;
$\mathbf{t_{ftg}}$ is the footing thickness, in feet;
$\mathbf{t_{wall}}$ is the wall thickness, in feet;
$\mathbf{S_g}$ is the unit weight of wall material (concrete is 150 pounds per cubic foot);

(See Figure VI-F17)

Formula VI-F2: Floodwall Weight

- the unit weight of the footing ($\mathbf{W_{ftg}}$);

$$W_{ftg} = B\, t_{ftg}\, S_g = \_\_\_\_ \text{lbs/LF}$$

where: $\mathbf{W_{ftg}}$ is the weight of the footing, in pounds;
$\mathbf{B}$ is the width of the footing, in feet;
$\mathbf{t_{ftg}}$ is the footing thickness, in feet;
$\mathbf{S_g}$ is the unit weight of wall material (concrete is 150 pounds per cubic foot)

(See Figure VI-F17)

Formula VI-F3: Footing Weight

- the unit weight of the soil over the toe (**W_{st}**);

FORMULA

$$W_{st} = C(D_t - t_{ftg})(\gamma_{soil}) = \_\_\_\_\_ \text{ lbs/LF}$$

where: **W_{st}** is the weight of the soil over the toe, in pounds;
C is the width of the footing toe, in feet;
D_t is the depth of the soil above the floodwall toe, in feet;
t_{ftg} is the footing thickness, in feet;
γ_{soil} is the unit weight of the soil, in pounds per cubic foot.

(See Figure VI-F17)

Formula VI-F4: Weight of Soil Over Floodwall Toe

The unit weight of the soil, γ_{soil}, can be obtained from the soil survey, engineering texts, or a geotechnical engineer.

- the unit weight of the soil over the heel (**W_{sh}**); and

FORMULA

$$W_{sh} = A_h(D_h - t_{ftg})(\gamma_{soil} - 62.4) = \_\_\_\_\_ \text{ lbs/LF}$$

where: **W_{sh}** is the weight of the soil over the heel, in pounds;
A_h is the width of the footing heel, in feet;
D_h is the depth of the soil above the heel, in feet;
t_{ftg} is the footing thickness, in feet;
γ_{soil} is the unit weight of the soil, in pounds per cubic foot.

(See Figure VI-F17)

Formula VI-F5: Weight of Soil Over Floodwall Heel

- the unit weight of the water above the heel ($\mathbf{W_{wh}}$).

$$\mathbf{W_{wh} = (A_h)(H - t_{ftg})(62.4) = \_\_\_\_ lbs/LF}$$

where: $\mathbf{W_{wh}}$ is the weight of the water above the heel, in pounds;
$\mathbf{A_h}$ is the width of the footing heel, in feet;
$\mathbf{H}$ is the floodproofing design depth, in feet;
$\mathbf{t_{ftg}}$ is the footing thickness, in feet;

(See Figure VI-F17)

Formula VI-F6: Weight of Water Above Floodwall Heel

The total gravity forces acting downward, $\mathbf{W_G}$, in pounds can be computed as the sum of the individual gravity forces:

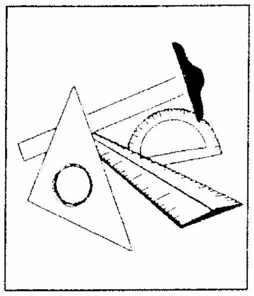
$$\mathbf{W_G = W_{wall} + W_{ftg} + W_{st} + W_{sh} + W_{wh} = \_\_\_\_ lbs/LF}$$

Formula VI-F7: Total Gravity Forces Per Linear Foot of Wall

Therefore the net vertical force, $\mathbf{F_v}$, is then calculated as:

$$\mathbf{F_v = W_G - F_b \geq 0}$$

Formula VI-F8: Net Vertical Force



Step 4: Check sliding.

This step involves the computation of the sliding forces, the forces resisting sliding, and the factor of safety against sliding. For a stable condition, the sum of forces resisting sliding should be larger than the sum of the sliding forces.

1. **Sliding Forces:** The sum of the sliding (lateral hydrostatic, hydrodynamic, and impact) forces, F_H, is computed as follows:

FORMULA

$$F_H = F_h + F_{dif} + (F_{dh} \text{ or } F_d) + (F_n \text{ or } F_s) = \_\_\_\_ \text{ lbs}$$

where: F_H is the cumulative lateral hydrostatic force acting at a distance H/3 from the point under consideration, in pounds;

F_h is the lateral hydrostatic force due to standing water in pounds; and

F_{dif} is the differential soil/water force acting due to combined free-standing water and saturated soil conditions, in pounds.

F_{dh} is the equivalent hydrostatic pressure due to low velocity flood flows, in pounds;

F_d is the hydrodynamic force against the structure due to high velocity flood flows, in pounds;

F_n is the normal impact force in pounds, and

F_s is the special impact force in pounds.

The computation of F_H, F_h, F_{dif}, F_{dh}, F_d, F_A, and F_s is presented in Formulas IV-4, IV-6, IV-10, IV-13, IV-14, and IV-15.

(See Figure IV-17)

Formula VI-F9: Sliding Forces

2. **Resisting Forces:** The forces resistant to sliding are the frictional force, $\mathbf{F}_{fr}$, between the bottom of the footing; the cohesion force, $\mathbf{F}_c$, between the footing and the soil; and the soil and the saturated soil force, $\mathbf{F}_p$, over the toe of the footing. These resisting forces are computed as follows:

    a. Frictional Force: The frictional force, $\mathbf{F}_{fr}$, between the bottom of the footing and the soil is a function of net vertical force, $\mathbf{F}_v$, times coefficient of friction, $\mathbf{C}_f$. The coefficient of friction, $\mathbf{C}_f$, between the base and the soil depends on the soil properties. (See Table VI-F1).

FORMULA

$$\mathbf{F}_{fr} = \mathbf{C}_f\ \mathbf{F}_v = \_\_\_\_\ \mathbf{lbs}$$

where: $\mathbf{F}_{fr}$ is the friction force between the footing and the soil, in pounds;

$\mathbf{C}_f$ is the coefficient of friction between the footing and the soil; and

$\mathbf{F}_v$ is the net vertical force acting on the footing, in pounds, as was previously presented in Formula VI-F8.

Formula VI-F10: Frictional Forces

b. Cohesion Force: The cohesion force between the base and the soil, $\mathbf{F_c}$, is obtained by multiplying the width of the footing, B, by the allowable cohesion value of the soil. This allowable cohesion value is usually obtained from a geotechnical analysis of the soil. The cohesion between the footing and the soil may be destroyed or considerably reduced due to contact from water. Due to potentially high variations in the allowable cohesion value of a soil, the cohesion is usually neglected in the calculations; unless the value of cohesion is ascertained by soil tests or other means, it should be taken as zero in the calculations.

$$\mathbf{F_c = C_s\,B = \_\_\_\_ \ lbs}$$

where: $\mathbf{F_c}$ is the cohesion force between the base and the soil in pounds;

$\mathbf{C_s}$ is the allowable cohesion in pounds per square foot (usually assumed to be zero), and

$\mathbf{B}$ is the width of the footing, in feet.

(See Figure VI-F17)

Formula VI-F11: Cohesion Force

c. <u>Saturated Soil Force Over the Toe</u>: The saturated soil force over the toe, $\mathbf{F_p}$, is calculated as:

FORMULA

$$F_p = \frac{1}{2}\left[k_p(\gamma_{soil} - \gamma) + \gamma\right]D_t^2 = \_\_\_\_ \text{ lbs}$$

where: $\mathbf{F_p}$ is the passive saturated soil force over the toe, in pounds;

γ_{soil} is unit weight of the soil (pounds per cubic foot); and

$\mathbf{D_t}$ is the depth of the soil over the floodwall toe, in feet.

$\mathbf{k_p}$ is the passive soil pressure coefficient

γ is the specific weight of water in lbs/ft^3.

(See Figure VI-F17)

Formula VI-F12: Saturated Soil Force Over Floodwall Toe

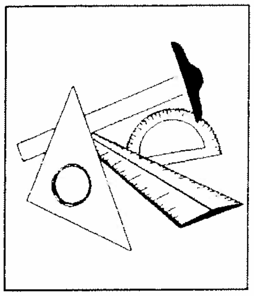
The passive soil pressure coefficient, $\mathbf{k_p}$, typically ranges from 2-5. Typical values are 2 for plastic clays, 3 for clayey silts and poorly graded gravels, and 3-4 or well graded sands. Consult a geotechnical engineer for more precise values.

The sum of the resisting forces to sliding, $\mathbf{F_R}$, is calculated as the sum of the individual resisting forces to sliding, as shown below.

FORMULA

$$F_R = F_{fr} + F_c + F_p = \_\_\_\_ \text{lbs}$$

Formula VI-F13: Sum of Resisting Forces to Sliding



3. **Factor of Safety Against Sliding:** For the stability of the wall, the sum of resisting forces to sliding, $\mathbf{F_R}$, should be larger than the sum of the sliding forces, $\mathbf{F_H}$. The ratio of $\mathbf{F_R}$ over $\mathbf{F_H}$ is called the Factor of Safety against sliding, $\mathbf{FS_{(SL)}}$, and is calculated as:

$$FS_{(SL)} = F_R/F_H = \_\_\_\_ \geq 1.5$$

where: $\mathbf{FS_{(SL)}}$ is the factor of safety against sliding (should be greater than 1.5);

$\mathbf{F_R}$ is the sum of the forces resisting sliding in pounds; and

$\mathbf{F_H}$ is the sum of the sliding forces (cumulative lateral hydrostatic force) in pounds.

Formula VI-F14: Factor of Safety Against Sliding

The factor of safety against sliding should be at least 1.5. If the factor of safety is determined to be less than 1.5, the designer should lower the footing, increase the amount of fill over the footing, and/or change the footing dimensions, then go back to Step 3 and try again (as is illustrated in the flow chart for design of floodwall).

Step 5: Check overturning.

The potential for overturning should be checked about the bottom of the toe (Figure VI-F5). For a stable condition, the sum of resisting moments, $\mathbf{M_R}$, should be larger than the sum of the overturning moments, $\mathbf{M_O}$. The ratio of $\mathbf{M_R}$ over $\mathbf{M_O}$ is called the Factor of Safety against overturning, $\mathbf{FS_{(OT)}}$.

1. **Overturning Moments**: The overturning moments are due to hydrostatic and hydrodynamic forces, impact loads, saturated soil, and the buoyancy forces acting on

When hydrodynamic input loads act on the floodwall sections parallel to the flow and the downstream facing wall, Formulas VI-F9 and VI-F15 will produce conservative results. Further detailed analysis may result in smaller sections and a corresponding reduction in cost.

the footing. The sum of the overturning moments, M_O, is calculated as:

FORMULA

$$M_O = F_h(H/3) + F_{dif}(D/3) + F_{b1}(2B/3) + \left[F_{dh}(H/2) \text{ or } F_d(H - D_h/2 + D_h)\right] + (F_nH \text{ or } F_sH) + F_{b2}(B/3) = \_\_\_\_ \text{ foot-lbs}$$

where:

- M_O is the sum of the overturning moments, in foot-lbs;
- F_h is the lateral hydrostatic force due to standing water, in pounds (Formula IV-4);
- F_{dif} is the differential soil/water force acting due to combined free-standing water and saturated soil conditions (Formula IV-6);
- F_{b1} is the buoyancy force, in pounds, due to hydrostatic pressure at the floodwall heel acting at a distance of B/3 from the heel, (Formula VI-F1);
- F_{b2} is the buoyancy force, in pounds, due to hydrostatic pressure at the floodwall toe, acting at a distance of B/3 from the toe, (Formula VI-F1);
- F_{dh} is low velocity force (Formula IV-10);
- F_d is hydrodynamic force (Formula IV-13);
- F_n is normal impact force (Formula IV-14);
- F_s is special impact force (Formula IV-15);
- B is the width of the footing, in feet;
- H is the height of the wall, in feet;
- D is the height of the soil above the heel, in feet; and
- D_h is the depth of the soil above the heel, in feet.

Formula VI-F15: Sum of Overturning Moments

2. **Resisting Moments:** The resisting moments are due to all vertical downward forces and the lateral force due to soil over the toe. The sum of resisting moments, M_R, is calculated as:

FORMULA

$$\mathbf{M_R = W_{wall}\,(C+(t_{wall}/2)) + W_{ftg}\,(B/2) + W_{st}\,(C/2) + W_{sh}\,(B-(A_h/2)) + W_{wh}\,(B-(A_h/2)) + F_p\,(D_t/3) = \_\_\_\_ \text{foot-lbs}}$$

where:

M_R	is the sum of the resisting moments in foot-lbs;
W_{wall}	is the weight of the wall, in pounds;
t_{wall}	is the wall thickness, in feet;
W_{ftg}	is the weight of the footing, in pounds, (Formula VI-F3);
B	is the width of the footing, in feet;
W_{st}	is the weight of the soil over the toe, in pounds, (Formula VI-F4);
C	is the width of the footing toe, in feet;
D_t	is the depth of the soil above the floodwall toe, in feet;
W_{sh}	is the weight of the soil over the heel, in pounds, (Formula VI-F5);
A_h	is the width of the footing heel, in feet;
W_{wh}	is the weight of the water above the heel, in pounds, (Formula VI-F6); and
F_p	is the passive saturated soil force over the toe, in pounds (Formula VI-F12).

(Refer to Figure VI-F17)

Formula VI-F16: Sum of Resisting Moments

3. **Factor of Safety Against Overturning**: As mentioned earlier, for a stable condition, the sum of resisting moments, $\mathbf{M_R}$, should be larger than the sum of the overturning moments, $\mathbf{M_O}$, resulting in a factor of safety greater than 1.0. However, the factor of safety against overturning, $\mathbf{FS_{(OT)}}$, should not be less than 1.5. If $\mathbf{FS_{(OT)}}$ is found to be less than 1.5, the designer should increase the footing dimensions, then go back to Step 3 and try again (see the flow chart for design of floodwall).

$$\mathbf{FS_{(OT)} = M_R/M_O = \_\_\_\_ \geq 1.5}$$

where: $\mathbf{FS_{(OT)}}$ is the factor of safety against overturning (should be greater than 1.5);

$\mathbf{M_R}$ is the sum of the resisting moments, in foot-lbs, (Formula VI-F15); and

$\mathbf{M_O}$ is the sum of the overturning moments, in foot-lbs, (Formula VI-F16).

Formula VI-F17: Factor of Safety Against Overturning

Step 6: Calculate eccentricity.

The final resultant of all the forces acting on the wall and its footing is a force acting at a distance, **e**, from the centerline of the footing. This distance, **e**, is known as eccentricity. The calculation of eccentricity is important to ensure that the bottom of the footing is not in tension. The eccentricity value is also needed for the calculation of soil pressures in Step 7. The eccentricity, **e**, is calculated as:

FORMULA

$$\mathbf{e = (B/2) - ((M_R - M_O)/F_v) = \_\_\_\_\_ \ feet}$$

where: **e** is the eccentricity, in feet;
B is the width of the footing, in feet;
$\mathbf{F_v}$ is the net vertical force acting on the footing, in pounds, (Formula VI-F8);
$\mathbf{M_O}$ is the overturning moment, in foot-lbs, (Formula VI-F15); and
$\mathbf{M_R}$ is the resisting moment, in foot-lbs, (Formula VI-F16).

(Refer to Figure VI-F17)

Formula VI-F18: Eccentricity

This eccentricity, **e**, should be less than 1/6 of the footing width. If **e** is found to exceed B/6, then change the footing dimensions, go back to Step 3, and try again (see flow chart for design of floodwall).

Step 7: Calculate soil pressures.

The soil pressures, **q**, are determined from the following formula.

$$q = (F_v/B)(1 \pm (6e/B)) = \_\_\_\_ \text{ lbs/ft}^2$$

where: **q** is the soil pressure created by the forces acting on the wall, in pounds per square foot;

F_v is the net vertical force acting on the footing, in pounds, (Formula VI-F8);

B is the width of the footing, in feet; and

e is the eccentricity, in feet (Formula VI-F18).

(Refer to Figure VI-F17)

Formula VI-F19: Soil Pressure

The maximum value of **q** should not exceed the allowable soil bearing capacity. The bearing capacity of soil varies with the type of soil, moisture content, temperature, and other soil properties. The allowable values should be determined by a geotechnical engineer. Some conservative allowable bearing values for a few soil types are given in Table VI-F1 Soil Factors for Floodwall Design. If the computed value of **q** is more than the allowable soil bearing value, increase the footing size, then go back to Step 3 and try again (see flow chart for design of the floodwall).

Step 8: Select reinforcing steel.

> The bending moment ($\mathbf{M_b}$) for sizing reinforcing steel in the vertical floodwall component is the product of the lateral hydrostatic force ($\mathbf{F_H}$) and the distance between the point of force application and the bottom of the vertical floodwall component ($\mathbf{H/3 - t_{ftg}}$).

Select an appropriate reinforcing steel size and spacing to resist the expected bending moment, $\mathbf{M_b}$. Figure VI-F18 illustrates a typical floodwall reinforcing steel installation. The cross-sectional area of steel reinforcing required can be computed using Formula VI-F20. This formula assumes use of steel with a $F_y = 60$ ksi.

FORMULA

$$\mathbf{A_s = \frac{\frac{M_b}{1000}}{1.76 d_f} = \_\_\_\_ \text{ in}^2\text{/ one-foot width of wall}}$$

where:

- $\mathbf{A_s}$ is the cross-sectional area of reinforcing steel required per foot width of wall, in square inches;
- $\mathbf{M_b}$ is the bending moment, in foot-lbs;
- **1000** is a factor used to convert foot-pounds to foot-kips; and
- $\mathbf{d_f}$ is the distance between the reinforcing steel and the floodwall face opposite retained material, in inches.

(Refer to Figure VI-F18)

Formula VI-F20: Cross-Sectional Area of Steel

d_f is typically the floodwall thickness minus 3-1/2" to allow a minimum of 3" between the reinforcing steel and the floodwall edge.

The selection of reinforcing steel in the footing portion of a floodwall is computed using Formula VI-F20 while modifying $\mathbf{M_b}$ for top and bottom steel considerations. For top steel, the moment is the product of the weight of soil and water over the heel ($\mathbf{w_{sh}+w_{wh}}$) and the heel length ($\mathbf{A_h}$) divided by 2.

The selection of bottom steel is a function of the soil bearing pressure. The moment can be computed by adding the soil bearing pressure at the toe edge of the vertical floodwall section to twice the maximum soil bearing pressure ($\mathbf{q + 2q_{max}}$) and multiplying this sum by toe length squared over 6 ($\mathbf{C^2/6}$). The soil bearing pressure at the toe edge of the vertical floodwall section ($\mathbf{q}$) can be computed by ratio from the calculations (**for** $\mathbf{q_{min}, q_{max}}$) shown in step 7.

Using the computed cross-sectional area of reinforcing steel, refer to ACI to select the most appropriate steel reinforcing bar size and spacing.

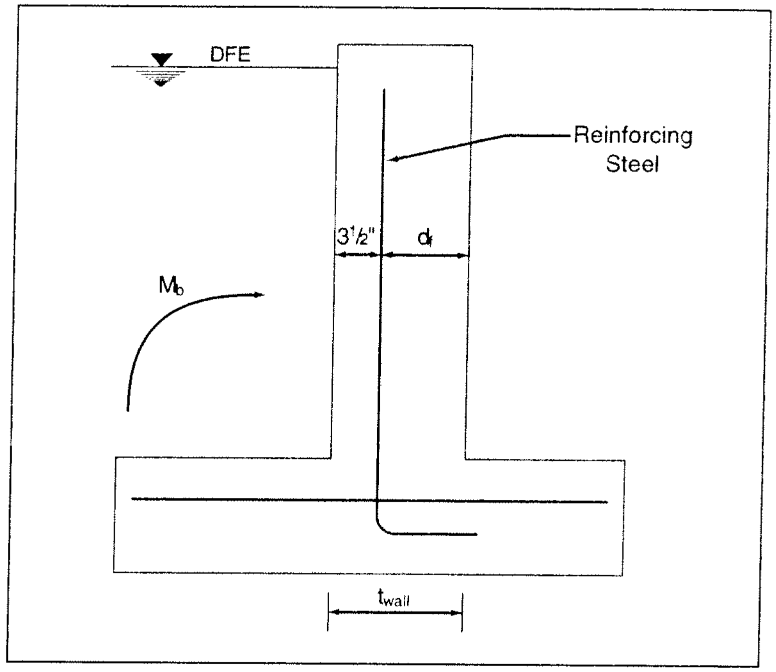
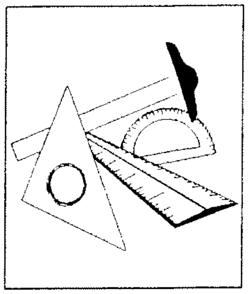


Figure VI-F18: Typical Reinforcing Steel Configuration



Floodwall Sample Calculation

FLOODWALL SAMPLE DESIGN

Objective:

Design a cantilever floodwall to protect a residence subject to 3 feet of flooding. Site soil conditions are as follows: Clean Dense Sand, Unit Weight = 120 lbs/ft^3; Allowable Soil Bearing Capacity = 2,000 lbs/ft^2; Equivalent Fluid Pressure of Soil = 78 lbs/ft^3; Coefficient of Friction (C_f) = 0.47; Passive Soil Pressure (k_p) = 3.69; and Cohesion = 0. The floodwall is in an area of potential normal impact loading and expected flood velocities are 5 fps.

Step 1: Assume wall height and footing depth (refer to Figure VI-F17).

$\mathbf{H}$ = 7.0 feet
$\mathbf{D}$ = 4.0 feet
$\mathbf{D_h}$ = 5.0 feet
$\mathbf{t_{ftg}}$ = 1.0 feet

Step 2: Determine dimensions (refer to Figure VI-F17).

$\mathbf{B}$ = 5.0 feet
$\mathbf{A_h}$ = 2.5 feet
$\mathbf{C}$ = 1.5 feet
$\mathbf{t_{wall}}$ = 1.0 feet

Wall and footing to be reinforced concrete having unit weight of 150 lbs/ft^3.

Floodwall Sample Calculation

Step 3: Calculate forces.

Determine Lateral Forces:

Formula IV-4	F_h = 1/2(62.4)(7)(7) = 1,528.8 lbs/LF.
Formula IV-6	F_{dif} = 1/2(78-62.4)(5)(5) = 195.0 lbs/LF.
Formula IV-9	**dh** = (1.25)(5)(5)/(2)(32.2) = 0.49 feet.
Formula IV-10	F_{dh} = (62.4)(0.49)(7) = 211.96 lbs/LF.
Formula IV-14	F_n= (1,000)(5)/(32.2)(1) = 155.28 lbs.
Formula IV-9	F_H= 1,528.80+195.00+211.96=1,935.76 lbs/LF.

Since F_n acts only at a single point, we will not include loading into the uniform lateral floodwall loading. Once the floodwall is sized, we will evaluate the wall perpendicular to flow to determine ability to resist the impact loading. If necessary this wall will be redesigned to resist impact loads. This process will avoid overdesigning of the entire floodwall.

Formula VI-F12	F_p = 1/2(3.69(120-62.4) + 62.4)(4)(4) = 2,199.55 lbs/LF.

Determine Vertical Forces:

Formula VI-F1	F_{b1} = 1/2(62.4)(5)(7) = 1,092.00 lbs.
Formula VI-F1	F_{b2} = 1/2(62.4)(5)(4) = 624.00 lbs.
Formula VI-F1	F_b = 1,092 + 624 = 1,716.00 lbs.
Formula VI-F2	W_{wall} = (7-1)(1)(150) = 900.00 lbs.
Formula VI-F3	W_{ftg} = (5)(1)(150) = 750.00 lbs.
Formula VI-F4	W_{st} = (2)(5-1)(120-62.4) = 720.00 lbs.
Formula VI-F5	W_{sh} = (4)(5-1)(120-62.4) = 921.60 lbs.
Formula VI-F6	W_{wh} = (2.5)(7-1)(62.4) = 936.00 lbs.
Formula VI-F7	W_G = 900 + 750 + 576 + 540 + 936 = 3,702.00 lbs.
Formula VI-F8	F_v = 3,702.00 - 1,716.00 = 1,986.00 lbs.

Floodwall Sample Calculation

Step 4: Check sliding.

Formula VI-F10	$F_{fr} = 0.47(1{,}986) = 933.42$ lbs.
Formula VI-F13	$F_R = 933.42 + 2{,}199.55 = 3{,}132.97$ lbs.
Formula VI-F14	$FS_{(SL)} = 3{,}132.97/1935.76 = 1.62$.

OK for sliding since
$1.62 > 1.5$ (recommended)

Step 5: Check overturning.

Formula VI-F15

$$M_O = (1{,}935.76)(7/3) + (195)(5/3) + (1{,}092)(10/3) + (624)(5/3) + (211.96)(7/2) = 9{,}314.05 \text{ foot-lbs.}$$

Formula VI-F16

$$M_R = (900)(1.5(1/2)) + (750.00)(5/2) + (540)(1.5/2) + (576)(5-(2.5/2) + (936)(5-(2.5/2)) + (2{,}199.55)(4/3) = 12{,}682.74 \text{ foot-lbs.}$$

Formula VI-F17

$$FS_{(OT)} = 12{,}682.74/9{,}314.05 = 1.36.$$

No good. Try increasing the footing size to overcome the overturning momement. Assume B = 7.0 feet; A_h = 4.0 feet; and C = 2.0 feet. This requires revision of Steps 3 and 4 for which the results are shown below. F_h, F_{dif}, F_{dh}, F_H, F_p, W_{wall} will not change. Recompute vertical forces.

Formula VI-F1	$F_{b1} = 1/2(62.4)(7)(7) = 1{,}528.80$ lbs.
Formula VI-F1	$F_{b2} = 1/2(62.4)(7)(4) = 873.60$ lbs.
Formula VI-F1	$F_b = 1{,}528.80 + 873.60 = 2{,}402.40$ lbs.
Formula VI-F2	$W_{wall} = (7-1)(1)(150) = 900.00$ lbs.
Formula VI-F3	$W_{ftg} = (7)(1)(150) = 1{,}050.00$ lbs.
Formula VI-F4	$W_{st} = (2)(5-1)(120-62.4) = 720.00$ lbs.
Formula VI-F5	$W_{sh} = (4)(5-1)(120-62.4) = 921.60$ lbs.

Floodwall Sample Calculation

Formula VI-F6 $W_{wh} = (4)(7\text{-}1)(62.4) = 1{,}497.60$ lbs.

Formula VI-F7 $W_G = 900.00 + 1{,}050.00 + 921.60 + 720.00 + 1{,}497.60 = 5{,}089.20$ lbs.

Formula VI-F8 $F_v = 5{,}089.20 - 2{,}402.40 = 2{,}686.80$ lbs.

Recheck Sliding

Formula VI-F10 $F_{fr} = 0.47(2{,}686.80) = 1{,}262.80$ lbs.

Formula VI-F13 $F_R = 1{,}262.80 + 2{,}199.55 = 3{,}462.35$ lbs.

Formula VI-F14 $FS_{(SL)} = 3{,}462.35/1{,}935.76 = 1.79$.

OK for sliding.

Recheck Overturning

Formula VI-F15 $M_O = (1{,}528.80)(7/3) + (195)(5/3) + (1{,}528.80)(2(7)/3) + (873.60)(7/3) + (211.96)(7/2) = 13{,}806.85$ foot-lbs.

Formula VI-F16 $M_R = (900)(2t(1/2)) + (1{,}050.00)(7/2) + (720)(2/2) + (921.60)(7\text{-}(4/2)) + (1{,}497.60)(7\text{-}(4/2)) + (2{,}199.55)(4/3) = 21{,}673.74$ foot-lbs.

Formula VI-F17 $FS_{(OT)} = 21{,}673.74/13{,}806.85 = 1.57$

OK for overturning since
$1.57 > 1.5$ (recommended)

Step 6: Determine eccentricity.

Formula VI-F18 $e = 7/2 - (21{,}673.74 - 13{,}806.85)/2{,}686.80 = 0.57 < 7/6$ OK

Floodwall Sample Calculation

Step 7: Check soil pressures.

Formula VI-F19

$$q = (2{,}686.80/7)(1 \pm 6(.57)/7)))$$
$$q_{min} = (2{,}686.80/7)(1 -(6(.57)/(7))) = 195.64 \text{ lbs/ft}^2$$
$$q_{max} = (2{,}686.80/7)(1+(6(.57)/(7))) = 572.64 \text{ lbs/ft}^2 < 2{,}000 \quad \text{OK}$$

Step 8: Select reinforcing steel.

For steel in the vertical floodwall section:

Formula VI-F20

$$A_s = (1935.76)(7/3-1)/1000/(1.76)(8.5) = 0.17 \text{ in}^2$$

For top steel in the footing section:

Formula VI-F20

$$A_s = ((921.60 + 936.00)(2.5)/2)/1000/(1.76)(8.5) = 0.13 \text{ in}^2$$

For bottom steel in the footing section:

ratio **q** from q_{min}, q_{max}

$$q = 572.64 - (1.5/8)(572.64-195.64) = 501.95 \text{ lbs/ft}^2$$

Formula VI-F20

$$A_s = ((1.5)^2/6)(501.95 + 2(572.64))/1000/(1.76)(8.5) = 0.04 \text{ in}^2$$

From American Concrete Institute Reinforced Concrete Design Handbook Table 9a: use #4 bars on 14 inch centers in the vertical floodwall section, use #4 bars on 18 inch centers for the top steel in the footing section, and use #2 bars on 12 inch centers for the botom steel in the footing section. Other ACI documents have similar information.

Floodwall Sample Calculation

Since this floodwall design situation also includes normal impact forces, we must check the wall perpendicular to the flow for this loading situation. However, since impact loads do not act uniformly along the wall, the factor of safety of sliding/overturning can be lowered as long as it is above 1.0. This check will change only F_H, M_O, $FS_{(SL)}$, $FS_{(OT)}$, and e.

Formula IV-14 $F_n = (1{,}000)(5)/(32.2)(1) = 155.28$ lbs.

Formula IV-9 $F_H = 1{,}528.80 + 195.00 + 211.96 + 155.28 = 2{,}091.04$ lbs/LF.

Formula VI-F14 $FS_{(SL)} = 3{,}462.35/2{,}091.04 = 1.65$.

OK for sliding since
$1.65 > 1.0$ (recommended)

Formula VI-F15 $M_O = (1{,}528.80)(7/3) + (195)(5/3) + (1{,}528.80)(2(7)/3) + (873.60)(7/3) + (211.96)(7/2) + (155.28)(7) = 14{,}893.81$ foot-lbs

Formula VI-F17 $FS_{(OT)} = 21{,}673.74/14{,}893.81 = 1.45$

OK for overturning since
$1.45 > 1.0$ (recommended)

Formula VI-F18 $e = 7/2 - (21{,}673.74 - 14{,}893.81)/2{,}686.80 = 0.97 < 7/6$ OK

OK for eccentricity. Therefore the wall as designed will withstand the anticipated impact loading. If the factors of safety for overturning/sliding and the eccentricity had not been acceptable, the footing should be resized or enlarged (B, A_h, and C).

(CONT'D)

Floodwalls

Table of Contents

This simplified approach uses assumed site conditions. The designer should be aware that the previous process is normally used in the design of most floodwalls. However, this design process can be shortened for floodwalls of less than three feet in height by assuming certain site-specific soil conditions and design parameters. Presented later in this section is a table of typical floodwall design sizes and reinforcement schemes that would be applicable in certain situations. The designer should be aware that unless the situation in question meets the assumptions and standard design criteria established herein, it would be prudent to complete the entire design process for the floodwall application.

FLOODWALL DESIGN - SIMPLIFIED APPROACH

The following Table VI-F3 presents general factors used in developing a standardized approach to floodwall design. If the soil conditions at the site in question do not reflect the assumed conditions below, the standard criteria approach cannot be utilized, and the detailed design process presented earlier in this section must be used.

Based on the stability requirements (assuming no cohesion), footing dimensions for various wall heights, footing depths, and two different soil types have been calculated. The calculation results are shown in Tables VI-F4 and VI-F5. The designer can utilize the following tables to specify floodwall/footing dimensions required for heights up to 7.0 feet, which reflect flooding levels from 1.0 to 4.0 feet (including a minimum of three feet of soil over the footing). Flooding levels can be computed as (H - D_t). It is important to note that these dimensions are very conservative and the designer may be able to reduce the dimensions.

In these calculations, the following assumptions have been made:

1 - Wall and footing are of concrete
2 - Wall thickness = 1' - 0"
3 - Footing thickness = 1' - 0"
4 - Minimal debris impact potential
5 - Minimal velocity (<5fps)
6 - Reinforcing consisting of #4 steel bars on 12-inch centers in both the wall and footing

Table VI-F2 **Assumed Soil Factors for Simplified Floodwall Design**

Soil Type	Allowable Bearing Pressure, lbs./ft.^{2}	k_p Passive Soil Pressure Coefficient	C_f Friction Factor	Equivalent Fluid Pressure for Saturated Soil	Unit Weight of Soil lbs/ft^3
Clean, dense sand and gravel GW, GP, SW, SP	2,000	3.70	0.55	75	120
Dirty sand and gravel of restricted permeability GM, GM-GP, SM, SM-SP	2,000	3.00	0.45	77	115

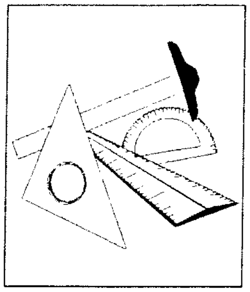


Table VI-F3 **Typical Floodwall Dimensions for Clean, Dense, Sand and Gravel Soil Types: (GW, GP, SW, SP)**

Height of Floodwall* H (ft)	Depth of Soil on Water* Side D_h (ft)	Depth of Soil on Land* Side D_t (ft)	Base Width* B (ft)	Heel Width* A_h (ft)	Toe Width* C (ft)
4' - 0"	3' - 0"	3' - 0"	2' - 6"	1'- 0"	6
5' - 0"	3' - 0"	3' - 0"	4' - 6"	2' - 6"	1'- 0"
	4' - 0"	3' - 0"	4' - 0"	2' - 0"	1'- 0"
	4' - 0"	4' - 0"	4' - 6"	2' - 6"	1'- 0"
6' - 0"	3' - 0"	3' - 0"	6' - 6"	3' - 6"	2' - 0"
	4' - 0"	3' - 0"	6' - 0"	3' - 6"	1' - 6"
	5' - 0"	3' - 0"	5' - 6"	3' - 0"	1' - 6"
	4' - 0"	4' - 0"	4' - 6"	2' - 6"	1' - 0"
	5' - 0"	4' - 0"	4' - 0"	2' - 6"	6"
7' - 0"	3' - 0"	3' - 0"	9' - 0"	6' - 6"	1' - 6"
	4' - 0"	4' - 0"	7' - 0"	3' - 6"	2' - 6"
	5' - 0"	4' - 0"	6' - 6"	3' - 6"	2' - 0"
	4' - 0"	3' - 0"	8' - 0"	5' - 0"	2' - 0"
	6' - 0"	3' - 0"	7' - 0"	4' - 6"	1' - 6"
	6' - 0"	4' - 0"	6' - 0"	3' - 6"	1' - 6"

*Refer to Figure VI-F17

Table VI-F4 **Typical Floodwall Dimensions for Dirty Sand and Gravel of Restricted Permeability Soil Types: (GM, GM-GP, SM, SM-SP)**

Height of Floodwall* H (ft)	Depth of Soil on Heel* D_h (ft)	Depth of Soil on Toe* D_t (ft)	Base Width* B (ft)	Heel Width* A_h (ft)	Toe Width* C (ft)
4' - 0"	3' - 0"	3' - 0"	2' - 6"	1' - 0"	0' - 6"
5' - 0"	3' - 0"	3' - 0"	5' - 0"	2' - 6"	1' - 6"
	4' - 0"	3' - 0"	4' - 6"	2' - 6"	1' - 0"
	4' - 0"	4' - 0"	4' - 0"	2' - 0"	1' - 0"
6' - 0"	3' - 0"	3' - 0"	8' - 0"	5' - 6"	1' - 6"
	4' - 0"	3' - 0"	7' - 6"	5' - 6"	1' - 0"
	5' - 0"	3' - 0"	7' - 0"	5' - 6"	0' - 6"
	4' - 0"	4' - 0"	5' - 6"	3' - 0"	1' - 6"
	5' - 0"	4' - 0"	5' - 0"	3' - 0"	1' - 6"
7' - 0"	4' - 0"	4' - 0"	8' - 0"	5' - 0"	2' - 0"
	5' - 0"	4' - 0"	7' - 0"	4' - 0"	2' - 0"
	6' - 0"	4' - 0"	6' - 6"	4' - 0"	1' - 5"

*Refer to Figure VI-F17

FLOODWALL APPURTENANCES

Floodwall appurtenances include drainage systems, stair details, wall facings, patios, existing structure connections (sealants), existing structure support (posts and columns), and closure details. Each will be discussed with illustrations, details, and photographs provided to help the designer develop details that meet the needs of their specific situation. The designer is reminded that it is likely that a local building code may have standards for the design and construction of many of these items.

Floodwall Closures

In designing floodwall closures, many of the principles discussed earlier in the dry floodproofing section apply. Watertight closures must be provided for all access openings such as driveways, stairs, and ramps, and seals should be provided for all utility penetrations. Figure VI-F19 illustrates typical floodwall closures. Structural analysis for the design of closures should follow the procedures outlined previously for shield design.

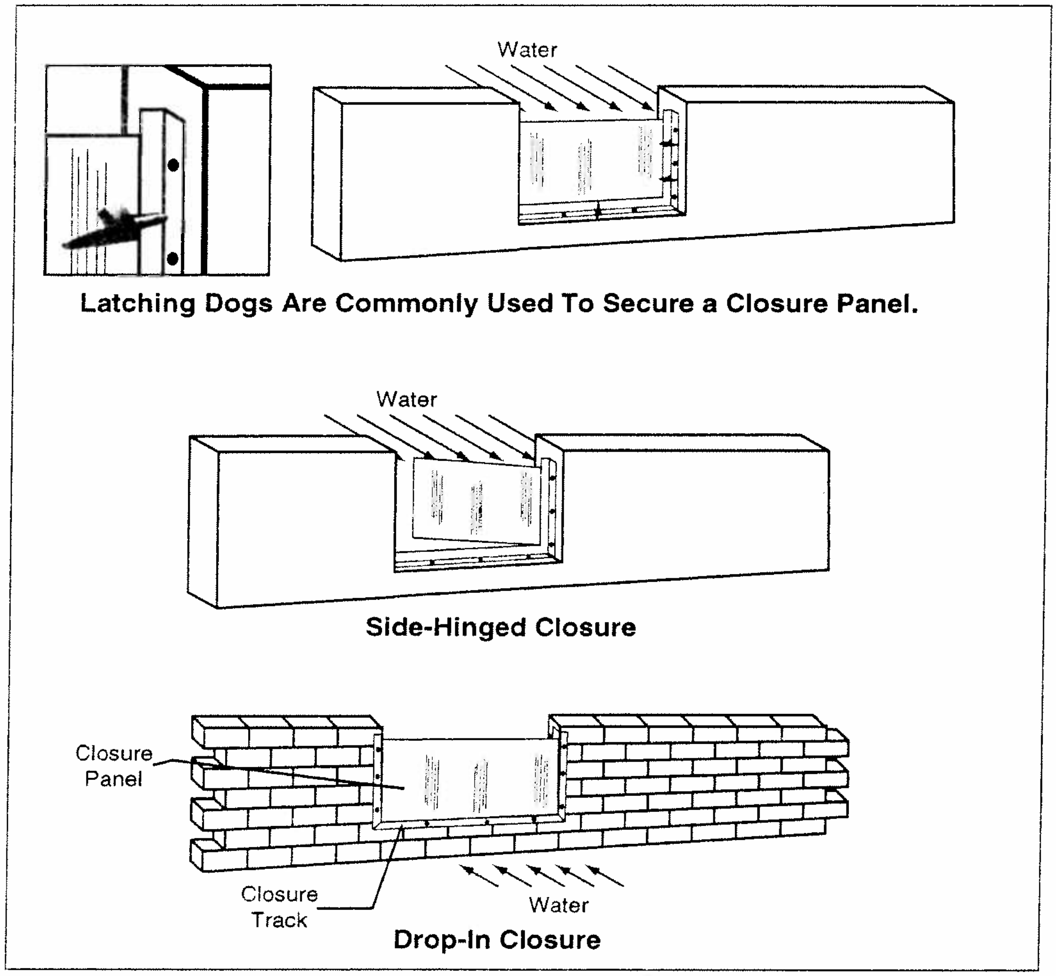


Figure VI-F19: Typical Floodwall Closures

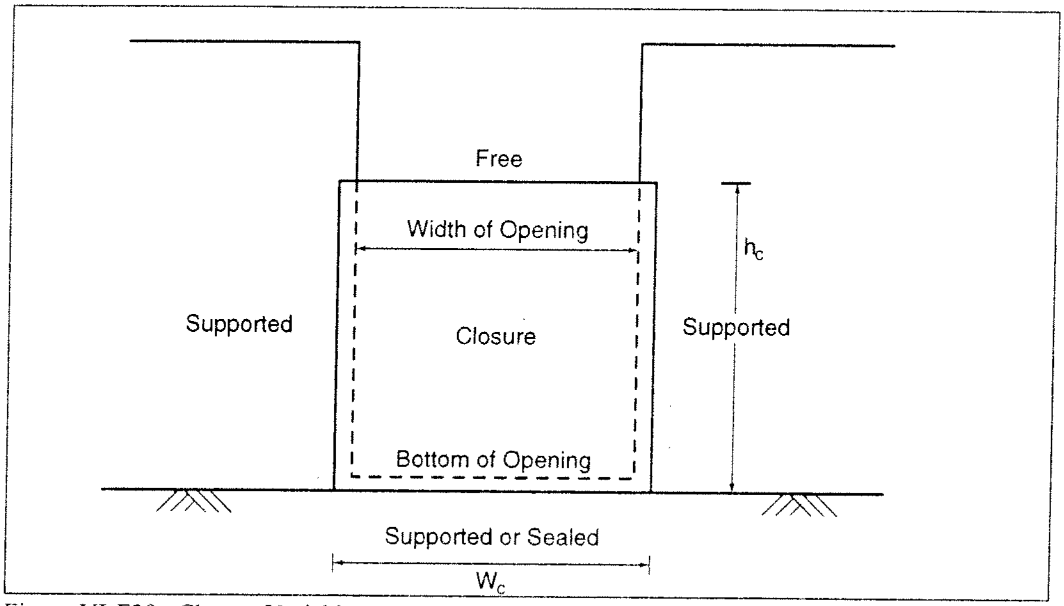


Figure VI-F20: Closure Variables

The type of closure used depends primarily on the size of the opening that needs to be protected. This will determine the type of material to be used and how the closure is to be constructed and operated.

Longer and larger closures, such as for a driveway, must be able to withstand significant flood forces, and therefore should be made of a substantial material. Normally this would be steel plate, protected against rust and corrosion. Heavy aluminum plate may also be used, although it will likely need to be reinforced. In either case, due to the weight of the closure, it is usually best that it be hinged so that it can swing into place. Hinging can be located along the bottom so the closure lies flat when not in use, or it can be placed along one side, so the closure can fold back out of the way.

For normal passage openings, aluminum is probably the most common material used. It is a lightweight material, allowing for easy fabrication and transport, and it is resistant to corrosion. Aluminum can buckle under heavy water pressure, so it may need some additional reinforcement.

For smaller openings, exterior grade plywood is also commonly used. It is relatively inexpensive and is easily fabricated. However, plywood is subject to warping if not properly stored. In addition, it will collapse under relatively low flood forces, and will usually require significant reinforcement, usually some type of wood frame.

Aluminum and plywood are both light enough to be used for temporary closures that can normally be stored in a safe location and installed only when floodwaters threaten. There are many different arrangements that can be used to install these movable closures. The more common methods include the "drop-in" shield that fits into a special slot arrangement and the "bolt-on" shield that is affixed over an opening. There are several different types of hardware that can be used to secure a closure in place, such as T-bolts, wing nuts on anchored bolts, or latching dogs.

It is absolutely essential that closures be made watertight. This is normally accomplished through the use of some type of gasket. Neoprene and rubber are materials commonly used, but there are a number of other materials readily available that perform equally as well.

The successful performance of a closure system also requires that it be held firmly against the opening being protected. Although the hydrostatic pressure of the water may help to hold the closure in place, floodwater surges can result in negative pressure that can pull off an improperly installed closure.

Whatever material is used, it must be of sufficient strength and thickness to resist bending and deflection failures. The ability of a specific material to withstand bending stresses may be substantially different from its ability to withstand deflection stresses. Therefore, to provide for an adequate factor of safety, the required closure thickness should be calculated twice: first taking into account bending stresses, and second taking into account deflection stresses. The resulting thicknesses should be compared and the larger value specified in the final closure design.

One method of determining the thickness of the closure for steel and aluminum is presented in *Formulas for Stress and Strain* by Roark and Young. For a flat plate supported on three sides, the plate thickness required due to bending stresses may be determined by the following formula:

Orientation of Openings: It is highly recommended that openings in floodwalls and levees not be placed on the upstream side. In the event that they are, Formulas VI-F21, VI-F22, VI-F23 and VI-F25 should be modified to include the expected hydrodynamic forces. Closures should not be used on upstream sides where impact loads are expected.

FORMULA

$$t = \sqrt{\frac{P_h + (P_{dh} \text{ or } P_d) W_c^2 \beta}{\text{Max } \sigma}} = \_\_\_ \text{ inches}$$

where:
- **t** plate thickness;
- P_h hydrostatic pressure due to standing water, in psi from Formula IV-4;
- W_c width of closure, in inches
- **Max σ** allowable stress for the plate material (from material handbooks), in psi; and
- β moment coefficient from Table VI-F5;
- P_{dh} **and** P_d are defined in Formulas IV-10 and IV-12.

Formula VI-F21: Plate Thickness due to Bending Stresses

Similarly, for a steel or aluminum flat plate supported on three sides, the plate thickness required due to deflection stresses may be determined by the following formula:

FORMULA

$$t = \sqrt[3]{\frac{360 \times P_h - (P_{dh} \text{ or } P_d) W_c^3}{E}} = \_\_\_ \text{ inches}$$

where:
- $\propto$ deflection coefficient from Table VI-F5; and
- E modulus of elasticity for the plate material (from material hand-books) in psi.

Formula VI-F22: Plate Thickness due to Deflection Stresses

The variables used in the above equations for plate thickness are illustrated in Figure VI-F20. Table VI-F5, Moment Coefficients details the moment and deflection coefficients as a function of the ratio of plate height to width.

Table VI-F5 **Moment (β) and Deflection (α) Coefficients**

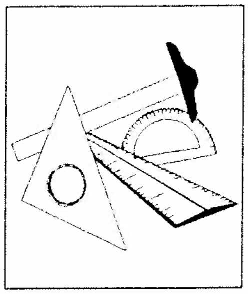
h_c/W_c*	0.05	0.67	1.00	1.50	2.00	2.50	3.00	3.50	4.00
α	0.11	0.16	0.20	0.28	0.32	0.35	0.36	0.37	0.37
β	0.03	0.03	0.04	0.05	0.06	0.06	0.07	0.07	0.07

*See Figure VI-F19

Allowable values for σ and E may be found for steel plates in *Manual of Steel Construction*, American Institute of Steel Construction, and for aluminum plates in *Aluminum Construction Manual*, the Aluminum Association.

The method of designing plywood closure plates is similar to that for steel and aluminum closure plates except that the varying structural properties of plywood make using a single formula inappropriate. Because these structural properties are dependent upon the grades of plywood sheet, the type of glue used, and the direction of stress in relation to the grain, determination of the thickness and grade required for a plywood closure is best achieved by assuming a thickness and grade of plywood and calculating its ability to withstand bending, shear, and deflection stresses. This involves calculating the actual bending, shear, and deflection stresses in the plywood closure plate for the thickness and grade specified. These actual stress values are then compared with the maximum allowable bending, shear, and deflection stresses (taken from *APA Plywood Design Specifications*).

If the actual stresses computed are less than the maximum allowable stresses for bending, shear, and deflection, then the thickness and grade specified are acceptable for that application. However, if either of the actual bending or



shear stresses or deflection exceeds the maximum allowable values, the closure plate is not acceptable and a new thickness and/or grade of plywood closure plate should be specified and the calculations repeated until all actual stresses are less than the maximum allowed. The following guidance has been prepared to illustrate one method of designing plywood closure plates. Note that a one-way horizontal span is assumed because the variability of plywood properties is dependent upon grain and stress direction.

> The designer is referred to *Plywood Design Specifications*, published by the Engineered Wood Association, for a detailed discussion of design guidelines.

Compute bending moment on horizontal one-way span (supported on two sides only).

FORMULA

$$M_b = \frac{(P_h + (P_{dh} \text{ or } P_d))W_c^2}{8} = \_\_\_\_ \text{in-lbs/in}$$

where: M_b is the bending moment in in-lbs/in;
P_h is the hydrostatic pressure due to standing water, in psi from Formula IV-4;
W_c is the width of the closure in inches; and
P_{dh} and P_d are defined in Formulas IV-10 and IV-12.

Formula VI-F23: Bending Moment

Check bending stress.

FORMULA

$$f_b = \frac{M_b}{KS} = \_\_\_\_ \text{psi}$$

where: f_b is the bending stress in psi;
M_b is the bending moment in in-lbs/in; and
KS is the effective section modulus from a reference in in^3/in.

Formula VI-F24: Bending Stress

If the calculated bending stress for the specified plate (f_b) is less than the maximum bending stress allowed (F_b) (from references), the closure plate is adequately designed for bending applications. If not, the closure should be redesigned and the calculation repeated.

Compute shear force.

FORMULA

$$V_s = \frac{(P_h + (P_{dh} \text{ or } P_d))W_c^{\,2}}{2} = \_\_\_\_ \textbf{ pounds}$$

where: V_s is the shear force in pounds;
P_h is the hydrostatic force in psi Formula IV-4;
W_c is the width of the closure plate in inches; and
P_{dh} and P_d are defined in Formulas IV-10 and IV-12.

Formula VI-F25: Shear Force

Check shear stress.

FORMULA

$$f_s = \frac{V_s}{C_{RS}} = \_\_\_\_ \textbf{ pounds}$$

where: f_s is the shear stress in pounds; and
C_{RS} is the rolling shear constant dimensionless.

Formula VI-F26: Shear Stress

If the calculated shear stress for the specified plate (f_s) is less than the maximum shear stress allowed (F_s), the closure plate is adequately designed for shear applications. If not, the closure should be redesigned and the calculations repeated.

Compute deflection for a single one-way span.

$$\Delta_b = \frac{(P_h + (P_{dh} \text{ or } P_d))(W_c + y)^4}{921.6(E)(I)} = \_\_\_\_ \text{ inches}$$

where: Δ_b is the computed deflection in inches;

P_h is the hydrostatic pressure, in psi, from Formula IV-4;

W_c is the unsupported width in inches;

y is a support width factor in inches;

E is the Modulus of Elasticity in psi;

I is the Effective Moment of Inertia in in^4/ft; and

P_{dh} and P_d are defined in Formulas IV-10 and IV-12.

Formula VI-F27: Plate Deflection for a One-Way Span

Check deflection.

A customary and acceptable level of deflection may be expressed as

$$\Delta_b \text{ (allowable)} = \frac{W_c}{240} = \_\_\_\_ \text{ inches}$$

where: Δ_b is the allowable deflection in inches; and

W_c is the unsupported width in inches.

Formula VI-F28: Allowable Deflection

If the calculated deflection (Δ_b) is less than the allowable deflection (Δ_b), the closure plate is adequately designed for deflection situations. If not, the closure should be redesigned and the calculations repeated.

Closure plates of plywood are limited to short spans and low water heights. It should also be noted that most plywood will deteriorate when exposed to high moisture. Therefore, plywood closure plates should be examined periodically and replaced as necessary.

Drainage Systems

When designing a floodwall system, the designer must verify that it will not cause the flooding of adjacent property by blocking normal drainage. Specific information and local requirements can be obtained from the local zoning commission, the building inspector, or the water control board. Before deciding on a design, the designer should check local building codes, floodplain and/or stormwater management ordinances, zoning ordinances, or property convenents that may prohibit or restrict the type of wall planned.

The flood protection design should be developed to divert both floodwater and normal rainfall away from the structure. By directing the floodwater and rainfall away from the structure, the designer can minimize potential erosion, scour, impacts, and water ponding. Typical design provisions include:

- Regrading the site
- Sloping applications
- Drainage system(s)

Regrading the site basically involves contouring. The surface can be contoured to improve the drainage and minimize floodwater turbulence. Ground covers or grasses, especially those with fibrous root systems, can be effective in holding soil against erosion and scour effects of floodwaters.

Sloping applications include providing a positive drainage for engineered applications such as patios, sidewalks and driveways. The material is slightly inclined, typically at a 1% to 2% grade, to an area designed for collection, which includes inlets, ditches, or an existing storm drain pipe system. Figures VI-F21 and VI-F22 show two patio drainage options, and Figure VI-F23 shows a floor drain section typically used to provide positive drainage for patio areas enclosed by floodwalls. These configurations can also be used with sump and sump pump installations.

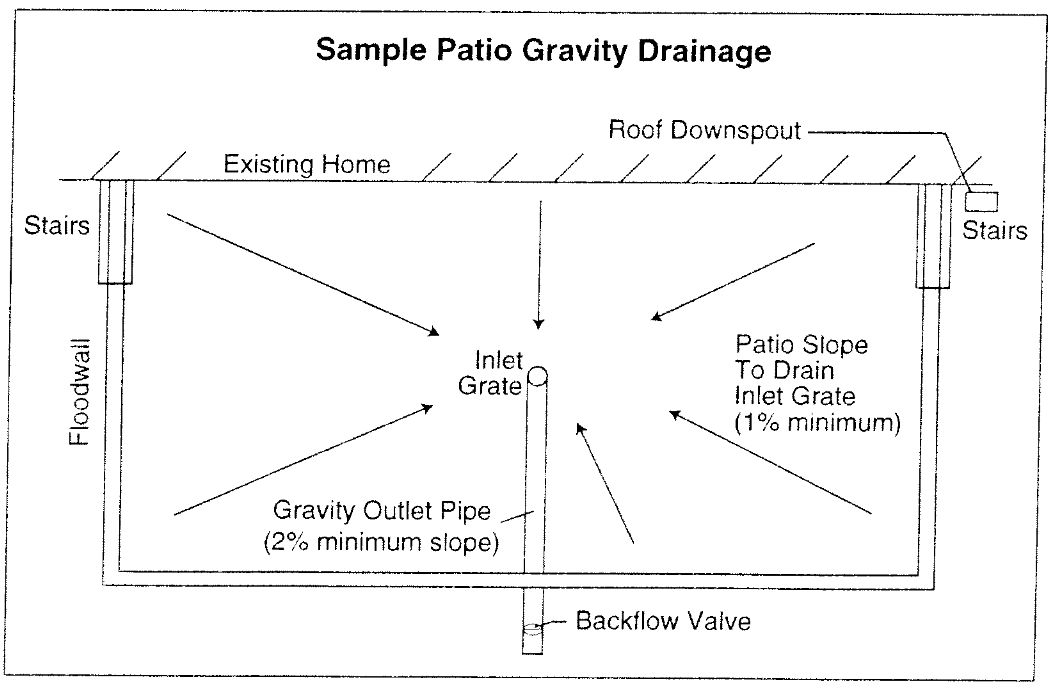


Figure VI-F21: Sample Patio Drainage to an Outlet

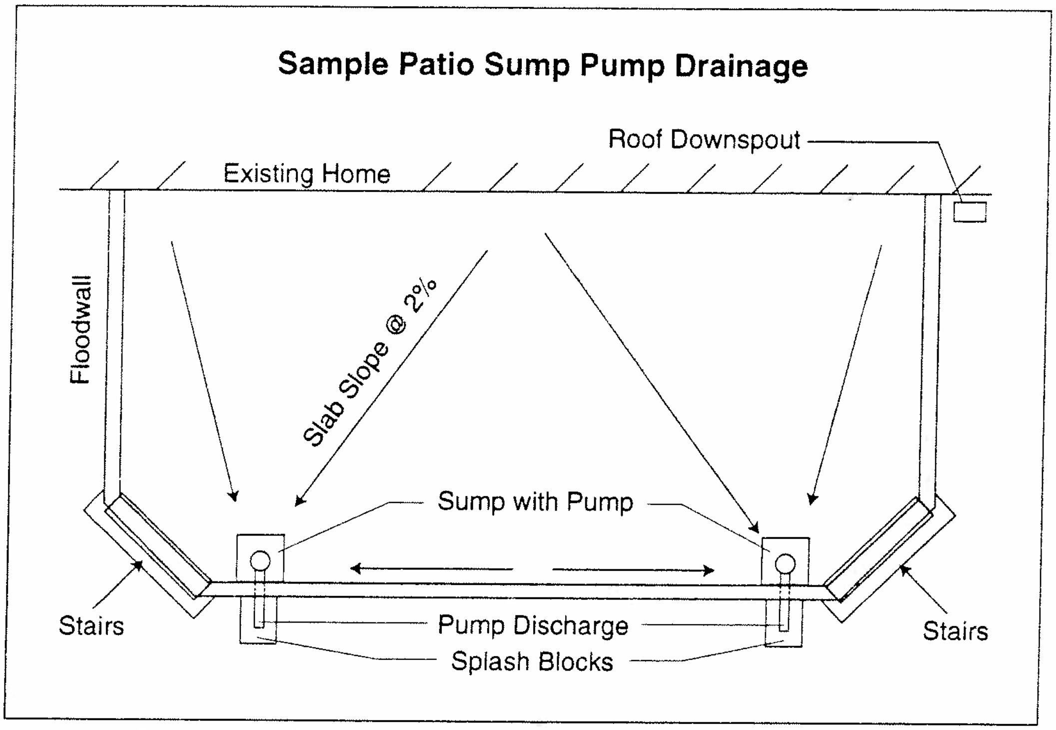


Figure VI-F22: Sample Patio Drainage to a Sump

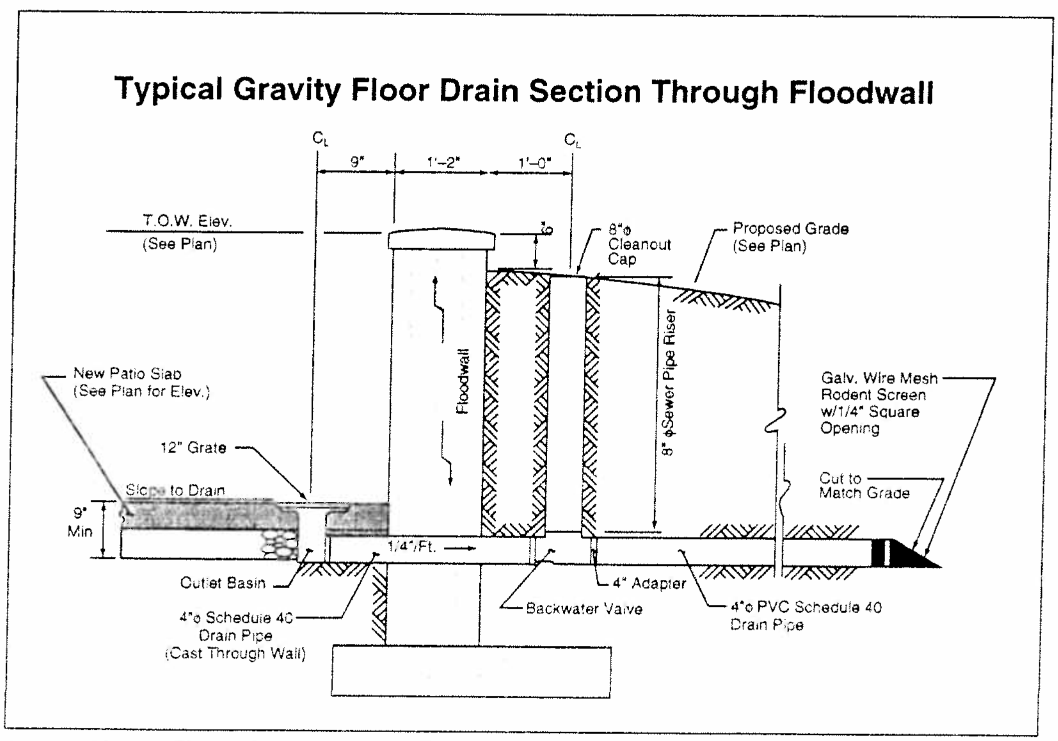


Figure VI-F23: Typical Gravity Floor Drain

Figure VI-F24: Typical Patio Sump Pump Installation

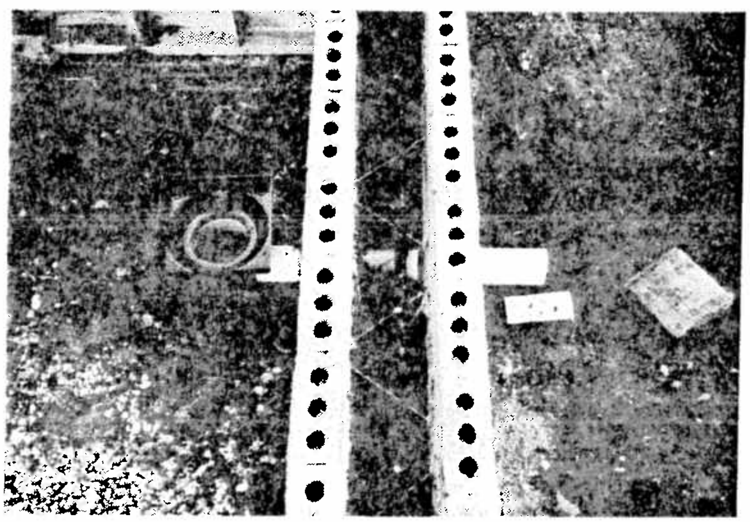


Figure VI-F25: Typical Patio Gravity Floor Drain Installation

Drainage systems are a series of pipes that collect and route interior drainage to a designated outfall. Usually the drainage operation is underground and works through a gravity process. However, when grading and sloping will not allow the gravity system to function, provisions for a pumping method, such as a sump pump, should be made. Information on the design of sumps and sump pump applications is provided in the Dry Floodproofing section of this chapter.

For example, in its simplified form, a gutter and downspout outlet, which can be found on almost all houses, is a type of storm drainage system. Provisions at the downspout outfall should also be developed in the site drainage design.

Included in the drainage system application is a backflow valve. The unit, sometimes referred to as a check valve, is a type of valve that allows water to flow one way but automatically closes when water attempts to flow in the opposite direction. Figure VI-F26 shows a typical flood-wall with a check valve for gravity drainage. The elevation of the drain outlet should be as high as possible to delay activating the backflow valve, while maintaining a minimum of 2% slope on the drain pipe.

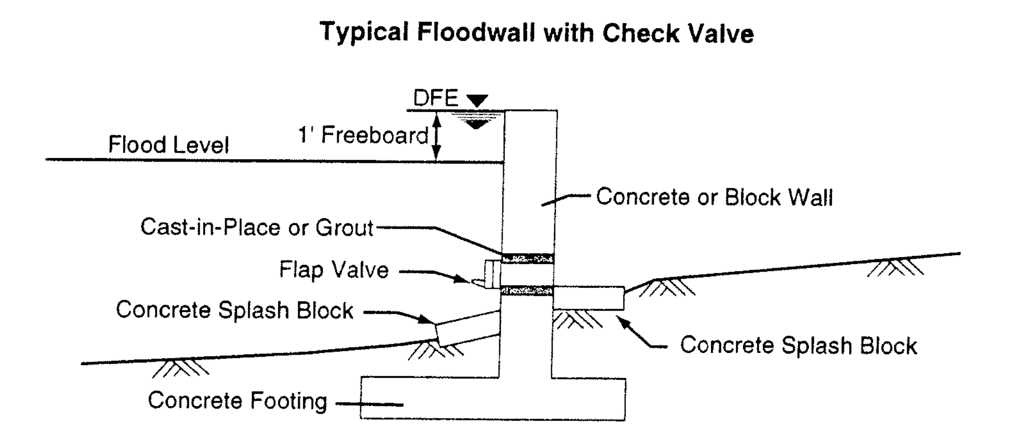
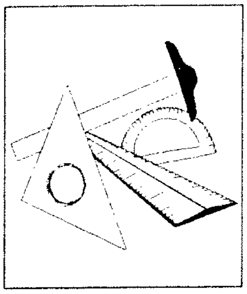


Figure VI-F26: Typical Floodwall With Check Valve

The success of the gravity drainage system is predicated on the fact that the floodwater will reach its maximum height after the rainfall at the site has lessened or stopped. Therefore, when the backflow valve is activated, little or no water will accumulate on the patio slab (usually after the rainstorm). However, should this condition not exist, the use of a sump pump and/or design of runoff storage within the enclosed area should be provided.



SEEPAGE AND LEAKAGE

Floodwalls should be designed and constructed to minimize seepage and leakage during the design flood. Without proper design considerations, floodwalls are susceptible to seepage through the floodwall; seepage under the floodwall; leakage between the floodwall and residence; and leakage through any opening in the floodwall.

Seepage Through the Floodwall

All expansion and construction joints shall be constructed with appropriate waterstops and joint sealing materials. To prevent excess seepage at the tension zones, the maximum deflection of any structural floor slab or exterior wall shall not exceed 1/500 of its shorter span. Figure VI-F27 illustrates the use of waterstops to prevent seepage through a floodwall.

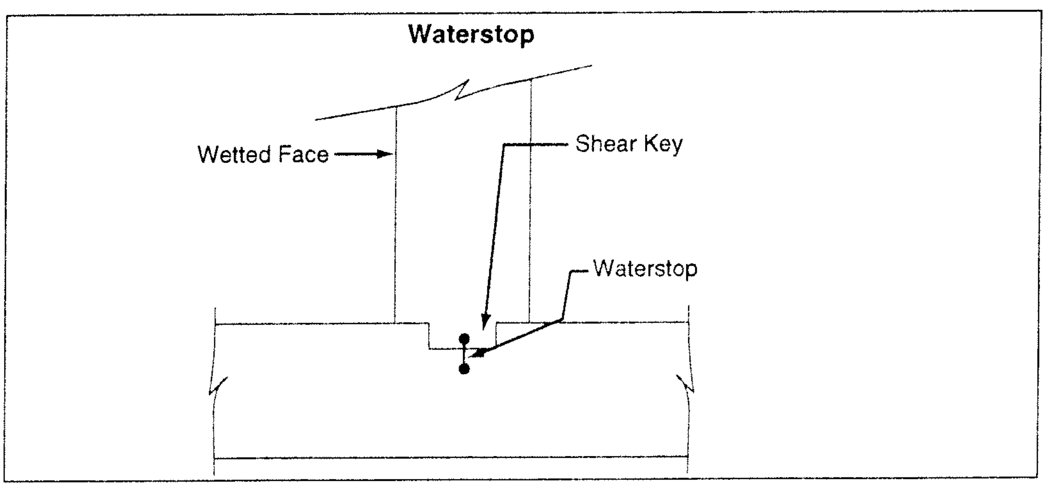


Figure VI-F27: Waterstop

Seepage Under the Floodwall

The structure design may also include the use of impervious barriers or cutoffs under floodwalls to decrease the potential for the development of full hydrostatic pressures and related seepage. These cutoffs must be connected to the impervious membrane of the building walls to operate effectively.

To meet these requirements, it may be necessary to provide impervious cutoffs to prevent seepage beneath the floodwall. This requirement is critical for structures that are designed on highly pervious foundation materials. It may also be necessary to construct a drainage system parallel to the interior base of the floodwall to collect seepage through or under the structure and normal surface runoff from the watershed. All seepage and storm drainage should be diverted to an appropriate number of sumps or gravity drains, or pumped to the floodwater side of the structure. Normal surface runoff (during non-flood conditions) must also be taken into account in the drainage system.

Leakage Between the Floodwall and Residence

> The effectivenss of house floodproofing can be increased by placing fill against the house to keep floodwaters from coming into direct contact with the structure.

The connection between the existing house wall and the floodwall is normally not a fixed connection, because the floodwall footing is not structurally tied to the house foundation footing. Therefore, a gap or expansion joint may exist between the two structures that offers the potential for leakage. This gap should be filled with a waterproof material that will work during seasonal freeze-thaw cycles.

One alternative, illustrated in Figure VI-F28, is to utilize a 1/2-inch bituminous expansion material, high-density caulking, and 1/2-inch polyurethene sealant.

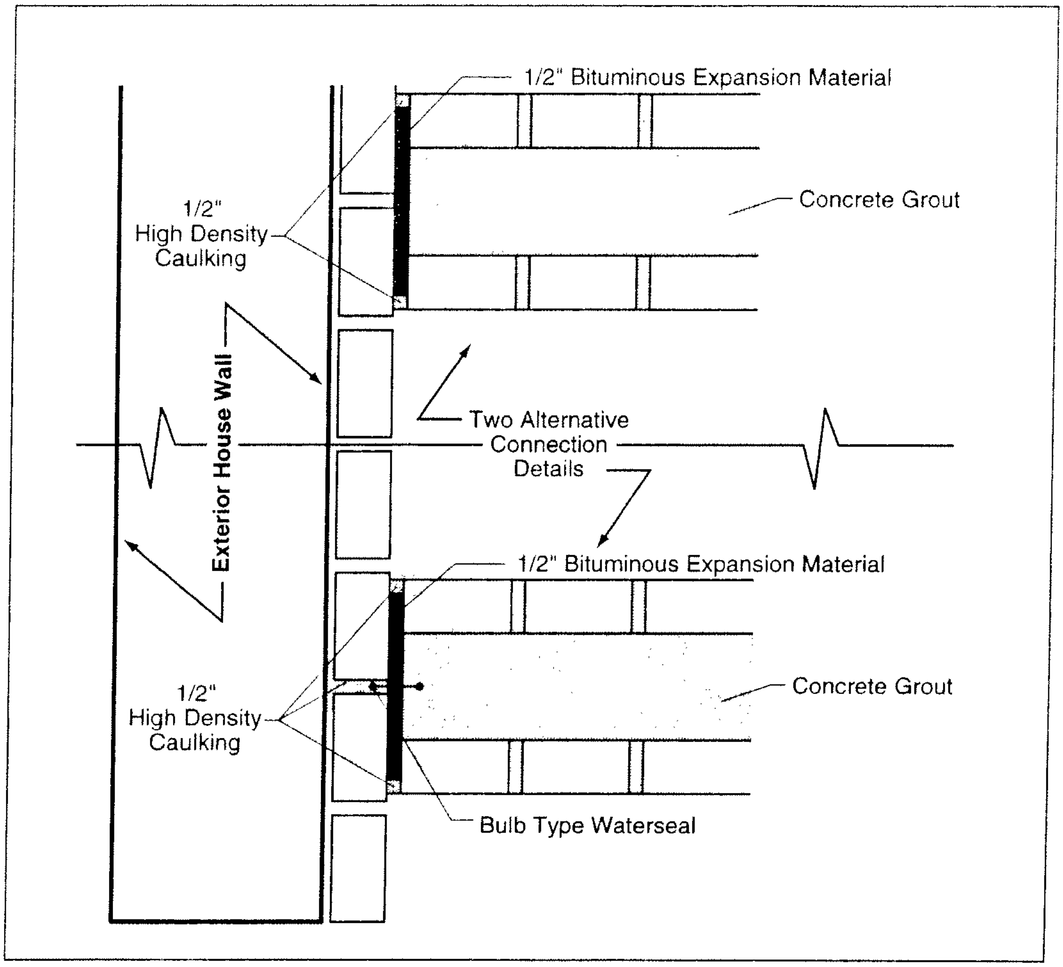


Figure VI-F28: Floodwall to House Connection

ARCHITECTURAL DETAILS

Floodwalls can be constructed in a variety of designs and materials. By taking into account the individual house design, topography, and construction materials, and with some imagination, the designer can build a floodwall to not only provide a level of flood protection, but also enhance the appearance of the home.

The floodwall design can be a challenge to landscape or to blend into the terrain. By using natural topography and employing various types of floodproofing techniques, such as waterproofing, sealants, or decorative bricks or blocks, the designer can make a floodwall not only blend in with the house and landscape, but also make an area more attractive by creating a privacy fence or by outlining a patio or garden area.

The two most common applications of cosmetic facing of a floodwall consist of brick facing and decorative block facing. This is illustrated in Figure VI-F29.

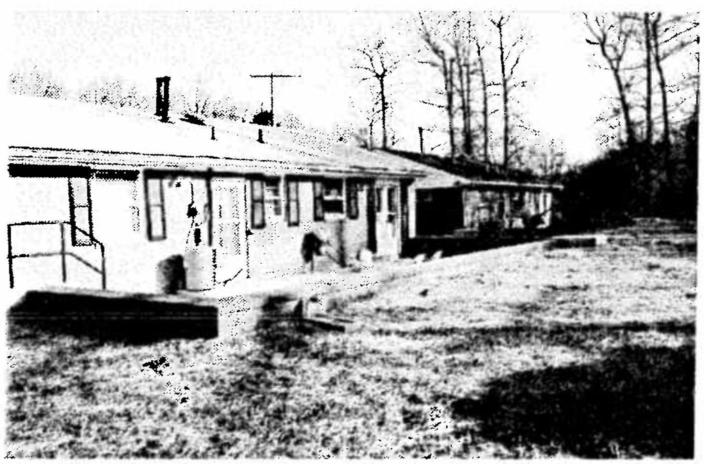


Figure VI-F29: Typical Cosmetic Facings

Typical floodwall design often incorporates the use of a patio, which is enclosed by the floodwall. A concrete slab-on-grade or decorative brick paving can be constructed between the house and the floodwall, which will create an attractive and useful feature. The slab-on-grade or brick paving can serve four very functional purposes:

- Patio area for the homeowner;
- Additional bracing for the floodwall;
- Positive drainage away from the building towards drainage collection points; and
- Impervious barrier inside the floodwall to reduce infiltration of water into the soil adjacent to the structure.

The patio floor or slab-on-grade is set four inches below the door openings to provide for a reasonable amount of water storage to accommodate rainfall and roof-gutter spillage that may occur after the floodwater has reached the elevation that will have closed the backflow valve on the patio drain. The concrete slab is sloped to a floor drain (or drains) which discharge, if existing grade allows, through a gravity pipe or sump pump installation.

In addition to designing patio applications, a qualified design professional can develop architectural and structural modifications that will accommodate existing/future wood decks or roof overhangs (illustrated in Figures VI-F30 and 31). These supports can bear on the floodwall's cap, provided additional structural modifications to the floodwall and foundations are furnished to sustain the increased load from above.

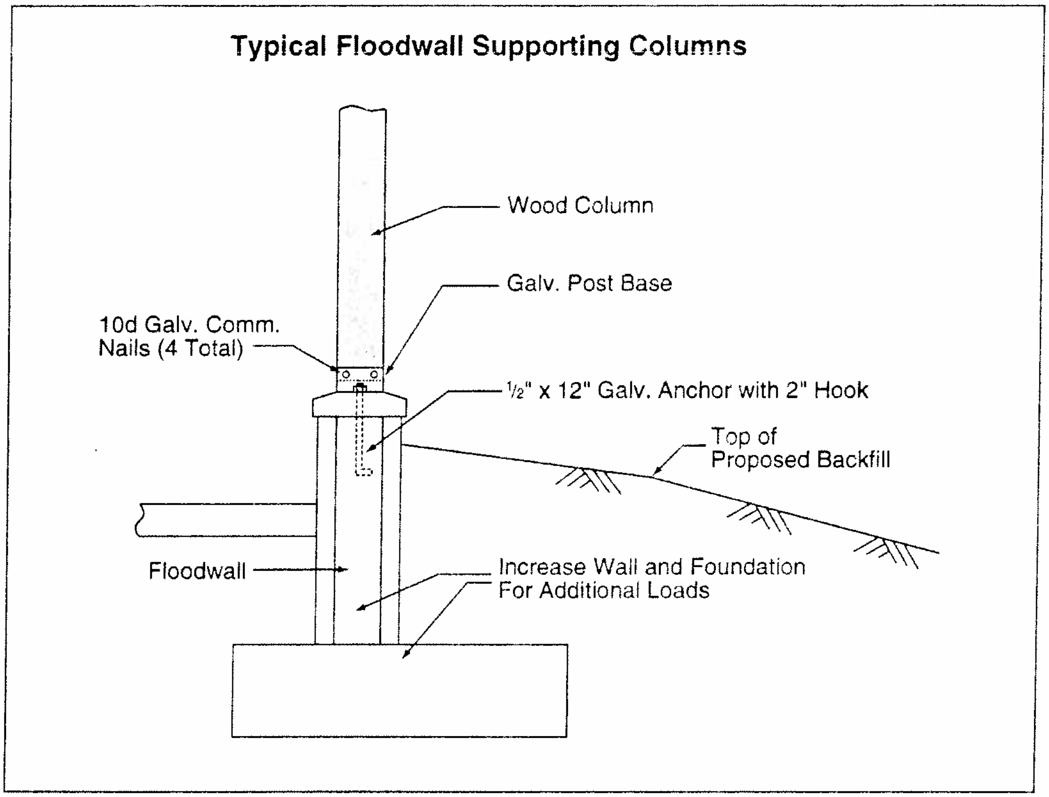


Figure VI-F30: Floodwall Supporting Columns

Figure VI-F31: Floodwall Supporting Columns

Residential access requirements, such as driveways, sidewalks, doors, and other entrances, will need to be examined during the design. These entrances may create gaps in the floodwall. Every effort should be made to design passages that extend over the top of the wall and not through it. A stile stairway over a floodwall provides access while not creating an opening in the floodwall.

The stile is a series of steps up and over the floodwall and to the designed grades, which thereby closes the floodwall gap and provides a permanent flood protection. Handrails, railings, and stair treads and other safety features must be incorporated into the stile stairway in accordance with local building codes.

Typical Step Detail

Scale: 3/4" = 1'-0"

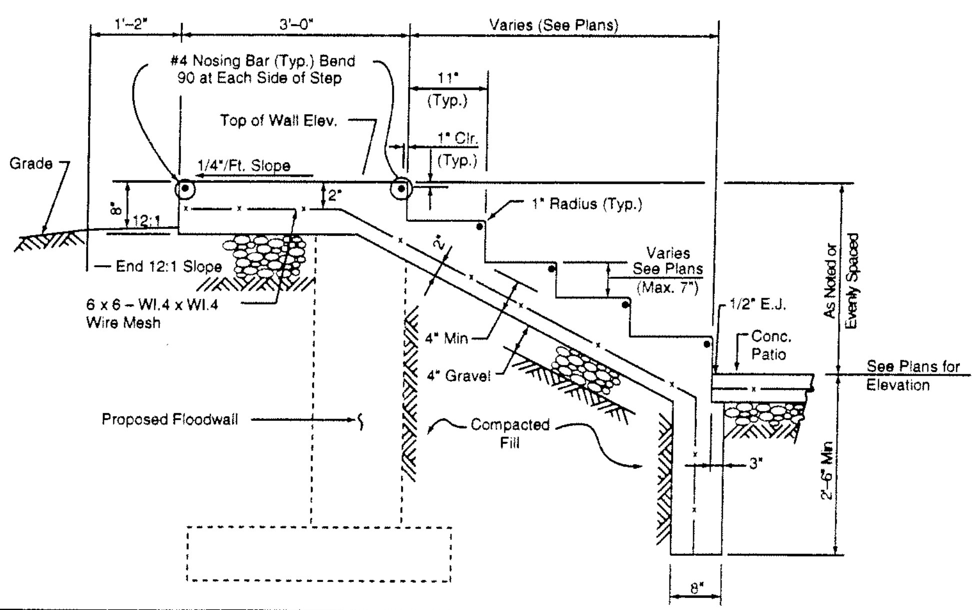


Figure VI-F32: Typical Step Detail

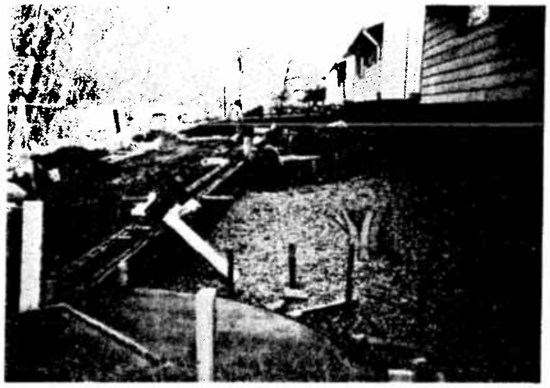


Figure VI-F33: Typical Floodwall Steps

Figure VI-F34: Typical Floodwall Steps

> Landscaping inside and overchanging a protected area may generate organic debris that could clog drains. Plants should be selected that do not result in clogged drains from falling leaves or fruit.

In addition to the architectural qualities the floodwall can provide, the entire site area can be finished with landscaping features such as planter boxes, trees, and shrubs. Vegetative cover and stone aggregate can also be utilized not only to enhance the flood protection, but also as a method of erosion and scour prevention. A qualified landscape architect should be consulted when selecting material coverage for a particular area. Roots, foliage, leaves, and even potential growth patterns of certain trees and shrubs should be accounted for in the selection of landscaping materials. Figure VI-F35 shows a typical landscaping alternative.

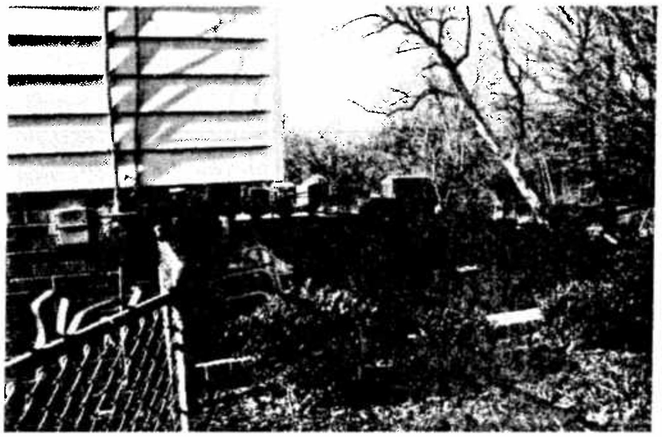


Figure VI-F35: Typical Floodwall Landscaping

MAINTENANCE CONSIDERATIONS

Once the flood protection has been constructed, a maintenance schedule should be adopted to ensure the system will remain operational during flooding conditions. Floodwalls should be inspected annually for structural integrity. The visual investigation should include a checklist and photographic log of:

- Date of inspection
- General floodwall observations involving wall cracking (length, width, locations), deteriorated mortar joints, misalignments, chipping, etc.
- Sealant observation, including displacement, cracking, and leakage.
- Overall general characteristics of the site including water ponding/leakage, drain(s), and drainage and site landscaping.
- Operation of the sump pump, generator/battery, and installation of any closures.
- Testing of drains and backflow valves

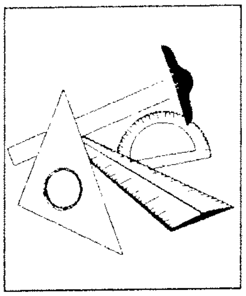
Additionally, the entire flood protection system should be inspected after a flood. A complete observation including a photographic record similar to the annual report should be developed and may also include:

- damages associated with impacts and flood,
- excessive scour and erosion damage,

- floodwater marks, and

- functional analysis regarding the flood protection system.

The following floodwall inspection worksheet (Figure VI-F36) can be used to record observations during the annual and post-flood inspections.



Owner Name: ____________________ **Prepared By:** ____________________
Address: ____________________ **Date:** ____________________
Property Location: ____________________

Floodwall Inspection Worksheet

FLOODWALL COMPONENT	YES	NO	OBSERVATIONS
Cracking in Wall			
Mortar Joint Separation			
Wall Misalignment			
Miscellaneous Chipping & Spalling			
Possible Leakage Spots			
Sealant Displacement			
Water Ponding			
Drains Functional			
Sump Pump Operational			
Landscaping			

Sketch Area:

General Observations and Summary:

Figure VI-F36: Floodwall Inspection Worksheet

CONSTRUCTION

During the construction of a floodwall, periodic inspections should be conducted to ensure that the flood protection measure has been built per the original design intent. As a minimum the designer, owner, or owner's representative should inspect and observe the following improvements:

- Confirm adequate slope drainage, including drain pipes, patio, and grading outside the floodwall;
- Confirm that floodwall foundation was prepared in accordance with plans and specifications;
- Confirm that sealants, waterproofing, and caulking were applied per the manufacturer's requirements for installation;
- Confirm that the sump pump is operational;
- Check sample brick or decorative block (before installation) for patterns or match to existing conditions; and
- Confirm that a maintenance requirement checklist was developed and used, which included all of the manufacturer's recommendations for passive flood protection applications, sealants, drains, etc.

Levees

Table of Contents

LEVEES

Levees are embankments of compacted soil that keep shallow to moderate floodwaters from reaching a structure. A well designed and constructed levee should resist flooding up to the design storm flood elevation, eliminating exposure to potentially damaging hydrostatic and hydrodynamic forces.

This chapter outlines the fundamentals of levee design and provides the designer with an empirical design suitable to a limited range of situations. The design criteria outlined in the USACE manual number EM1110-2-1913, entitled *Design and Construction of Levees,* are complex and intricate because they must provide for a wide variety of design conditions that are not always applicable to residential levees. These additional factors could result in construction costs that are considerably higher than the value of the benefits (damages avoided) associated with construction. If certain design parameters are controlled, the costs should be greatly reduced, allowing the individual homeowner to consider this retrofitting technique an economically feasible option.

FIELD INVESTIGATION

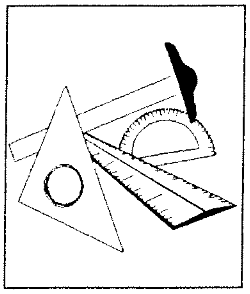
Under NFIP regulations, levees cannot be used to bring non-compliant structures into compliance.

Placement of levees in the floodway is not allowed under local floodplain regulations.

Certain conditions must exist before levees can be considered a viable retrofitting option. The questions that should be asked before proceeding any further are listed below:

- Does the natural topography around the structure in question lend itself to this technique?

  A significant portion of the cost associated with the construction of a levee hinges upon the amount of fill material needed. If the topography around the structure is such that only one or two sides of the structure need to be protected, a levee may be economical.



A settled height of six feet is the maximum elevation recommended for individual residential levees.

- Is a suitable impervious fill material readily available?

  A suitable impervious fill material, such as a CH, CL, or SC, as defined in American Society for Testing and Materials (ASTM) designation D-2487, entitled *Classification of Soils*, is required to eliminate concerns of seepage and stability.

- Do local, state, or federal laws, regulations, or ordinances restrict or prevent the construction of a levee?

  Coordination with local, state, and federal officials may be necessary to determine if the levee retrofitting option is permissible. Certain criteria exist prohibiting construction within a FEMA-designated floodway, the main portion of a stream or watercourse that conveys flow during a storm.

- Will the construction of a levee alter, impede, or redirect the natural flow of floodwaters?

  Previous calculations from Chapter IV to determine both the depth and velocity of flood flows around the structure in question should be checked to ensure that the levee will not result in increased flood hazards upstream. Also, in many cases the local floodplain administrator may require an analysis of the proposed modification to the floodplain.

- Will flood velocities allow for the use of this technique?

  If the flood velocities along the water side of the levee embankment exceed eight feet per second, the cost of protecting against the scour potential may become so great that a different retrofitting technique should be considered.

The designer of a levee should be aware that the construction of a levee may not reduce the hydrostatic pressures against a below-grade foundation. Seepage underneath a levee and the

If the assumptions listed in this chapter are not applicable to the site being considered, an experienced levee designer should be consulted or another method considered.

natural capillarity of the soil layer may result in a water level inside the levee that is equal to or above grade. This condition is worsened by increased depth of flooding outside the levee and increased flooding duration. Unless this condition is relieved, the effectiveness of the levee may be compromised. This condition, which involves the intersection of the phreatic line with the foundation, is illustrated in Figures VI-F11 and VI-F12.

It is important that the designer check the ability of the existing foundation to withstand the saturated soil pressures that would develop under this condition. The computations necessary for this determination are provided in Chapter IV.

The condition can be relieved by installation of foundation drainage (drainage tile and sump pump) at the footing level, and/or by extending the distance from the foundation to the levee. The land side seepage pressures can also be decreased by placing backfill against the flood side of the levee to extend the point where floodwaters submerge the soil away from the structure, but the effectiveness of this measure depends on the relative characteristics of the soils investigation. The design of foundation drains and sump pumps is presented in Chapter VI Dry Floodproofing section. An experienced geotechnical engineer should compute the spacing required to obviate the problem.

DESIGN

STANDARD CRITERIA

These design recommendations are conservative. Alternative parameters for a specific site may be developed by an engineer qualified in levee design.

The following parameters are established to provide a conservative design while eliminating several steps in the USACE design process, thereby minimizing the design cost. These guidelines pertain to the design and construction of localized levees with a maximum settled height of six feet. Techniques of slope stability analysis and calculation of seepage forces are not addressed. The recommended side slopes have been selected, based on experience, to satisfy requirements for stability, seepage control, and maintenance. The shear strength of suitable impervious soils compacted to at least 95 percent of the Standard Laboratory density as determined by ASTM Standard D-698 will be adequate to assure stability of such low levees, without the need for laboratory or field testing or calculation of safety factors.

The minimum requirements for crest width and levee side slopes are defined below. In combination with the toe drainage trench (which will be defined later in this section) and the cutoff effect provided by the backfilling of the inspection trench, these minimum requirements will provide sufficient control of seepage, and do not require complex analyses. Flatter land side slopes are recommended for a levee on a sand foundation to provide a lower seepage gradient, because a sand foundation is more susceptible to seepage failure than a clay foundation.

Maximum Settled Height of Six Feet

This is a practical limit placed due to available space and material costs.

Minimum Crest Width of Five Feet

This is required to minimize seepage concerns and allow for ease of construction and maintenance.

Floodwater Side Slope of 1 Vertical on 2.5 Horizontal

This is required to minimize the scour and erosion potential, to provide adequate stability under all conditions including rapid drawdown situations, and to facilitate maintenance.

Land Side Slope

The land side slope may vary based upon the soil type used in the levee. If the levee material is clay, a land side slope of one vertical to three horizontal is acceptable. If the levee material is sand, a flatter slope of one vertical to five horizontal is recommended to provide a lower seepage gradient.

One Foot of Freeboard

This is required to provide a margin of safety against overtopping and allow for the effects of wave and wind action. These forces create an additional threat by raising the height of the floodwater.

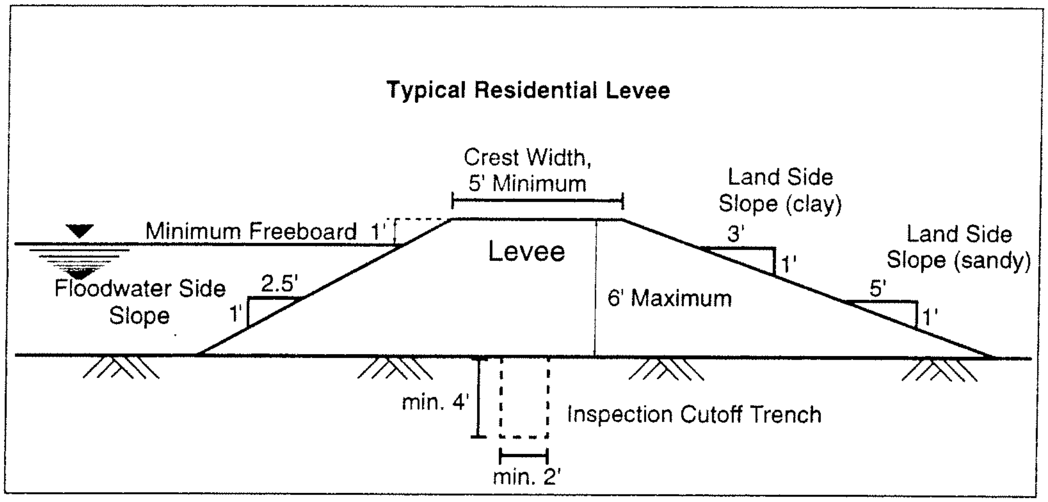


Figure VI-L1: Typical Residential Levee

INITIAL PHASES

Because of the importance of the characteristics of the soil that makes up the levee foundation, the excavation of an inspection trench is required. The minimum dimensions of the inspection trench are shown in Figure VI-L1. The inspection trench, which shall run the length of and be located beneath the center of the levee, provides the designer with information that will dictate the subsequent steps in the design process. The mandatory requirement of an inspection trench is fundamental to the assumptions made for the rest of the design process. The inspection trench will accomplish the following objectives:

Locate Utility Lines That Cross Under the Levee

Once identified, these must be further excavated and backfilled with a compacted impervious material to prevent development of a seepage path beneath the levee along the lines.

Provide "Cut-Off" for Levee Foundation Seepage

The trench itself will be backfilled with a highly impervious soil, such as a CH, CL, or SC, as previously referenced, to create an additional buffer against levee foundation seepage.

Identify Foundation Soil Type

The construction of the inspection trench should provide the designer with a suitable sample to identify the foundation soil type through the use of the Unified Soil Classification System, (USCS). This variable will further direct the design of the levee.

Clay Foundation

If, after inspection, it is determined that the in situ foundation material is composed of a clay soil, as defined by the NRCS, a land side slope of 1 vertical on 3 horizontal should be utilized.

Sandy Foundation

If, after inspection, it is determined that the in situ foundation is composed of a sandy soil, as defined by the NRCS, a land side slope of 1 vertical on 5 horizontal should be utilized.

SEEPAGE CONCERNS

Duration of flooding is a critical consideration in the design of levee seepage control measures. The longer the duration of flooding (i.e., the longer floodwaters are in contact with the levee), the greater the potential for seepage and the greater the need for seepage control measures such as cutoffs, drainage toes, and impervious cores.

If inspection determines that the foundation consists of a deep deposit of sand or gravel that will permit seepage under the shallow inspection trench, a deeper trench would be required, especially if the protected structure has a basement founded in a NRCS-defined sand or gravel. This scenario may make the use of a levee uneconomical.

Long duration flooding may negatively impact the ability of the drainage toe and inspection trench to control the seepage through and under the levee.

Two types of seepage must be considered in the design of a residential levee system: levee foundation seepage and embankment seepage. The amount of seepage will be directly related to the type and density of soils in both the foundation and the embankment of the levee. While the installation and backfilling of the inspection trench with impervious material will help reduce concerns of foundation seepage, further steps must be taken to minimize any embankment seepage for levees between three and six feet in height. The mandatory inclusion of a drainage toe will control the exit of embankment seepage while also controlling seepage in shallow foundation layers.

The inclusion of a drainage toe for a levee of varying height will be limited to those areas with a height greater than three feet. If the levee height varies due to the natural topography, a drainage toe will be required only for those portions of the levee that have a height greater than three feet.

The major reason for the inclusion of these measures is to relieve the pressure of seepage through or under the levee so that piping may be avoided. Piping is the creation of a flowpath for water through or under a soil structure such as a levee, dam, or other embankment, resulting in a pipe-like channel carrying water through or under the structure. Piping can lead to levee failure. Piping becomes a more serious problem as the permeability of the foundation soil increases.

The drainage toe should be sized as shown in Figure VI-L2, and should be filled with sand conforming to the gradation of standard concrete sand as defined by ASTM standards.

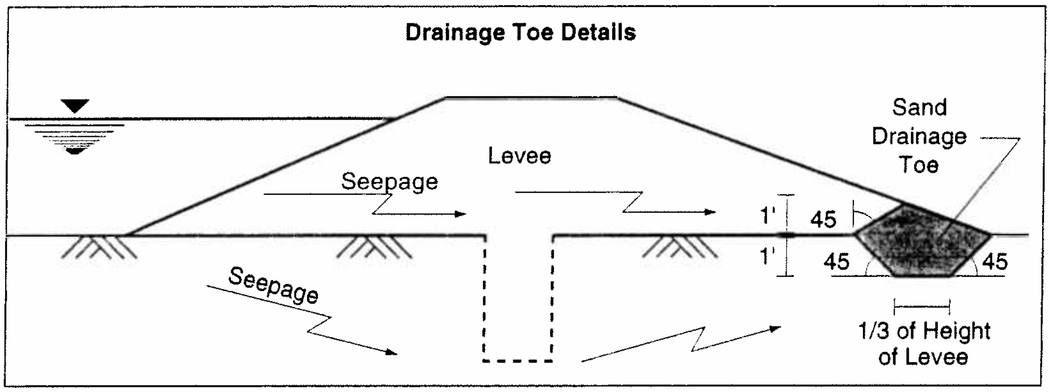


Figure VI-L2: Drainage Toe Details

SCOURING/SLOPE PROTECTION

The floodwater side of the levee embankment may require protection from erosion caused by excessive flow velocities. For flow velocities of up to three feet per second, a vegetatively stabilized or sodded embankment will generally provide adequate erosion protection. Some vegetative covers, such as Bermuda grass, Kentucky bluegrass, and tall Fescue, provide erosion protection from velocities of up to five feet per second. The grasses should be those that are suitable for the local climate. An alternative or supplement to a vegetative cover is the use of a stone protection layer. The layer should be placed on the entire floodwater face of the levee and be sized in accordance with Table VI-L1:

These values are from USACE Manual *Design and Construction of Levees.*

Table VI-L1 **Stone Protection Layer Guidance**

Velocities Against Slope	Minimum Diameter of Stone
< 2 fps	0.5 inches
< 5 fps	2.0 inches
< 8 fps	9.0 inches

INTERIOR DRAINAGE

> Guidance on estimating interior drainage quantities is presented in Chapter IV.

Constructing a levee around a house will not only keep floodwater out, but also will act to keep seepage and rainfall inside the levee unless interior drainage techniques are utilized. One method of draining water that collects from rain and from seepage through and under a levee is to install drain pipes that extend through the levee. While this will allow for drainage by gravity, the drains must be equipped with flap gates, which close to prevent flow of floodwaters through the pipe. The flap gates will open automatically when interior floodwaters rise above exterior floodwaters.

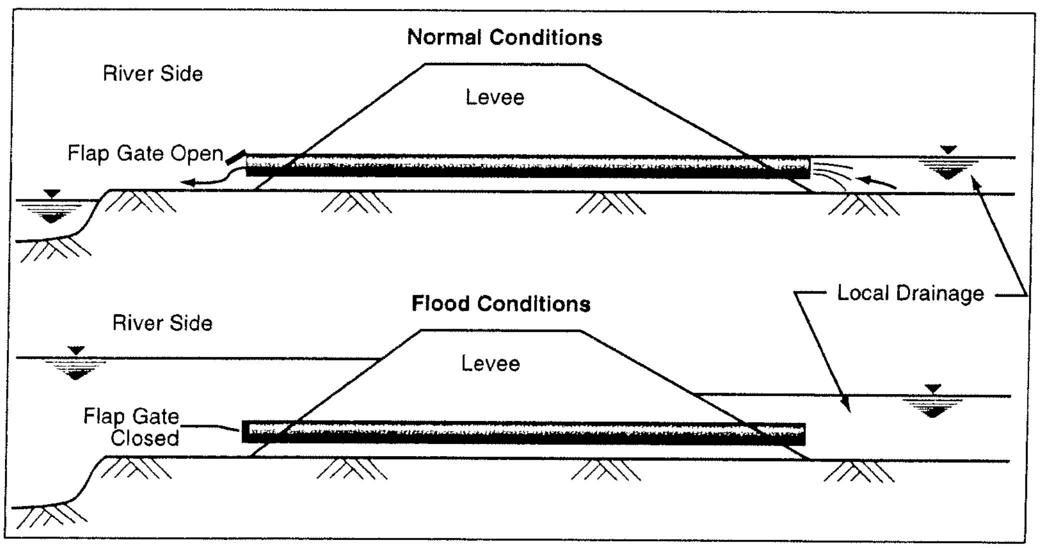


Figure VI-L3: Drain Pipe Extending through Levee

To ensure that water from precipitation or seepage within a leveed area is removed during flooding, a sump pump should be installed in the lowest area encompassed by the levee. All interior drainage measures should lead to this pump, which will discharge the flow up and over the levee. The sump pump should have an independent power source so that it will stay in operation should there be an interruption of electrical power, a common event during a flood.

An alternative to the use of a sump pump (for minor storms), is the creation of an interior storage area that will detain all interior flow until the floodwaters can recede. See Figure VI-L4. Typically the storage area is sized for the 2- or 10-year recurrence interval event.

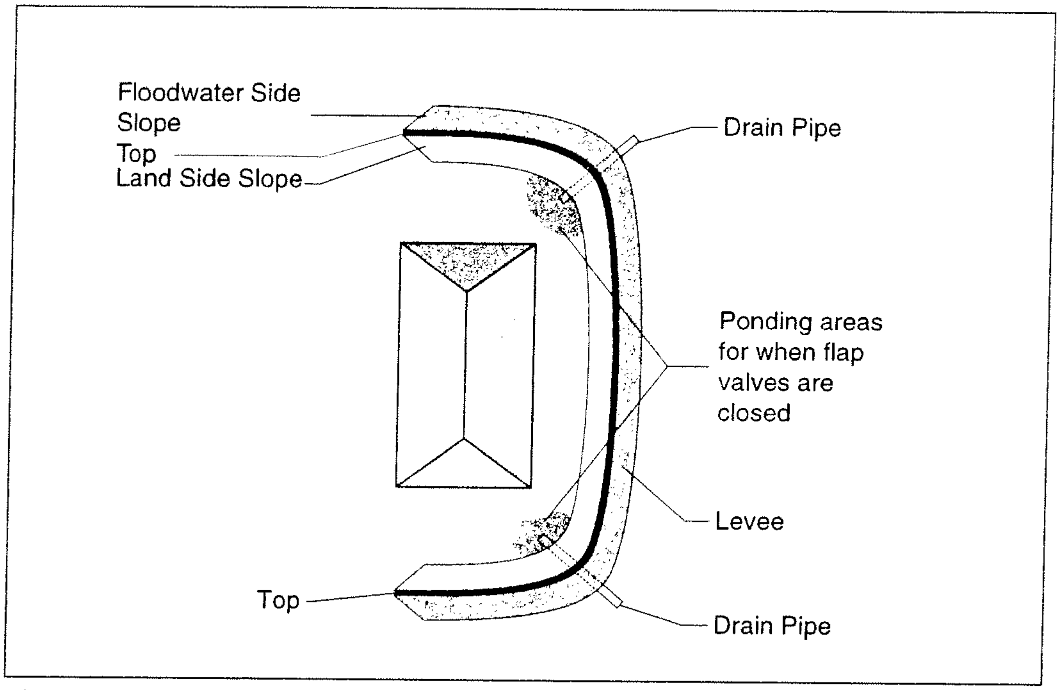


Figure VI-L4: Interior Storage Area

MAINTENANCE

Levee maintenance should include keeping the vegetation in good condition and preventing the intrusion of any large roots from trees or bushes or animal burrows, since they can create openings or weak paths in the levee through which surface water and seepage can follow, enlarging the openings and causing a piping failure. Planting of trees and bushes is not permitted on the levee.

Any levee design should include a good growth of sod on the top and slopes of the levee to protect against erosion by wind, water, and traffic, and to provide a pleasing appearance. Regular mowing, along with visual inspection several times a year, should identify critical maintenance issues.

COST

The accuracy of a cost estimate is directly related to the level of detail in a quantity calculation. The following example provides a list of the common expenses associated with the construction of a residential levee. Unit costs vary with location and wholesale price index. To obtain the most accurate unit prices, the designer should consult construction cost publications or local contractors. The designer should also budget an additional five percent of the total construction capital outlay annually for maintenance of the levee.

Table VI-L2, Cost Estimate Example, illustrates the estimated cost (in 2000 dollars) for construction of a three-foot- high, 216-foot-long levee, which was built in 1985 to protect a 1,600-SF house in Montgomery County, Maryland.

Table VI-L2 **Cost Estimate Example**

Item	Cost
Strip Topsoil & Clear & Grub	$522.00
Dig Inspection Trench	$1,170.00
Import Fill	$2,808.00
Compact Fill	$936.00
Riprap	$4,212.00
Drain Tile (4" PVC)	$335.00
Check Valve	$1,404.00
Sewer Gate Valve	$1,872.00
Sump Pump	$1,560.00
Discharge Piping	$156.00
Total Cost	$14,975.00

Table VI-L3 **Levee Cost Estimating Worksheet**

Owner Name: ____________________ **Prepared By:** ______________
Address: ____________________ **Date:** ______________
Property Location: ______________________________

Item	Unit	Unit Cost 1999 Dollars	# Units Needed	Item Cost
Clearing & Grubbing	Acre	$4,550.00		
Stripping Topsoil	Cubic Yards	$0.51 to $1.75		
Seeding	T.S.F.*	$37.50 to $49.50		
Sod	T.S.F.*	$460.00 to $715.00		
Haul Fill (1-5 miles round trip)	Cubic Yards	$4.50 to $11.00		
Haul Fill (5-15 miles round trip)	Cubic Yards	$7.00 to $21.00		
Import Fill	Cubic Yards	$8.50 to $12.00		
Compact Fill	Cubic Yards	$0.75 to $2.00		
Riprap/Stone Slope Protection	Cubic Yards	$39.50		
Dig Inspection Trench - 2' x 4'	Linear Feet	$4.22 to $11.66		
Steel Drain Gate Valve	Each	$615.00 to $1,925.00		
Steel Drain Check Valve	Each	$565.00 to $1,200.00		
Sump and Sump Pump (with back up battery)	Each	$850.00 to $1,400.00		
Drain Tile 4"-6" DIA PVC	Linear Feet	$7.80 to $9.40		
Drain Tile 8"-10" DIA PVC/RCP	Linear Feet	$10.05 to $12.10		
Discharge Piping for (1"-2" DIA PVC) Sump Pump	Linear Feet	$3.73 to $4.55		
			Total Cost	

*T.S.F. = Thousands of Square Feet

CONSTRUCTION

To prepare for the construction of a levee, all ground vegetation and topsoil should be removed over the full footprint of the levee. If sod and topsoil are present, they should be set aside and saved for surfacing the levee when it is finished.

SOIL SUITABILITY

Most types of soils are suitable for constructing residential levees. The exceptions are very wet, fine-grained, or highly organic soils, defined as OL, MH, CH, OH type soils by the NRCS. The best are those with a high clay content, which are highly impervious. Highly expansive clays should also be avoided because of potential cracking due to shrinkage.

COMPACTION REQUIREMENTS

As the levee is constructed, it should be built up in layers, or lifts, each of which must be individually compacted. Each lift should be no more than six inches deep before compaction (see Figure VI-L5). Compaction to at least 95 percent of standard laboratory density should be performed at or near optimum moisture content with pneumatic-tired rollers, sheepfoot rollers, or other acceptable powered compaction equipment. In some situations, certain types of farm equipment can effect the needed compaction.

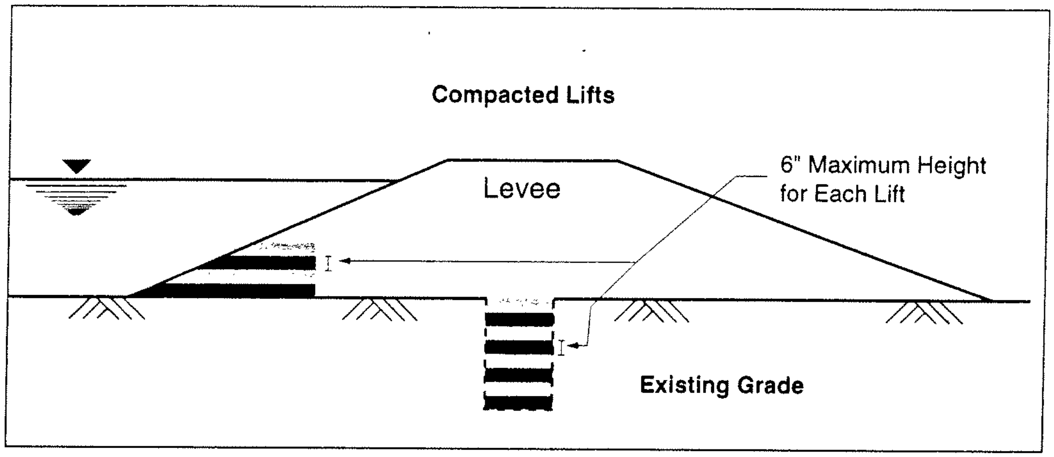


Figure VI-L5: Compacted Lifts

SETTLEMENT ALLOWANCE

The levee should be constructed at least five percent higher than the height desired to allow for soil settlement.

> Settlement allowances vary by geographic region and geologic conditions. Therefore, a five percent allowance may not be applicable in all situations. Consult the state or local flood-plain management officials for further information.

BORROW AREA RESTRICTIONS

A principle concern for the construction of the levee is the availability of suitable fill for levee construction, but caution should also be taken as to the location of the fill borrow area. For the purpose of this manual a general rule is to avoid utilizing a borrow area within 40 feet of the landward toe of the levee.

ACCESS ACROSS LEVEE

The complete encirclement of a structure with a levee can create access problems not only for the homeowner but also for emergency vehicles. If the levee is low enough, additional fill material can be added to provide a flat slope in one area for a vehicle access ramp running over the levee as shown in Figure VI-L6. Care should be taken to prohibit high volumes of traffic across the levee, which could result in the formation of ruts or the wearing away of the vegetative cover.

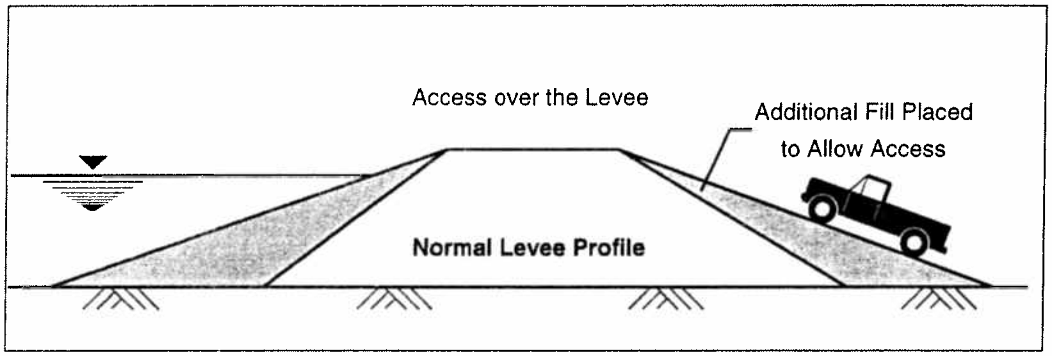


Figure VI-L6: Access over the Levee

If it is necessary to have a gap in the levee, this can be closed during flooding through the use of a gate or closure structure. Additional details are provided in the section of Chapter VI entitled Dry Floodproofing. It should be noted that the use of a closure structure requires human intervention. If the structure in question is susceptible to flood hazards with little or no warning time, or if human intervention cannot be guaranteed, the use of a closure is not recommended.

Relocation

Table of Contents

RELOCATION

Relocation is the retrofitting measure that can offer the greatest security from future flooding. It involves moving an entire structure to another location, usually outside the floodplain. Selection of the new site is usually conducted by the homeowner, often in consultation with the designer to ensure that critical site selection factors such as floodplain location, accessibility, utility service, cost, and, of course, homeowner preference meet engineering and local regulatory concerns. Relocation as a retrofitting measure not only relieves future anxiety about flooding, but also offers the opportunity to eliminate future flood insurance premiums.

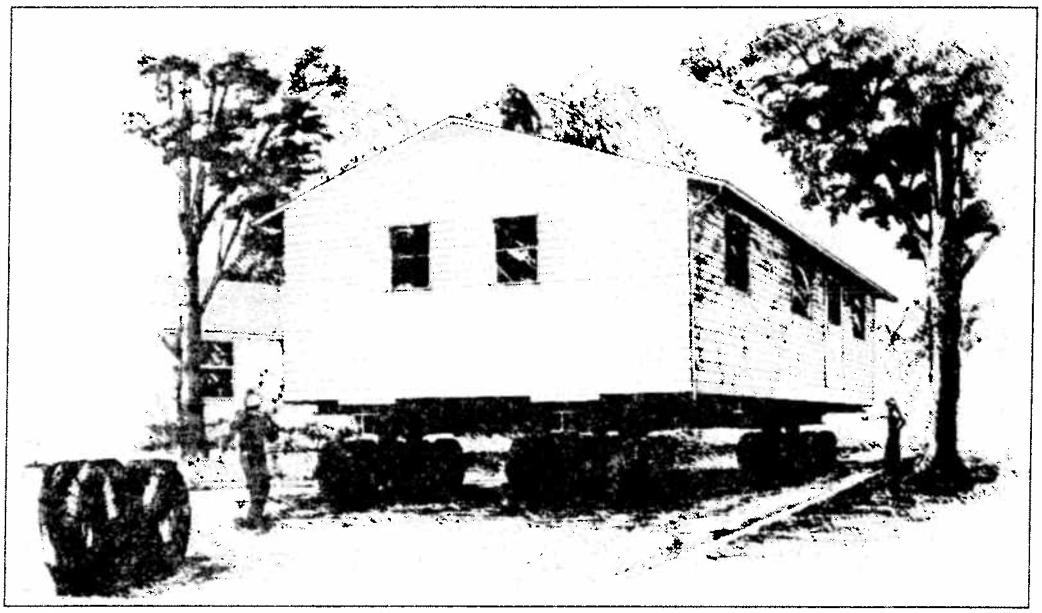


Figure VI-R1: House Relocation

The relocation process, as illustrated in Figure VI-R2, is fairly straightforward, but there are a number of design considerations to be addressed before embarking on this retrofitting measure. The steps involved with the relocation of a structure are discussed in more detail throughout this chapter:

Step 1: Selection of a House Moving Contractor

↓

Step 2: Analysis of Existing Site and Structure

↓

Step 3: Selection, Analysis and Design of New Site

↓

Step 4: Preparation of the Existing Site

↓

Step 5: Analysis and Preparation of the Moving Route

↓

Step 6: Preparation of the Structure

↓

Step 7: Moving the Structure

↓

Step 8: Preparation of the New Site

↓

Step 9: Restoration of the Old Site

Figure VI-R2: Relocation Process

STEP 1 - SELECTION OF A HOUSE MOVING CONTRACTOR

The selection of a moving contractor is one of the most important decisions a homeowner will make and may ultimately have the greatest impact on the success of the project. The designer can assist the homeowner in selecting an experienced home moving contractor. Some of the key elements of this selection (outlined in the Relocation Contractor Selection Checklist, Figure VI-R3) include:

EXPERIENCE

The designer/homeowner should visit recent projects the contractor has completed and talk to owners who recently went through the process to develop an opinion on the quality of work done by the contractor.

FINANCIAL CAPABILITY

The homeowner/designer should determine whether and to what extent the contractor is licensed, insured, and bonded. A prudent homeowner will consider the potential risk of a failed project before enlisting the assistance of a contractor.

The International Association of Structural Movers (ISM) may be contacted at: P.O. Box 1213, Elbridge, NY 13060, (315) 689-9498, or www.iasm.org to obtain information on house relocation companies for a retrofitting project.

PROFESSIONALISM AND REPUTATION

The designer/homeowner may wish to check the contractor's reputation with the state licensing board, the local Better Business Bureau, local officials, and/or the International Association of Structural Movers (ISM). A critical question is whether or not the contractor is licensed to work in your area.

The designer/homeowner should also interview several contractors to determine:

- how well they may be able to work with this individual;
- the extent of the contractor's knowledge; and
- what confidence may be had in the contractor's ability to complete the relocation project.

COST OF SERVICES

While this should not be the sole determinant of contractor selection, cost of services is an important aspect of the relocation process. To ensure a comparison of similar levels of effort, the designer/homeowner should develop a detailed scope of services to be provided and have each contractor prepare a bid from the same scope of services. Remember, the most qualified contractor may not always have the highest cost and conversely, the least qualified contractor may not have the lowest cost.

Relocation/Elevation Contractor Selection Checklist

1. Experience of the Contractor:

   Recent, successful house relocation/elevation projects? Yes______ No ______

   Satisfied clients providing good references? Yes______ No ______

   Met time schedules? Yes______ No ______

   Cleaned up and restored old site? Yes______ No ______

   Quality product through your visual inspection of recent projects? Yes______ No ______

2. Financial Stability of Contractor:

   Bonded? Yes______ No ______; Amounts:__________________

   Licensed? Yes______ No ______; Amounts:__________________

   Insured? Yes______ No ______; Amounts:__________________

3. Professionalism and Reputation of Contractor:

   State Licensing Agency:____

   Better Business Bureau:____

   Local Officials:____

   International Association of Structural Movers: ______________________________

   Results of the Interview:____

4. Cost of Services:____

   ____

5. Summary of References:____

   ____

   ____

Figure VI-R3: Relocation/Elevation Contractor Selection Checklist

STEP 2 - ANALYSIS OF EXISTING SITE AND STRUCTURE

The designer should help the homeowner to ensure that the contractor conducts an analysis of the existing site and structure to determine the critical criteria for the relocation of the structure. These criteria will include:

- Does sufficient space exist around the structure for the installation of lifting beams and truck wheels?
- Can the structure be lifted as one piece or must it be separated into sections?
- Depending upon the final assessment of the structure's conditions, how much bracing will be required to successfully move this structure?
- Will this structure survive the lift and a move of the distance proposed by the homeowner?
- Which utilities must be disconnected and where?
- What local regulations govern demolition of the remaining portions of the structure (foundation and paved areas) and to what standard must the site be restored?

Usually this analysis is conducted by the moving contractor and not by the homeowner's designer. However, it is important that the designer/homeowner coordinate and communicate with the contractor regarding the aforementioned issues.

The contractor usually has experience in analyzing the existing structure to determine:

- the size and placement of lifting beams, jacks, and lateral or cross beams;
- whether the structure should be elevated/moved in one or several pieces.

The final decision on these items may not be made until an evaluation of the moving route is conducted.

If the selected contractor is not familiar with these factors, the homeowner and designer might reconsider their contractor selection.

LIFTING BEAM PLACEMENT

Each of the following factors affecting the placement of lifting beams must be taken into consideration during the elevation and relocation process:

- size and shape of the house;
- existing framing and structural parameters;
- deflection limitations; and
- distribution of the structure's weight.

The major consideration for the placement of lifting beams is to limit cracking due to excessive deflections during preparation, moving, and settling in place. The lifting beams, in tandem with cross or lateral beams, must provide sufficient support for the structure. When the house is removed from the foundation, the lifting and lateral beams should provide as stable a support as the original foundation.

Deflection of any portion of the structure is normally a result of the manner in which the weight of the house is distributed, the location of the jacks under the lifting beams, and the rigidity of the lifting beam. Proper placement of lifting beams, jacks, and lateral beams will protect against cracking of both the interior and exterior finishes, as well as ensure the integrity of the entire house.

A second consideration concerning the installation of lifting beams is to ensure that they are located so that the house can be attached to truck wheel sets forming a trailer.

The route to be taken during the relocation of the house dictates the physical size and weight limitations of the structure, due to the horizontal and vertical clearances from obstructions. The house may have to be cut into sections, which are moved separately to negotiate the available route. Lifting beams, therefore, would have to be placed for each section to be moved. The entire elevation framing must also be rigid enough to take the forces associated with movement.

The weight of heavier construction materials on certain portions of the structure, such as brick veneer, chimneys, and fireplaces, causes additional deflection and warrants special attention when determining the lifting beam system. Even with minimal deflection, brick construction is subject to cracking. Therefore, extra precautions will be needed in the form of additional beam support or removal of the brick for possible later replacement.

The size and shape of the house also affect the placement and number of lifting beams. A simple rectangular floor plan allows for the easiest and most straightforward type of elevation project. Generally, placement of the longitudinal lifting beams, with lateral beams located as required, is the system utilized for the elevation process. Larger or more complex shapes, such as L-shaped or multi-level homes, necessitate additional lifting beams and jacks to provide a stable lifting support system. Every consideration of the load based upon the size and shape of the structure should be incorporated into the design and layout of the lifting beam system.

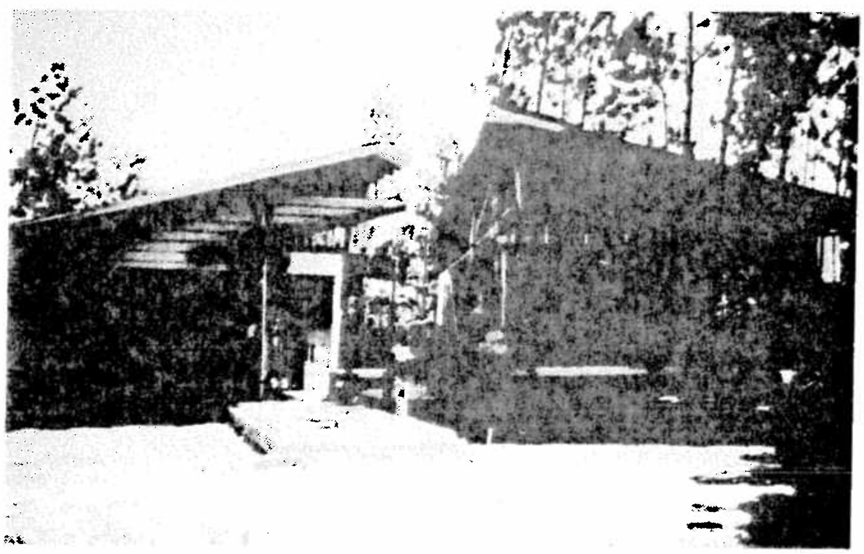


Figure VI-R4: When a house is too large to be relocated in one piece, careful planning is necessary in order to cut the structure in pieces and move the pieces separately.

STEP 3 - SELECTION, ANALYSIS, AND DESIGN OF THE NEW SITE

> Information on site design standards may be obtained from the local building official, or, if there is none, from a HUD publication entitled, *Proposed Model Land Development Standards and Accompanying State Enabling Legislation*, 1993 Ediition.

The selection of a new site for a relocated house will require the examination of potential sites with regard to:

- floodplain location;
- utility extension feasibility;
- accessibility; and
- permitting feasibility.

The process is similar to selecting a lot upon which to design and build a new home. Local building codes and approval processes must be followed. In some instances, the homeowner may be required to upgrade existing mechanical, electrical, and plumbing systems to meet current code requirements.

SITE ACCESS

An important consideration in the selection of a new site is the accessibility of the site for both the house moves and the new site construction crews. Severe site access constraints can increase the cost of the measure and/or require cleaning and grading activities, which may diminish the site characteristics the homeowner initially desired.

PERMITS

The designer/homeowner should make certain that when the house is moved to the new lot, it will conform to all the zoning and construction standards in effect at the time of relocation. The designer should contact the local regulatory officials to determine the design standards and submission process requirements that govern development of a new site. All permits required for construction at the new site and for transporting the structure to the new site should be obtained prior to initiating the relocation process.

STEP 4 - PREPARATION OF THE EXISTING SITE

The initial preparation of the site includes clearing all vegetation from the area in and around the footprint of the house. This is done to clear a path beneath the structure to allow the insertion of beams for lifting supports. These pathways should be deep enough to allow for the movement of both people and machinery.

Figure VI-R5: Clearing Pathways Beneath the Structure for Lifting Supports

STEP 5 - ANALYSIS AND PREPARATION OF THE MOVING ROUTE

Once the relocation site has been selected, a route for transport must be analyzed and selected. This route should be chosen carefully and planned well in advance of the design of the new site or the undertaking of any relocation process activities at the existing site.

IDENTIFY ROUTE HAZARDS

> Timing may be critical in areas that have heavy traffic during morning and evening rush hours. Homes are often relocated during the late evening and early morning hours.

Make certain that the house, as it will be moved, will navigate the following:

- narrow passages, such as road cuts and widths;
- bridge weight limits and widths;
- utility conflicts, such as light poles, and electric and telephone lines;
- fire hydrants;
- road signs;
- traffic signals; and
- tight turns around buildings, bridges, and overpasses.

Care should be taken to ensure that the structure will clear all overhead utility lines. Many of these can be lifted during the move, but utility companies sometimes require the presence of their employees and will charge for this service. In some instances an overland (non-road) route may be the best alternative.

OBTAIN APPROVALS

It may be necessary to obtain moving permits, not only for the area from which the structure is being moved, but also in jurisdictions through which the move is passing. Approvals for transport in a public right-of-way may be required from local governments, highway departments, and utility companies. Often approvals may be necessary from private landowners whose properties are either crossed or affected by the move.

The time required to obtain approvals and the complexity of information some parties may require in order to provide approvals may vary widely. The designer/contractor and homeowner should investigate this approval process early in the relocation effort to minimize potential delays due to obtaining permits.

COORDINATE ROUTE PREPARATION

The moving contractor should be responsible for the necessary coordination made along the moving route. This includes:

- the raising or relocation of utilities by utility companies;
- any road/highway modifications, such as traffic lights, signage, temporary bridges, etc; and
- clearing/grubbing of overland areas, where necessary.

The moving contractor should also be responsible for making sure that these facilities are returned to their normal operating condition as soon as the move is completed.

STEP 6 - PREPARATION OF THE STRUCTURE

The steps involved in preparing a structure to be moved are described below.

DISCONNECT UTILITIES

The first step in preparing the structure is to disconnect all the utilities connected to the structure. Specific requirements governing the capping, abandoning, and/or removal of specific utilities should be available from the local utility companies and/or the local regulatory officials.

CUT HOLES IN FOUNDATION WALL FOR BEAMS

From beneath the structure, the pathways for lifting beams are cut in the existing foundation.

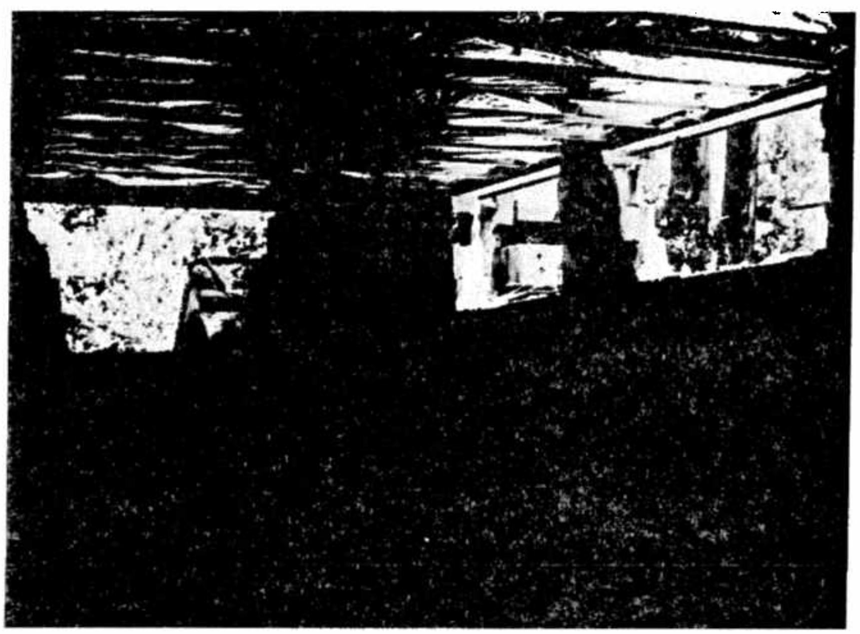


Figure VI-R6: Pathways for Lifting Beams

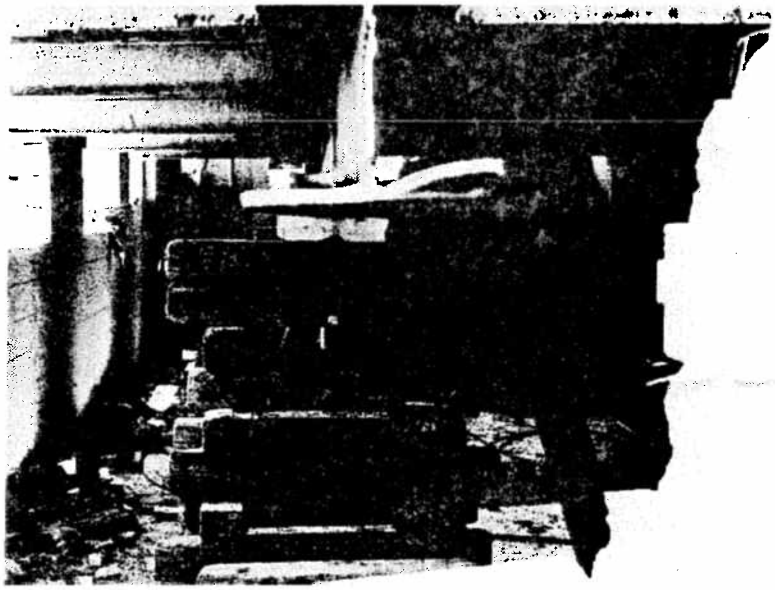


Figure VI-R7: Beams Supported by Cribbing are Placed at Critical Lift Points

INSTALL BEAMS

Lifting and lateral beams are placed beneath the structure at all critical lift points and support cribbing is added as the structure is separated from its old foundation.

INSTALL JACKS

Jacks are used to lift the structure from its foundation. Various types of jacking systems may be employed as long as gradual and uniform lifting pressures are utilized to lift the structure.

INSTALL BRACING AS REQUIRED

Bracing may need to be installed to maintain the integrity of the structure.

SEPARATE STRUCTURE FROM FOUNDATION

The structure now stands free from its former foundation.

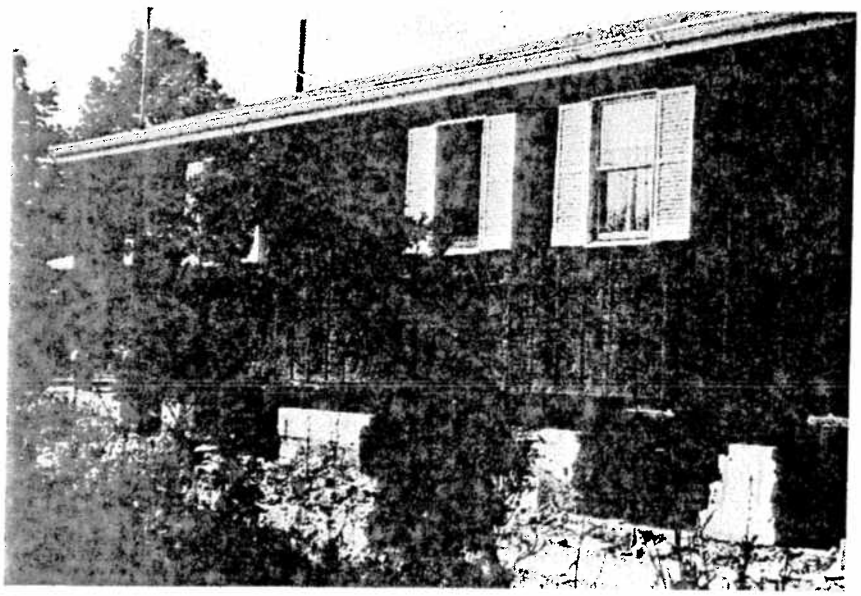


Figure VI-R8: Structure is Separated from Foundation

STEP 7 - MOVING THE STRUCTURE

Once the structure has been raised, it is transported to the new site. This process is outlined below.

EXCAVATE/GRADE TEMPORARY ROADWAY

Excavation and grading of a temporary roadway is done at one end of the structure. The truck wheels, which will form the trailer that will be used to move the house, are brought to the site and placed beneath the lifting and lateral beams.

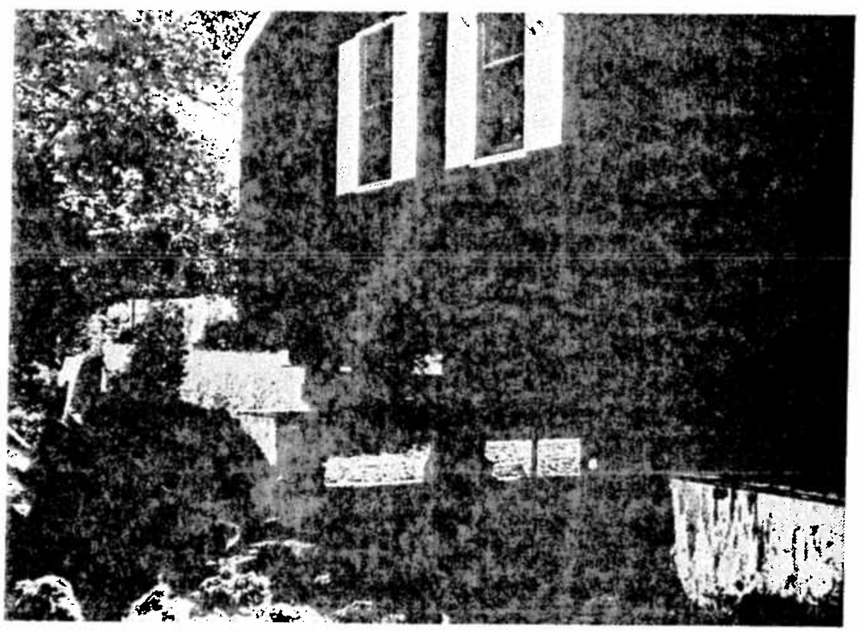


Figure VI-R9: Excavation of Temporary Roadway

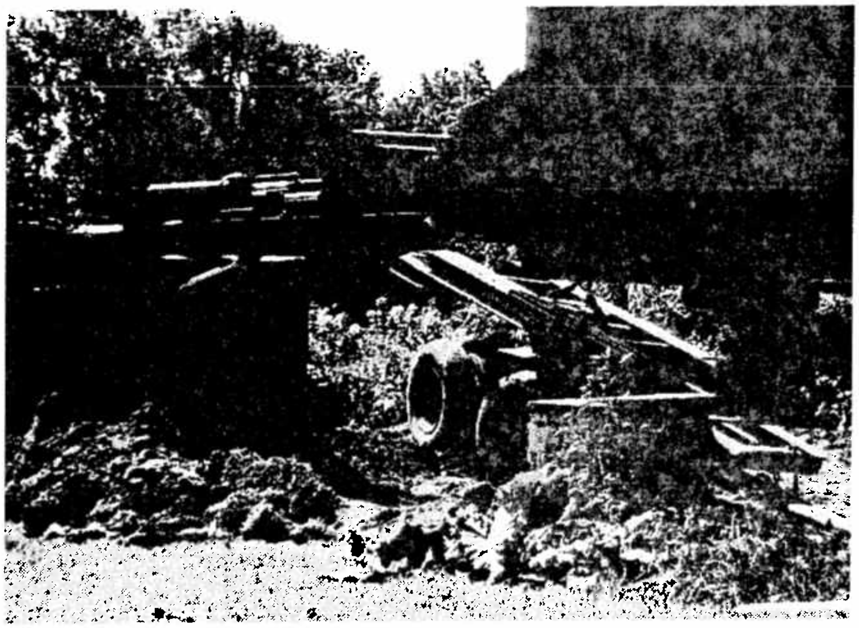


Figure VI-R10: Trailer Wheel Sets are Placed Beneath the Lifting Beams

ATTACH STRUCTURE TO TRAILER

The house is attached to the truck wheels and then attached to the tractor/dozer in preparation for the moving of the structure from its original site. The tractor/dozer is used to pull the house to street level, while workers continually block the wheels to prevent sudden movement. At street level, the house is stabilized and a truck is connected to the trailer for the journey to the new site.

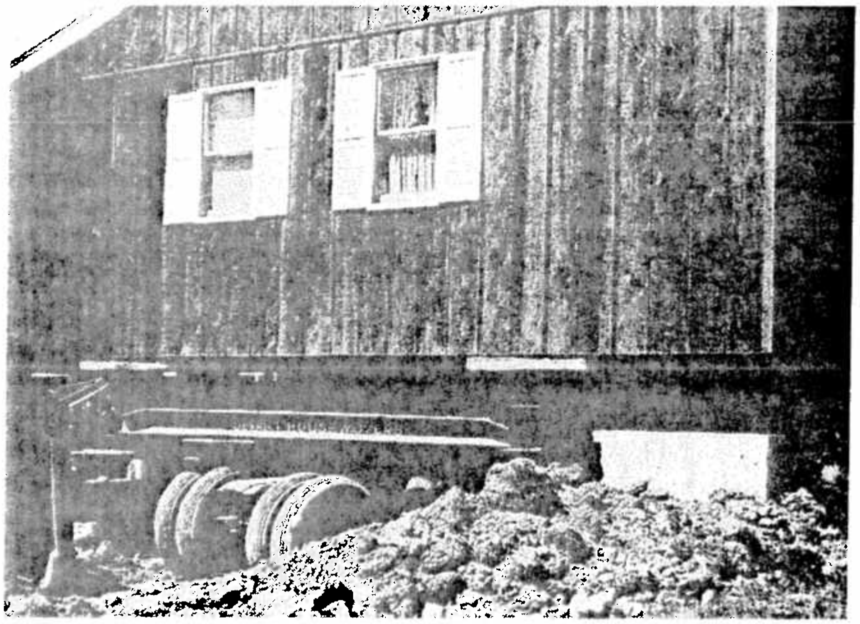


Figure VI-R11: House is Lowered onto Trailer Wheel Sets

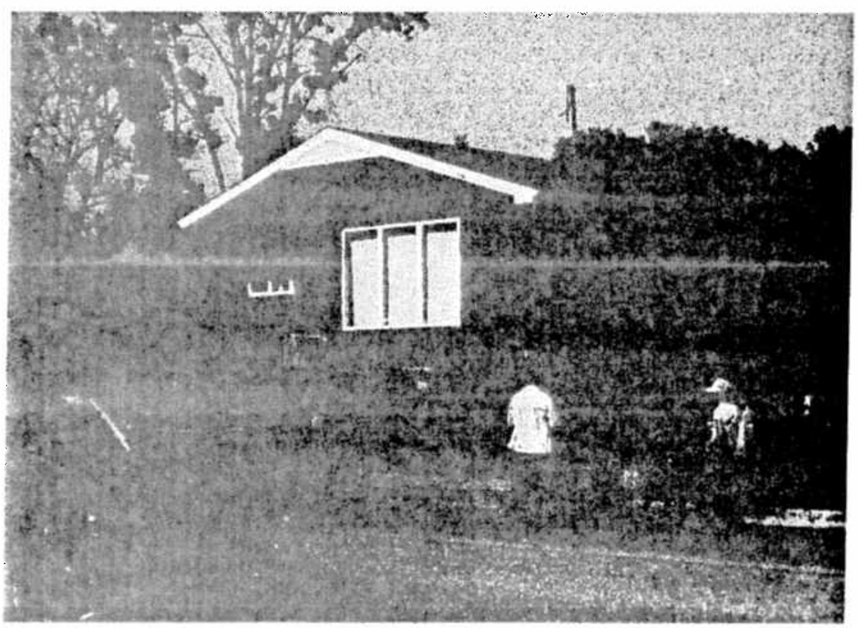


Figure VI-R12: Trailer is Used to Pull House to Street

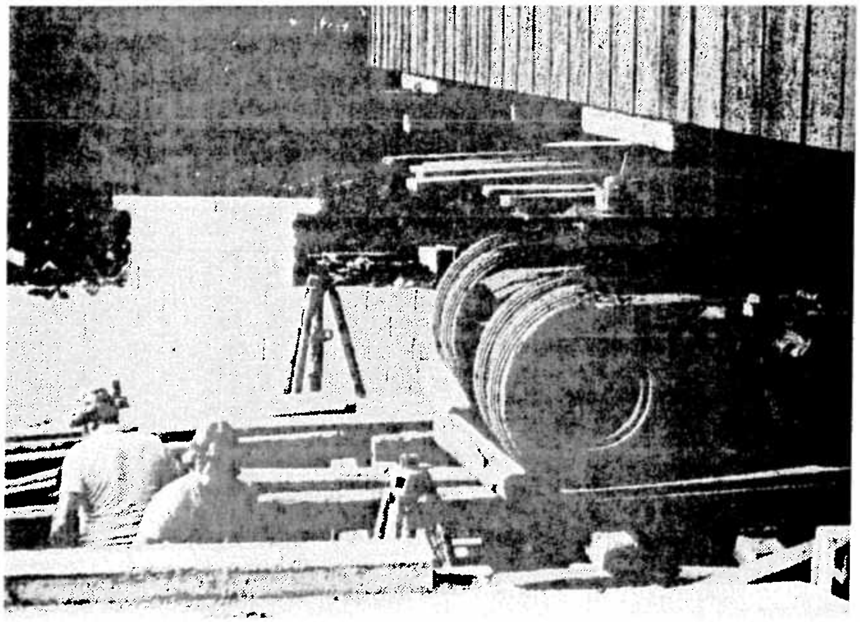


Figure VI-R13: As house is pulled to street level, workers continually block wheels to prevent sudden movement.

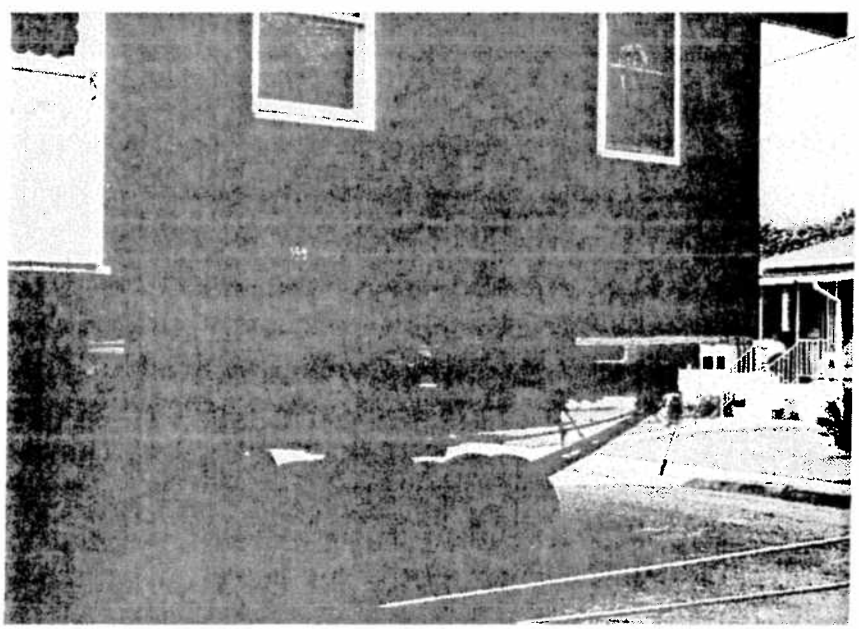


Figure VI-R14: House is Stabilized and Connected to Trailer

TRANSPORT STRUCTURE TO NEW SITE

With connections to the truck completed, the actual transport of the structure to the new site begins.

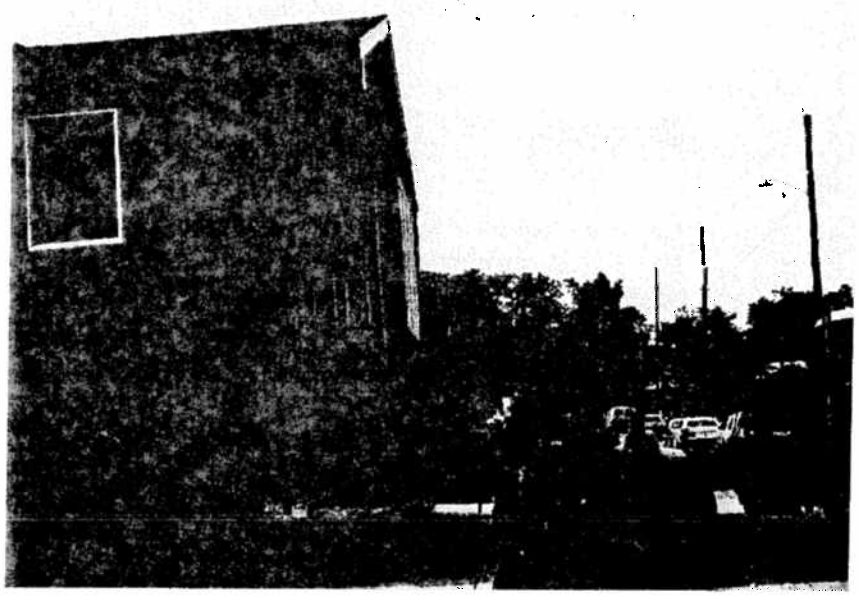
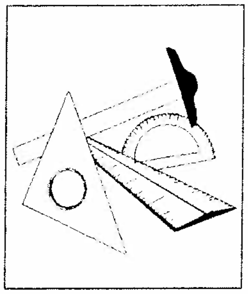


Figure VI-R15: Journey to New Site Begins



STEP 8 - PREPARATION OF THE NEW SITE

The new site is prepared for the arrival of the structure.

DESIGN FOUNDATION

The steps needed to design the new foundation have been defined in the Elevation portion of this chapter.

DESIGN UTILITIES

Utilities must be available to be brought directly to the structure at the new site. Construction should be accomplished in accordance with the approved set of design documents prepared for the new site and any building permit conditions specified by local officials (as explained in Step 3).

EXCAVATION AND PREPARATION OF NEW FOUNDATION

At the new site, excavation and preparation of the foundation are underway.

Figure VI-R16: Foundation Preparation at New Site

CONSTRUCTION OF SUPPORT CRIBBING

Support cribbing is put in place to allow the structure to be jacked up and the truck wheel sets are removed. With support cribbing in place, materials for completion of the foundation are readied.

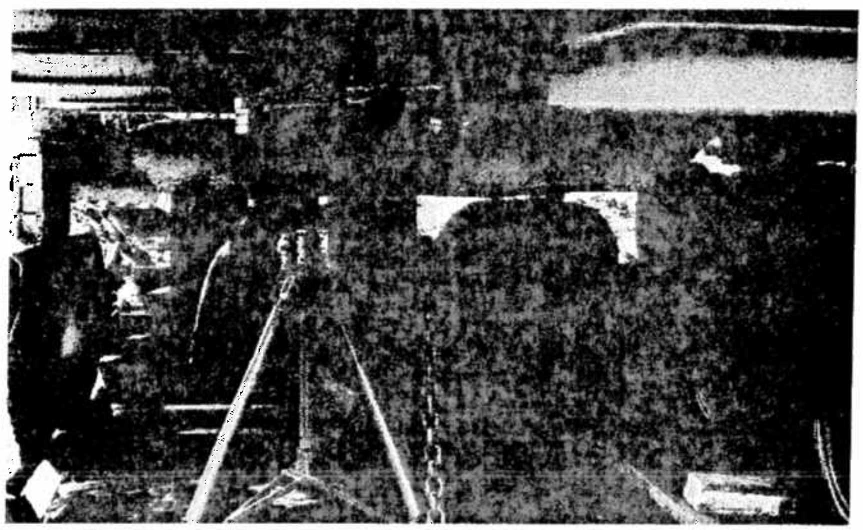


Figure VI-R17: Support Cribbing is Placed

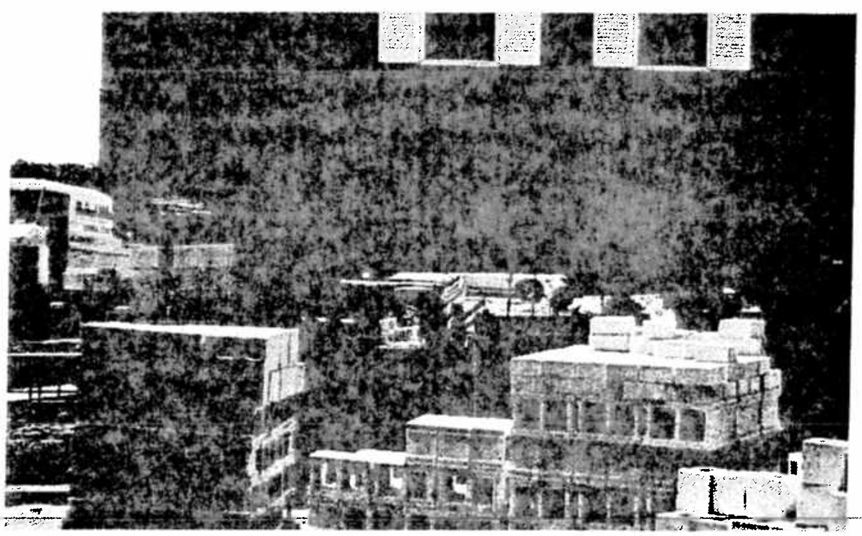


Figure VI-R18: Materials for New Foundation are Readied

CONSTRUCTION OF FOUNDATION WALLS

The foundation wall construction begins.

LOWER STRUCTURE ONTO FOUNDATION

Once the desired height of the new wall is reached, the house is lowered onto its new foundation, cribbing is removed, and foundation walls are completed.

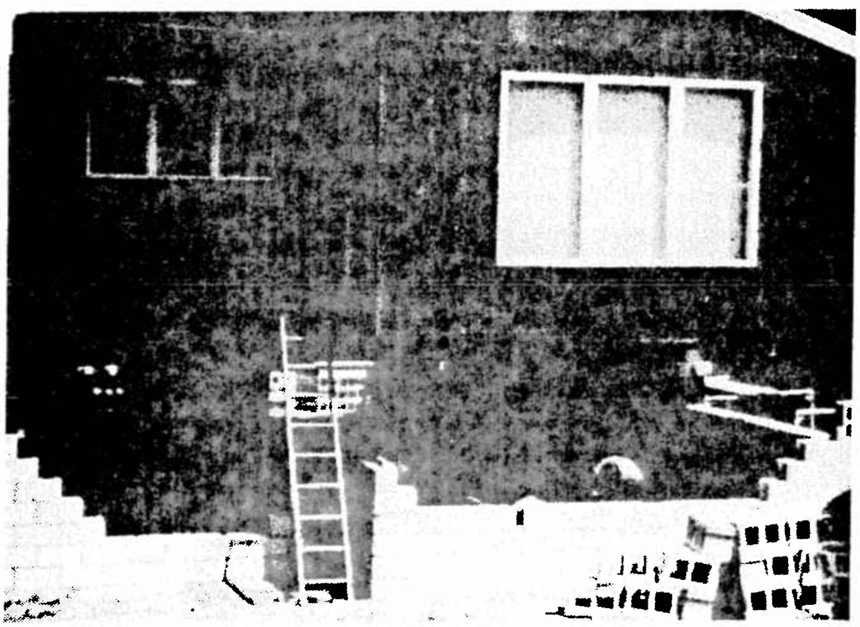


Figure VI-R19: New Foundation Wall Construction Begins

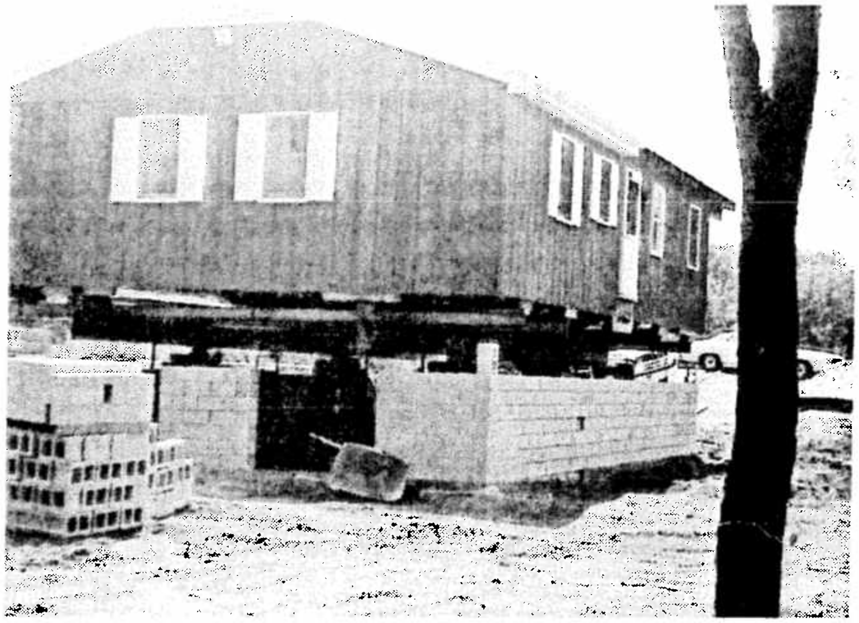


Figure VI-R20: Once foundation walls are completed, house is lowered and connected to foundation.

LANDSCAPING

Finishing touches, like preparing the foundation for back-filling, are done to blend in the house with its new environment.

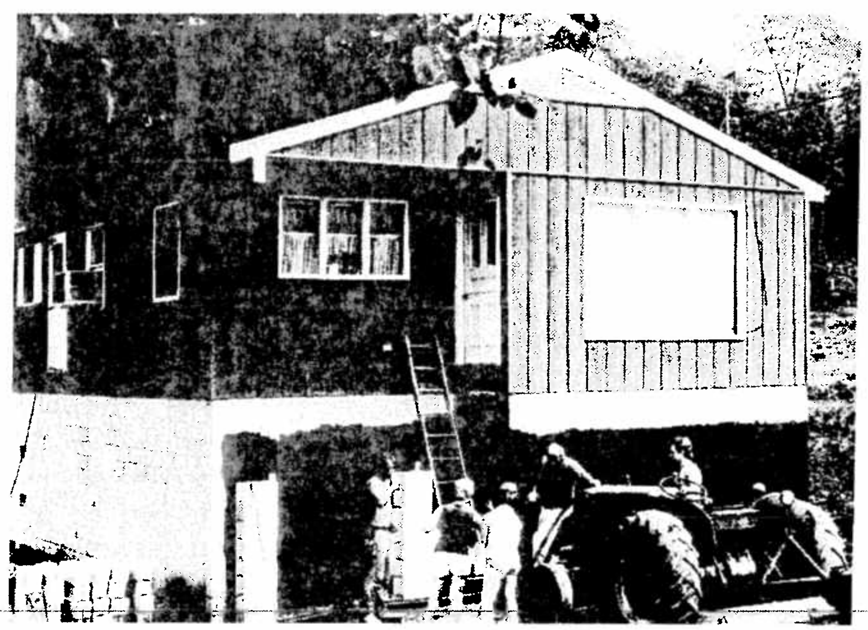


Figure VI-R21: Final Preparations for Backfilling and Landscaping

STEP 9 - RESTORATION OF OLD SITE

Once the structure is removed from the site, certain steps need to be taken to stabilize the site in accordance with local regulations. Many homeowners have sold or deeded these abandoned properties to local municipalities for the development of parkland and/or open space. In any case, permits for the demolition of the old site, remaining foundation, and remaining utility systems, as well as grading and site vegetative stabilization are normally required.

DEMOLISH AND REMOVE FOUNDATION AND PAVEMENT

The old basement may have to be backfilled to eliminate any potential hazard. Check local regulations to see if old foundation and utility connections have to be removed.

DISCONNECT AND REMOVE ALL UTILITIES

Material from drained septic, oil, and gas storage tanks must be disposed of in a safe and legal manner.

Following up on the disconnection and capping of utility services previously discussed in Step 2, the homeowner may be required to remove all existing utility systems from the site. Septic tanks and oil/gas storage tanks on site may be governed by specific environmental guidelines, which must be followed to ensure that leakage to groundwater sources does not occur. Depending upon the age and condition of the tanks, the homeowner may be required to drain and remove these tanks, or drain and stabilize the underground tanks against flotation.

The homeowner may also be required to test the soil around an underground tank to determine if leakage has occurred. If leakage is confirmed, the homeowner is usually responsible for cleaning the contaminated soils. When facing this situation, the homeowner should contact a qualified geotechnical or environmental engineer. Specific requirements governing the capping, abandoning, and/or removal of specific utilities should be determined from the local utility companies and/or the local regulatory officials.

GRADING AND SITE STABILIZATION

The old site may have to be regraded after all the excavation and movement by the heavy equipment. The lot will need to be stabilized with vegetation as appropriate to its intended future use.

Wet Floodproofing

Table of Contents

WET FLOODPROOFING

Wet floodproofing can be defined as permanent or contingent measures applied to a structure and/or its contents that prevent or provide resistance to damage from flooding by allowing floodwaters to enter the structure. The basic characteristic that distinguishes wet floodproofing from dry floodproofing is that it allows internal flooding of a structure as opposed to providing essentially watertight protection.

Flooding of a structure's interior is intended to counteract hydrostatic pressure on the walls, surfaces, and supports of the structure by equalizing interior and exterior water levels during a flood. Inundation also reduces the danger of buoyancy from hydrostatic uplift forces. Such measures may require alteration of a structure's design and construction, use of flood-resistant materials, adjustment of building operation and maintenance procedures, relocation and treatment of equipment and contents, and emergency preparedness for actions that require human intervention. This section examines:

- protection of the structure;
- design of openings for intentional flooding of enclosed areas below the DFE;
- use of flood-resistant materials;
- adjustment of building operation and maintenance procedures;
- the need for emergency preparedness for actions that require human intervention; and
- design of protection for the structure and its contents including utility systems and appliances.

Wet floodproofing is appropriate for basements, garages, and enclosed areas below the flood protection level.

PROTECTION OF THE STRUCTURE

The NFIP allows wet flood-proofing only in limited situations. The most common application is with pre-FIRM structures not subject to substantial damage and/or substantial improvement criteria. Structures in the pre-FIRM category can utilize any retrofitting method. However, for new structures or those that have been substantially damaged or are being substantially improved, application of wet floodproofing techniques is limited to the following situations:

- **Enclosed areas below the BFE that are used solely for parking, building access, or limited storage.** These areas must be designed to allow for the automatic entry and exit of flood-waters through the use of openings, and be constructed of flood-resistant materials.

- **Attached Garages.** A garage attached to a residential structure, constructed with the garage floor slab below the BFE, must be designed to allow for the automatic entry and exit of flood-waters. Openings are required in the exterior walls of the garage or in the garage doors. In addition, the areas below the BFE must be constructed with flood-resistant materials.

- **FEMA has advised communities that variances to allow**

(continued on next page)

As with dry floodproofing techniques, developing a wet floodproofing strategy requires site-specific evaluations that may necessitate the services of a design professional. The potential for failure of various structural components (foundations, cavity walls, and solid walls) subjected to inundation is a major cause of structural damage.

FOUNDATIONS

The ability of floodwater to adversely affect the integrity of structure foundations by eroding supporting soil, scouring foundation material, and undermining footings necessitates careful examination of foundation designs and actual construction. In addition, it is vital that the structure be adequately anchored to the foundation. Uplift forces during a flood event are often great enough to separate an improperly anchored structure from its foundation should flood-waters reach such a height.

CAVITY WALLS

Wet floodproofing equalizes hydrostatic pressure throughout the structure by allowing floodwater to enter the structure and equalize internal and external hydrostatic pressure. Thus, any attempt to seal internal air spaces within the wall system is not only technically difficult, but also contrary to the wet floodproofing approach. Provision must be made for the cavity space to fill with water and drain at a rate approximately equal to the floodwater rate of rise and fall. Insulation within cavity walls subject to inundation should also be a type that is not subject to damage from inundation. The design of foundation openings to equalize hydrostatic pressure is covered in the next section.

(continued)

wet floodproofing may be issued for certain categories of structure. Refer to FEMA's Technical Bulletin 7-93, *Wet Floodproofing Requirements for Structures Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program.*

SOLID WALLS

Solid walls are designed without internal spaces that could retain floodwater. Because these walls can be somewhat porous, they can absorb moisture and, to a limited degree, associated contaminants. Such intrusion could cause internal damage, especially in a cold (freeze-thaw) climate. Therefore, where solid walls are constructed of porous material, the retrofitting measures should include both exterior and interior protective cladding to guard against absorption.

DESIGN OF OPENINGS IN FOUNDATION WALLS FOR INTENTIONAL FLOODING OF ENCLOSED AREAS BELOW THE DFE

For additional information on the regulations and design guidelines concerning foundation openings, please refer to FEMA Technical Bulletin 1-93, *Openings in Foundation Walls for Buildings Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program.*

In buildings that are constructed on extended solid foundation walls or that have other enclosures below the DFE (that are not designed to resist flooding), it is important that the foundation contain openings that will permit the automatic entry and exit of floodwaters. (See Figures VI-W1 and VI-W2.)

These openings allow floodwaters to reach equal levels on both sides of the walls and thereby lessen the potential for damage from hydrostatic pressure. While not a requirement for existing buildings built prior to a community's joining the NFIP, NFIP regulations require these openings for all new construction and substantial improvements of existing buildings in SFHAs.

The minimum criteria for design of these openings is as follows:

- A minimum of two openings shall be provided on different sides of each enclosed area, having a total net area of not less than one square inch for every square foot of enclosed area subject to flooding. This is not required if openings are engineered and certified.

- The bottom of all openings shall be no higher than one foot above grade.

- Openings must be equipped with screens, louvers, valves, or other coverings or devices that permit the automatic entry and exit of floodwaters.

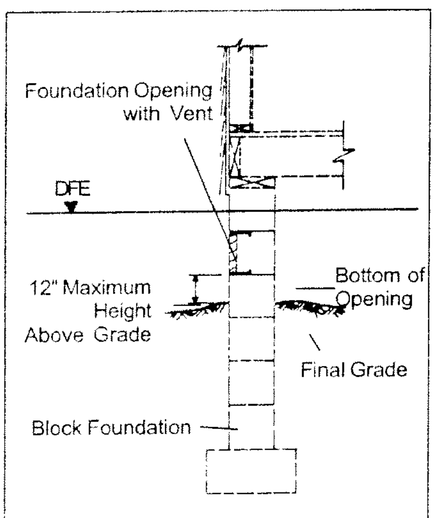


Figure VI-W1: Typical Opening for Solid Foundation Wall

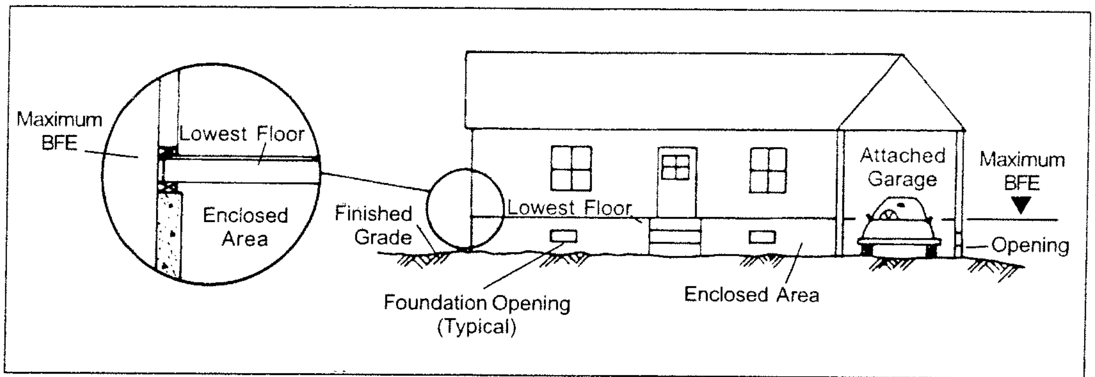


Figure VI-W2: NFIP-Compliant Residential Building Built on Solid Foundation Walls with Attached Garage

USE OF FLOOD-RESISTANT MATERIALS

> Detailed guidance is provided in FEMA Technical Bulletin 2-93, *Flood-Resistant Materials Requirements for Buildings Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program.*

> Additional information on these elements can be obtained from FEMA Technical Bulletin 7-93, *Wet Floodproofing Requirements for Structures Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program.*

In accordance with the NFIP, all materials exposed to floodwater must be durable, resistant to flood forces, and retardant to deterioration caused by repeated explsure to floodwater. Interior building elements such as wall finishes, floors, ceilings, roofs, and building envelope openings can also suffer considerable damage from inundation by floodwaters, which can lead to failure or an unclean situation. The exterior cladding of a structure subject to flooding should be nonporous, resistant to chemical corrosion or debris deposits, and conducive to easy cleaning. Interior cladding should be easy to clean and not susceptible to damage from inundation. Likewise, floors, ceilings, roofs, fasteners, gaskets, connectors, and building envelope openings should be constructed of flood-resistant materials to minimize damage during and after floodwater inundation.

Generally, these performance requirements indicate that masonry construction is the most suited to wet floodproofing in terms of damage resistance. In some cases, wood or steel structures may be candidates, provided that the wood is pressure treated or naturally decay-resistant and steel is galvanized or protected with rust-retardant paint.

BUILDING OPERATION AND MAINTENANCE PROCEDURES AND EMERGENCY PREPAREDNESS PLANS

The operational procedure aspect of applying floodproofing techniques involves both the structure's functional requirements for daily use and the allocation of space with consideration of each function's potential for flood damage. Daily operations and space use can be organized and modified to minimize damage caused by floodwater.

FLOOD WARNING SYSTEM

Because wet floodproofing will, in most cases, require some human intervention when a flood is imminent, it is extremely important that there be adequate time to execute such actions. This may be as simple as monitoring local weather reports, the National Weather Service alarm system, or a local flood warning system.

INSPECTION AND MAINTENANCE PLAN

Every wet floodproofing design requires some degree of periodic maintenance and inspection to ensure that all components will operate properly under flood conditions. Components of the system, including valves and opening covers, should be inspected and operated at least annually.

EMERGENCY OPERATION PLAN

This type of plan is essential when wet floodproofing requires human intervention, such as adjustments to or relocation of contents and utilities. A list of specific actions and the location of necessary materials to perform these actions should be developed.

PROTECTION OF UTILITY SYSTEMS

Utility systems include heating and air conditioning systems, appliances, electrical/plumbing systems, and water service/sewer facilities.

The purpose of the retrofitting methods in this section is to prevent damage to building contents and equipment caused by contact with floodwaters by isolating these components from floodwaters. Isolation of these components can take the form of relocation, elevation, or protection in place.

ELEVATION

The most effective method of protection for equipment and contents is to elevate and/or relocate (permanently or temporarily) threatened items out of harm's way. The interior of the structure must be organized in a way that ensures easy access, facilitates relocation, and meets current building code requirements.

Refer to FEMA 348: *Protecting Building Utilities from Flood Damage* for details on the protection of utility systems.

Both inside and outside of the flood-prone structure, elevation of key components may be achieved through the use of existing or specially constructed platforms or pedestals. Contingent elevation can be accomplished by the use of hoists or an overhead suspension system. Relocated utilities placed on pedestals are subject to wind and earthquake damage and must be secured to resist wind and seismic forces.

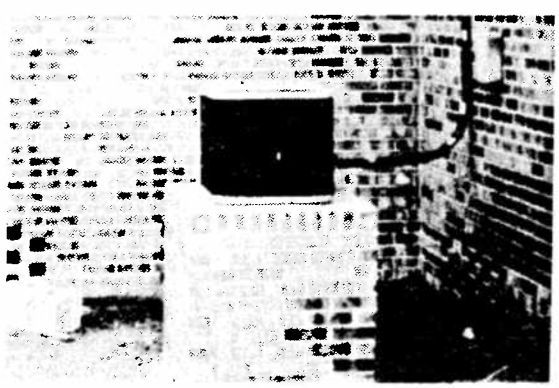


Figure VI-W3: Elevated Air Conditioning Compressor

IN-PLACE PROTECTION

Some components can be protected in place through a variety of options, such as:

- anchors and tie-downs to prevent flotation;
- low barriers or shields; and
- protective coatings.

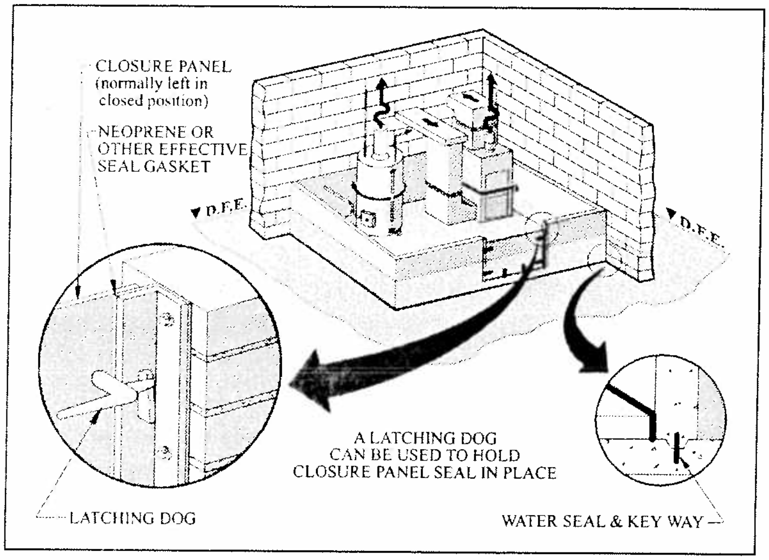


Figure VI-W4: Flood Enclosure Protects Basement Utilities from Shallow Flooding (FEMA 348)

Regardless of the method of protection, any adjustments or modifications to retrofit building utility systems should be completed in accordance with local building code requirements.

Utility systems as used here are mechanical, electrical, and plumbing systems including water, sewer, electricity, telephone, cable TV, natural gas, etc. The recommendations presented in this section are intended for use individually or in common to mitigate the potential for flood-related damage.

FIELD INVESTIGATION

Basic process for protection of utility systems:

Field Investigation
↓
Design
↓
Construction

Detailed information must be obtained about the existing structure to make decisions and calculations concerning the feasibility of using wet floodproofing. Use the Building/ Building System Data Sheets (Figures VI-3 and VI-4) as a guide to record information.

Once this data is collected, the designer should answer the questions contained in Figure VI-W5, Field Investigation Worksheet, to confirm the measure(s) selected and develop a preliminary concept for the installation of wet floodproofing measures.

Once a conceptual approach toward wet floodproofing has been developed, the designer should discuss the following items with the homeowner:

- Previous flood history, flood depths, and equipment/ systems impacted by the floods.
- Plan of action as to which equipment can be relocated and which equipment will have to remain located below DFE.
- Length of power outages, water shut-off, or fuel shut-off for work to be completed.
- Specific scope of items to be designed.
- Note any unsafe practices or code violations or exceptions to current codes.

Owner Name: ________________________ Prepared by: ________________________

Address: ____ Date: __________

Property Location: ____

Flood Resistant Retrofitting Field Investigation Worksheet

Design Flood Elevation (DFE) ______________

HVAC System

- Can all equipment be protected in-place? ___Yes ___ No
- Is it feasible to install a curb or "pony" wall around equipment to act as a barrier? ___Yes ___No
- Is it feasible to construct a waterproof vault around equipment below the DFE? ___ Yes ___ No
- Can reasonably sized sump pumps keep water away from the equipment? ___ Yes ___ No
- Can equipment feasibly be relocated:

  -- To a pedestal or balcony above the DFE? ___ Yes ___ No

  -- To a higher level on the same floor level? ___ Yes ___ No

  -- To the next floor level? ___Yes ___No
- Is space available for the equipment in the alternate location? ___ Yes ___ No
- Can existing spaces be modified to accept equipment? ___ Yes ___ No
- Is additional space needed? ___ Yes ___ No
- Do local codes restrict such relocations? ___ Yes ___ No

Fuel System

- Can all equipment be protected in-place? ___ Yes ___ No
- Is the tank properly protected against horizontal and vertical forces from velocity flow and buoyancy? ___Yes ___ No
- Is it feasible to install a curb or "pony" wall around equipment to act as a barrier? ___ Yes ___ No
- Can reasonably sized sump pumps keep water away from the equipment? ___Yes ___No

Figure VI-W5: Field Investigation Worksheet (page 1 of 3)

- Do local code officials and the gas company allow the meter to be relocated to a higher location? ___Yes ___No
- Can equipment feasibly be relocated:
  - -- To a pedestal or balcony above the DFE? ___ Yes ___ No
  - -- To a higher level on the same floor level? ___ Yes ___ No
  - -- To the next floor level? ___Yes ___ No
- Is space available for the equipment in the alternate location? ___ Yes ___ No
- Can existing spaces be modified to accept equipment? ___ Yes ___ No
- Is additional space needed? ___ Yes ___ No
- Do local codes restrict such relocations? ___ Yes ___ No

Electrical System

- Is it feasible to relocate the meter base and service lateral above the DFE? ___ Yes ___ No
- Is it feasible to relocate the main panel and branch circuits above the DFE? ___Yes ___ No
- Is it feasible to relocate appliances, receptacles, and circuits above the DFE? ___ Yes ___ No
- Is it feasible to relocate light switches and receptacles above the DFE? ___ Yes ___ No
- Can ground fault interrupter protection be added to circuits below the DFE? ___ Yes ___ No
- Can service lateral outside penetrations be sealed to prevent water entrance? ___ Yes ___ No
- Can cables and/or conduit be mechanically fastened to prevent damage during flooding? ___ Yes ___ No
- Can splices and connections be made water resistant or relocated above the DFE? ___ Yes ___ No
- Do local code officials and electric companies allow the elevation of the meter? ___ Yes ___ No

Sewage Management Systems

- Can the on-site system be protected in-place? ___ Yes __ No
- Is it feasible to anchor the tank? ___ Yes ___ No
- Can the distribution box and leech field be protected from scour and impact forces? ___ Yes ___ No
- Can the supply lines be properly protected from scour and impact forces? ___Yes ___No

Figure VI-W5: Field Investigation Worksheet (page 2 of 3)

- Can backflow prevention valves be used to minimize flow of sewage into the building? ___ Yes ___ No
- Can equipment feasibly be relocated? ___ Yes ___ No
- Can the system be moved to a higher elevation on the property? ___ Yes ___ No
- Can the tank be relocated to a higher elevation or indoors? ___ Yes ___ No
- Can the drains and toilets be relocated above the DFE? ___ Yes ___ No
- Is space available for the equipment in the alternate location? ___ Yes ___ No
- Can existing spaces be modified to accept equipment? ___ Yes ___ No
- Is additional space needed? ___Yes ___ No
- Do local codes restrict such relocations? ___ Yes ___ No

Potable Water Systems

- Can the well be protected in-place? ___ Yes ___ No
- Is it feasible to install a curb or "pony" wall around equipment to act as a barrier? ___ Yes ___ No
- Is it feasible to construct a waterproof vault around equipment below the DFE? ___ Yes ___ No
- Can the wellhead and tank be protected from scour and impact forces? ___ Yes ___ No
- Can the supply lines be properly protected from scour and impact forces? ___ Yes ___ No
- Can backflow prevention valves be used to minimize flow of floodwaters into the water source? ___ Yes ___ No
- Can the equipment feasibly be relocated? ___ Yes ___ No
- Can the well be moved to a higher elevation on the property? ___ Yes __ No
- Can the electric controls for the well be protected from inundation? ___ Yes ___ No
- Can the tank be relocated to a higher elevation or indoors? ___ Yes ___ No
- Can the taps be relocated above the DFE? ___ Yes ___ No
- Is space available for the equipment in the alternate location? ___ Yes ___ No
- Can existing spaces be modified to accept equipment? ___ Yes ___No
- Is additional space needed? ___Yes ___ No
- Do local codes restrict such relocations? ___ Yes ___ No

Figure VI-W5: Field Investigation Worksheet (page 3 of 3)

DESIGN OVERVIEW

This section presents the process of designing and implementing measures to retrofit existing building utility sytems. Retrofitting may involve a combination of elevating and/or protecting in place. The general design process involved with wet floodproofing is listed below:

Determine the DFE

↓

Establish system component vulnerability

↓

Develop alternatives (elevate or protect in place)

↓

Verify with homeowner and code officials

↓

Construct/implement

We will examine elevation and protection in place alternatives for the following systems:

- Electrical systems;
- Heating, ventilating, and air conditioning (HVAC) systems;
- Fuel supply/storage systems;
- Water systems; and
- Sewer systems

ELECTRICAL SYSTEMS

Electrical system components can be seriously damaged by floodwaters when either active or inactive. Silt and grit accumulates in devices not rated for complete submergence and destroys the insulation value of the device. Current circuit breakers and fuses are designed to protect the wiring conductors and devices from overload situations, including short circuit or ground fault conditions. Floodwaters seriously affect operation of these devices.

Most homes were not designed to mitigate potential flood damage to electrical equipment; however, there are retrofitting steps that will provide permanent protection for the electrical system.

- The chief concern is to raise or relocate equipment and devices above the DFE.

- A second step is to seal outside wall penetrations, anchor cables and raceway, and mechanically protect the wiring system in flood-prone locations.

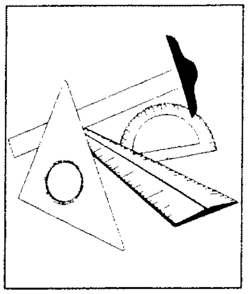
- A third step is to seal out moisture. Electrical system problems occur as moisture permeates devices causing corrosion.

- A fourth step necessary for retrofitting is the addition of Ground Fault Interrupting Circuit (GFIC) breakers, which deactivate circuits when excessive current leakage is encountered. This step ultimately assists life safety protection and may be required by local code.

Each residence presents the designer with a unique set of characteristics including age, method of construction, size, and location. There are different combinations of systems that may need to be modified. When it is not feasible to elevate in place, the following information provides the design considerations and details that govern the retrofitting of electrical equipment and circuits below the DFE.

- Receptacles and switches should be kept to a minimum and elevated as high as is practical.
- Circuit conductors must be UL listed for use in wet locations.
- Wiring should be run vertically for drainage after being inundated.
- Receptacles and switches should be installed in non-corrosive boxes with holes punched in the bottom to facilitate drying. The receptacles will have to be replaced after inundation by floodwaters.
- Lighting fixtures should be connected via simple screw base porcelain lampholders. This will allow for speedy removal of lamp or fixture, and the lampholder can be cleaned and reused.
- Sump pumps and generators should have cables long enough to reach receptacles above the DFE.
- All circuits below the DFE should be protected by GFIC breakers.
- Circuits serving equipment below the DFE should be placed on separate GFICs, clearly marked in the breaker box. This allows power to be turned off to circuits below the DFE without affecting the rest of the home.
- Wiring splices below DFE should be kept to a minimum. If conductors must be spliced, use crimp connectors and waterproof with heat shrink tubing or grease packs over the splice.

Retrofitting measures, using techniques similar to those discussed under Electrical Systems, should be considered for telephone and cable TV exterior service lines, indoor wiring, outlet jacks, wall plates, etc.



HEATING, VENTILATING, AND AIR CONDITIONING (HVAC) SYSTEMS

HVAC system equipment (i.e., furnaces, boilers, compressors) should be elevated/relocated above the DFE or protected within a watertight enclosure whenever possible. However, the protection of HVAC system equipment requires consideration of several factors. Some general points to include when evaluating potential retrofitting measures are:

> In a post-flooding situation, the designer may recommend replacing old equipment with a new one that meets current codes, is more energy/cost-efficient, and fits in the desired location. In some cases, the old equipment may be replaced with a lateral or in-line equipment, installed in the attic to protect it from flooding.

- adequate space and structural support for relocated equipment;
- maintenance of required equipment clearances and maintenance access dictated by code and/or manufacturer;
- provision of adequate combustion air for fuel-burning equipment;
- modification and/or maintenance of proper venting of fuel-burning equipment;
- necessity of non-combustible construction materials;
- necessity of eliminating ductwork below the DFE whenever possible;
- suitability of protective partitions or vaults;
- reconfiguration of ductwork;
- consideration of duct construction material; and
- modification of hot water or steam circulation piping.

> Galvanized steel ductwork is less susceptible than ductboard or similar materials to damage from flooding. Generally, if flooded, ducts made up of ductboard are not reusable.

FUEL SUPPLY/STORAGE SYSTEMS

In conjunction with the retrofitting of HVAC equipment, the designer must consider rerouting and/or extending fuel supply lines (i.e., fuel oil, natural gas, and LPG) when equipment is relocated. The following should be considered with respect to fuel supply/storage systems:

- Extension of fuel supply lines to relocated equipment;
- Use of flexible connections;
- Adequate support and anchorage to resist hydrostatic and hydrodynamic forces that act on tank. This can be accomplished by:
  - -- elevation of tanks on structural fill (Figure VI-W6);
  - -- elevation of tanks on a braced platform;
  - -- anchoring of supply lines to the downstream side of structural members;
- Relocation of fuel tank because of equipment relocation; and
- Use of automatic cut-off valves.

WATER SYSTEMS

The primary threats that floodwaters pose to water systems are contamination and velocity flow damage. Contamination by floodwaters may occur through infiltration into on-site water wells, public water supplies, open faucets, or broken pipes. In flood-prone areas that experience high velocity flow, damage may occur from the effects of the velocity, wave action, and/or debris impact. Some factors to consider when retrofitting water systems include:

Ensurance of all fuel, water, and sewer pipes and tanks being adequately protected to prevent damage caused by erosion, scour, buoyancy, debris impact, velocity flow, and wave action should be verified during the retrofitting design process.

Guidance concerning the anchoring of septic tanks is applicable to other types of underground storage tanks.

- minimizing plumbing fixtures below the DFE;
- adequate space for elevating components;
- modification of fixtures to prevent backflow;
- protection of system components from high velocity flow;
- suitability of protective partitions or vaults; and
- modification of the well top using watertight casing.

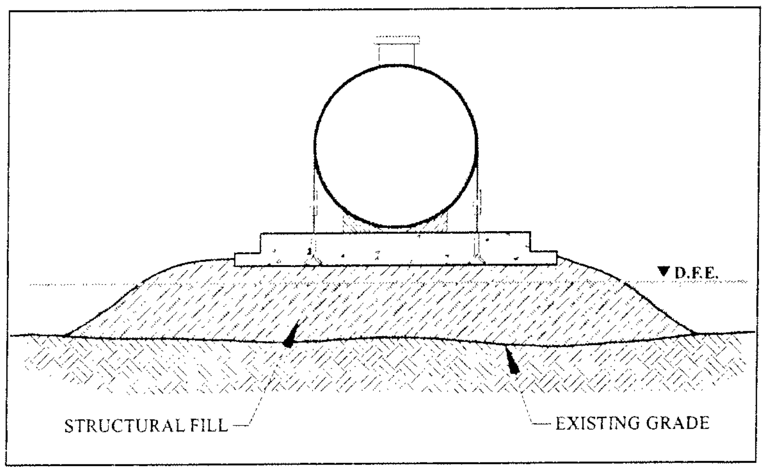


Figure VI-W6: Fuel Tank Elevated on Structural Fill (FEMA 348)

SEWER SYSTEMS

The main dangers associated with the flooding of sewer systems are back-up of sewage, damage of system components, and contamination of floodwaters. Because these dangers could result in serious health risks, minimization of their occurrences could reduce clean-up expense and hazards. Retrofitting of sewer systems to eliminate or minimize the dangers include considering the following possible options:

- relocation of collection components to higher elevation;
- installation and/or maintenance of a check or sewer backflow prevention valve;
- installation and/or maintenance of combination check and gate valves (see Figure VI-W7);
- installation of an effluent ejector pump (see Figure VI-W7);
- provision of a back-up electrical source;
- sealing of septic tank to prevent contamination; and
- adequate anchorage of septic tank to withstand buoyancy forces.

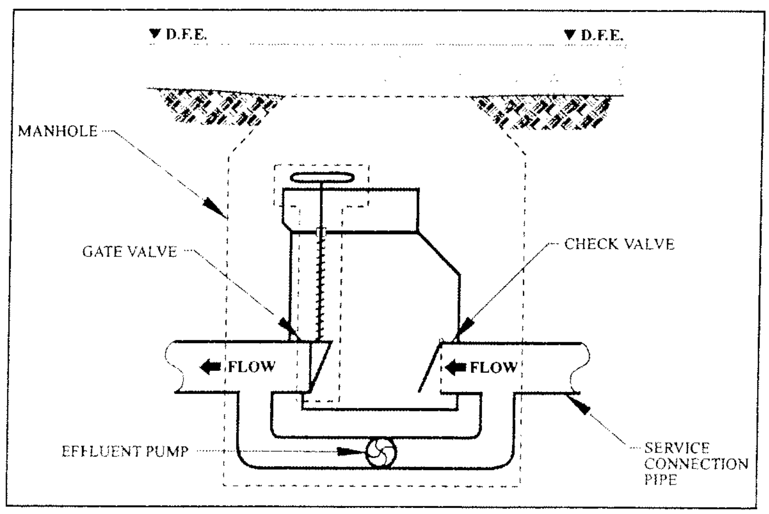


Figure VI-W7: Backflow Valve--A Check Valve and Gate Valve With An Effluent Pump Bypass (FEMA 348)

CALCULATION OF BUOYANCY FORCES

> When subject to flood forces, ruptured storage tanks containing natural gas or oil also pose the additional risk of explosion or environmental contamination.

> To minimize buoyancy forces, fuel tanks should be "topped off" prior to flooding.

> For a more detailed analysis of buoyancy forces please refer to FEMA 348.

The anchorage of any tank system consists of attaching the tank to a resisting body with enough weight to hold the tank in place. The attachment, or anchors, must be able to resist the total buoyant force acting on the tank. The buoyant force on an empty tank is the volume of the tank multiplied by the specific weight of water. It is usually advisable to include a factor of safety of 1.3, as is shown in the following net buoyancy force computation:

FORMULA

$$\mathbf{F_b = 0.134\, V_t \gamma FS - W_t}$$

where:

$\mathbf{F_b}$	is the net buoyancy force of the tank, in pounds;
$\mathbf{V_t}$	is the volume of the tank in gallons;
0.134	is a factor to convert gallons to cubic feet;
γ	is the specific weight of flood water surrounding the tank (generally 62.4 lb/ft^3 for fresh water and 64.1 lb/ft^3 for salt water);
FS	is a factor of safety to be applied to the computation, typically 1.3 for tanks; and
$\mathbf{W_t}$	is the weight of the tank.

Formula VI-W1: Net Buoyancy Force on a Tank

The volume of concrete required to offset the buoyant force of the tank can be computed as follows:

FORMULA

$$V_c = F_b/(S_c - \gamma)$$

where: V_c is the volume of concrete required, in cubic feet;
F_b is the net buoyancy force of the tank in pounds;
S_c is the effective weight of concrete, typically 150 pounds per cubic foot; and
γ is the specific weight of water (62.4 lb/ft^3).

Formula VI-W2: Concrete Volume Required to Offset Buoyancy

To resist this buoyant force, a slab of concrete with a volume, V_c, is usually strapped to the tank to resist the buoyant load.

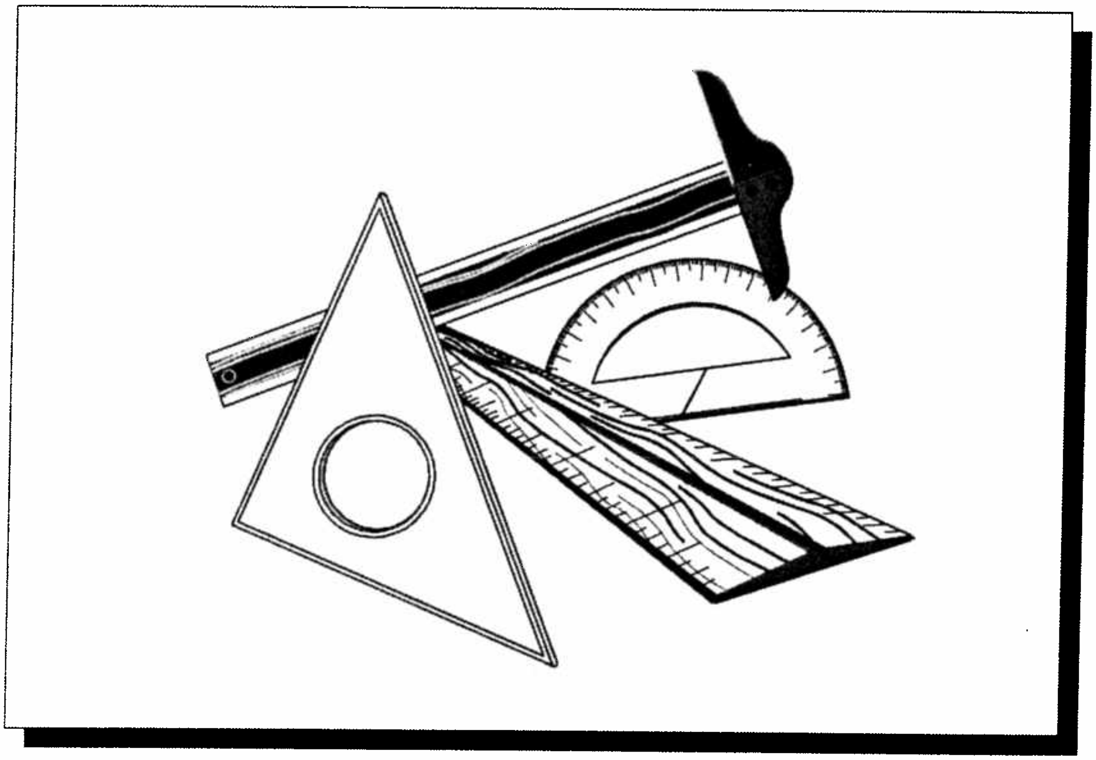
CONSTRUCTION/IMPLEMENTATION

The retrofitting of utility systems, both elevating and protecting in place, must conform to the requirements set forth in local and state building codes, floodplain ordinances, and equipment manufacturer's installation instructions. Additionally, any applicable international and/or national codes or guidelines, such as the International Electrical Code and *Guidelines for Handling Water Damaged Electrical Equipment,* by the National Electrical Manufacturers Association (NEMA), should be observed.

The successful construction and implementation of wet floodproofing measures should include the use of flood-resistant materials and consider operation and preparedness planning as outlined earlier in this chapter.

CHAPTER VI

GENERAL DESIGN PRACTICES



Featuring:

Field Investigation
Analysis of Existing Structure
Design
Construction

GENERAL DESIGN PRACTICES

FIELD INVESTIGATION

- Local Building Requirements
- Hazard Determination
- Documentation of Existing Building Systems
- Homeowner Preferences

ANALYSIS OF EXISTING STRUCTURE

- Structural Reconnaissance
- Footings and Foundation Systems
- Lateral Loads
- Vertical Loads
- Capacity vs. Loading

DESIGN AND CONSTRUCTION

- Elevation
- Relocation
- Dry Floodproofing
- Wet Floodproofing
- Floodwalls
- Levees

Chapter VI: General Design Practices

Table of Contents

Elevation

Relocation

Dry Floodproofing

Wet Floodproofing

Floodwalls

Levees

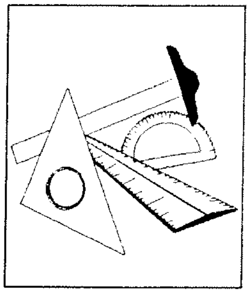
GENERAL DESIGN PRACTICES

Chapter IV introduced the analyses necessary to quantify the flood- and non-flood-related hazards that control the design of a specific retrofitting measure. The objective of Chapter VI is to apply the anticipated loads developed in Chapter IV to the existing site/ structure and design an appropriate retrofitting measure.

This chapter covers the process of designing each retrofitting measure and developing construction details and specifications, providing the designer with tools to tailor each retrofitting measure to local requirements and homeowner preferences. Separate sections for elevation, relocation, dry floodproofing, wet floodproofing, floodwalls, and levees are presented.

The design of these retrofitting measures is a straightforward but technically intensive approach that will result in the generation of construction plans that may receive a building permit and mitigate potential flood and other natural hazards. This design process is illustrated in Figure VI-1.

Many elements of the design process (field investigation, homeowner coordination, maintenance considerations, and analysis of existing structure) are common to many of the retrofitting measures, warranting a general discussion of these elements.



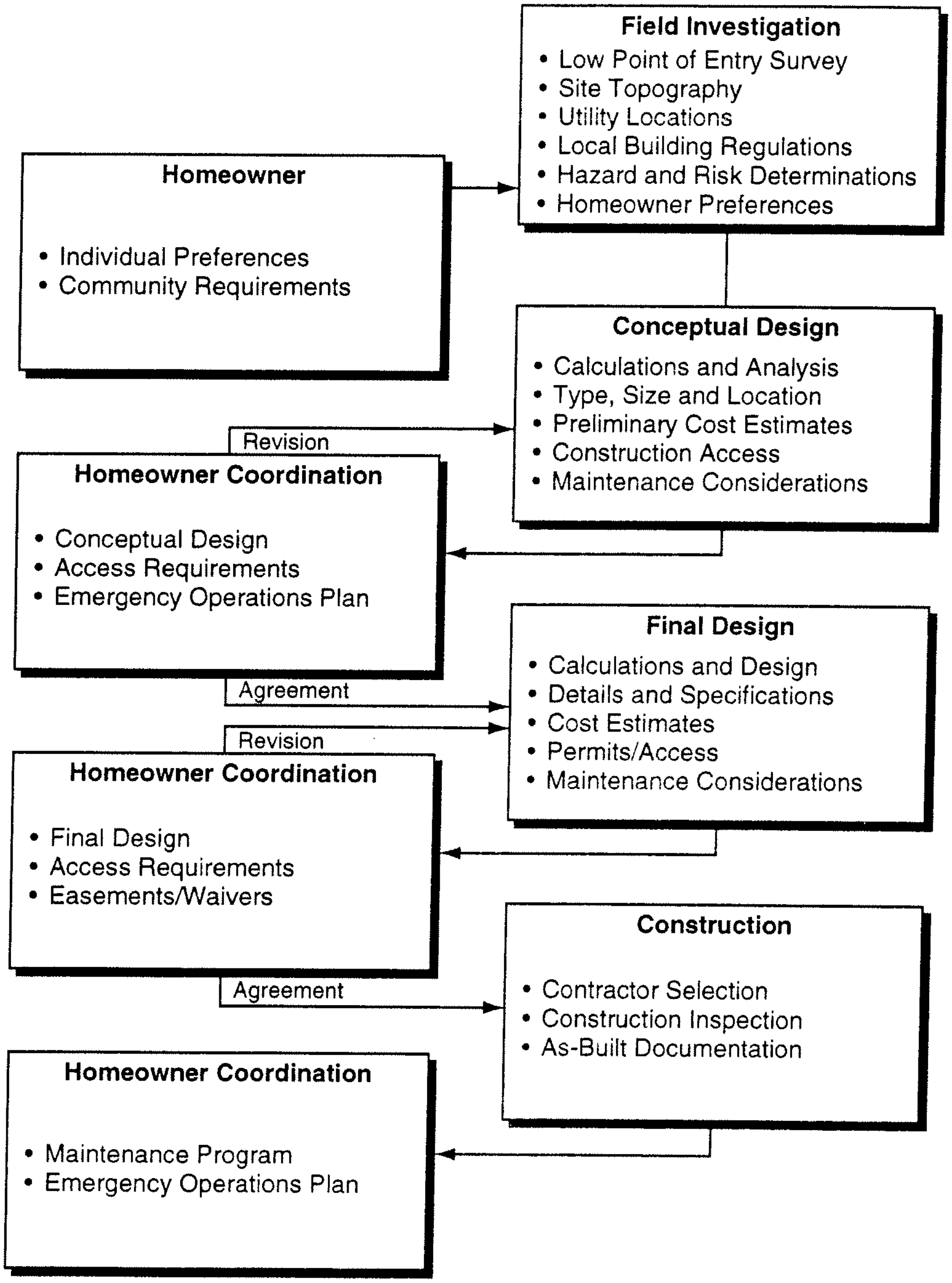


Figure VI-1: Design Process

FIELD INVESTIGATION

Detailed information must be obtained about the site and existing structure to make decisions and calculations concerning the design of a retrofitting measure. The designer should obtain the following information prior to developing retrofitting measure concepts for the owner's consideration:

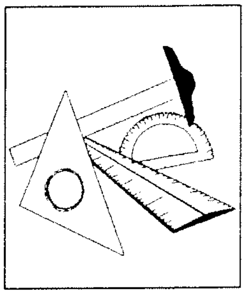
- local building requirements;
- surveys;
- final hazard determinations;
- documentation of existing structural, mechanical, electrical, and plumbing systems; and
- homeowner preferences.

LOCAL BUILDING REQUIREMENTS

Close coordination with the local building code official is critical to obtaining approval of a retrofitting measure design. The designer should review the selected retrofitting measure concept with the local building official to identify local design standards or practices that must be integrated into the design. This discussion may also identify, and provide an opportunity to resolve, issues where construction of the retrofitting measure may conflict with local building regulations.

SURVEYS

A detailed survey of the site should be completed to supplement the information gathered during the Low Point of Entry Determination (discussed in Chapter III) and to



identify and locate structure, site, and utility features that will be needed for the design of the retrofitting measure.

Structure Survey

The structure survey is a vertical elevation assessment at potential openings throughout the structure, whereby flood-waters may enter the residence. It may include:

- basement slab elevation;
- windows, doors, and vents;
- mechanical/electrical equipment and meters;
- finished floor elevation of the structure;
- drains and other floor penetrations;
- water spigots, sump pump discharges, and other wall penetrations;
- other site provisions that potentially may require flood protection such as storage tanks and outbuildings; and
- the establishment of an elevation reference mark on or near the house.

Topographic Survey

> Field surveys for design purposes should be performed by a state registered Professional Land or Property Line Surveyor.

A detailed retrofitting design should not be developed without a site plan or map of the area. A state registered Professional Land or Property Line Surveyor can prepare a site plan of the area, incorporating the Low Point of Entry Determination information, as well as general topographic and physical features. The entire site and/or building lot should be mapped for design purposes. A typical topographic and site survey is shown in Figure VI-2. General surveying practices should be observed, but as a minimum the site plan should include:

- spot elevations within potential work areas;
- one-foot or two-foot contours, depending on degree of topographic relief;
- property lines, easements, and/or lines of division;
- perimeter of house and ancilliary structures (sheds, storage tanks);

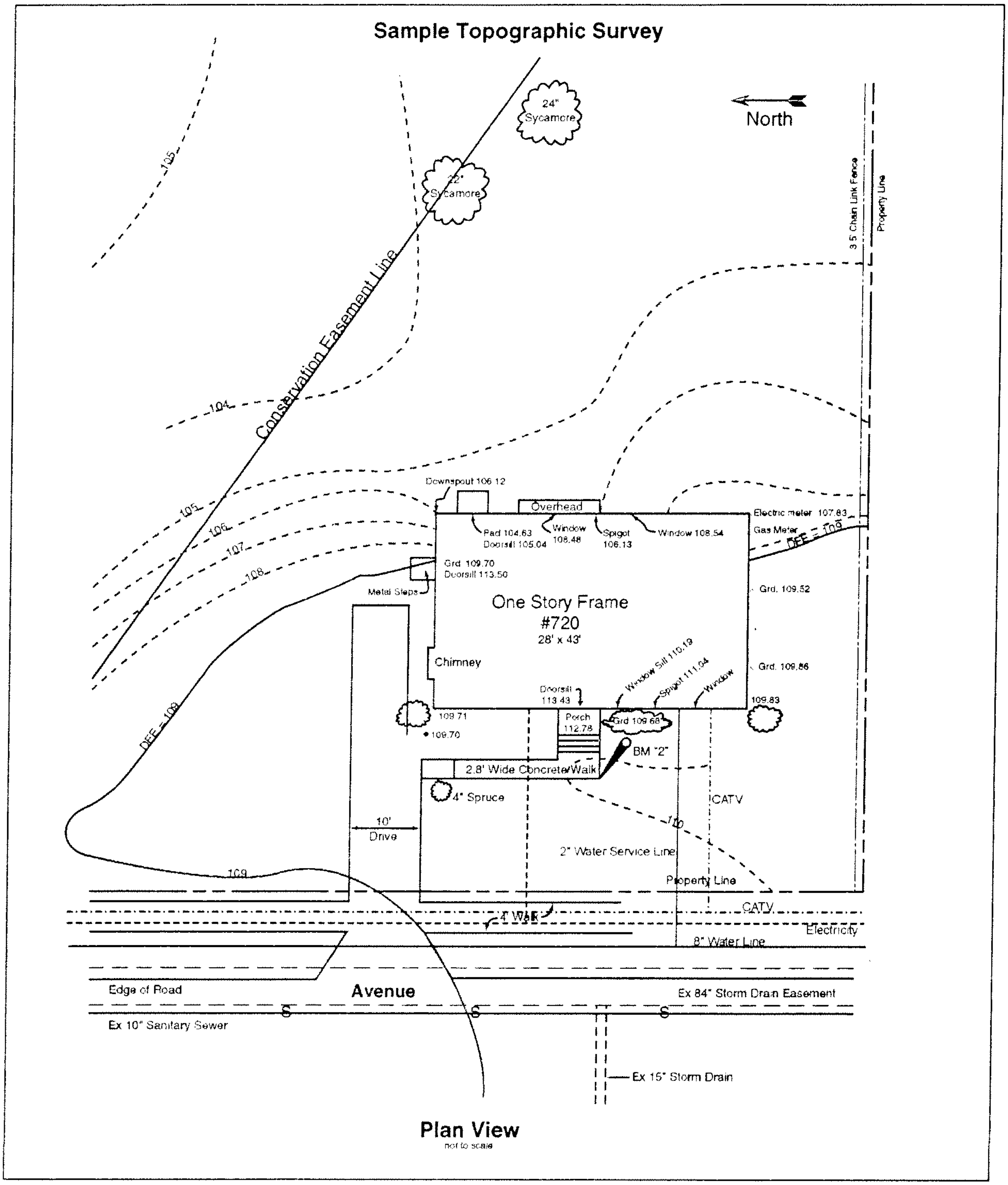


Figure VI-2: Topographic and Site Survey

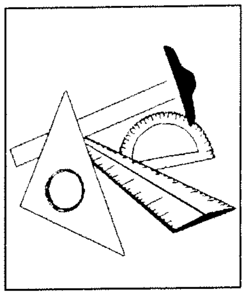
- driveways, sidewalks, patios, mailbox, fences, light poles, etc.;
- exposed utility service (meters, valves, manholes, etc).;
- road or streets;
- downspout locations;
- trees, shrubs, and other site landscaping features;
- building overhangs and chimney;
- window, door, and entrance dimensions;
- mechanical units such as A/C and heat pumps; and
- other appropriate flood data.

Additionally, the site plan should extend at least 50 to 100 feet beyond the estimated construction work area. The purpose of extending the site map beyond the estimated work limits is to insure that potential drainage and/or grading problems can be resolved. Construction site access for materials and equipment as well as sediment and erosion control measures may also have an effect on the adjacent work area. Local building code mapping issues should also be addressed.

Site Utilities Survey

Contact local utility companies regarding the location of underground utilities before construction begins.

As part of the field investigation, above- and below-ground site utilities should be identified. Above-ground utilities, such as power lines, manhole covers, electric meters, etc., can be located both horizontally and vertically on the topographic map. Underground utilities, such as sanitary and storm drain lines, wells and septic tanks, and electric or



gas service, will require an investigation through the appropriate utility agency. Local utility companies and county, municipal, and building code officials will be able to assist in the identification of the underground utilities. Sometimes a copy of the topographic map and area can be submitted to the utility agency, who will prepare a sketch of their underground service. A checklist of underground services includes:

- water main and sanitary sewer pipes;
- water and sanitary service pipes;
- cable television;
- gas lines;
- storm drain pipes;
- water wells;
- electric service;
- telephone cables; and
- other local utility services.

In some instances, exact horizontal and vertical locations of the utility service may be required. A small hole, more commonly referred to as a test pit, can be dug to unearth the utility service in question. Typically this service is performed by a licensed contractor or the utility provider.

By identifying the utility services and units, provisions can be developed during the detailed design that will protect these utilities and keep them operational during a flood. Design provisions for utility relocation, encasement, elevation, anchoring, and, in some instances, new service, can be prepared.

If the design flood elevation is less than the 100-year flood elevation, the retrofitting measure may violate FEMA standards. Check with the local building official or the FEMA Regional Office for clarification.

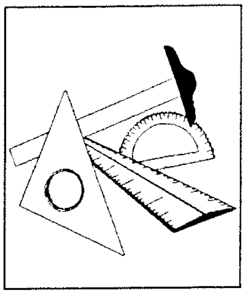
HAZARD DETERMINATIONS

The designer (with the homeowners) should review the risk determinations previously conducted in Chapter III and confirm the flood protection design level and required height of the retrofitting measure selected. Not merely a function of expected flood elevation, freeboard, and low point of entry, this analysis should consider the protection of all components below the design elevation (i.e. below-grade basement walls and associated appurtenences).

The analysis of flood- and non-flood-related hazards was presented in detail in Chapter IV. The designer should utilize the calculation templates presented there to finalize expected design forces.

DOCUMENTATION OF EXISTING BUILDING SYSTEMS

Documentation of the condition of the existing structure is an important aspect of the design of elevation, relocation, and dry and wet floodproofing measures. This topic was introduced in Chapter III as reconnaissance designed to provide preliminary information on the condition of an existing structure and its suitability for the various retrofitting methods.



Since the data sheets provided in this book are generalized for residential housing applications and ask for information that may not be applicable to a specific retrofitting measure, the designer should exercise judgment in collecting the information cited on the checklists.

As the design of a specific elevation, relocation, or dry and wet floodproofing measure is begun, the designer should conduct a detailed evaluation of the type, size, location, and condition of the existing mechanical, electrical, and plumbing systems. The enclosed Mechanical, Electrical, Plumbing, and related Building Systems Data Sheet (Figure VI-3) can be used to document the results of this examination.

(Note: Collect only the data necessary for your project)

Owner Name: ______________________ **Prepared By:** ______________
Address: _________________________ **Date:** ____________________
Property Location: ____

A. EXTERIOR UTILITIES AND APPURTENANCES

Water
- ☐ On-site well or spring
- ☐ Public water system

  Water Purveyor's Name: ______________________________

Sanitary
- ☐ On-site septic and drain field
- ☐ Public sewerage

Storm
- ☐ On-site
- ☐ Public sewerage

Incoming Electrical Service
- ☐ Overhead ☐ Underground
- ☐ Voltage ☐ 120/240 volt 10 ☐ 120/208 volt 10
- ☐ Direct Burial Size: ______________________
- ☐ Service Entrance Cable Amps: ______________________
- ☐ PVC Conduit
- ☐ RGS Conduit

  Transformer #: ______________________________
- ☐ Power Co: ______________________________

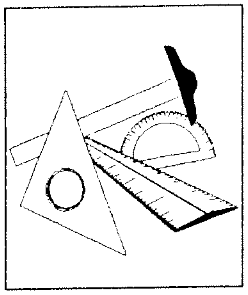
  Power Meter #: ______________________________
  Contact: ______________________________
  Estimated Transformer Rating: ______________________
  Fault Current Rating: ______________________

Telephone Service
- ☐ Company: ______________________________
- ☐ Overhead ☐ Underground
- ☐ Cable Pair
- ☐ Pedestal ☐ Grounded
- ☐ Direct Burial

Cable TV
- ☐ Company: ______________________________
- ☐ Overhead ☐ Underground # of channels: ____________
- ☐ PVC CATV #: ______________________
- ☐ Direct Burial ☐ RGS: Contact: ______________________

Page 1 of 3

Figure VI-3: Mechanical, Electrical, Plumbing and Related Building Systems Data Sheet



Other Utilities

☐ Natural Gas
Utility Company Name: ______________________________
Location of service entrance: ______________________________
Meter Location: ______________________________
☐ LPG
Utility Company Name: ______________________________
Location of gas bottle: ______________________________
How is tank secured? ______________________________
☐ Oil
Oil Supplier:
☐ Above ground tank ☐ Underground tank
Size ______ gallons
Location ______________________________
Vent terminal ______________________________
Elevation: ____ feet or elevation above grade? _____ feet
Fill cap type: ______________________________

B. DOMESTIC PLUMBING

Water

☐ Location of service entrance ______________________________
Main service valve? ☐ Yes ☐ No
Backflow preventer? ☐ Yes ☐ No
Type of water pipe ☐ Copper ☐ Iron ☐ Plastic
☐ Domestic water heater
☐ Gas BTU/HR ______________
☐ Oil GAL/HR ______________
☐ Other Specify units ______________
Size: ______________ gallons
Location: ______________________________
☐ Sanitary Drainage
Floor served? ______________
Fixtures below BFE ☐ Yes ☐ No
Backwater valve installed in fixtures below BFE? ☐ Yes ☐ No
Backwater valves needed (if none exist) ☐ Yes ☐ No
☐ Storm Drainage
Basement floor drains connected? ☐ Yes ☐ No
Is storm combined w/sanitary? ☐ Yes ☐ No

C. HEATING SYSTEM

Type ☐ Central System ☐ Space heaters

Central System
☐ Warm air ☐ Hot water ☐ Steam
Warm Air Furnace
Location: ☐ Basement ☐ 1st Floor ☐ ___floor ☐ Attic Page 2 of 3

Figure VI-3: Mechanical, Electrical, Plumbing and Related Building Systems Data Sheet (continued)

Type: ☐ Upflow ☐ Downflow ☐ Horizontal ☐ Low Boy
Fuel: ☐ Natural Gas ☐ LPG ☐ Electric ☐ Coal ☐ Wood
Burner: ☐ Atmospheric ☐ Fan assisted
Condensing: ☐ Yes ☐ No
Venting: ☐ Natural draft ☐ Forced draft ☐ Direct vent
Air Distribution: ☐ Gravity ☐ Ducted
☐ Sheet metal ductwork
☐ Flexible, non-metallic runouts
☐ Fiberglass ductboard
☐ Location
Air Outlets: ☐ Floor ☐ Low sidewall ☐ High sidewall ☐ Ceiling ☐ 2nd floor
Hot Water/Steam:
Boiler: ☐ Hot Water ☐ Steam
Location: ☐ Basement ☐ 1st Floor ☐ ___floor ☐ Attic
Fuel: ☐ Natural Gas ☐ LPG ☐ Electric ☐ Coal ☐ Wood
Terminal Units: ☐ Baseboard ☐ Radiators ☐ Other

In-Space Heating Equipment
Gas ☐ Room heater☐ Vented ☐ Unvented
☐ Wall Furnace ☐ Conventional ☐ Direct vent
☐ Floor Furnace
Oil/Kerosene: ☐ Vaporizing oil pot heater ☐ Powered atomizing heater
☐ Portable kerosene heater
Electric Heaters: ☐ Wall ☐ Floor ☐ Toe space ☐ Baseboard
Radiant Heat: ☐ Panels ☐ Embedded fireplace ☐ Portable cord and plug
Solid Fuel Heaters: ☐ Simple fireplace ☐ Factory built ☐ Radiant
☐ Circulating ☐ Freestanding
Stoves: ☐ Conventional ☐ Advanced design ☐ Fireplace insert
☐ Pellet stove

D. COOLING SYSTEM
Type ☐ Central ☐ In-space Conditioners

Central Systems ☐ Split system A/C ☐ Unitary A/C ☐ A-Coil add-on
☐ Split system heat pump
Split Systems:
Indoor unit location: ☐ Basement ☐ 1st Floor ☐ ___floor ☐ Attic
Type: ☐ Upflow ☐ Downflow ☐ Horizontal
Air distribution: ☐ Sheet metal ductwork
☐ Fiberglass ductboard
☐ Flexible non-metallic runouts
Air outlets: ☐ Floor ☐ Low sidewall
☐ High sidewall ☐ Ceiling

Outdoor unit location: ______________________________

In-space Air Conditioners: ☐ Window air conditioners
☐ Ductless split systems

Page 3 of 3

Figure VI-3: Mechanical, Electrical, Plumbing and Related Building Systems Data Sheet (continued)

HOMEOWNER PREFERENCES

A detailed discussion of homeowner preferences was presented in Chapter III. The designer should confirm the homeowner's preferences regarding:

- retrofitting measure type, size, and location(s);
- project design desires/preferences;
- limitations on construction area;
- estimated construction budget; and
- potential future site improvements.

Once the designer has collected the above-mentioned information, a conceptual design of the proposed retrofitting measure can be discussed with the homeowner.

At this time the designer should also review and confirm coordination and future maintenance requirements with the homeowner to ensure that the selected retrofitting measure is indeed suitable.

Homeowner Coordination

Homeowner coordination is similar for each of the retrofitting methods and involves reviewing design options, costs, specific local requirements, access and easement requirements, maintenance requirements, construction documents, and other information with the homeowner and regulatory officials to present the alternatives, resolve critical issues, and obtain necessary approvals.

Maintenance Programs and Emergency Action Plans

Development of appropriate maintenance programs for retrofitting measures is critical to the continued success of retrofitting efforts. Refer to FEMA Technical Bulletin 3-93 *Non-Residential Floodproofing—Requirements and Certification for Buildings Located in Special Flood Hazard Areas in Accordance with the NFIP* for additional guidance concerning minimum recommendations for Emergency Operations Plans and Inspection and Maintenance Plans. While this bulletin was prepared for non-residential structures, it contains sound advice for the development of inspection, maintenance, and emergency operation plans.

Design information presented in this chapter relates to field investigation, design calculations and construction details, and construction issues. Since many of the key elements in the field investigation phase were discussed above, only those issues that are critical to the design and successful construction of the particular retrofitting measure are included here.

ANALYSIS OF EXISTING STRUCTURE

> Existing structures should be designed to resist the primary modes of building failures (uplift, overturning, sliding) as well as structural collapse. Particular attention should be focused on building foundations and structural connections. Retrofitting measures must be designed to protect the structures from building failure.

The ability of an existing structure to withstand the additional loads created as a result of retrofitting is an important design consideration. Accurate reconnaissance of the foundation and estimates of the capacity of various structural systems are the first steps in the design of retrofitting measures. The objective of this analysis is to identify the extent to which structural systems must be modified or redesigned to accommodate a retrofitting measure such as elevation, relocation, dry and wet floodproofing, levees, or floodwalls. The steps involved in this analysis include:

- structural reconnaissance;
- determination of the capacity of the existing footing and foundation system;
- analysis of the loads imposed by the retrofitting measure; and
- comparison of the capacity of the existing structure to resist the additional loads imposed by the retrofitting measure.

STRUCTURAL RECONNAISSANCE

In order to determine whether a structure is suited to the various retrofitting measures being considered, the type and condition of the existing structure must be surveyed. Some structural systems are more adaptable to modifications than others. Some retrofitting methods are more suited for, or specifically designed for, various construction types. Of the retrofitting methods discussed, elevation, dry floodproofing, and relocation most directly affect a home's structure. Floodwalls and levees are designed to prevent water from reaching the house and thus should not have an impact on the structure. Wet floodproofing techniques have a lesser impact on the structure due to equalization of pressures, and also require analysis of the existing structure.

Several sources of information concerning the details of construction that were used in a structure include:

- construction drawings from the architect, engineer, or builder. These are usually the best and most reliable resource for determining the structural systems and the size of the members;
- information available from the building permits office;
- plans of any renovations or room additions and a recent record of existing conditions;
- contractors who have performed recent work on the house, such as plumbing, mechanical, electrical, or other kinds;
- a home inspection report, if the home has been recently purchased. While these reports are not highly detailed, they may give a good review of the condition of the house and point out major deficiencies.

If the aforementioned information is not available, the designer (with the permission of the owner) should determine the type and size of the critical structural elements. The structural reconnaissance worksheet provided as Figure VI-4 can be used to document this information.

Owner Name: ______________________	**Prepared By:** ______________________
Address: ______________________	**Date:** ______________________
Property Location: ______________________	

Structural Reconnaissance Worksheet

Sketch and Description of Existing Structure:

Item	Material	Size	Condition (Excellent, Good, Fair, Unacceptable)	Notes
Footing	Concrete			
Foundation Wall	Concrete			
	Concrete Masonry			
	Brick Masonry			
Walls	Wood Frame			
	Masonry			
	Metal Frame			
Floor System	Wood Joist			
	Post and Beam			
	Wood Truss			
Roof System	Truss			
	Rafter			
Exterior Finishes	Wood Siding			
	Brick Veneer			
	Stucco			
Interior Finishes	Drywall			
	Plaster			
	Wood			

Figure VI-4: Structural Reconnaissance Worksheet

Elevating a house exposes it to greater vertical loads from increased wind loadings and additional weight, and horizontal and shear loads from increased wind forces. Figure VI-5 illustrates the various loads that affect a foundation system.

FOOTINGS AND FOUNDATION SYSTEMS

The foundation system of a house (footings and foundation walls) serves several purposes. It supports the house by transmitting the building loads to the ground, and it serves as an anchor against uplift and against forces caused by wind, seismic, flooding, and other loads. Foundation walls (below grade) restrain horizontal pressures from adjacent soil pressures. The foundation system anchors the house against horizontal, vertical, and shear loads from water, soil, debris, seismic, snow, and wind hazards. Retrofitting measures such as elevation change the dynamics of the forces acting on a house.

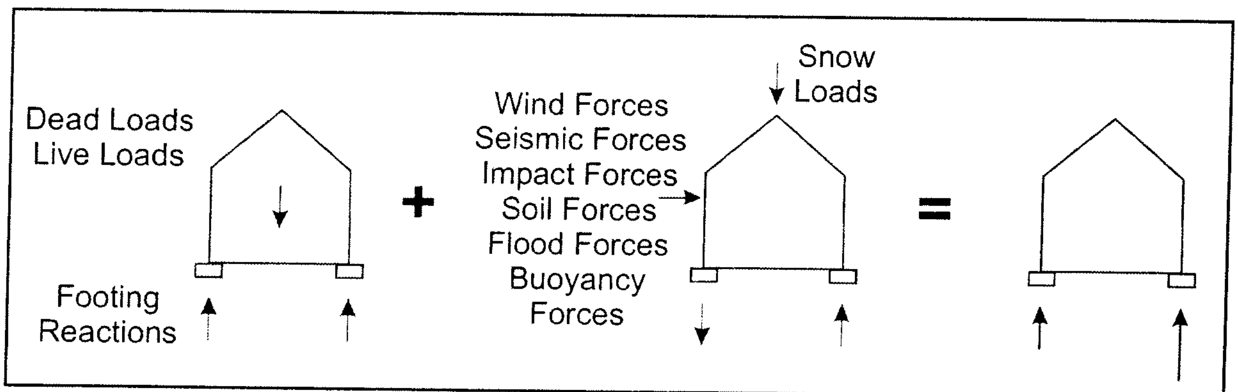
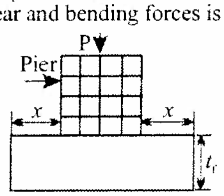


Figure VI-5: Foundation System Loading

The load carrying capacity of residential footings, particularly strip footings, are usually limited by the bearing capacity of the soil. For spread footings (i.e., isolated footings that may be spread over a relatively large area), the footing may be controlled by the structural capacity of the footing itself. Thus, spread footings typically require reinforcing bars near the bottom of the footing. In general, when x exceeds 1.5 times t_f in the figure below, an analysis of shear and bending forces is required. Refer to ACI 318 for investigations of such concrete footings.



Perimeter drainage systems may be used if the bearing soil is adversely affected by saturation. Often soils under bearing pressure will not become saturated due to low permeability. Each situation should be evaluated separately.

When older foundation systems (such as stone) are encountered, the designer should consult the local code on what procedures/applications are allowable. The compressive strength of stone walls is so variable that professional testing and specialized expertise is usually required.

Bearing Capacity of Footings

Footings are designed to transmit building loads to the ground and should be placed completely below the maximum frost penetration depth. The size of the footing can be determined by the formula below:

FORMULA

$$A_f = P/S_{bc} = \_\_\_\_ \text{ ft}^2$$

where: A_f is the bearing area of the footing in square feet;
P is the load in pounds; and
S_{bc} is the allowable soil bearing capacity in pounds per square foot. See Table VI-2.

Formula VI-1: Determining Footing Size

An existing footing should be checked to determine its maximum loading condition. Rearranging the above formula will provide the maximum load for the existing footing.

FORMULA

$$P_{max} = A_f S_{bc} = \_\_\_\_ \text{ lbs}$$

where: P_{max} is the load in pounds;
A_f is the bearing area of the footing (in square feet); and
S_{bc} is the allowable soil bearing capacity in pounds per square foot. See Table VI-2.

Formula VI-2: Maximum Loading of Existing Footing

$$W_f = b_f S_{bc} = \_\_\_\_ \text{ lbs/ft.}$$

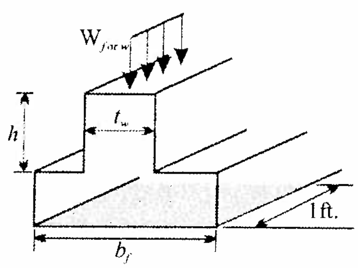
where: W_f is the total weight per linear foot the footing will support; and

b_f is the width of footing in feet.

Formula VI-3: Bearing Capacity of Existing Strip Footing

In conducting this computation, it is important to confirm the size and depth of the footing and bearing capacity of the soil to assure that the existing conditions meet current codes. In the absence of reliable information, excavation may be required to confirm the depth, size, and condition of the existing footing.

W acts downward as shown below.



The designer should also check the existing footing to ensure that it has a perimeter drainage system to prevent saturation of the soil at the footing. If one does not exist, the designer should consider including this feature in the design of the retrofit.

Bearing Capacity of Foundation Wall

The bearing capacity of an existing concrete masonry foundation wall can be estimated if the designer knows the size and grade of the block, using the following formula.

Use of Formula VI-4 is limited and should be verified using ACI 530 and local building codes for design applications.

$$W_w = F_c A = \_\_\_\_ \text{ lbs/ft.}$$

where: W_w is the total weight per linear foot the wall will support;

F_c is the bearing capacity of the masonry from Table VI-1;

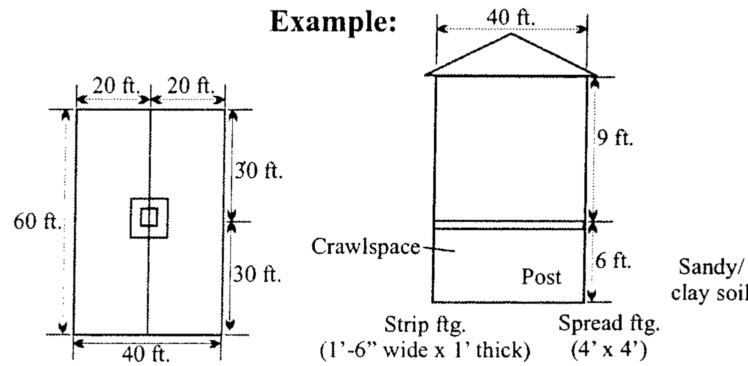
A is the cross sectional area per linear foot of wall = t_w (12")
where: t_w thickness of wall in inches.

Formula VI-4: Bearing Capacity of an Existing Concrete Masonry Foundation Wall

American Concrete Institute (ACI) 530 provides maximum height or length to thickness ratios. Height or length is based on the location of the lateral support elements that brace the masonry and permit the transfer of loads to the resisting elements. Nominal wall thickness may be used for $\mathbf{t}_w$. Table VI-2: Wall Lateral Support Requirements, provides maximum slenderness ratio values for bearing and non-bearing walls.

By changing the value of the bearing capacity according to the conditions identified on the site, the designer can determine the approximate weight that the foundation wall will support. If the type of block and mortar is unknown, the most conservative values should be used. Intrusive methods of investigation must be employed to determine footing depth, thickness, reinforcement, condition, or drainage. Technology exists for investigation of walls using x-ray, ultrasound, and other methods; however, these methods may be too costly for residential retrofitting projects.

Example:



PLAN **SECTION**

<u>Roof</u> Wood trusses, sheathing, ceiling, asphalt singles, insulation-15 psf, load type D. Snow (per building code)-20 psf, load type S.

<u>Floor</u> 2x10 framing, plywood, hardwood/ carpet flr.-12 psf, load type D. Residential dwelling-40 psf, load type L.

<u>Walls</u> Above grade; 2x4 framing, drywall, insulation, sheathing, 4" brick-55 psf, load type D. Below grade; 8" concrete block (hollow units) with some grout, 4" brick-95 psf, load type D.

<u>Foundation Wall/Footing</u> Roof-W_D=15 psf (40')/2 = 300 plf, load type D, W_S=20 psf (40')/ 2=400 plf, load type S. Floor-W_D=12 psf (20')/2=120 plf, load type D. Wall: $(W_D)_{A.G.}$ = 55 psf (9') = 495 plf, load type D; $(W_D)_{B.G.}$ = 95 psf (6;) = 570 plf, load type D. Footing: $(W_D)_{FTG}$ = (1.5' x 1')(150 pcf) = 225 plf, load type D. Total: D + L + S = W = 2,510 plf.

<u>Foundation Wall</u> Area of 8" concrete block = 42.24 in^2. Gross cross section per linear foot. Table VI-1: Assume F_C = 60 psi. Formula VI-4: $W_w = F_c A$ = 60 psi (42.24 in^2) = 2,534 lbs./ft. Since W_w = 2,534 plf > W = 2,510 plf, the bearing capacity of the wall is capable of supporting the vertical loads placed on the wall*
*Actual wall load = (2,510 - 225) = 2,285 plf at base of wall.

<u>Strip Footing</u> 1'-6" wide strip footing - b_f = 1.5'. Table VI-2: Assume S_{bc} = 2,000 psf. Formula VI-3: $W_f = b_f S_{bc}$ = 1.5' (2,000 psf), W_f = 3,000 lbs./ft. Since W_f = 3,000 plf > W = 2,510 plf, the bearing capacity of the footing is capable of supporting the vertical loads placed on the wall.

<u>Spread Footing</u> Tributary area on post - A = (20')(30') = 600 ft.^{2}. 4'x4' footing - A_f = 4'(4'), A_f = 16 ft.^{2}. S_{bc} = 2,000 psf (same as before). P = (Uniform Load) (Trib. Area, A), P_D = 12 psf (600 ft.^{2}) = 7,200 lbs. P_L = 40 psf (600 ft^2) = 24,000 lbs. Total: D + L P_{TL} = 31,200 lbs.
Formula VI-2: $P_{max} = A_f S_{bc}$, P_{max} = (16 ft.^{2}) (2,000 psf) = 32,000 lbs. Since P_{max} = 32,000 lbs. > P_{TL} = 31,200 lbs., the bearing capacity of spread footing is capable of supporting the loads applied.

The approximate bearing capacity of concrete and reinforced concrete materials may be quite variable due to regional differences in concrete mix, aggregate, reinforcing practices, and other factors. In general, the approximate bearing capacity of concrete/reinforced concrete is substantially greater than masonry block: a conservative estimate ranges from 500 to 1,000 pounds per square inch. Additional information on the capacity and strength of concrete mixtures can be obtained from the American Concrete Institute (ACI) 318.

Values shown in Table VI-1 are guidelines. Additional information on the capacity and strength of masonry construction can be obtained from ACI 530.

Values shown in Table VI-2 are guidelines. They are derived from the 2000 International Building Code and other sources.

Table VI-1	**Approximate Bearing Capacity for Masonry Walls, F_c, on gross cross section (lb/in^2)[1]**
Solid masonry of brick and other solid units of clay or shale; sand-lime or concrete brick	100 to 115
Grouted masonry, of clay or shale; sand-lime or concrete	100 to 115
Masonry of hollow units	55 to 75
Stone: cut granite cut limestone, marble cut sandstone, cast stone rubble; rough, random, or coursed	 640 to 720 400 to 450 320 to 360 100 to 120

1.) Minimum thickness:
Masonry bearing walls;
-one story - 6 inches,
-more than one story - 8 inches
Rubble stone walls;
-rough, random, or coursed - 16 inches
Note: See ACI 530-95 if dimensions stated above are not met.

Table VI-2	**Allowable Soil Bearing Capacity, S_{bc}[1]**
Type of Soil	**S_{bc} Range, pounds per square foot**
1. Rock	6,000+
2. Sandy gravel, gravel	2,500 to 3,500
3. Sand, silty sand, clayey sand, silty gravel, clayey gravel	1,500 to 2,500
4. Clay, sandy clay, silty clay, clayey silt	1,000 to 2,000

1.) Experience with local conditions should be used to modify these values when appropriate.

LATERAL LOADS

> For additional information concerning the performance of various structural systems, refer to the U.S. Army Corps of Engineers research study entitled *Flood Proofing Tests,* August, 1988.

> For masonry walls, use ACI 530 to determine allowable stress information. For plywood shear walls, the Engineered Wood Association offers allowable load information. For reinforced concrete walls, consult ACI 318. For non-reinforced concrete walls, consult Chapter 22 of ACI 318.

> Lateral loads are distributed to the shear walls via the diaphragms of the floor or roof. Distribution is based upon relative stiffnesses of the walls. Use extreme care in the design of diaphragm-to-wall connections. Most codes require that an additional eccentricity (factor of safety) be considered in the location of the resultant of the lateral loads.

The ability of exterior foundation walls and interior structural walls to withstand flood-related and non-flood-related forces is dependent upon the wall size, type, and material. Interior and exterior walls are checked for failure from overturning, bending, and shear (horizontal, vertical, and diagonal). If the stress caused by the expected loading is less than the code-allowable stress for the expected failure mode, the wall design is acceptable. Conversely, if the stresses caused by the expected loadings are greater than the code-allowable stresses for the expected failure mode, the design is unacceptable and reinforcing is required.

Due to the large number of wall types and situations that can be encountered that would make a comprehensive examination of this subject unwieldy for this manual, only procedural and reference information for lateral load resistance is provided. The process of analyzing foundation and interior walls is outlined below:

Step 1: Determine the type, size, material, and location of the walls to be analyzed.

Step 2: Determine the code-allowable overturning, bending, and shear stresses for the wall in question.

For additional information on loading conditions for exterior and shear walls, refer to *ASCE 7*.

Step 3: Compare the stresses caused by the expected loadings versus code-allowable stresses (capacities) for each wall being analyzed. If the stresses caused by the expected loadings are less than the code-allowable stresses, the design is acceptable; if not, reinforcement is required or another method should be considered.

VERTICAL LOADS

In addition to the loads imposed by floodwaters, other types of loads must be considered in the design of a structural system, such as building dead loads, live loads, snow loads, wind loads, and seismic loads (if applicable). Flood, wind, and seismic loads were discussed earlier in Chapters III and IV. This section deals with the computation of dead loads, live loads, and snow loads.

Dead Loads

Dead loads are the weight of all permanent structural and nonstructural components of a building, such as walls, floors, roofs, ceilings, stairways, and fixed service equipment. The sum of the dead loads should equal the unoccupied weight of the building. The weight of a house can be determined by quantifying the wall and surface areas and multiplying by the weights of the materials or assemblies. A list of the weights of some construction types is provided in Table VI-3. In addition to the weight of the structure, any furnishings and equipment located in the house must be added to the total. The worksheet provided at Figure VI-6 can be used to make a preliminary estimate of the weight of a structure. To use Figure VI-6, the designer should:

Step 1: Determine the construction of the various components of the building, quantify them, and enter this information in the second column;

Step 2: Look up the weight of these assemblies and enter that figure into the third column;

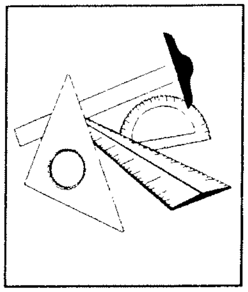
Step 3: Multiply the quantities by the unit weights to obtain the construction component weights, and enter the result in the fourth column;

Step 4: Add these component weights in column four to obtain an estimate of the total weight of the structure. Enter the result in the box at the bottom of column four.

Values provided in Table VI-3 are typical guideline values. See ASCE 7 for additional information.

Table VI-3 **Weights of Construction Types**

Construction	Weight, lb/ft² surface area
Wood stud wall, 2x4, interior, ½-in drywall 2S	8
Interior, wood or metal 2x4s, plaster 2S	19
Exterior, drywall; 4-in batt insul.; wood siding	11
Exterior, drywall; 4-in batt insul.; 4-in brick (MW)	47
Exterior, drywall; 4-in batt insul.; 8-in concrete block	60-65
Metal stud wall, 2x4, interior, ½-in drywall 2S	7
Exterior, drywall; 4-in batt insul.; 1-in stucco	23
Metal stud wall, exterior, drywall; 4-in batt insul.; 2-in drywall	18
Exterior, drywall; 4-in batt insul.; 3-in granite or 4-in brick	55
Plaster, per face, wall, or ceiling, on masonry or framing	8
Ceramic tile veneer, per face	10
Masonry wall, 4-in brick, MW, per wythe	39
4-in conc. block, heavy aggregate, per wythe	30
8-in conc. block, heavy aggregate, per wythe	55
Glass block wall, 4-in thick	18
Glass curtain wall	10-15
Floor or ceiling, 2x10 wood deck, outdoors	8-10
Wood frame, 2x10, interior, unfinished floor; drywall ceiling	8-10
Concrete flat slab, unfinished floor; susp. ceiling	80-90
Concrete pan joist (25 in o.c., 12-in pan depth, 3-in slab), unfinished floor; susp. ceiling	90-100
Concrete on metal deck on steel frame, unfinished floor; susp. ceiling	65-70
Finished floors, add to above:	
Hardwood	3
Floor tile	10
1½-in terrazzo	25
Wall-to-wall carpet	2
Roof, sloping rafters or timbers, sheathing; 10-in batt insul.; ½-in drywall	12-15
Built-up 5-ply roofing, add to above	6
Metal roofing, add to above	3-4
Asphalt shingle roofing, add to above	4
Slate or tile roofing, ¼-in thick, add to above	12
Wood shingle roofing, add to above	3-5
Insulation, batt, per 4-in thickness	1
Insulation, rigid foam boards or fill, per inch thickness	0.17
Stairways:	
Concrete	80-95
Steel	40-50
Wood	15-25



Owner Name: ____________________ Prepared By: ________________________

Address: ________________________ Date: ______________________________

Property Location: ____

Building Weight Estimating Worksheet

Construction Type (1)	Surface Area (2)	Weight (lbs/sf) of Surface Area (3)	Weight Component (4)
Walls			
Exterior			
Interior			
Floors			
First			
Second			
Attic			
Roof			
Special Items			
Fireplace*			
Chimney*			
Structure Weight			
Furnishings			
Total Weight			

Figure VI-6: Building Weight Estimating Worksheet

*Do not include if chimney/fireplace has a separate foundation.

Check local codes for guidance on acceptable live loads. In the absence of code information use *ASCE 7*.

Live Loads

Live loads are produced by the occupancy of the building, not including environmental loads such as wind loads, flood loads, snow loads, earthquake loads, or dead loads. For residential one- and two-family dwellings, a typical floor live load is a uniformly distributed load of 40 pounds per square foot.

FORMULA

$$LL = A\,L_o = \_\_\_\_ \text{ lbs}$$

where: **LL** is the live load in pounds;
A is the area of each floor of the residence in square feet; and
L_o is the minimum uniformly distributed live load in pounds per square foot.

Formula VI-5: Calculation of Live Load

Roof Snow Loads

The roof snow load varies according to the geography, roof slope, and thermal, exposure, and importance factors. Local building codes should be consulted to find the snow load and how to apply it to the structure. Take particular care to account for drift and unbalanced snow loads. If no local code is available, the designer should refer to *ASCE 7* for this information. In areas of little snowfall, codes may require a minimum roof snow load.

Calculation of Vertical, Dead, Live, and Snow Loads

Dead, live, and snow loads act vertically downward and are carried by the load-bearing walls or the columns to the foundation system. The load-bearing walls support any vertical load in addition to their own weight. The amount of the dead load carried by a wall or column is calculated based on the partial area of the roof and floor system (tributary areas) that are supported by that wall or column plus its own weight (self weight). The tributary areas are illustrated in Figures VI-7 and VI-8 and determined as follows:

For the load-bearing walls, the tributary area is the area bounded by the length of the wall perpendicular to the floor joists or roof trusses multiplied by half the span length of the joist or truss.

FORMULA

$$A_w = lw/2 = \_\_\_\_ \text{ ft}^2$$

where: A_w is the wall tributary area in square feet;
l is the length of the wall in feet; and
w is the span length between walls or the wall and center girder in feet.

Formula VI-6: Calculation of Tributary Area for Load-bearing Walls

FORMULA

$$A_g = l\,(a+b)/2 = \_\_\_\_ \text{ ft}^2$$

where: A_g is the center girder tributary area in square feet;
l is the length of the wall in feet; and
$a+b$ is the span length between the center girder and walls in feet.

Formula VI-7: Calculation of Tributary Area for Center Girder

For columns the tributary area is the area bounded by imaginary lines drawn halfway between the column and the adjacent load-bearing wall or column in each direction.

FORMULA

$$A_t = (w/2)(l/2) = \_\_\_\_ \text{ ft}^2$$

where: A_t is the column tributary area in square feet;
l is the length of the wall surrounding the column in feet; and
w is the span length between walls surrounding the column in feet.

Formula VI-8: Calculation of Tributary Area for Columns

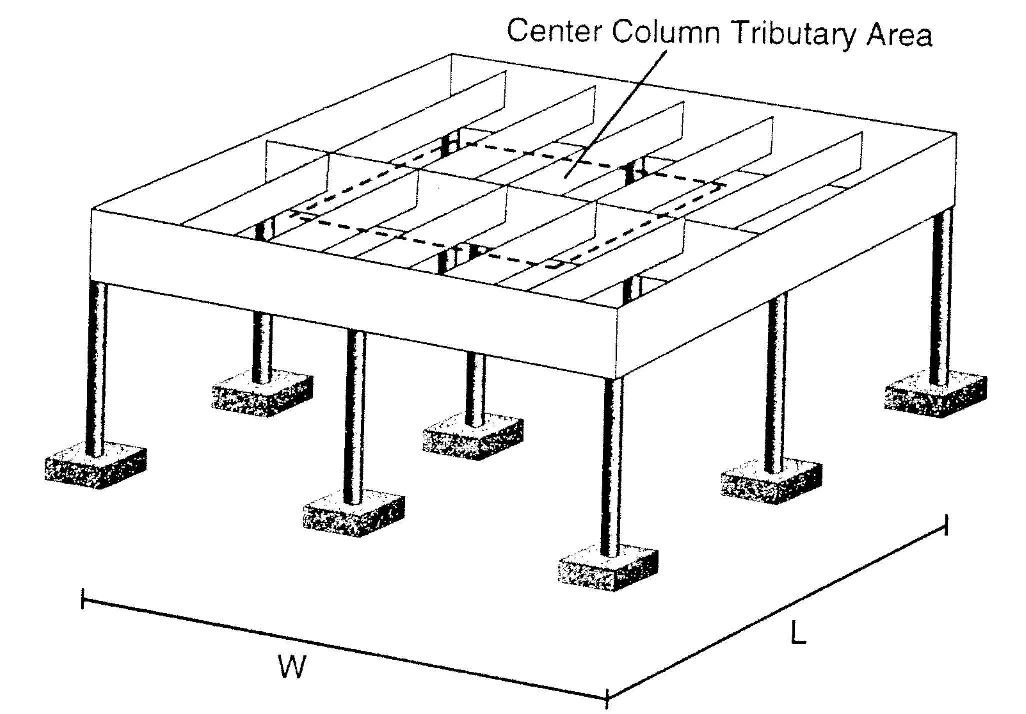


Figure VI-7: Column Tributary Area

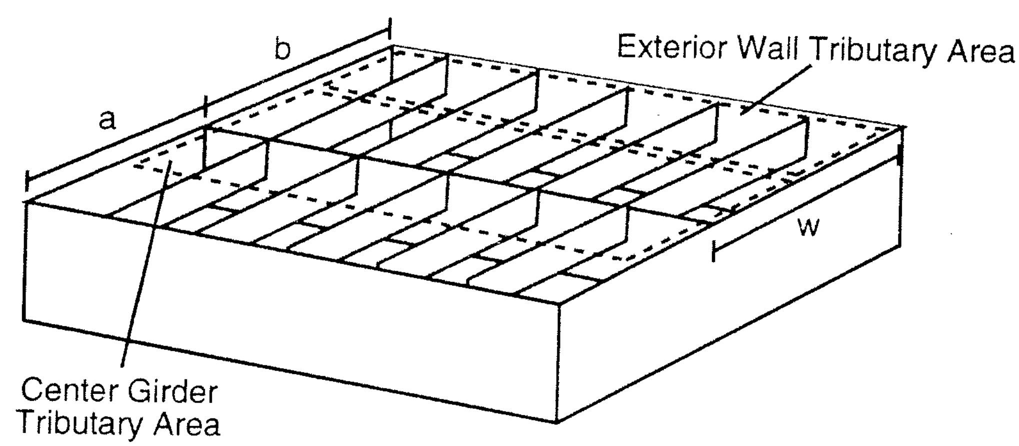


Figure VI-8: Wall/Girder Tributary Area

To calculate the loads, follow the steps below:

Step 1: Inspect the roof and the floor construction to identify load-bearing walls. Mark the direction, the span length, and the supporting walls or columns for the roof trusses and floor joists.

Step 2: Calculate the roof and the floor tributary areas for each load-bearing wall and column.

Step 3: For each load-bearing wall and column, multiply the tributary areas by the dead, live, and snow loads to find the total loads.

FORMULA

$$\mathbf{TL_{dis} = (DL + LL + SL)\, A_t = \_\_\_\_\ lbs}$$

where: **TL_{dis}** is the total dead, live, and snow loads acting on a specific wall or column in pounds;

DL is the dead load in pounds per square foot (from Figure VI-6);

LL is the live load in pounds per square foot (from Formula VI-5);

SL is the snow load in pounds per square foot (from code); and

A_t is the tributary area of the wall or column in square feet (from Formulas VI-6 and VI-8). (When analyzing walls use A_w instead of A_t.)

Formula VI-9: Calculation of Wall/Column Loads

Step 4: Calculate the self weight of the wall or column. Add any overbearing soil and foundation weight to the total. This information can be taken from the calculation template shown in Figure VI-6.

FORMULA

$$\mathbf{SW = SA\ W_u + OSW + FW = \_\_\_\_\ lbs}$$

where: **SW** is the self weight of the component in pounds;

SA is the section area of the component in square feet; and

W_u is the unit weight of the component in pounds per square foot of surface.

OSW overbearing soil weight in pounds

FW foundation weight in pounds

Formula VI-10: Calculation of the Self Weight of the Wall/ Column

Step 5: Add all the above calculated loads to find the load carried by the wall or column to the foundation or footing.

FORMULA

$$\mathbf{TL = SW + TL_{dls} = \_\_\_\_ lbs}$$

where: **TL** is the total load carried by the wall or column to the footing or foundation in pounds;

SW is the self weight of the component in pounds; and

TL_{dls} is the total dead, live, and snow loads acting on a specific wall or column in pounds.

Formula VI-11: Calculation of Total Load Carried by the Wall or Column to the Footing or Foundation

CAPACITY VERSUS LOADING

The next step is to examine the capacity of the existing foundation component or system versus the expected loading from a combination of dead, live, flood, wind, snow, and seismic loads. This analysis will provide an initial estimate of the magnitude of foundation modifications necessary to accomplish an elevation or relocation project.

International building codes (IBC and IRC) require the analysis of a variety of loading conditions and then base the capacity determination on the loading condition that presents the most unfavorable effects on the foundation or structural member concerned.

Designers should refer to ASCE 7-98 when conducting load combination analysis.

It is the purpose of the load combinations to identify critical stresses in structural members (or nonstructural members) and critical conditions used to design the support system. Since every conceivable situation cannot be covered by standard load cases, sound engineering judgment must be used.

Load Combination Scenarios

ASCE 7-98 prescribes how to analyze flood loads in concert with other loading conditions. This guidance involves the use of two methods—allowable stress design and strength design. In the case of allowable stress design, design specifications define allowable stresses that may not be exceeded by load effects due to unfactored loads, that is, allowable stresses contain a factor of safety.

In strength design, design specifications provide load factors, and, in some instances, resistant factors.

The analysis of loading conditions may be checked using either method provided that method is used exclusively for proportioning elements of that construction material. The designer should consult ASCE 7-98 for guidance in analyzing the multi-hazard loading conditions described below:

The following symbols are used in defining the various load combinations.

D Dead Load

E Earthquake Load

F Load due to fluids with well defined pressures and maximum heights

F_a Flood Load

H Load due to weight and lateral pressure of soil and water in soil

L Live Load

L_r Roof Live Load

R Rain Load

S Snow Load

T Self-Straining Force

W Wind Load

These symbols are based upon information from ASCE 7-98 but do not match exactly as several symbols had to be revised to accommodate symbols already used in this manual. Refer to ASCE 7-98 for clarification and additional information.

STRENGTH DESIGN METHOD

When combining loads using the strength design methodology, structures, components, and foundations should be designed so that their strength equals or exceeds the effects of the factored loads in the following combinations:

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W)$
4. $1.2D + 1.6W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + 0.5L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E \text{ or } 1.6H$

Exception 1: The load factor on **L** in combinations (**3**), (**4**), and (**5**) shall equal 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 lb/ft^2 (pounds force per square foot).

Exception 2: The load factor on H shall be set equal to zero in combinations (6) and (7) if the structural action due to H counteracts that due to W or E. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.

Each relevant strength limit state shall be investigated. Effects of one or more loads not acting should be investigated. The most unfavorable affects from both wind and earthquake loads should be investigated, where appropriate, but they need not be considered to act simultaneously. When a structure is located in a flood zone, the following load combinations shall be considered:

1. In V Zones or coastal A Zones, $1.6W$ in combinations (4) and (6) shall be replaced by $1.6W + 2.0F_a$.
2. In non-coastal A Zones, $1.6W$ in combinations (4) and (6) shall be replaced by $0.8W + 1.0F_a$.

This material is taken directly from ASCE 7-98.

ALLOWABLE STRESS METHOD

When combining loads using the allowable stress method, the loads should be considered to act in the following combinations, whichever produces the most unfavorable effect on the building, foundation; or structural member being considered. This material is taken directly from ASCE 7-98.

1. D
2. $D + L + F + H + T + (L_r \text{ or } S \text{ or } R)$
3. $D + (W \text{ or } 0.7E) + L + (L_r \text{ or } S \text{ or } R)$
4. $0.6D + W + H$
5. $0.6D + 0.7E + H$

The most unfavorable effects from both wind and earthquake loads should be considered, where appropriate, but they need not be assumed to act simultaneously. Buildings and other structures should be designed so that the overturning moment due to lateral forces (wind or flood) acting singly or in combination does not exceed two-thirds of the dead load stabilizing moment unless the building or structure is anchored to resist the excess moment. The base shear due to lateral forces should not exceed two-thirds of the total resisting force due to friction and adhesion unless the building or structure is anchored to resist the excess sliding force. Stress reversals should be accounted for where the effects of design loads counteract one another in a structural member or joint.

When a structure is located in a flood zone, the following load combinations shall be considered:

1. In V Zones or coastal A Zone, $1.5F_a$ shall be added to other loads in combinations (3) and (4), and E shall be set equal to zero in (3).
2. In non-coastal A Zones, $0.75F_a$ shall be added to combinations (3) and (4), and E shall be set equal to zero in (3).

This material is taken directly from ASCE 7-98.

Analyzing the existing structure's capacity to resist the expected loads is sometimes a long and tedious process, but it must be done to ensure that the structure will be able to withstand the additional loadings associated with various retrofitting measures.

The objective of this analysis is to verify that:

- the existing structure is able to withstand the anticipated loadings due to the retrofitting measure being considered;

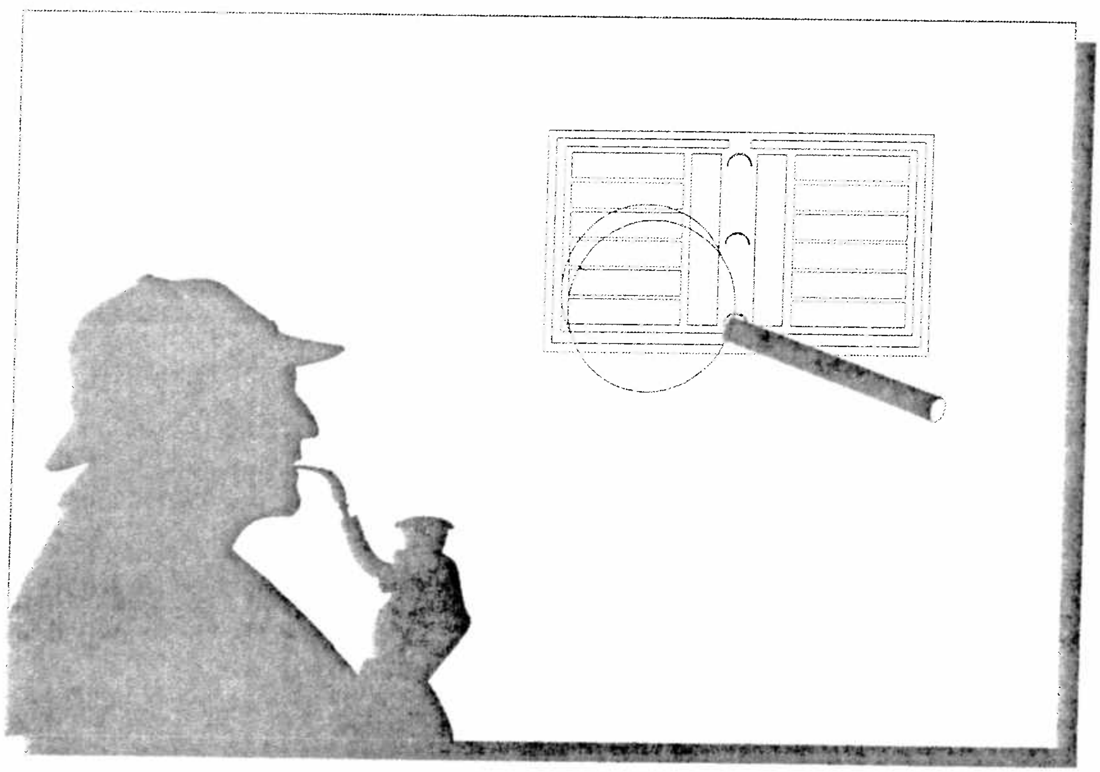
- the existing structure is unable to withstand the anticipated loadings due to the retrofitting measure being considered and requires reinforcement or other structural modification; and/or

- the retrofitting measure should be eliminated from consideration.

Using the information presented here, the designer should be able to conduct the analyses to implement the stated objective and identify the measures/modifications that must be designed.

CHAPTER VII

CASE STUDIES



Featuring:

Elevation
Relocation
Levees and Floodwalls
Wet Floodproofing
Dry Floodproofing

Case Studies

Table of Contents

CASE STUDIES

This chapter presents case studies of actual structural and nonstructural retrofitting measures. The studies illustrate some of the procedures presented in previous chapters in actual practice. The cases include elevation, relocation, small levees and floodwalls, and wet and dry floodproofing methods.

The case studies were extracted from the following reports:

Flood Proofing Technology in the Tug Fork Valley, US Army Corps of Engineers, National Flood Proofing Committee, April, 1994.

A Flood Proofing Success Story Along Dry Creek at Goodlettsville, Tennessee, US Army Corps of Engineers, National Flood Proofing Committee, September, 1993.

Raising and Moving the Slab-on-Grade House with Slab Attached, US Army Corps of Engineers, National Flood Proofing Committee, 1990.

Bailey Creek Flood Prevention, Resource, Conservation and Development Project, Madison, Connecticut, Connecticut Department of Environmental Protection, New Haven County Soil and Water Conservation District, US Department of Agriculture Natural Resources Conservation Service.

Henson Creek Floodplain Study, Prince George's County, Maryland, Department of Environmental Resources, Watershed Protection Branch.

ELEVATION

This section presents two case studies that identify procedures, methodology, and design parameters used to elevate houses. Case Study #1 illustrates the elevation of houses on masonry walls, masonry piers, and wood posts in the Tug Fork Valley, West Virginia. Case Study #2 illustrates the elevation of homes on a crawlspace in Goodlettsville, Tennessee.

CASE STUDY #1
Elevating Houses on Masonry Walls, Masonry Piers and Wood Posts
Tug Fork Valley, West Virginia

The Tug Fork Valley is located on the border of southern West Virginia and northeastern Kentucky (see Figure VII-1.1). The April 1977 flood provided the impetus for formulating a flood damage reduction plan, which used both structural and nonstructural measures to achieve a cost-effective and socially acceptable solution to the flooding problems in the valley.

As a result of the April 1977 flood, Congress enacted legislation within the Energy and Water Development Act of 1980. This Act was unique in that it authorized the Chief of Engineers to take whatever measures were necessary and advisable to reduce flood damages at federal expense. In effect, the Act provided a fertile legislative environment for the formulation and implementation of an array of both structural and nonstructural measures in the Tug Fork Valley.

Retrofitting Options

Structures located in the floodplain that would suffer damages to the first habitable floor during a recurrence of a flood of the magnitude of the April 1977 flood were eligible for either voluntary retrofitting or acquisition. Eligibility for retrofitting required that:

- the structure would suffer damages to the first floor or to mechanical systems below the first floor;
- the structure not be located within the regulatory floodway;

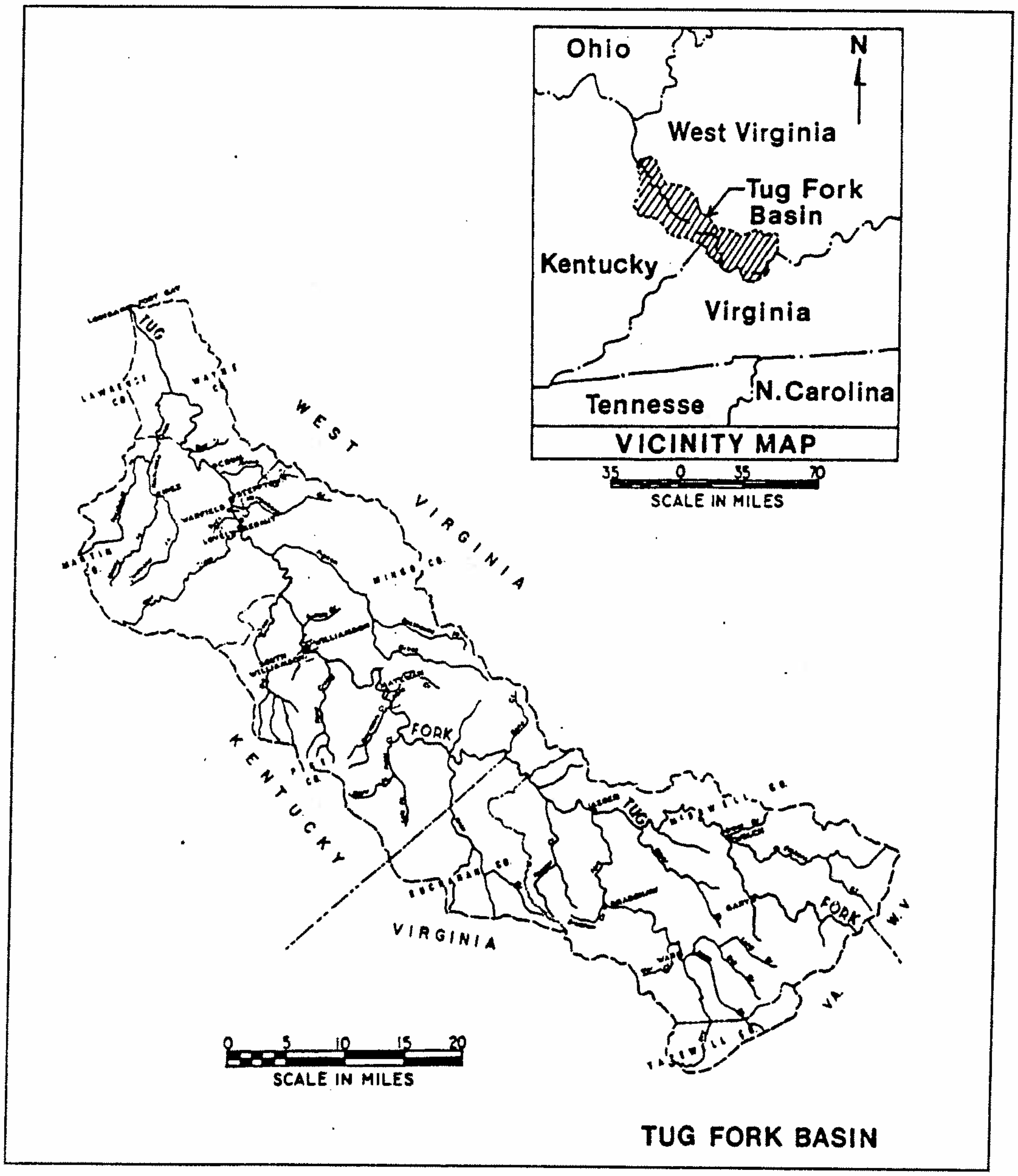


Figure VII-1.1: Tug Fork Valley

- raising the structure to an elevation one foot above the April 1977 flood level would not place the first floor more than 12 feet above the adjacent ground surface; and

- the structure was structurally sound and could be raised safely.

The method chosen for retrofitting was based upon engineering feasibility and cost-effectiveness. The options available included the following:

- elevation of the livable area on a solid masonry wall foundation, masonry pier, or wood post/beam foundation;

- construction of a veneer wall against the structure with sealed openings at entrances (see Case #8);

- construction of floodwalls or levees around an individual or group of structures; or

- construction of a replacement floodproofed structure on-site.

For those structures for which elevation was the most cost-effective option, the owner was required to execute an agreement, prior to start of construction, that restricted future use of the enclosed lower area below the elevated first floor. Future enforcement of owner operation and maintenance of the retrofitting construction and owner compliance with the restrictive agreements was transferred to the local government sponsor following the final construction inspection.

Retrofitting Design Parameters

The following series of design parameters was developed for the retrofitting program:

- **The Design Flood -** Established by legislation as the April 1977 flood or the 100-year flood level if it was higher at the project site.

- **Freeboard -** A one-foot freeboard for elevated structures was measured from the elevation of the design flood to the bottom of the subfloor material or floor slab of the first floor.

- **Veneer Wall Design -** The maximum height for the design of a veneer wall is dependent upon the strength of the existing structure walls and the soil conditions supporting the structure (see Case #8).

- **Height of Rise -** The height limit for elevating structures was determined to be 12 feet from the adjacent ground surface. This tall height limit resulted in a substantial savings in program costs by reducing the number of structures for which acquisition/relocation was the only option.

- **Floodwater Velocity** - Hydrologic and engineering studies for foundation designs showed that retrofitting structures by elevation or veneer wall could only occur where floodwater velocities did not exceed eight feet per second.

- **Structure Condition** - Structures found to be deteriorated beyond a point where limited rehabilitation would not permit safe elevation were not raised.

- **Adjacent Structures -** In some situations, portions of adjacent structures were temporarily demolished in order to place steel lifting beams for raising the structure to be elevated. Justified temporary demolition costs were reimbursed as a part of the total construction costs.

Retrofitting Costs

Retrofitting existing structures by elevation can be a complicated and labor-intensive process. The factors described above all contribute to the cost of elevating an existing structure. The key factors influencing the cost of retrofitting by elevation include:

- size, condition, and construction type (frame or masonry) of the structure;

- the height of elevation required and the type of foundation needed to support the structure;

- the need for structure rehabilitation;

- the type, condition, and location of mechanical and utility systems;

- requirements for structure access, including handicapped access; and

- access to the site.

Table VII-1.1 below shows the percentage contribution to elevate a structure.

Table VII-1.1 **Retrofitting Cost for Structure Elevation**

Construction Items	Percent of Total Construction
Structure Lifting	27
Foundations	21
Mechanical and Utilities	9
Carpentry and Finishings	14
Site Work, Mobilization, and Cleanup	29
	100

From "Floodproofing Technology in the Tug Fork Valley," U.S. Army Corps of Engineers

Applied Retrofitting Technology

The choice of a particular foundation for an elevated structure and the basic design of the supporting foundation were critical cost and coordination elements in the retrofitting program. Several factors influenced the basic design and application of foundations in the Tug Fork Valley, including:

- floodplain location of the structure and the inherent hydraulic characteristics of that location;
- height of house raising required to reach the design flood elevation with freeboard;
- type of building construction such as frame or masonry;
- use and condition of structure;
- architectural character of the structure; and
- cost effectiveness of the solution.

Generally, three types of supporting foundations were used to raise the structures in the Tug Fork Valley:

- reinforced solid masonry wall;
- masonry pier construction; and
- wood post and beam.

Elevation Using a Masonry Wall Foundation

The majority of structures completed in the first three approved phases of the Tug Fork Valley project were raised on reinforced masonry wall foundations. The decision to use this type of foundation was based upon the architectural styles of structures located in those project areas and the increased support strength needed in areas of higher flood-water velocity.

Normally, existing foundations and footings on eligible structures were deteriorated due to repeated flooding or were unsuitable as a base for the new walls due to poor construction. For this reason, most, if not all, portions of the existing footing and foundation walls were demolished during the raising process. Where possible, the existing footing and portions of the existing foundation walls were used as a base for the extended masonry wall.

The basic design of the reinforced masonry wall foundation (see Figure VII-1.2) consisted of a continuous perimeter wall of concrete block (8 x 8 x 16 or 8 x 12 x 16 inch block) resting upon an appropriately sized (12 x 18 or 12 x 24 inch) reinforced concrete footing. The masonry wall contained vertical steel reinforcing grouted into every third cell of the concrete block.

The vertical steel was placed in two-foot lengths with 12-inch lap spacings. All concrete block cells were grouted solid below grade, and block sealer was applied to the exterior block face below grade to prevent moisture penetration. The exterior surface of the block was painted with a coating of block filler and two coats of latex paint (owner's choice of colors). The vertical steel was tied to the footing reinforcing and a continuous bond-beam course positioned near the top of the foundation wall. Generally, number four steel rebar was used in the footing as vertical reinforcing, and in the bond-beam course.

In addition to the vertical reinforcing, steel reinforcing (standard truss "dur-o-wal") was added to alternating horizontal mortar joints. Steel anchor bolts were extended into grouted block cells from the bond-beam course to the new sill plate, or steel strapping was included in the grouted block cells and attached to the existing joists for anchoring the first floor to the new foundation (see Figure VII-1.2).

In those limited cases where the existing footing was suitable as a base for the new foundation, the existing footing was drilled, new number four steel reinforcing bars were grouted in, and a strip footing cap was poured on top of the old footing before laying new foundation block. A continuous grout layer was placed on top of all footings before laying the initial block course.

In cases where the structure had an existing below-grade basement, the existing basement wall was removed two feet below grade and a new footing was constructed on top of the existing wall before laying the new foundation block. The existing basement floor was fractured and the basement area was filled with compacted free-draining material to the elevation of the exterior grade. Interior supporting masonry or steel pipe columns, when required, were founded on unfractured portions of the existing basement floor or on new footings and extended to the required design height (see Figure VII-1.3).

An integral part of the solid wall foundation design was the equalization of hydrostatic water pressures between the interior enclosure and the exterior flood heights. With the exception of one structure (see Case #8) the entire Tug Fork Valley retrofitting program was based upon elevation with flooding below the first floor.

In the case of the solid masonry wall foundation system, openings to allow filling and drainage of the enclosed area were designed based upon FEMA criteria (one square inch of free opening per one square foot of enclosed floor space). The design used on 88 percent of the structures elevated on masonry wall foundations was a 2 x 2 foot square galvanized sheet metal louver, providing 50-percent free opening with alternating louvers for both filling and drainage of the enclosure.

Louvers were placed within eight inches of the interior grade and at least two louvers were used in each enclosure, regardless of the enclosed square footage. Owners were allowed to press-fit one-inch thickness styrofoam panels into the louvered opening from the interior to reduce cold air penetration into the enclosed area beneath the first raised floor (see Figure VII-1.4). In the event of flooding, these panels would dislodge at low water pressure and permit hydraulic equalization to occur.

In the case of the other foundation designs (wood post/beam and masonry pier) the area beneath the first floor was not entirely enclosed or was enclosed with wood lattice, allowing free passage of floodwater both into and out of the space without louvers.

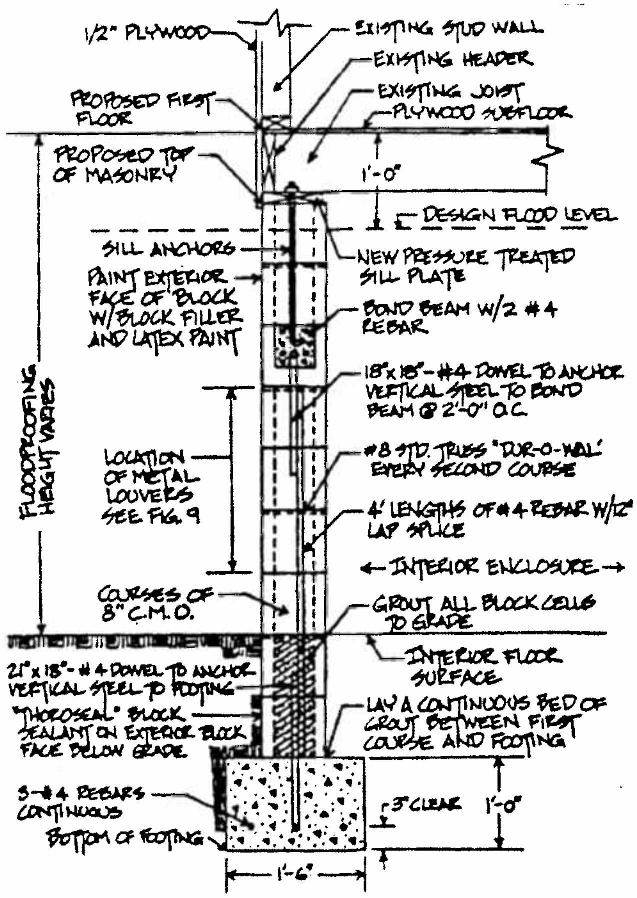


Figure VII-1.2: Typical Wall Detail Section

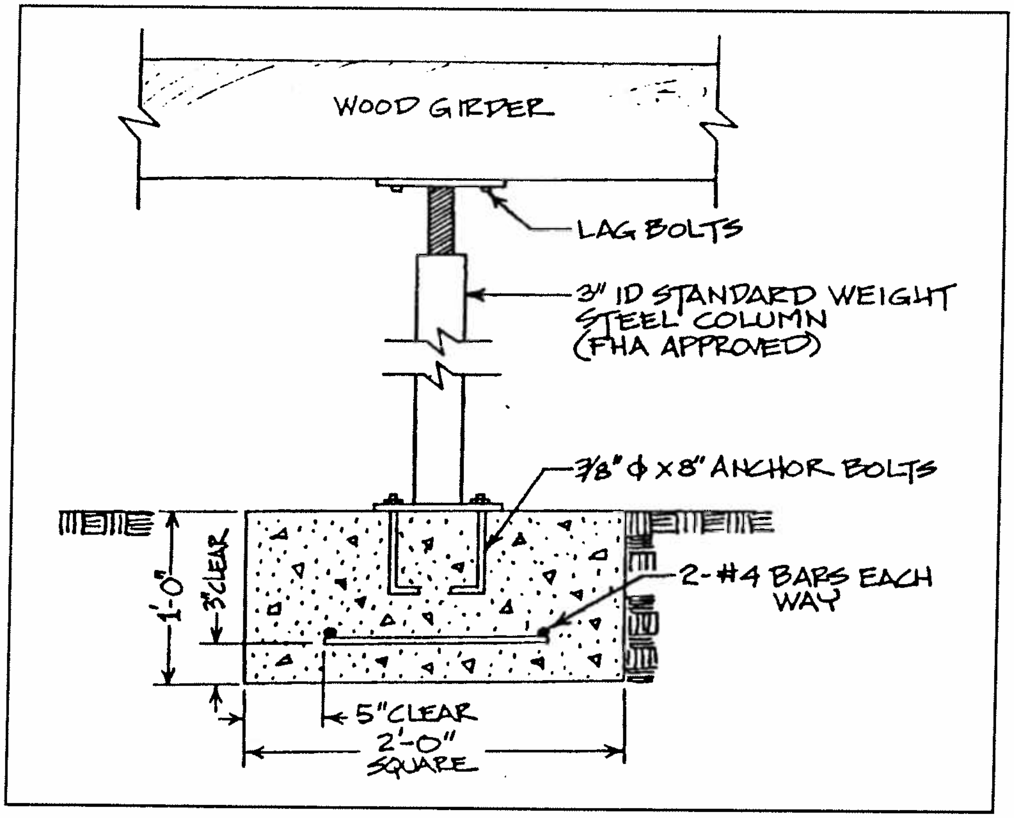


Figure VII-1.3: Interior Column Detail

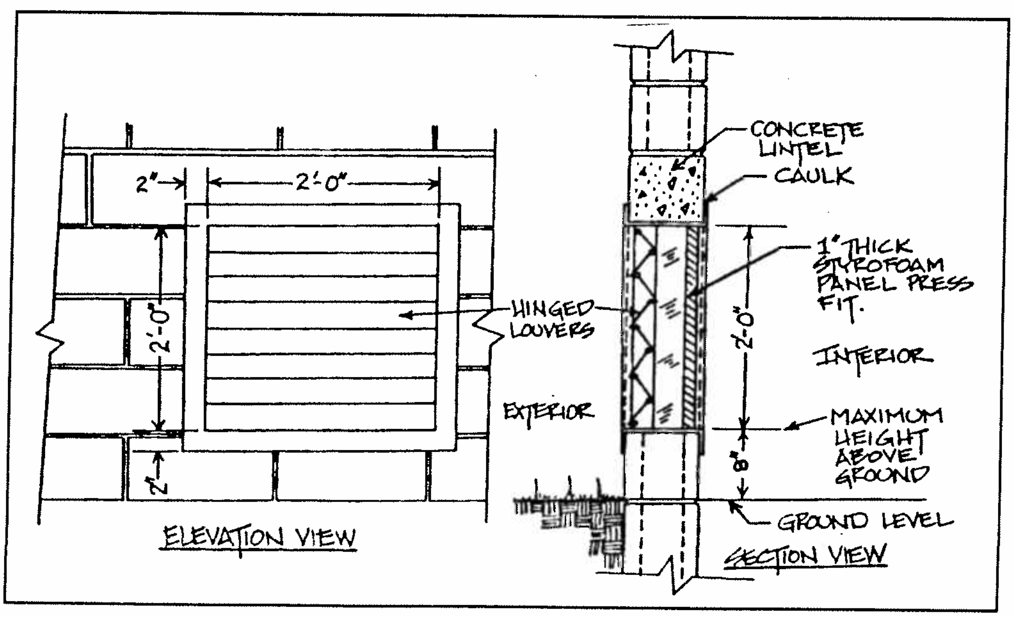


Figure VII-1.4: Flood Louver Detail

Elevation Using a Masonry Pier Foundation

One residence in the Tug Fork Valley program was raised approximately 11 feet on masonry piers. A steel frame structure was designed to span the masonry piers and support the existing floor system, which was in poor condition from past flooding damages (see Figure VII-1.5). All of the masonry piers were individually designed to fit the structure and the expected hydrostatic and hydrodynamic loading at the site.

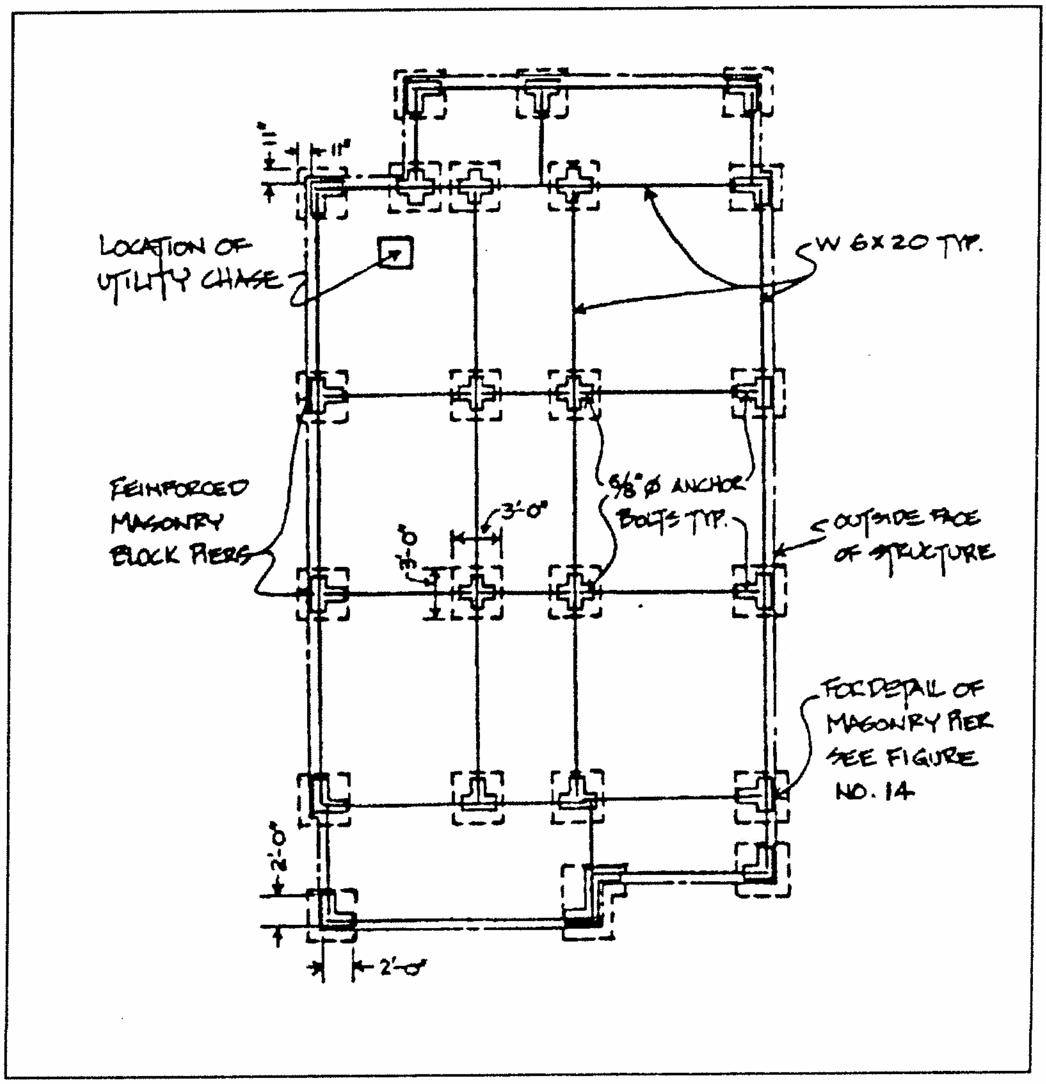


Figure VII-1.5: Masonry Pier Plan

The piers were constructed of 8 x 8 x 16 inch concrete masonry block founded on concrete footings. All cells of the block pier were grouted solid with concrete. Vertical steel reinforcing was placed in all piers with ladder-style masonry joint reinforcing in alternating horizontal joints. Number five reinforcing steel bars were used for footings and vertical reinforcing as shown in Figure VII-1.6.

Utilities were collected into a single insulated pipe chase constructed to resist flood damages (see Figures VII-1.7 and VII-1.8). The structure floor was fully insulated to reduce the increased heating demands caused by unimpeded air flow beneath the structure. The perimeter of the masonry pier foundation was clad with treated wood planking and wood lattice to reduce the visual impacts of this design.

Two additional factors that require consideration in the elevation process are weather and safety. Weather-related problems were solved, in part, by installing plastic skirting around the bottom of the raised structure. Once the plastic skirting was installed, the area beneath the structure was protected from precipitation and could be heated to a temperature that protected utilities and allowed concrete and mortar work to proceed.

Safety was most important during the construction activities of the retrofitting program. Contractors, inspectors, Corps of Engineers personnel, and the staff of the state housing agencies were informed of the inherent construction dangers. Standard precautions regarding the use of personal safety equipment (helmets, safety footwear, eye and ear protection, etc.), the use and storage of potentially hazardous solvents and fluids, fire protection, use of heavy equipment and power tools, and control of the job site perimeter were discussed frequently with contractors. The safety efforts resulted in the successful retrofitting of 136 structures without a single serious injury or fatality.

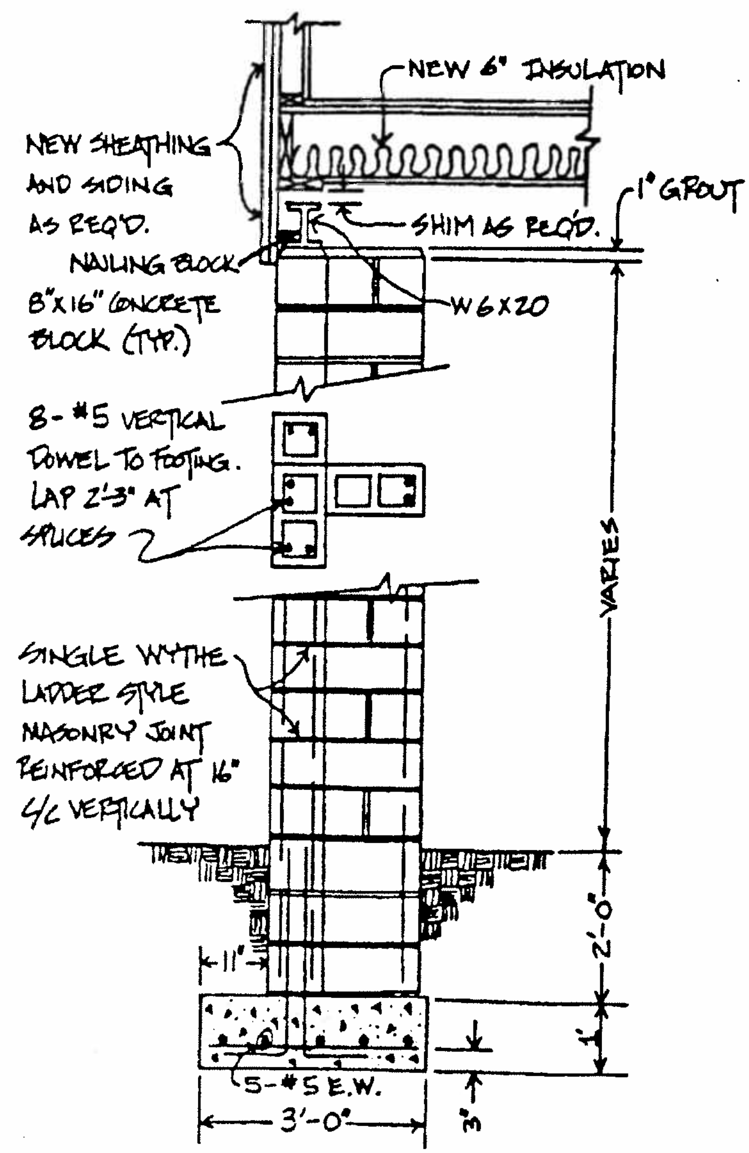


Figure VII-1.6: Masonry Pier Detail Section

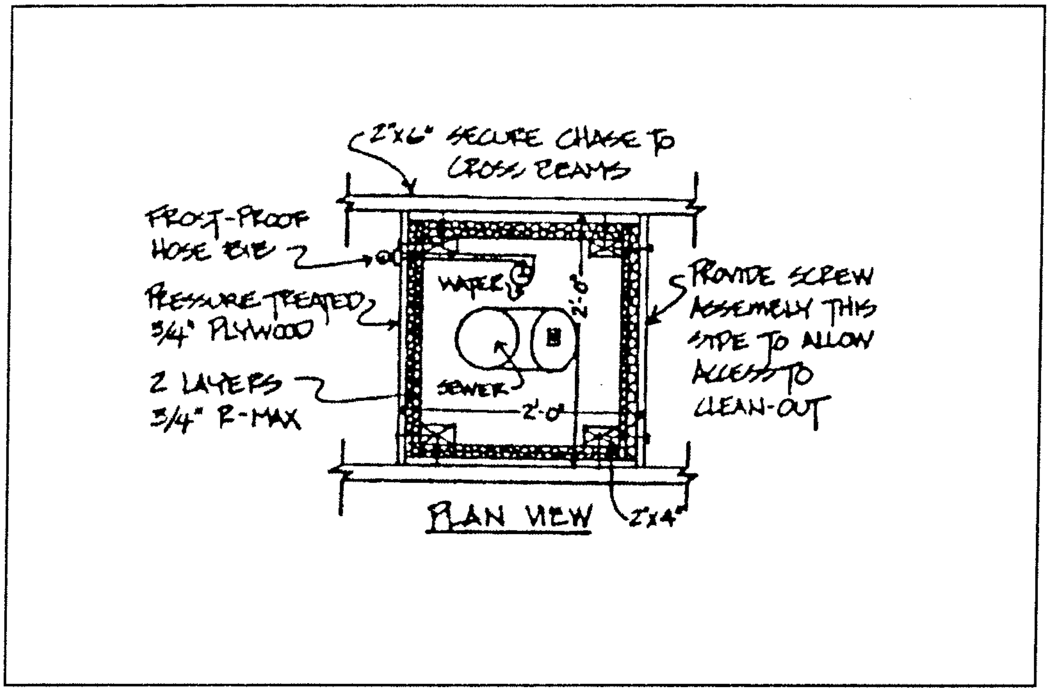


Figure VII-1.7: Insulated Utility Pipe Chase Detail

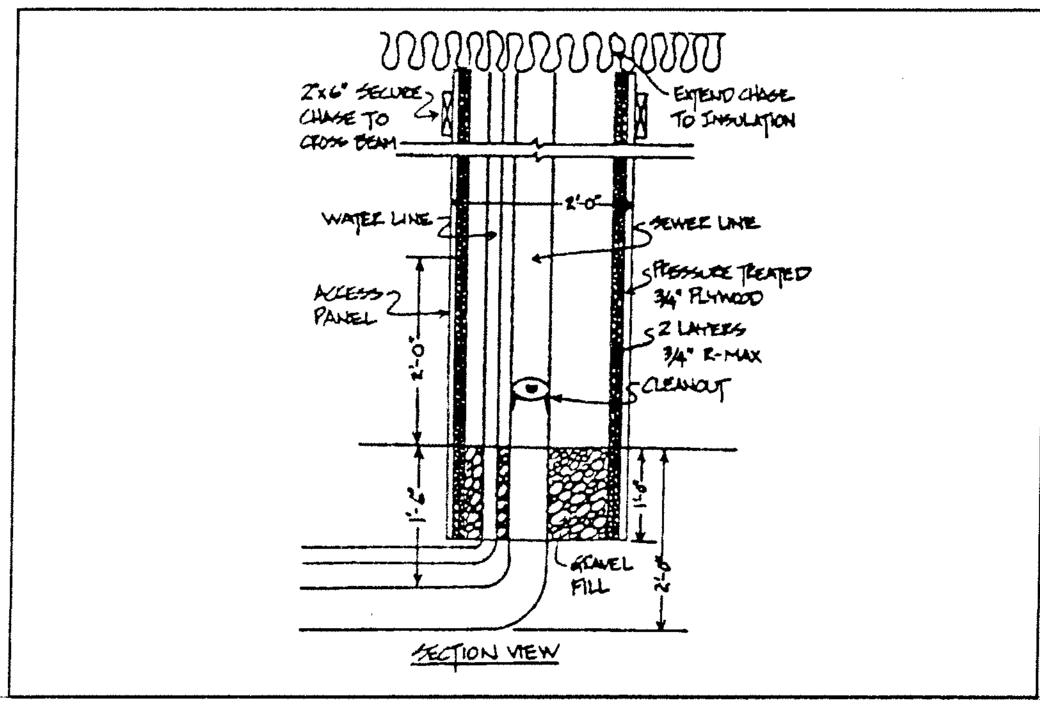


Figure VII-1.8: Pipe Chase Detail Section

Elevation Using a Wood Post and Beam Foundation

Two frame residences were raised using this design. The basic design uses eight-inch diameter round or square pressure-treated wood posts founded at least four feet deep with a continuous six-inch concrete encasement below grade. Spacing of posts is dependent upon structure size and configuration, size and number of supporting beams required, soil bearing capacity, and legal uses of the area below the raised first floor.

The superstructure consisted of pressure-treated wood beams positioned to support the main bearing walls of the structure. Pressure-treated wood sill plates were placed between the post/beam framework and the structure's floor system. The beams were connected to the notched posts using galvanized bolts, washers, and nuts. Additional lateral and horizontal wood bracing was added to resist lateral wind and floodwater loading. Figure VII-1.9 shows the basic design elements of the wood post/beam foundation.

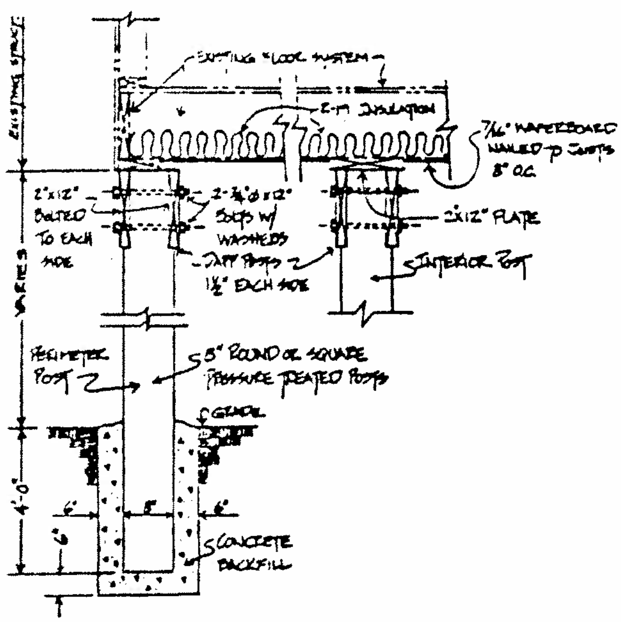


Figure VII-1.9: Wood Post/Beam Detail Section

Materials used for aesthetic treatment were resistant to water damage and did not impede high water flows. Rather than using breakaway walls that may require replacement after a flood event, the panels were hinged at the top to swing in the direction of the flood flow, thus reducing hydrodynamic loading on the foundation, reducing the obstruction of floodwater, and reducing operation and maintenance cost for the owner.

STRUCTURE LIFTING PROCESS

One of the most important and relatively expensive elements in elevating structures is the process of physically lifting the structure to the design elevation. Structure lifting contractors were employed as both subcontractors and prime contractors depending on their management, insurance, and financial capabilities in the retrofitting program.

Several elements contributed to the successful elevation of structures in the program. First, each lifting contractor was required to submit for review a lifting plan that described the number and placement of support beams, cribbing supports, and any special support systems for porches or building additions required to raise the structure.

Prior to lifting a structure, a survey was made of the structure interior to locate critical stress points and concentrated weights. Critical areas in residences included bathrooms, kitchens, interior supporting walls, floor slabs, fireplaces, chimneys, and room additions. Each of these areas received special attention in the lifting plan due to the presence of non-flexible wall and floor coverings, which were subject to cracking. Also required in the lifting plan was the proposed hydraulic jacking system, which allowed collective or individual control of hydraulic jacks located within the cribbing supports. As a by-product of the elevation process, the unified hydraulic jacking system determined the weight of the structure, which proved useful in foundation design. Use of the unified hydraulic jacking system facilitated the elevation of most structures in the program to the design flood height in a single work day.

Two additional factors that require consideration in the elevation process are weather and safety. Weather-related problems were solved, in part, by installing plastic skirting around the bottom of the raised structure. Once the plastic skirting was installed, the area beneath the structure was protected from precipitation and could be heated to a temperature that protected utilities and allowed concrete and mortar work to proceed.

CASE STUDY #2
Elevating Homes on Crawlspace, Dry Creek Goodlettsville, Tennessee: 1989-1990

This case study is included because it represents a departure from the traditional way the Corps of Engineers has elevated houses by the standard government process of "plans and specs - advertisement - sealed bid - award - construction," where the homeowner has little or no input, and the contractor's work is directed and inspected by the Corps of Engineers. The goal of the Dry Creek Project was to reduce the Corps of Engineers' involvement and increase homeowner participation. This was accomplished by changing the standard procedure and allowing the homeowners to select their own contractors and direct the work. In very simple terms, the Corps of Engineers said to each homeowner, "We will give you technical assistance; then you get your house raised and we will pay for it."

The project is located about ten miles north of downtown Nashville, Tennessee. Dry Creek is the boundary between the city of Goodlettsville and metropolitan Nashville (see Figure VII-2.1). The purpose of the project was to reduce damages as a result of flooding in the Gateway Subdivision, where 46 homes were within the 100-year floodplain. Nineteen of the homes were eligible for elevation. The project began in March 1989 and was completed in June 1990.

Project Implementation

Project implementation began with an information phase. Each homeowner was given a package explaining the house elevating program in general, the Corps of Engineers' role, and the homeowner's responsibilities. The homeowners were also given information to pass along to prospective contractors.

Scope of Work, Proposals, and Contract

The homeowners were required to obtain at least three proposals from contractors of their choice and submit them to the Corps of Engineers. It was emphasized to the owners that their meetings with the contractors were very important since that would be their opportunity to exchange ideas and recommendations, and to gain familiarity with the contractors. The Corps of Engineers supplied estimating forms for the contractors in the information packages.

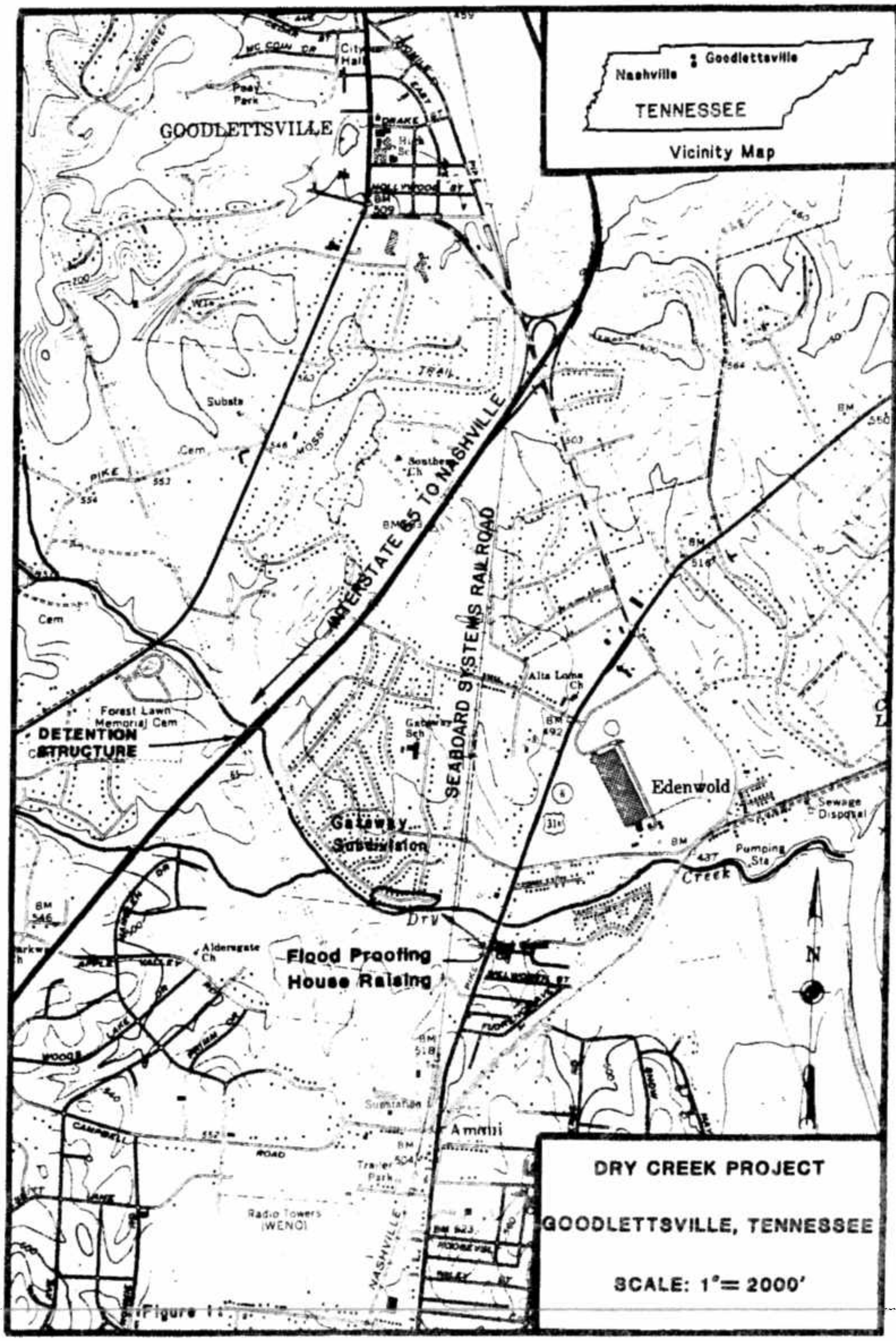


Figure VII-2.1: Dry Creek Project

The Corps of Engineers' project manager and a cost engineering representative measured and inspected each home so that cost estimates could be developed. Following a review of the particular aspects of each home, the project manager and the cost engineer's representative independently developed estimates for each home. Since plans and specifications were not prepared, the Corps of Engineers essentially developed generic "fair and reasonable" estimates for each home. After two Corps of Engineers estimates were prepared, a single amount was agreed upon (usually the average of the two), and that value became the government cost estimate.

Before the offer to the homeowner was finalized, the Corps of Engineers reviewed the contractor's proposal to verify (as much as possible) the assumed scope of work. On occasion, the government estimate was adjusted after review of the proposals. After the government estimate was finalized, a Memorandum of Record was prepared to document the costing process. The Corps of Engineers' "offer" included construction costs and a $200 legal allowance to the homeowner.

The next step was the homeowner's negotiation of a contract with the selected contractor. Without exception, the Corps of Engineers' offer was less than the lowest contractor proposal, but all the homeowners were able to negotiate an agreement within the Corps of Engineers' allowance. After the Homeowner-Contractor contract was executed, it was forwarded to the Corps of Engineers for review. The review was to insure that the fundamental requirements were covered, and other major items of work were agreed upon, such as the size of porches and decks, sidewalks, driveways, landscaping, etc.

The last step prior to construction was the execution of the Corps-Homeowner Agreement. It was very simple, with only four Corps requirements:

- the house must be raised at least one foot above the 100-year flood elevation as specified by the Corps of Engineers;
- the construction must pass the codes inspection by the City of Goodlettsville;
- a provision for flow through the foundation was required to eliminate hydrostatic pressure; and
- the homeowner must execute a covenant provided by the Corps and later recorded at the courthouse stating that the space below the new first floor would never be converted into living space. The space could be used for parking, building access, and storage only.

Construction

All the homes in the program were one-story brick veneer, in sound structural condition. The homes were approximately 1,000 to 1,475 square feet, and the required elevation heights ranged from two to six feet. All homes had crawlspaces under the main portion of the structure. Several residences had finished garages on slabs about 1.5 feet lower than the first floor; the slabs were not raised.

Costs

The cost of elevating the 19 homes in place ranged from $25,900 to $35,350 each, including government administrative cost. Table VII-2.1 below identifies the cost of retrofitting each structure. The major variables that influenced the costs were the number of entrances/exits, height of the elevation, foundation perimeter, size of existing porches, offsets, and finished garages. Administrative costs of about $4,000 per structure were incurred.

Table VII-2.1 **DRY CREEK FLOOD PROOFING PROJECT SUMMARY***

SIZE of HOUSE (sq. ft.)	RAISE HEIGHT (ft.)	CONST. COST**, ***	COMMENTS
1000	5.33	$26,200	3 exits
1000	6.00	$29,500	3 exits
1000	5.33	$29,500	3 exits
1000	4.67	$29,500	3 exits, A/C
1420	4.67	$35,000	3 exits, finished garage, offset
1450	4.00	$35,350	2 exits, A/C, fin. garage, offset, paved drive, big porch
1430	3.33	$34,050	2 exits, fin. garage, offset, fireplace, paved drive, 2 big porches
1475	4.00	$33,000	3 exits, offset
1425	3.33	$32,600	2 exits, garage, offset, paved drive,alum.siding, big front porch
1425	2.67	$31,000	2 exits, garage, offset, big front porch
1450	2.00	$30,800	2 exits, finished garage, large attached carport
1065	4.67	$29,700	2 exits, offset
1275	2.00	$30,200	2 exits, finished utility room (on slab), A/C, partial stone face
1450	2.00	$31,800	2 exits, finished garage w/ false ceiling, C/L fence
1400	2.00	$31,800	2 exits, finished garage w/ false ceiling, A/C
1450	2.00	$28,500	front porch, garage (rehang 2 doors & window, interior steps)
1014	2.00	$25,900	2 exits, paved driveway
1000	2.00	$27,200	2 exits, attached utility room,wood fence, concrete patio
1450	2.00	$31,600	2 exits, finished garage w/ false ceiling, large front porch

* Brick veneer houses in sound structural condition with crawl spaces.

** Includes $4,000 per structure for Corps of Engineers' administrative costs.

*** 1989-1990 prices.

The steps listed below were typical.

- Building permit and electrical and plumbing permits were obtained.
- A pre-construction inspection and inventory was conducted by some contractors and homeowners at the Corps of Engineers' suggestion.
- Site work in advance of the elevation took from three to five days. This included brick removal and disposal, dismantling fences and moving shrubbery to allow access for the mobile equipment, knocking holes in the foundation walls, cutting garage slabs to allow placement of the house lifting beams, and other miscellaneous activities.
- On the day of the actual house elevating, water and sanitary drainage lines were disconnected and the owners vacated the home.
- The elevation was usually accomplished with synchronized hydraulic jacking systems and timber cribbing. This activity took about one to two hours per vertical foot.
- Temporary utility reconnections were made and temporary steps were built.
- The remainder of the work can be characterized as "routine" home construction activities. The time involved for the construction varied greatly, from two weeks to three months. Factors impacting the time included the weather, capability of the contractor, and availability of subcontractors.

Inspection, Approval, and Payment

Because the contractor worked directly for the homeowner, the Corps of Engineers did not direct the work. The only formal "inspection" by the Corps of Engineers was to certify that the terms of the Corps-Homeowner agreement were met prior to payment. The Goodlettsville Building Code Department provided the "quality control" for the construction (along with the homeowners). Payment was made by check and was issued jointly to the homeowner and the contractor for the amount specified in the Corps-Homeowner agreement.

Using Dry Creek as an Estimating Tool

As discussed earlier, the homes on Dry Creek were structurally sound, brick veneer, one-story homes with crawlspaces. The homes ranged from 1,000 to 1,475 square feet. Building materials and skilled labor were readily available, and there was a competitive environment within the local contractor community. This does not mean that the Dry Creek costs are not representative; it means that extracting cost data from this project for use elsewhere should be done with caution and with an understanding of the applicability of such cost data.

A number of factors influence the cost of retrofitting a home; some include: size of structure, height of raise, condition of the home, number of entrances, size of porches, fireplaces, type of construction (brick veneer vs. frame), access, additions or offsets, and others. For homes in fair condition or better (no serious structural deficiencies), the dominant factors are usually the size of the home and the raise height. After the Dry Creek retrofitting project was completed, the cost data was evaluated to see if any relationships could be derived that might be used as a planning-level estimating tool. An equation was developed that computes the Dry Creek house-raising costs. The variables in the equation are size of structure and raise height, and the equation takes the form:

FORMULA

$$\textbf{Computed cost} = \mathbf{K} + (\mathbf{K_s})(\textbf{size}) + (\mathbf{K_h})(\textbf{raise height}) = \$\_\_\_\_\_\_$$

Where:	**K**	is 11,360;
	$\mathbf{K_s}$	is 12.6/square feet;
	$\mathbf{K_h}$	is 970/raise height;
	size	is square feet of the ground floor, including attached garage; and
	raise height	is raise height in feet.

Formula VII-2.1: Dry Creek House Raising Costs

The following Cost Analysis Table (Table VII-2.2) shows the actual cost, the computed cost using this formula, and the percent difference for each house raised in the Dry Creek Retrofitting Project.

The above equation should give reasonable planning-level estimates for screening alternatives. Anyone using the equation or its results should recognize the limitations of this method. The equation should not be applied to situations that are drastically different from those at Dry Creek. Specifically, the equation should not be used on homes in poor (unsound) condition or homes on slab.

Table VII-2.2 **COST ANALYSIS TABLE**

STRUCTURE NUMBER	SIZE (square feet)	RAISE HEIGHT (feet)	ACTUAL COST*	COMPUTED COST**	PERCENT DIFFERENCE (Computed vs. Actual)
1	1000	5.33	$26,200	$29,130	+10
2	1000	6.00	$29,500	$29,780	+ 1
3	1000	5.33	$29,500	$29,130	- 1
4	1000	4.67	$29,500	$28,490	- 4
5	1420	4.67	$35,000	$33,782	- 4
6	1450	4.00	$35,350	$33,510	- 5
7	1430	3.33	$34,050	$32,608	- 4
8	1475	4.00	$33,000	$33,825	+ 2
9	1425	3.33	$32,600	$32,545	0
10	1425	2.67	$31,000	$31,905	+ 3
11	1450	2.00	$30,800	$31,570	+ 2
12	1065	4.67	$29,700	$29,309	- 1
13	1275	2.00	$30,200	$29,365	- 3
14	1450	2.00	$31,800	$31,570	- 1
15	1400	2.00	$31,800	$30,940	- 3
16	1450	2.00	$28,500	$31,570	+10
17	1014	2.00	$25,900	$26,076	+ 1
18	1000	2.00	$27,200	$25,900	- 5
19	1450	2.00	$31,600	$31,570	0

* Includes $4,000 per structure for Corps of Engineers' administrative costs

** Computed Cost Where K = 11,360; K_s = 12.6; K_h = 970

EXAMPLE:

House No. 5:

COMPUTED COST = K+(K_s)(size of house in square feet) + (K_h)(raise height in feet)
= **11,360 + (12.6)(size of house) + (970)(raise height)**
= **11,360 + (12.6)(1420) + (970)(4.67)**
= **$33,782**

Conclusions

The Dry Creek retrofitting project was a success. The project objectives were achieved: retrofit the homes in a cost-efficient manner and maximize homeowner satisfaction.

There was nothing unique about retrofitting the homes along Dry Creek; no new construction techniques were developed, and no unusual techniques were used. The uniqueness of the project was the administrative philosophy. This philosophy was to "keep things simple, and stay out of the way as much as possible."

Unless there are special conditions, plans and specifications are not required for elevation projects, and the Corps of Engineers' presence is not necessary to direct and inspect the work. Special conditions can include multi-hazard concerns such as velocity, debris impact, high wind, and/or seismic activity for example. A straightforward agreement was created with the necessary conditions to insure that retrofitting objectives were met. The Corps of Engineers allowed the homeowners to make decisions regarding their homes and work with the contractors of their choice. Cost-efficiency was achieved by limiting the administrative cost throughout the process.

The Homeowners at Dry Creek included factory workers, bankers, single parents, elderly couples, and others. Approximately two years after project completion, the Corps of Engineers sent a questionnaire to each of the 19 homeowners requesting their opinions about the project and how it was administered. Twelve of the homeowners returned the questionnaire. The results indicate that they favored the high level of homeowner involvement that the project provided. The results of the post-project questionnaire are shown in Table VII-2.3

Table VII-2.3 **Post - Project Questionnaire Results**						
	1	2	3	4	5	0
I'm glad I was given the opportunity to choose my own contractor.	9	2	1			
I'm glad I was allowed to direct the work and make decisions concerning the final appearance and function of my house.	9	1				2
I don't feel my responsibilities (soliciting proposals, negotiating with my contractor agreements, etc.) were too much handle.	9	2				1
I think the Corps exercised about the right amount of control over the project.	9	2		1		
I think the overall appearance of my home is at least as good as before my house was raised.	9	2	1			
I think the value of my home increased by having it raised.	9	2		1		
Overall, I consider the house raising project a success.	8	4				
All things considered, I'm glad I had my house raised.	10	2				

KEY					
Strongly Agree				Strongly Disagree	No Response
1	2	3	4	5	0

Customer satisfaction is always important, particularly when something as personal as elevating an individual's home is involved. The best formula is to allow the homeowner as much freedom and flexibility as possible while maintaining control of the "federal interest," cost, and project integrity. The procedures used in the Dry Creek Project should be considered when cost efficiency and customer satisfaction are project objectives. Figures VII-2.2 through VII-2.7 are examples of homes raised during the Dry Creek project.

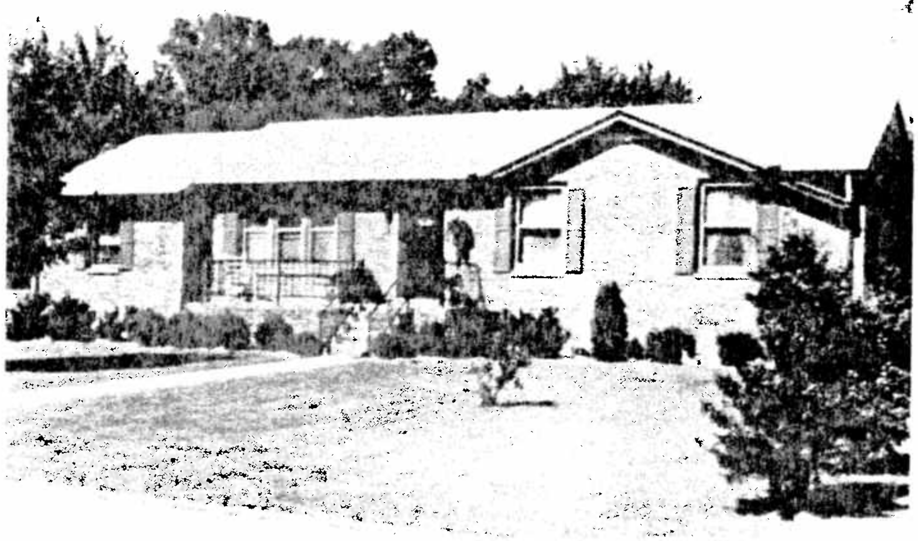


Figure VII-2.2: Typical Home Raised About Two Feet

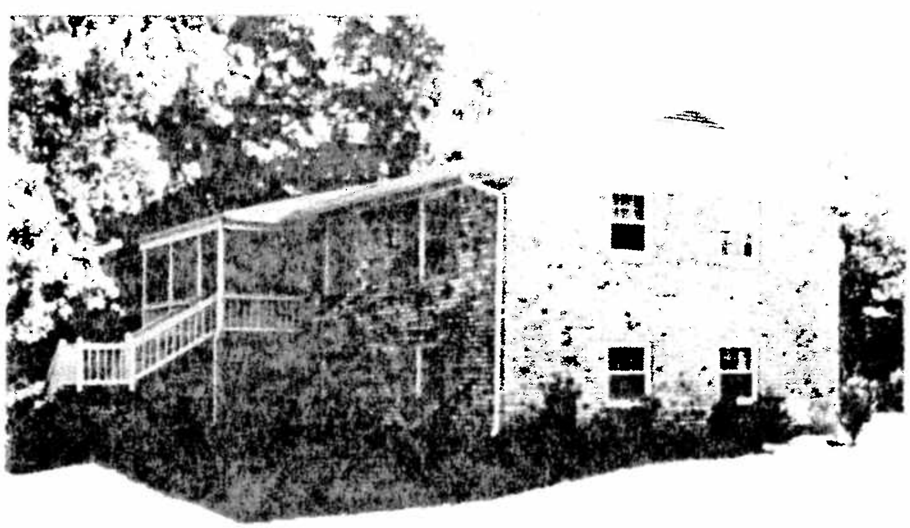


Figure VII-2.3: Typical Home Raised About Five Feet

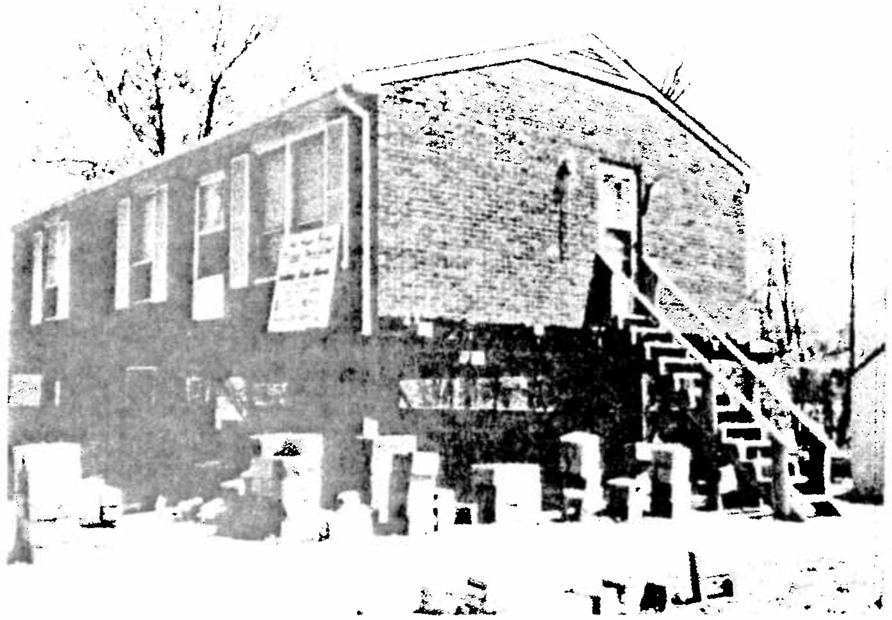


Figure VII-2.4: Example of a Home Raised With the Brick Veneer in Place - During Construction

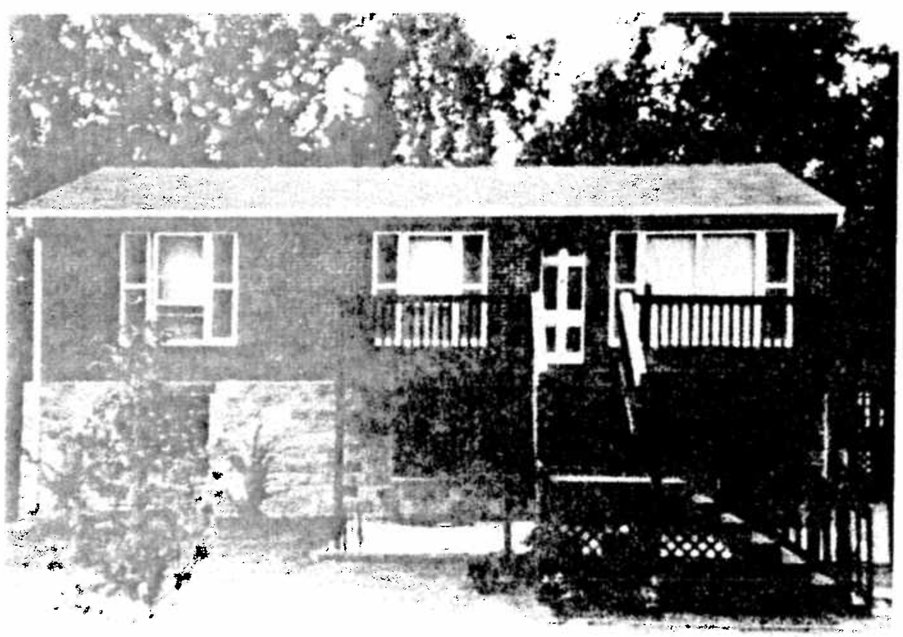


Figure VII-2.5: Example of a Home Raised With the Brick Veneer in Place - Completion

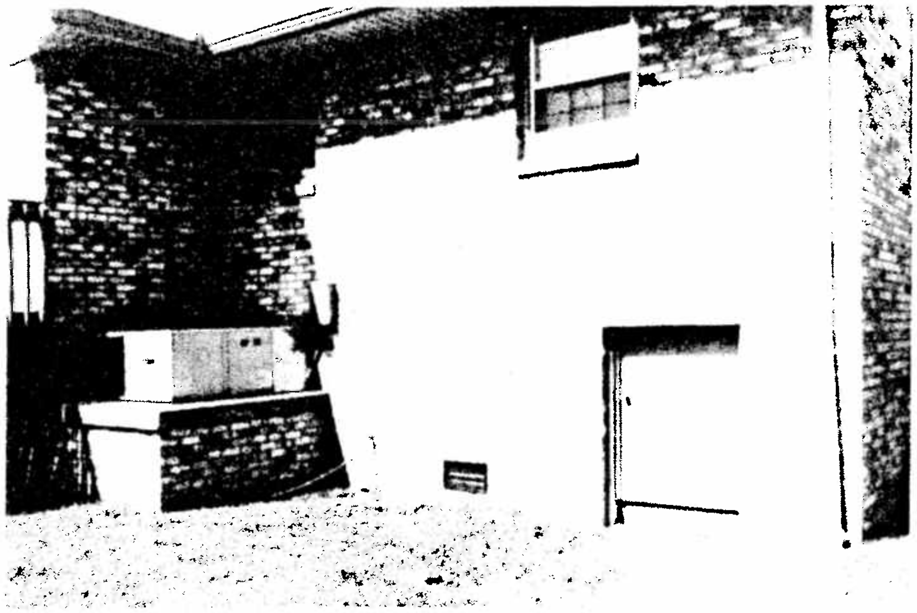


Figure VII-2.6: Provision for equalization of hydrostatic head was accomplished with foundation vents and/or flexible flaps on crawlspace access door.

Figure VII-2.7: Example of a Home Raised With Air Conditioner Compressor Unit on Elevated Platform

HOUSE RELOCATION

This section presents a case study that identifies procedures, methodology, and design parameters used to raise and move a slab-on-grade house with the slab attached.

CASE STUDY #3
Relocating a Slab-On-Grade House With Slab Attached
Tampa, Florida: 1990

Many approaches to flood protection and flood loss reduction have been developed and used with varying degrees of success, including raising existing structures above expected flood levels, or relocating them to flood-free areas. Those approaches are relatively simple for structures originally constructed on piers; however, they are not as well recognized as economically viable practices for structures on concrete slab foundations. In the case of slab foundations, there are two practical possibilities: detaching the structure from the floor slab, or moving the entire structure with the slab attached. The latter practice is not widely known and understood, and is often believed to be infeasible. It is, however, technically feasible, is often economically feasible, and presents many advantages in the hands of an experienced structural mover.

The procedures and techniques described here are based primarily on those employed by a professional structural mover operating in the Tampa, Florida area. Other professionals in the field may employ different but equally effective methods. No undertaking of this magnitude should be attempted without the advice and assistance of professional structural movers and structural engineers or architects.

Keeping the slab attached has a number of advantages over the detached-from-slab approach. In the case of raising the structure in place, or moving it only a short distance so that temporary utility connections can be maintained, a major advantage to the homeowner is the possibility of continued residence in and use of the house during the process. The presence of the floor slab adds greatly to the structural integrity of the building or building segments during the move, and somewhat simplifies the internal shoring and bracing required. The presence of the slab is especially advantageous, if not absolutely essential, for some types of construction, such as concrete block.

Raising and Moving the Structure

A system used extensively in Florida, where construction with concrete block is widely practiced, involves excavating the soil from beneath the structure, inserting a system of two heavy steel longitudinal beams and numerous closely-spaced cross members, and cutting the plumbing connections and any footings or piers encountered. Procedures will vary somewhat from structure to structure, and must be planned on a case-by-case basis. The slab-on-grade is typically designed to be continuously supported by the underlying soil. This demands careful planning for the systematic removal of the soil and for supporting the slab throughout the process as shown in Figure VII-3.1. Special care is required for concentrated loads such as fireplaces and chimneys as indicated in Figure VII-3.2.

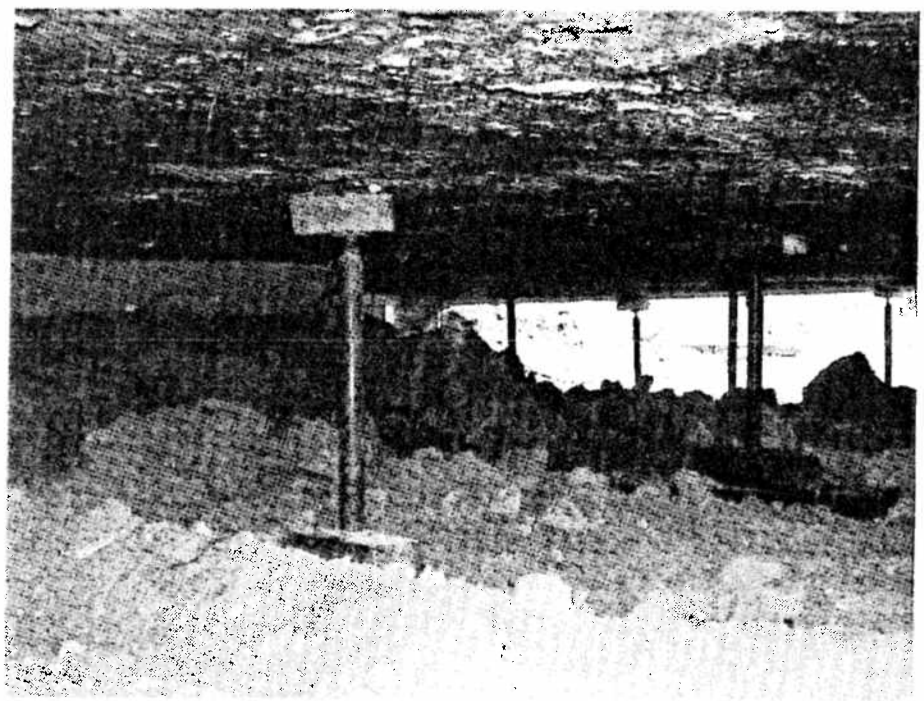


Figure VII-3.1: Temporary Supports for the Slab

Hydraulic jacks are placed at three points beneath the steel beam system, two near one end of the structure beneath each of the main longitudinal beams, and one at the other end of the structure midway between the two longitudinal beams. The lifting points are thus positioned to form an isosceles triangle in the horizontal plane of the slab. The three-point lift minimizes the possibility of cracking of the slab due to twisting or differential movement.

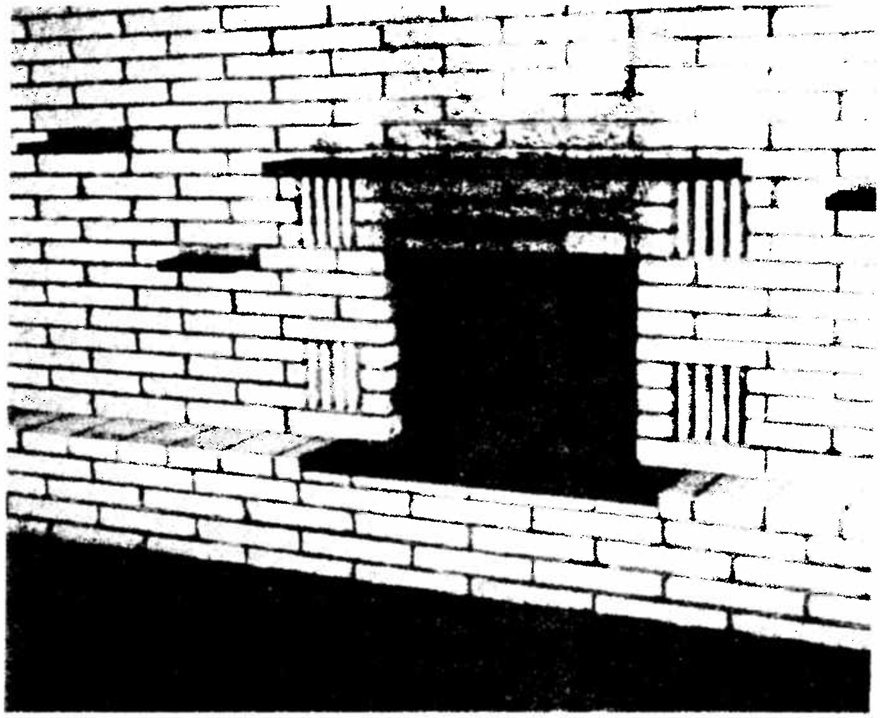


Figure VII-3.2: Fireplaces Require Special Attention

If the structure is to be raised in place without relocation, once it is raised to the desired elevation the jacks are replaced with timber cribbing. If it is to be moved to another location, large wheeled dollies are inserted at the two jacking points under the main beams, and the hauling equipment takes the load at the third jacking point, centered between the main beams. At the new location, the moving equipment is replaced by timber cribbing supporting the structure at the desired elevation, and the new foundation is constructed beneath it. Figure VII-3.3 shows one of the timber cribbing supports placed beneath a main longitudinal beam.

If piers or portions of grade beams must be removed, they are first scored along appropriate cut lines with an air saw equipped with a concrete blade, then broken with a hammer. Figures VII-3.4 and VII-3.5 show where previously existing piers have been cut away. Any reinforcing steel encountered is cut with a torch as shown in Figure VII-3.6.

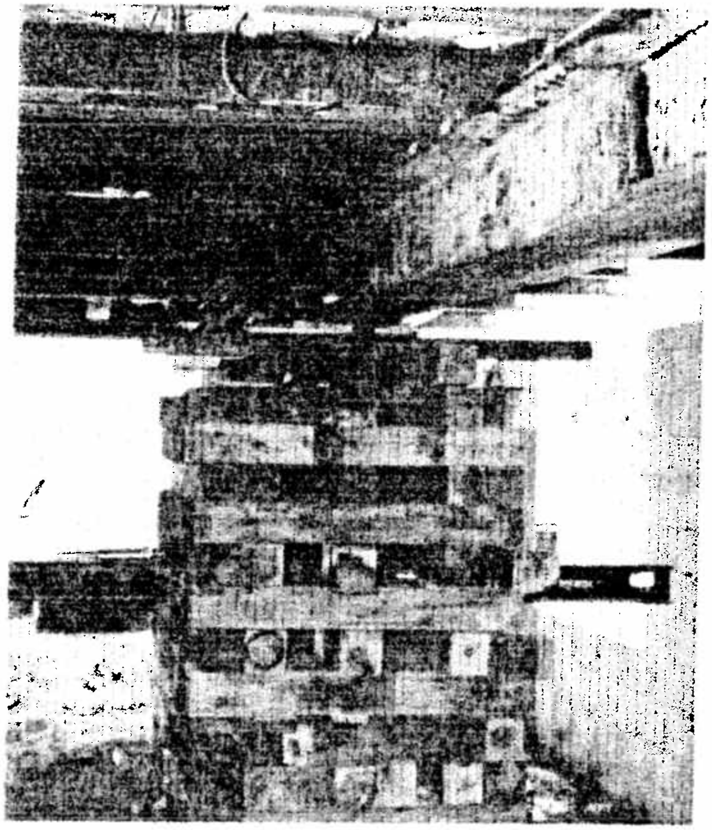


Figure VII-3.3: Timber Cribbing

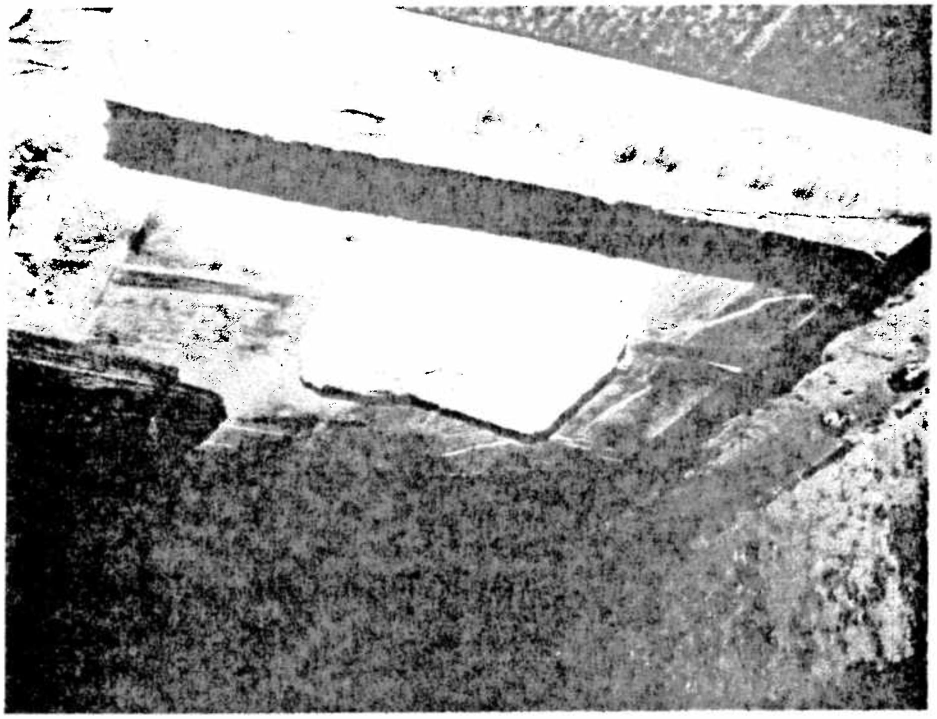


Figure VII-3.4: Piers Cut Away Using Air Saw

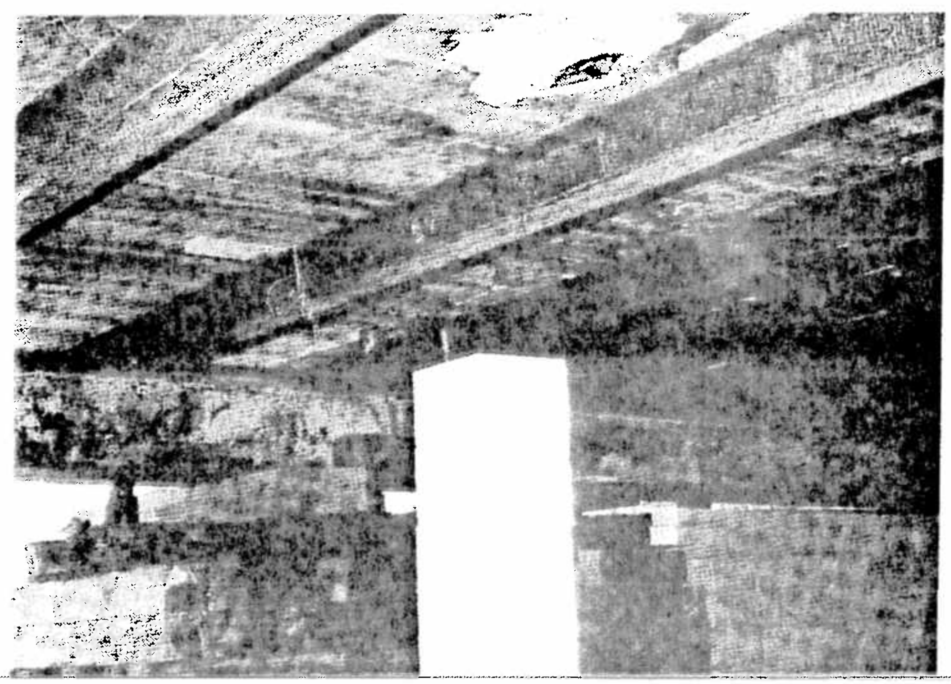


Figure VII-3.5: Piers From Original Foundation

Figure VII-3.6: Cutting Reinforcing Steel

If the structure's size or shape prevents raising or moving it one piece, it can be cut into manageable segments. If the structure is too tall for vertical clearances available along the route, the roof can be partially or completely removed. It is frequently necessary to remove chimneys for this reason. It may also be advantageous to remove the floor from attached garages, many of which are constructed at a slightly lower elevation than the remainder of the house.

Cuts in walls are made between studs in frame construction. In concrete block construction, a whole section of blocks may be removed (see Figures VII-3.7 through VII-3.10) and replaced at the new site, sometimes incorporating a new pilaster at the location of the removed blocks.

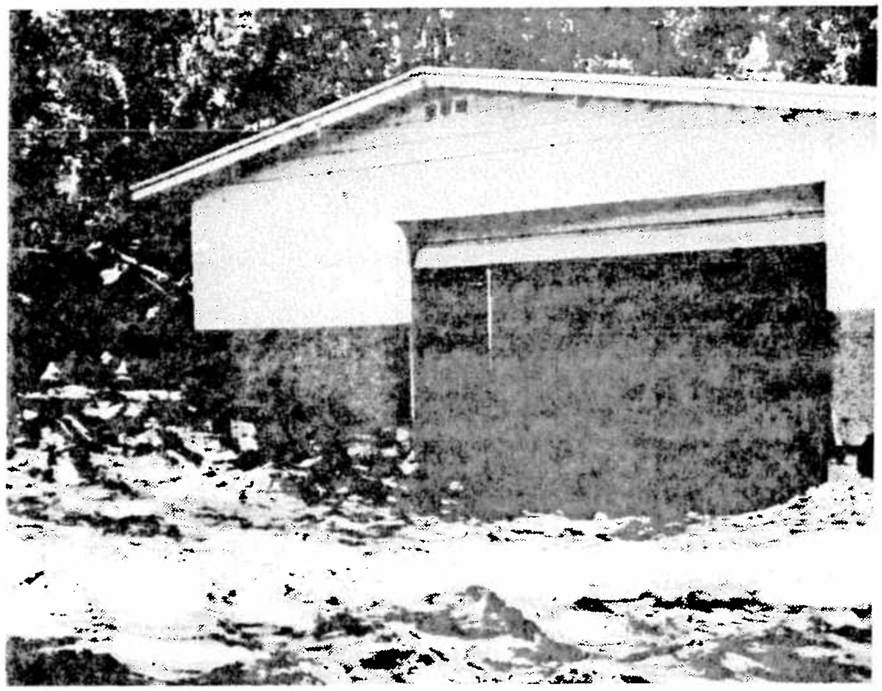


Figure VII-3.7: Garage Floor Slab Removed

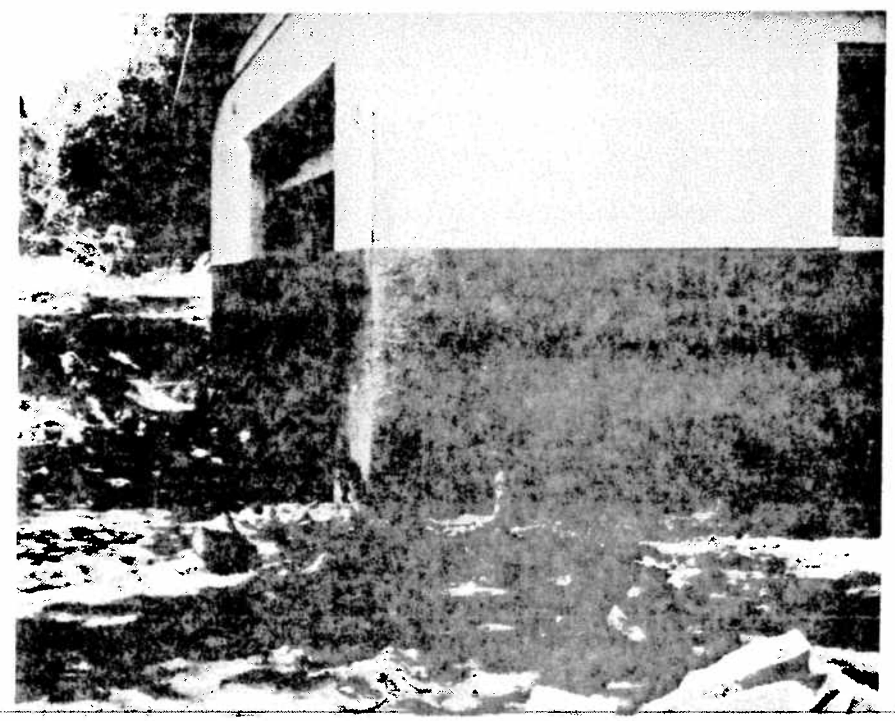


Figure VII-3.8: Holes in Garage Wall to Insert Steel Beams

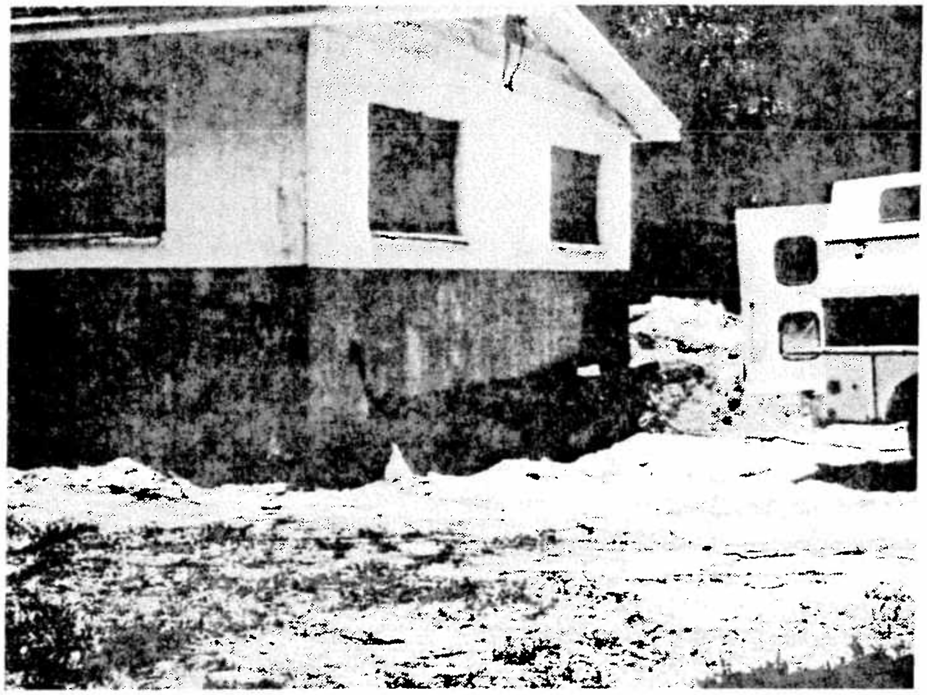


Figure VII-3.9: Excavation Below Slab to Allow Access

Figure VII-3.10: Excavation and Tunneling Completed

Vertical cuts through roofs are usually made between rafters or joists, or immediately alongside a rafter or joist. Reconnections at cuts between rafters are made with 2 x 6 or 2 x 8 timbers laid flat against the underside of the roof. Reconnections of cuts immediately adjacent to a rafter can be made by nailing additional rafters to the old rafter.

Cuts through the slab are made with "street saws" equipped with diamond blades. Usually no attempt is made to reconnect the slab at the new site. The joints will merely be sealed with grout. New foundation piers can be located directly under slab cuts to prevent differential movement of the two edges. Figure VII-3.11 shows a cut through a slab prior to raising the structure.

According to experienced structural movers, about two weeks are required for the average residential structure for initial site preparations, excavation and tunneling, and jacking. This time can be substantially increased by site conditions such as large trees preventing or limiting access by the excavating and earth moving equipment, the need for dewatering, the presence of rock, etc. Construction of the new foundation, reconnecting utilities and air conditioning equipment, architectural adjustments, and final site cleanup and landscaping involve additional time.

Figure VII-3.11: Slab Cut With a Street Saw

Additional time is required if the structure is to be cut into sections, (see Figure VII-3.24) moved to a new location, and reassembled. This additional time is highly variable, depending on the design of the structure involved, distance of the move, and difficulty of the route. Speed of the equipment along the route can be as high as 20 miles per hour under extremely favorable road conditions, but usually ranges between three and eight miles per hour.

Raising in Place

The following steps are generally required, although not necessarily in the sequence presented. The operations listed below assume continued occupation of the home during the process.

- Obtain the necessary building permits and arrange with utility providers for necessary disconnections, reconnections, and inspections. Regulations vary greatly from jurisdiction to jurisdiction.

- Prepare site as required to allow access for necessary equipment. This includes removal and protection of trees and shrubs, removal of fences, etc.

- Excavate around the perimeter of the slab to allow access for subsequent operations. Excavation is carried to an elevation below the base of the perimeter grade beams.

- Excavate and tunnel under the foundation to allow placement of support beams. Excavation and tunneling are accomplished both manually and mechanically. Specialized earthmoving equipment has been developed to facilitate this process. One such piece of equipment, termed a "long nose bucket" or a "snoot" by its developer, is designed for attachment to a front end loader. The "snoot" is pushed under the slab to remove the earthen materials. This equipment and its use are shown in Figures VII-3.12 through VII-3.14.

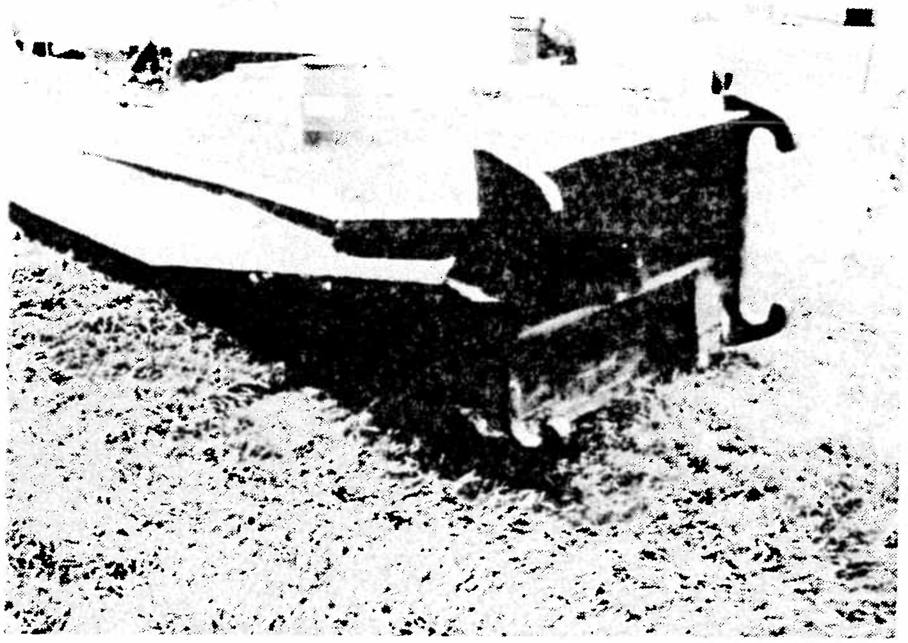


Figure VII-3.12: Long Nosed Shovel Attachment

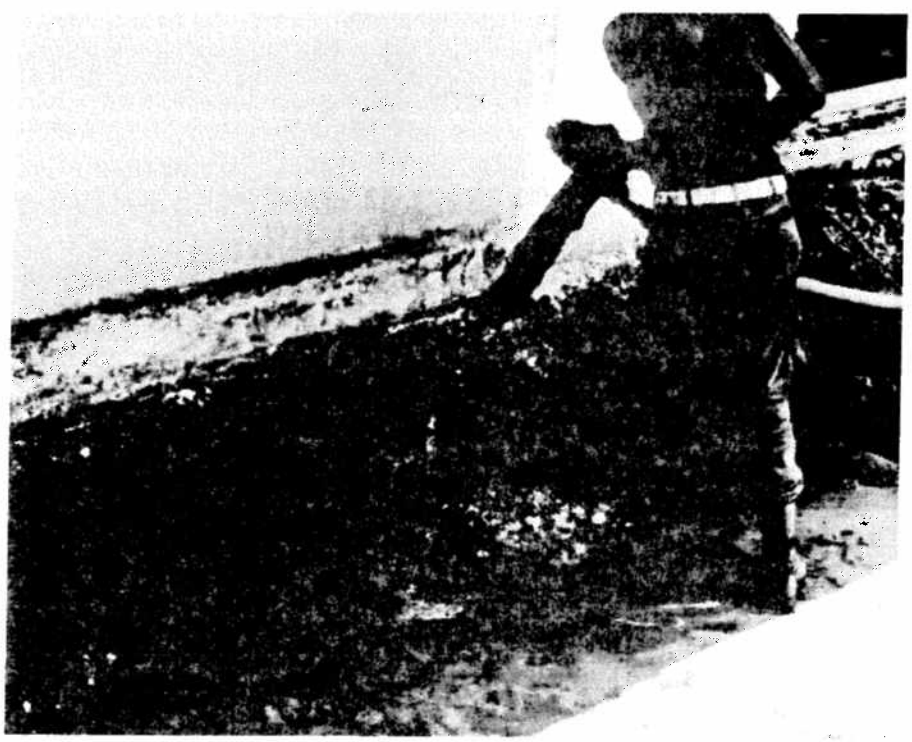


Figure VII-3.13: Perimeter Grade Beam Being Removed

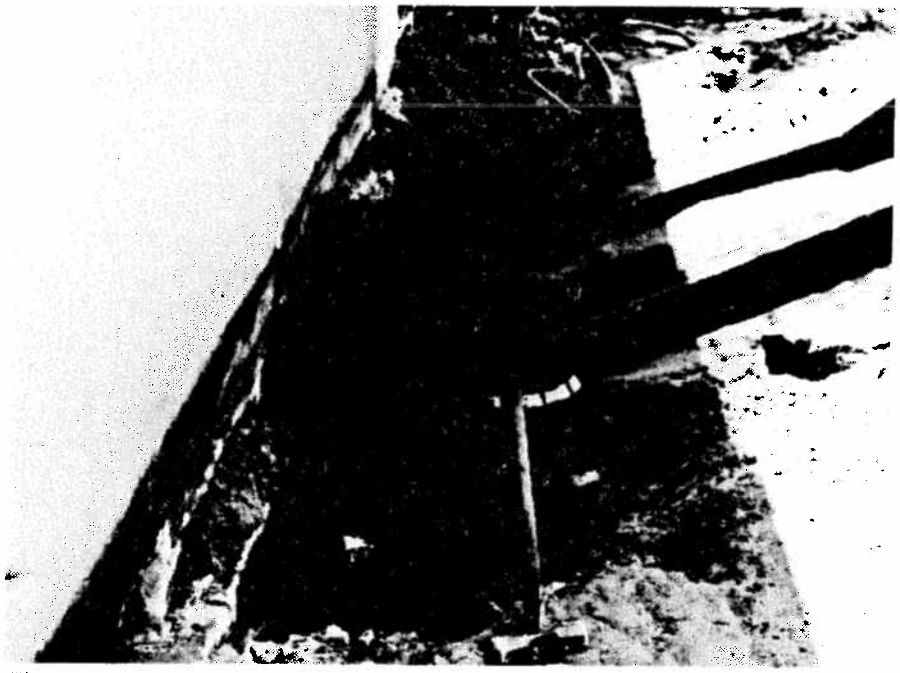


Figure VII-3.14: Snoot Being Used to Tunnel Under Slab

- Provide temporary access facilities to the structure. (Temporary entrance, steps, landings, etc.)

- Provide temporary, flexible utility connections. Water, electricity, telephone, and natural gas are generally above-ground connections and relatively simple. Sanitary sewer connections will generally require excavation, usually in connection with the excavation and tunneling under the slab.

- Detach driveways, sidewalks, porches, and garage, if applicable, or remove the slabs from these areas.

- Remove or secure fragile home furnishings. Most of the contents can remain in the home throughout the raising process.

- Place support beams and jacks. A system of main beams and smaller cross beams is used. The main beams are placed under the structure and positioned on jacks. The cross beams are placed over the main beams and jacked upward until close to the slab, then shimmed against the underside of the slab. Unevenness in the underside of the slab is compensated by the shims and wedges as shown in Figures VII-3.15 and VII-3.16.

- Elevate the structure and support it on temporary cribbing as shown in Figure VII-3.4.

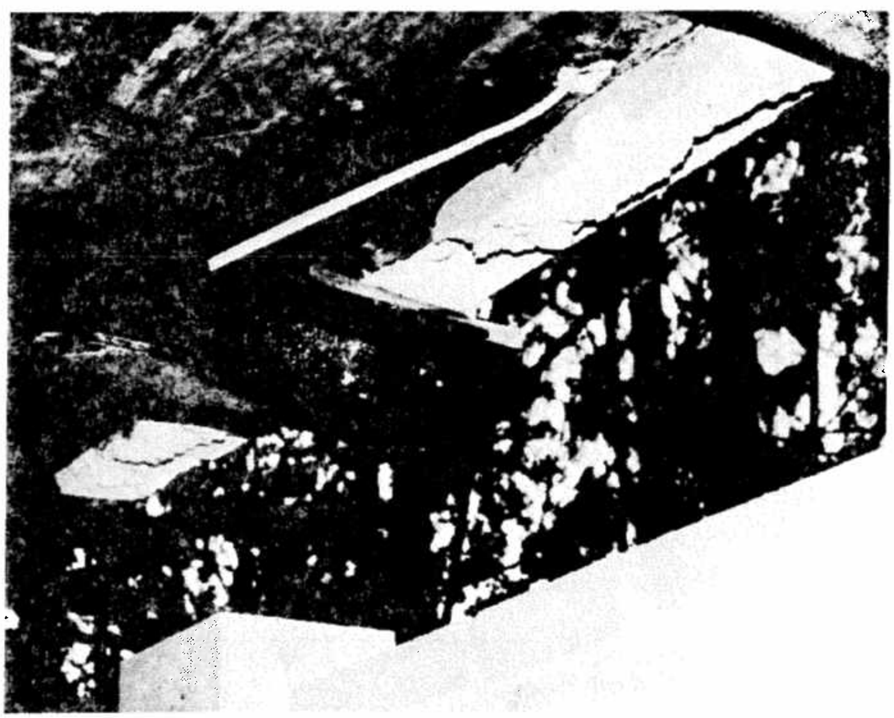
- Construct the new foundation as shown in Figures VII-3.18 through VII-3.21.

- Elevate and reconnect the air conditioning equipment, if any.

- Permanently reconnect the utilities.

- Construct and install architectural and aesthetic adjustments, as required. This will include new entrances and closing in under the elevated floor slab, which must give consideration to floodplain regulations such as a requirement for openings.

- Restore the site, including landscaping.



FigureVII-3.15: Shims Used on Underside of Slab

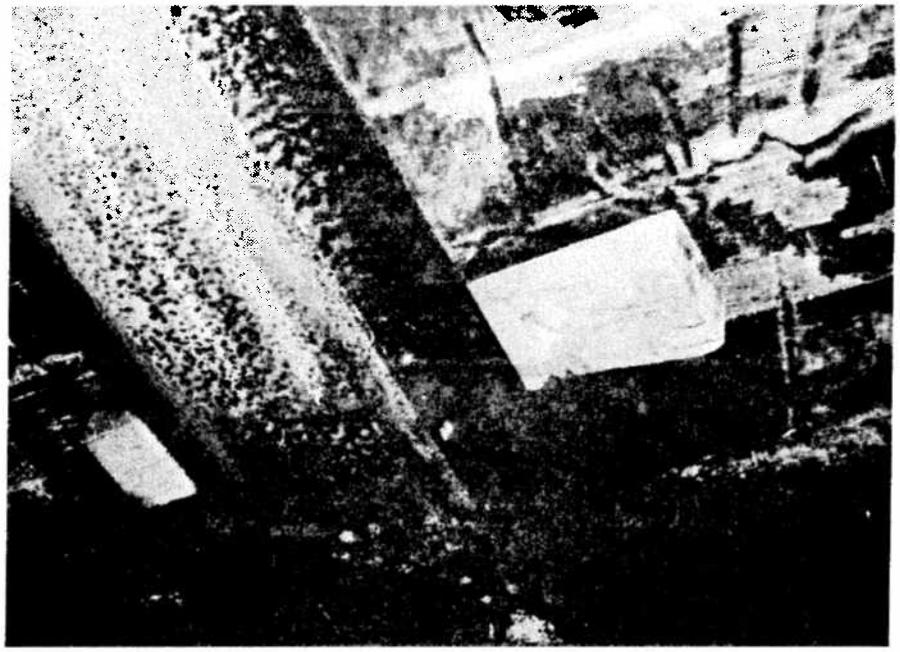


Figure VII-3.16: Wedges Used on Underside of Slab

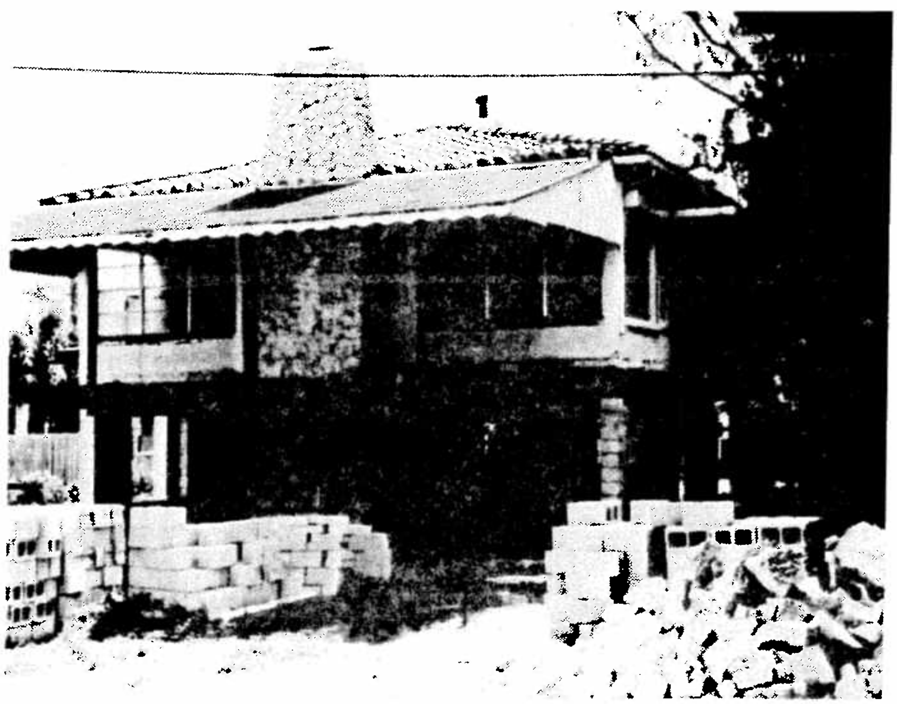


Figure VII-3.17: Relocated Concrete Block Home

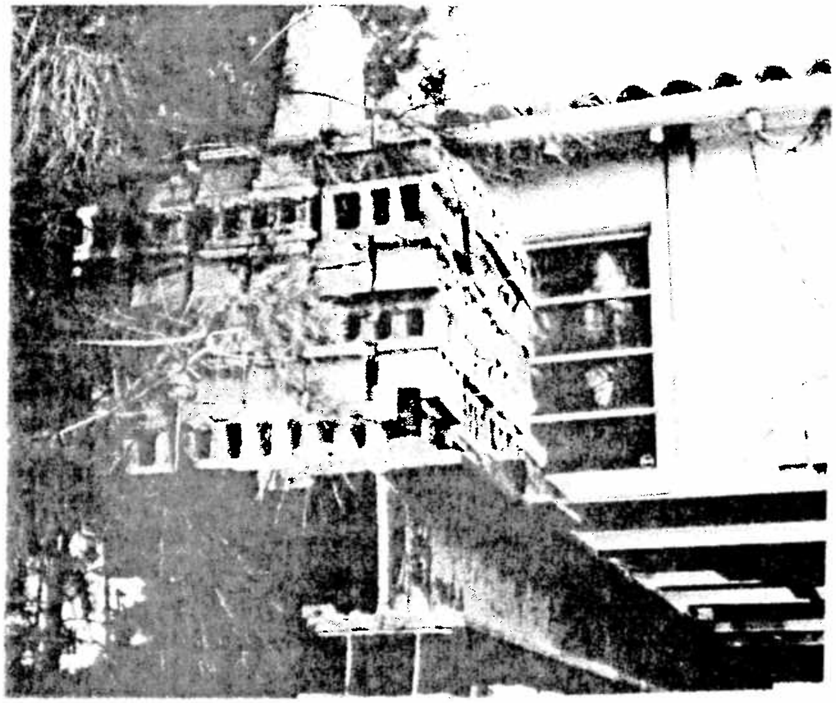


Figure VII-3.18: Concrete Block Counterbalance

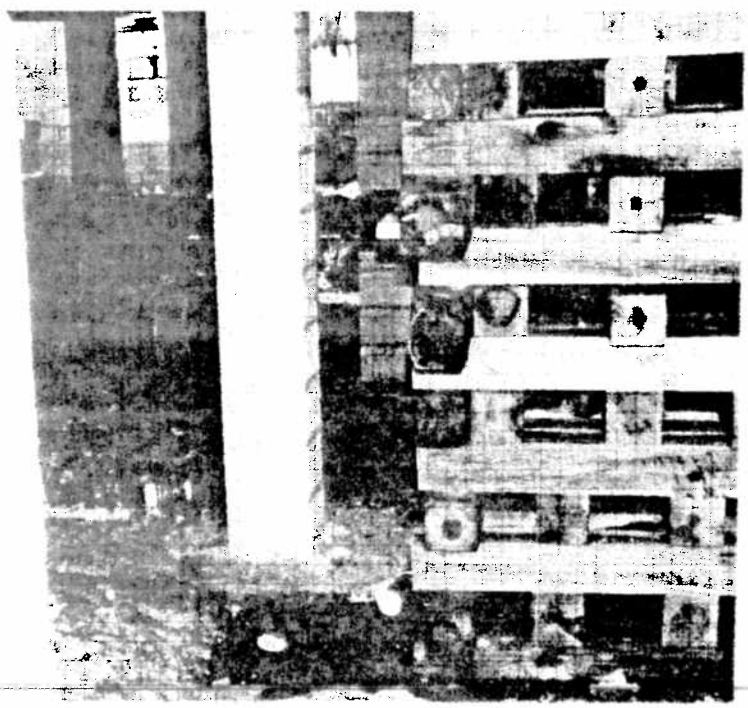


Figure VII-3.19: New Piers and Wood Cribbing

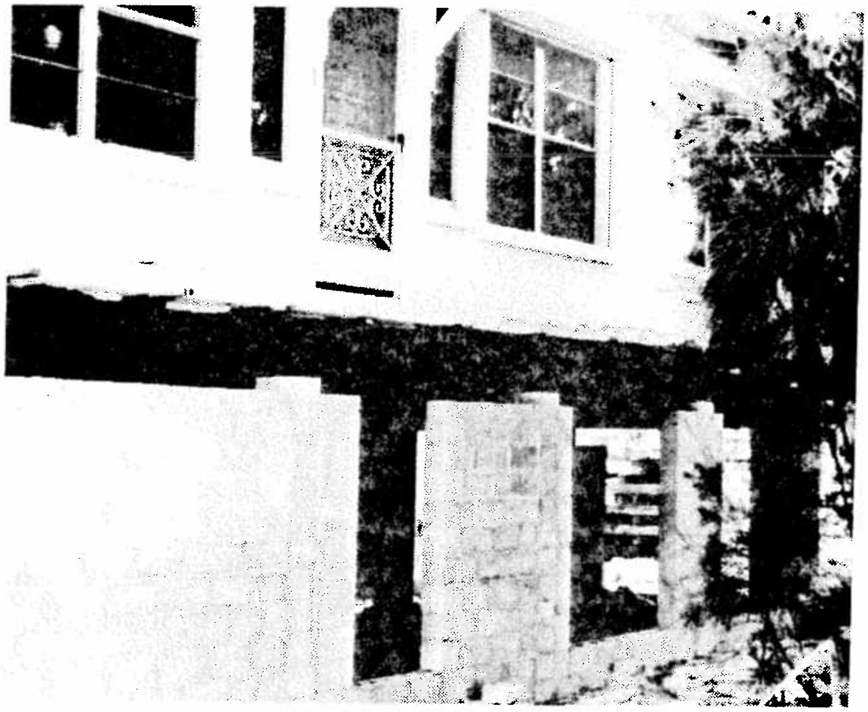


Figure VII-3.20: Exterior Concrete Masonry Block Wall

Figure VII-3.21: Breakaway Exterior Walls

Relocation

Relocating the structure entails all or most of the operations required for elevating in place, plus some additional procedures related to the move to a new location. Temporary utility connections are usually not required as it is generally not possible to continue living in the home during the moving process. If the structure must be moved in sections, most or all of the contents must be removed and stored, possibly including even carpets, plumbing fixtures, water heaters, air conditioning systems, etc. With those exceptions, all of the operations are similar to elevating in place. Additional operations that would be required are listed below.

- Investigate possible routes to the new location and arrange for necessary permits and utility company assistance along the selected route.

- Prepare the new site, including installation of utility service. Timing of utility service construction must be planned to avoid damage from heavy equipment during the house moving process.

- Cut the structure into sections small enough for the route, placing interior shoring and weatherproofing the openings as shown in Figures VII-3.22 and VII-3.23. Vertical clearance limitations may require removal of roof sections. Cut locations must be carefully chosen to minimize damage and maximize internal support. Cuts through hallways can minimize damage to interior walls. Cutting through roofs can be delayed until the final cut to minimize weather damage.

- Place the dollies and hauling equipment at the jacking points of each section, and move them to the new location as shown in Figure VII-3.24.

- Reassemble the structure at the new site.

- Construct new walks and driveways.

Figure VII-3.22: Interior Shoring

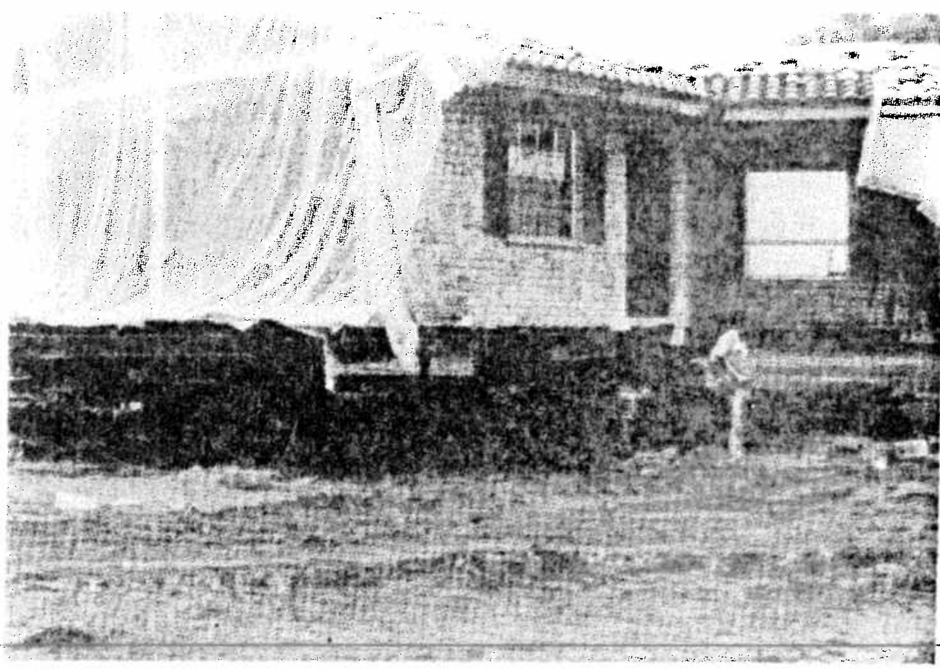
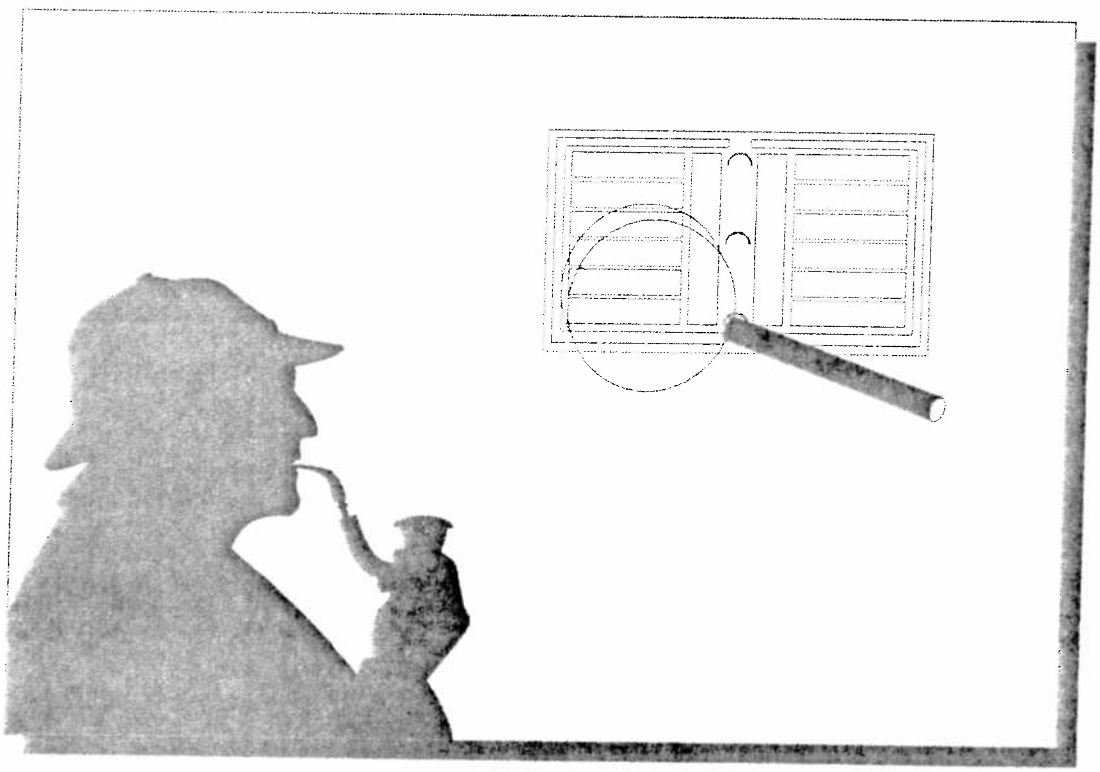


Figure VII-3.23: Plastic Sheeting for Weather Protection

CHAPTER VII (Cont'd)

CASE STUDIES



Featuring:

Elevation
Relocation
Levees and Floodwalls
Wet Floodproofing
Dry Floodproofing

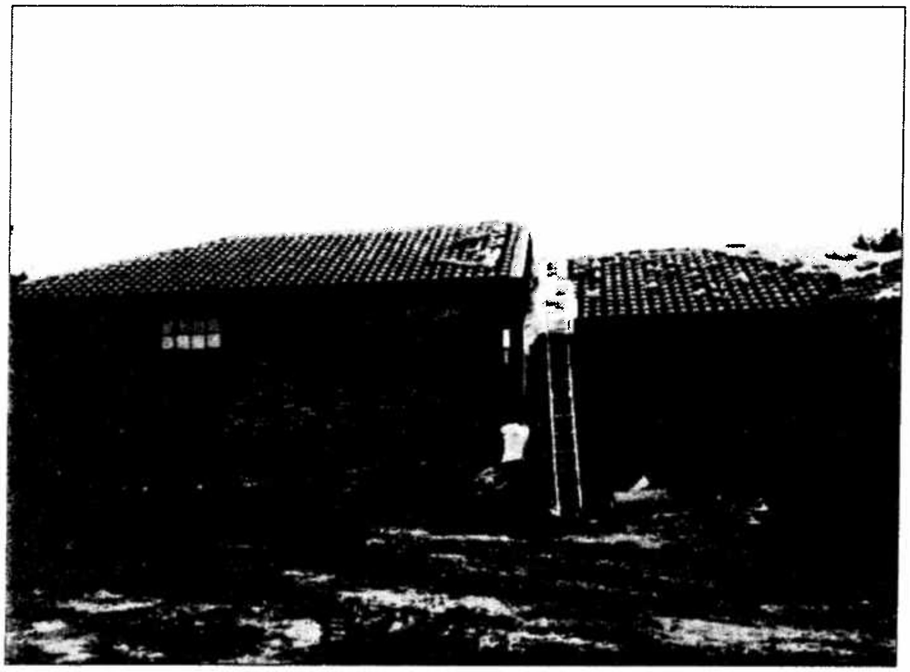


Figure VII-3.24: One Section of the House Has Been Raised, Prior to Insertion of Dollies

Foundation Design Considerations

The system described requires the use of construction materials and methods suitable for the limited vertical clearance provided beneath the raised or relocated structure. Although it might be possible to move an elevated structure onto an already constructed foundation of driven piles, it would undoubtedly be extremely difficult and expensive, as would building a new foundation to fit the under surface of an existing slab prior to moving the slab into place. Attachment of the old slab to a timber pile foundation would also present difficult problems. The usual practice, therefore, is to move the structure to the desired location and elevation, and construct the new foundation beneath it. Reinforced concrete or concrete block are the most commonly used construction materials.

Other than the restrictions on materials dictated by the presence of the structure overhead, foundation design considerations would be no different than for new construction. Although intended primarily for use in coastal high hazard areas, excellent information and recommendations on design of foundations for elevated structures is contained in the *Coastal Construction Manual* published in 1986 by the Federal Emergency Management Agency (FEMA).

The publication suggests a number of foundation types and materials suitable for construction of raised structures in coastal high hazard areas. Several of those suggested would lend themselves to construction in the restricted space beneath a raised or relocated structure. Among them are reinforced concrete or reinforced masonry unit (concrete block) piers on spread footings, or on grade beams under concrete slab; and reinforced concrete or reinforced masonry unit shear walls parallel to the likely direction of flow of floodwaters or waves.

Design of the new foundation should consider wind and wave forces, and the potential for erosion and scour. Also, in coastal high hazard areas, careful attention should be given to the connections between the new foundation and the raised slab.

The design should take into account the fact that the original slab was intended to be continuously supported on the underlying soil. Unsupported spans of floor slab should probably be limited to ten feet or less, and piers should be spaced as required to insure integrity of the slab. Some designers recommend four inches on center.

Elevating the Structure Cost Considerations

Costs include site preparation, excavation and tunneling, removal of unwanted slab areas, utility disconnections (and temporary flexible connections, if required), jacking and leveling, utility reconnections, and site cleanup. Information from structural movers experienced with the process indicates that the basic cost of these procedures would be about $12.00 per square foot of foundation area for a 1,200- to 1,800-square-foot one-story residence. Costs per square foot would increase somewhat for either smaller or larger structures, and for multistory structures. There is a practical lower limit to the time for initial site preparation, excavation, jacking, and mobilization costs, all of which increase the cost per square foot for the smaller structures. Larger structures require more time and labor for the increased volume of material to be excavated. Within limits, up to 10 to 12 feet, the height to which the structure is to be elevated does not significantly affect the cost. Costs are affected, however, by site conditions such as large trees preventing or limiting access by the excavating and earth moving equipment, the need for dewatering, the presence of rock, etc.

Construction of the New Foundation and Attaching the Elevated Structure Cost Considerations

Foundations for elevated residential structures can be constructed by a variety of methods, and with a variety of materials. The costs are dependent on site conditions, materials used, and labor costs, and differ between different regions of the country. Reinforced concrete grade beams ranging in size from 8 x 16 to 24 x 24 inches cost from $7.70 to $27.50 per linear foot. Reinforced concrete masonry unit piers, typically 8 x 16 or 12 x 12 inches, could cost from $2.00 to $14.00 per linear foot including the footing. Reinforced concrete piers 12 x 24 inches could cost from $14.00 to $48.00 per linear foot of elevation.

New or Raised Utilities, and Raised Air-Conditioning Equipment Cost Considerations

Again according to the *FEMA Coastal Construction Manual*, raising the water utility costs $4.00 to $8.80 per foot; the sewer utility costs $6.00 to $16.50 per foot; the gas utility costs $4.00 per foot; and the electrical utility costs $3.00 per foot. Varying permit requirements, protection against freezing, etc., may influence costs for these items in various regions of the U.S.

Architectural Modification Cost Considerations

These include enclosing the area beneath the raised floor slab (with breakaway walls if required), new entranceways, stairs, landings, porches, and patios, new sidewalks, and driveways, etc. Breakaway walls would cost about $0.75 per square foot of lattice work, $1.50 to $2.00 per square foot for stud wall and plywood sheathing, and $2.70 to $3.10 per square foot for block walls. Landscaping and site restoration costs are highly variable.

Cost Estimates

Detailed cost estimates for elevating a hypothetical residential structure in place two feet and ten feet above grade are shown in Tables VII-3.1 and VII-3.2 The estimates assume the structure to be 36 x 36 feet, single story (1,296 square feet), with a detached garage. The foundation is assumed to be typical slab-on-grade with a perimeter grade beam and interior beams beneath bearing walls poured monolithically.

The new foundation consists of 14-inch-square reinforced concrete piers with two-feet-square by eight-inch-deep footings set three feet ten inches below grade. The piers are nine feet on centers both ways, for a total of 25 piers.

The project site is assumed to have no unusual or difficult soil conditions, and to have adequate clearances for equipment and operations. The equipment required for elevating structures is highly specialized and expensive.

Major costs in the procedures described above are involved in mobilization and demobilization of this equipment. Some reductions in the cost per residence can be realized if work on more than one structure can be undertaken within a reasonably limited area and within a limited time. The sample cost estimates assign these mobilization costs to one structure.

The cost estimates reflect 1988 price levels in the Houston-Galveston area of the Texas Gulf coast. These cost estimates were derived from various sources, primarily the *FEMA Coastal Construction Manual*, and *Means Site Work Cost Data 1988*. The estimates are intended only to indicate the general range of costs involved in a slab raising project for comparison with other possible flood protection measures or with new construction, and should not be used as a basis for estimates for specific projects. Costs in addition to those shown would be incurred for landscaping, and for temporary housing during the construction if the work prevented remaining in residence during the process.

Conclusions

Raising and moving a slab-on-grade structure with the slab attached can be both practical and cost-effective when undertaken by competent, experienced, and adequately equipped structural movers. In some cases, the procedure may provide the only practical retrofitting option. Each structure will have highly individual engineering and architectural characteristics affecting economic feasibility and aesthetic desirability. Some advantages of this procedure include:

- continued occupancy and use of the structure during the process;
- avoiding or simplifying interior shoring and bracing, better preserving the structural integrity of the building; and
- the technique is applicable to some construction materials not otherwise feasible to move or raise, such as concrete masonry block.

Table VII-3.1 **Detailed Cost Estimate Elevation of a 36 x 36 (1296 sf) One-Story Home 2 Feet Above Ground**

ITEM	QUANTITY	UNIT	PRICE	COST
Elevation of Home	1,296	SF	12	15,552
New Concrete Pier Foundation				
(see note)	25	EA	170	4,250
Pier-Slab Connection	25	EA	21.20	530
Hurricane Clips	50	EA	1.50	75
Raise Water, Sewer & Gas	2	LF	28.14	56
Elevate Air Conditioner				
Wooden Deck, 5x5 treated	25	SF	7.54	188
Extend Downspouts	1	LS	30	30
Architectural Modifications				
New Front Porch (see note)	36	SF	7.54	271
Wood Front Stair & Rails	2	LF	239	478
Concrete Stair Pad	1	EA	50	50
New Back Porch (see note)	100	SF	7.54	754
Wood Back Stair & Rails	2	LF	239	478
Concrete Stair Pad	1	EA	50	50
Wooden Lattice	288	SF	1	288
Painting, Decks & Stairs	125	SF	0.39	49
Painting Lattice, Spray	288	SF	0.14	40
New Sidewalks	30	LF	5.52	166
Subtotal				**23,306**
Contingencies	25%			5,826
Subtotal Construction Cost				29,132
Engineering & Design	5%			1,457
Supervision & Administration	3%			874
Total Construction Cost				**$31,463**
Const. Cost Per Sq. Foot of Slab	**$24**			

NOTES:

Concrete piers, 9' c.c., 14x14x5'-10" pier in place. Includes 2' x 2' x 8" footing in place, 4 #4 bars each way.

New Front Porch: Wood Deck, 6' x 6', treated 2x6.

New Back Porch: Wood Deck, 10' x 10', treated 2x6.

Source: U.S. Army Corps of Engineers

Table VII-3.2 **Detailed Cost Estimate Elevation of a 36 x 36 (1296 sf) One-Story Home 10 Feet Above Ground**

ITEM	QUANTITY	UNIT	PRICE	COST
Elevation of Home	1,296	SF	12	15,552
New Concrete Pier Foundation				
(see note)	25	EA	190	4,750
Pier-Slab Connection	25	EA	21.20	530
Hurricane Clips	50	EA	1.50	75
Raise Water, Sewer & Gas	10	LF	28.14	281
Elevate Air Conditioner				
Wooden Deck, 5x5 treated	25	SF	7.54	188
Extend Downspouts	1	LS	150	150
Architectural Modifications				
New Front Porch (see note)	36	SF	7.54	271
Wood Front Stair & Rails	10	LF	239	2,390
Concrete Stair Pad	1	EA	50	50
New Back Porch (see note)	100	SF	7.54	754
Wood Back Stair & Rails	10	LF	239	2,390
Concrete Stair Pad	1	EA	50	50
Wooden Lattice	1,440	SF	1	1,440
Painting, Decks & Stairs	150	SF	0.39	58
Painting Lattice, Spray	1,440	SF	0.14	202
New Sidewalks	30	LF	5.52	166
Subtotal				**29,298**
Contingencies	25%			7,325
Subtotal Construction Cost				36,623
Engineering & Design	5%			1,831
Supervision & Administration	3%			1,099
Total Construction Cost				**$39,552**
Const. Cost Per Sq. Foot of Slab	**$31**			

NOTES:

Concrete piers, 9' c.c., 14x14x13'-10" pier in place. Includes 2' x 2' x 8" footing in place, 4 #4 bars each way.

New Front Porch: Wood Deck, 6' x 6', treated 2x6.

New Back Porch: Wood Deck, 10' x 10', treated 2x6.

Source: U.S. Army Corps of Engineers

SMALL LEVEES AND PERIMETER FLOODWALLS

This section presents two case studies that identify procedures, methodology, and design parameters used to construct small, low-level levees and perimeter floodwalls. Case Study #4 illustrates a variety of measures used in Madison, Connecticut, and Case Study #5 illustrates a perimeter floodwall in Prince George's County, Maryland.

CASE STUDY #4
Floodwalls, Levees, and Perimeter Drains
Bailey Creek, Madison, Connecticut

This case study discusses retrofitting methods that were successfully used to protect houses located along Bailey Creek, in Madison, Connecticut. The Bailey Creek Flood Prevention, Resource, Conservation and Development (RC&D) Project was sponsored by the Town of Madison, Connecticut, Department of Environmental Protection, and New Haven County Soil and Water Conservation District, in cooperation with King's Mark RC&D and the US Department of Agriculture, Natural Resources Conservation Service. Construction was completed in 1991 and worked successfully during Hurricane Bob (August 1991).

General Design Criteria

The following design criteria were applied to all the retrofitted houses along Bailey Creek:

- minimum protection from flooding is the 100-year level.;
- freeboard for floodwalls less than three feet high will be 0.5 feet;
- freeboard for earth levees less than three feet high will be 1.0 feet;
- a cementitious waterproof coating is applied to all walls up to the design flood level;
- concrete floodwall footings must be 42 inches below the ground surface (primarily for frost protection);
- a 4,000-watt generator is required to power the sump pump, emergency lights, well pump, and other emergency equipment;

- drainage and sump pumps are installed within the protected area; and
- existing poured concrete foundation walls and floors are assumed to be structurally sound enough to withstand three feet of hydrostatic pressure. However, the floors were only able to withstand 1.5 feet.

Project Summary

The projects required continuous inspection during construction, and were expensive. The Natural Resources Conservation Service was directly involved with the contractor to limit homeowner- requested changes and for quality assurance and contract compliance. Since installation, the measures have experienced significant flooding conditions and have proven to be very successful.

Engineering Analysis Summary

SURFACE WATER FLOODING

The flooding threat to one of the homes consisted of surface water flooding of the basement and attached garage. Figure VII-4.1 depicts the preexisting surface water problems. The basement floor was 2.5 feet below the 100-year flood level, causing water to enter the basement. Figure VII-4.2 depicts the engineering solutions developed to retrofit the house and garage.

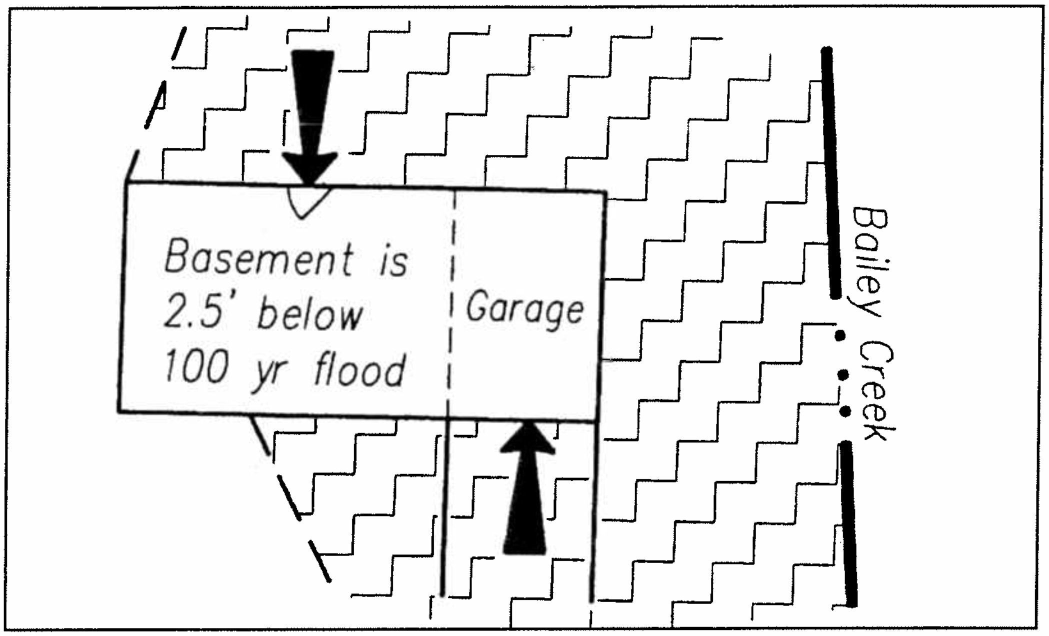


Figure VII-4.1: Surface Water Problem (Before)

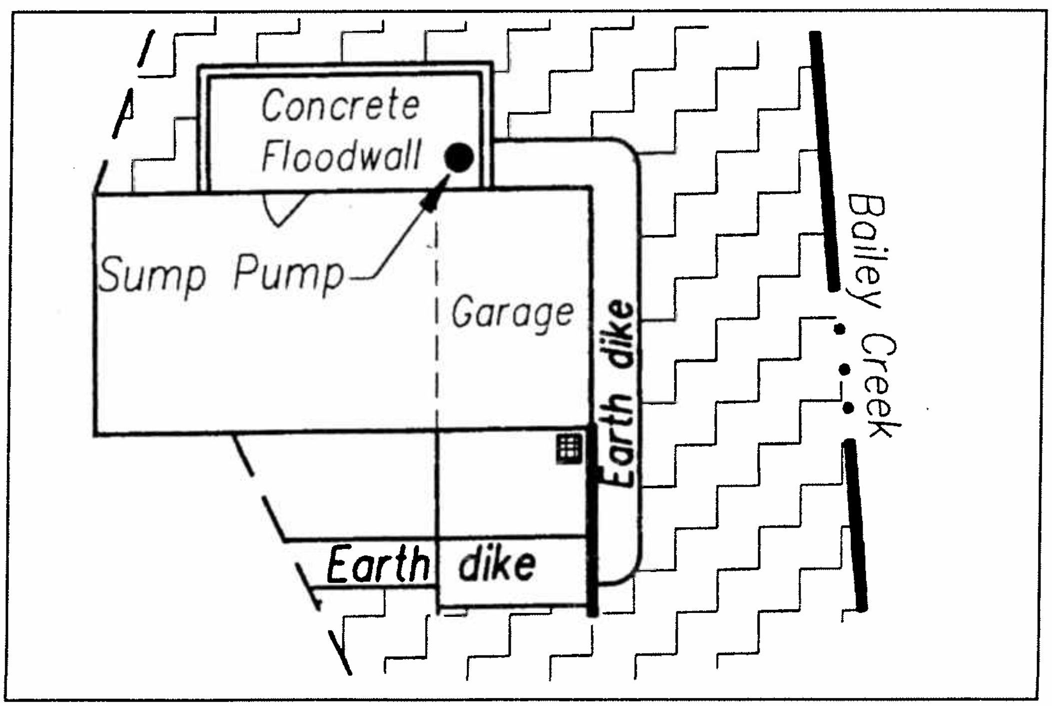


Figure VII-4.2: Surface Water Problem (After)

The site required the construction of floodwall, levee, and sump pump with an underground drain as shown on the site plan at Figure VII-4.3. A concrete floodwall (see Figure VII-4.4) was constructed around the existing patio and deck and a sump pump installed (see Figure VII-4.5). An earth levee was built to protect three sides of the house. The levee was built to a height ranging from 0.5 feet to 3.0 feet with a top width ranging from two to five feet. An earth backfill with a four-inch perforated drain (see Figure VII-4.6) was placed along one corner of the existing foundation to complete the encirclement. The earth backfill was filled to a top elevation of 11.6 NGVD with a three-foot-wide top width. The project was completed at a total cost of $25,000. Figure VII-4.7 and VII-4.8 show the completed parts floodwall and earth backfill.

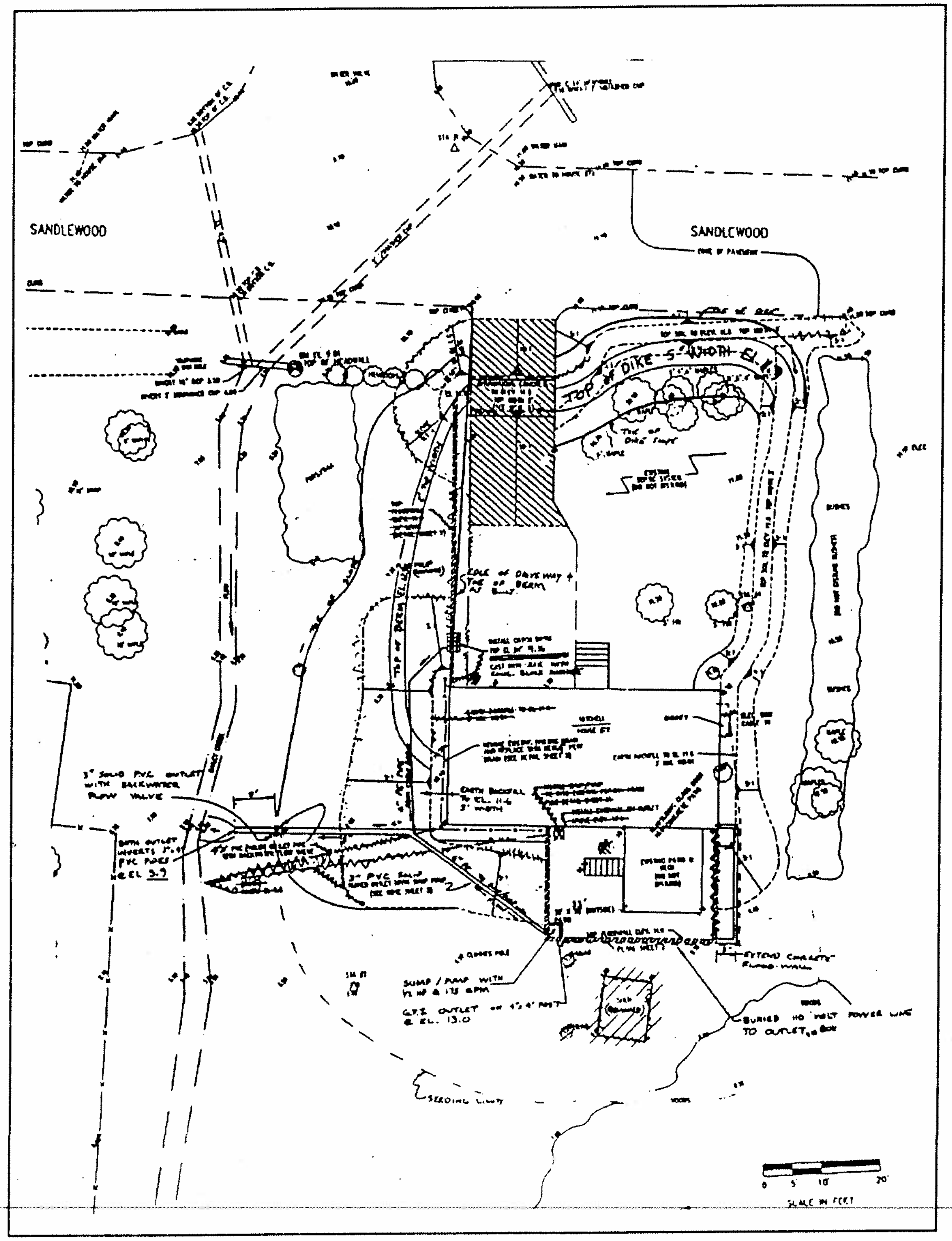


Figure VII-4.3: Site Plan

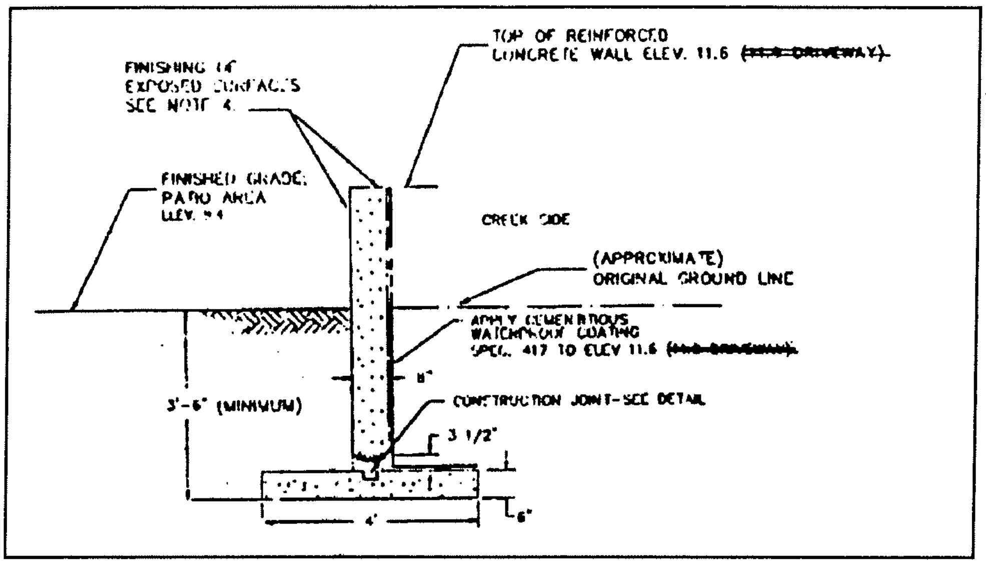


Figure VII-4.4: Typical Detail Section Floodwall

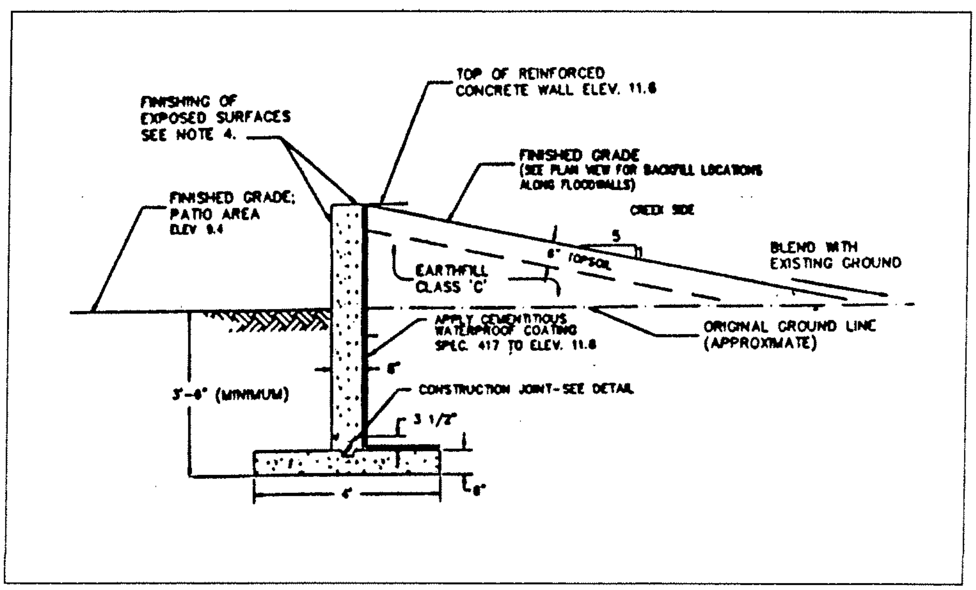


Figure VII-4.5: Typical Detail Section of Backfilled Floodwall

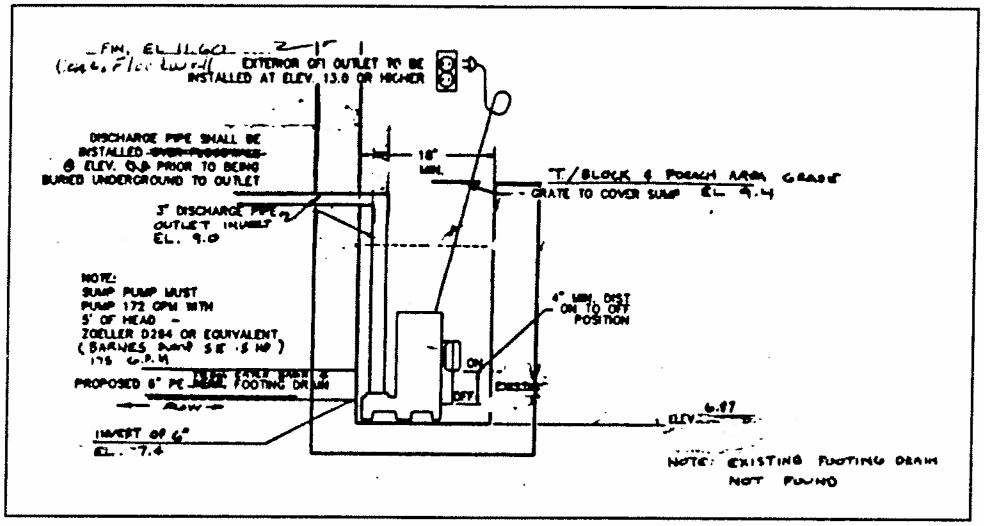


Figure VII-4.6: Patio Area Sump Pump Detail

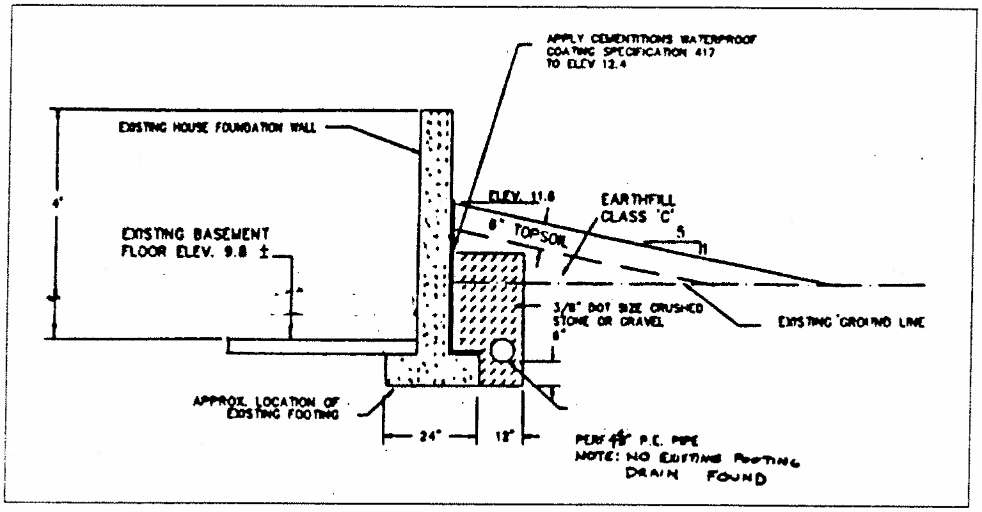


Figure VII-4.7: Footing Drain Detail

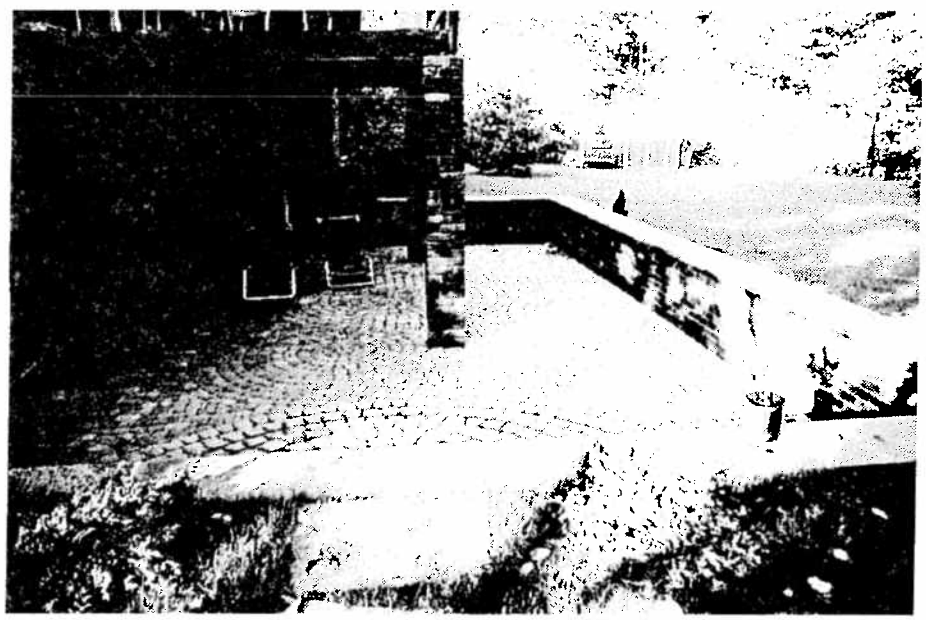


Figure VII-4.8: Completed Patio Floodwalls

SUBSURFACE WATER FLOODING

The threat to another house, on the other side of Bailey Creek, consisted of surface and subsurface flooding. Figure VII-4.9 depicts the location of the house with respect to Bailey Creek and the engineering solution developed to retrofit the house. Figure VII-4.11 depicts details on the sump pump and drain installation. An earth backfill against the existing foundation was constructed on the Bailey Creek side of the house with an elevation of 11.6 feet NGVD and a top width of three feet. A drain (see Figure VII-4.10) and sump pump (see Figure VII-4.11) were constructed inside the basement to handle subsurface water. This project was completed at a total cost of $9,280.

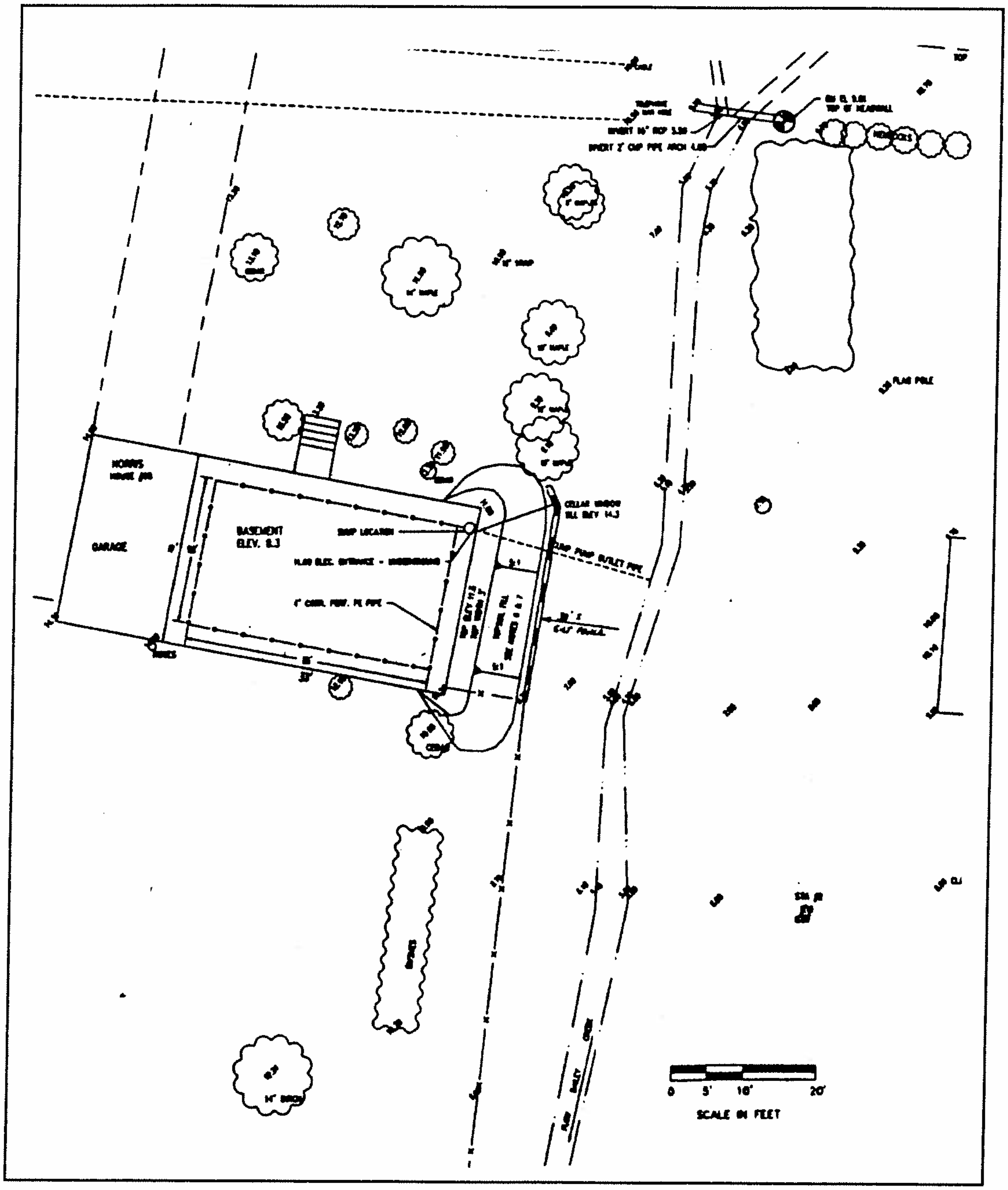


Figure VII-4.9: Site Plan: House on Opposite Side of Bailey Creek and Engineering Solutions

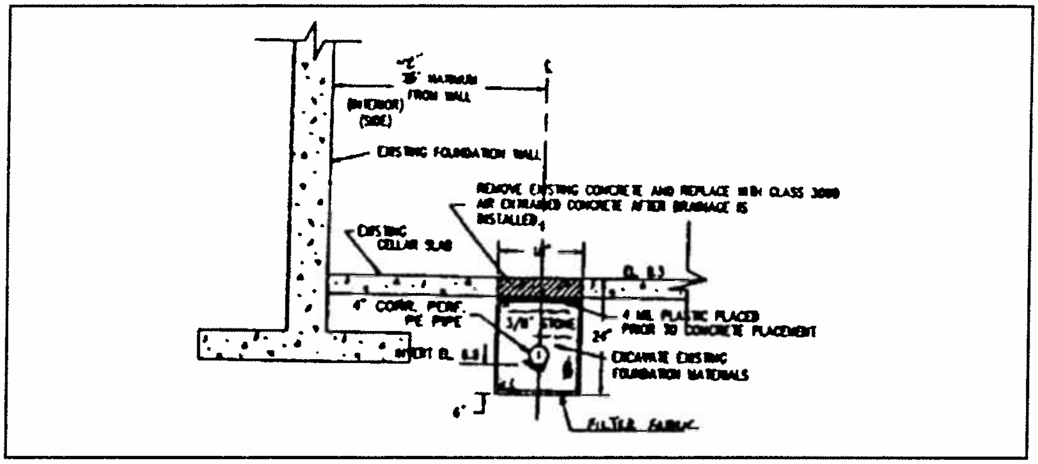


Figure VII-4.10: Typical Drain Detail

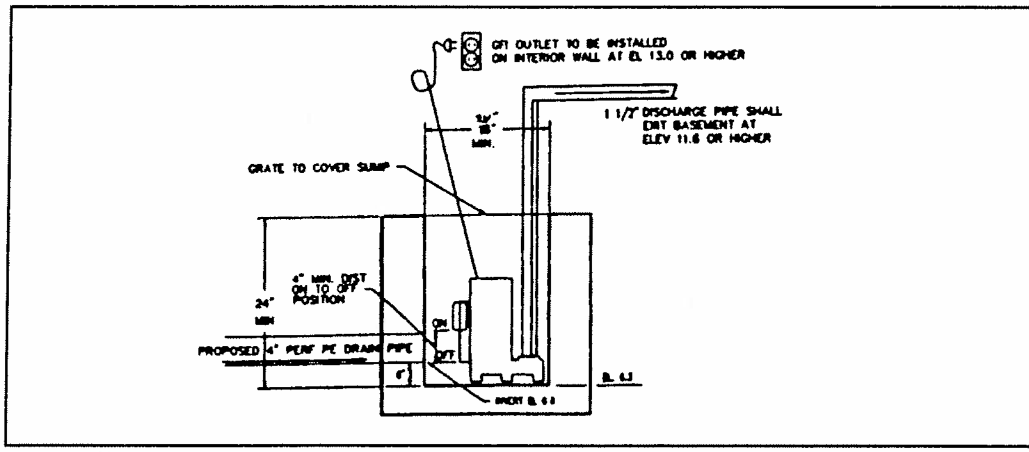


Figure VII-4.11: Sump Pump and Sump Detail

CASE STUDY #5
Perimeter Floodwall
Henson Creek, Prince George's County, Maryland

This case involves the construction of a floodwall around the perimeter of a slab-on-grade house located along Henson Creek in Prince George's County, Maryland. The actions taken (sponsored by the Prince George's County, Maryland, Department of Environmental Resources, Watershed Protection Branch) were in keeping with the county's policy to protect houses within the 100-year floodplain and/or remove the threat of flooding to these private residences.

The Henson Creek watershed area is a relatively narrow watershed, ranging from 2.5 to 3.0 miles in width and about 11 miles in length. Its combined drainage area, which includes tributary flows, is in the range of about 30 square miles. Various areas along Henson Creek were subject to flooding, and the problems were expected to increase because of development growth within the watershed boundaries.

The initial analysis was conducted to examine the feasibility of widening and improving Henson Creek channel for the purpose of flood control. In an effort to remove affected houses from the 100-year floodplain, five alternative designs were investigated. Four of the studies involved the hydraulic analysis of an existing culvert, and widening and improving the creek's banks. The fifth alternative was to retrofit individual houses.

Based on the results of the alternatives evaluated, home retrofitting was the most cost-effective solution to provide 100-year flood protection. The four designs involving culvert structure modification were rejected due to costs that ranged from $1,245,000 to $3,095,000. The retrofitting of individual houses (elevation, floodwalls, wet and dry floodproofing measures) was estimated at $246,800.

Retrofitting Methodology

DETERMINATION OF FLOOD DEPTH

Computer analysis through the use of HEC-2 and TR-20 modeling was used to determine water-surface elevations that would result from a 100-year flood based on ultimate watershed development. Cross sections were located at critical locations and at predetermined distances along the stream channel. The flood depth at a particular structure (residence) was interpolated from the water-surface elevations at the nearest cross section both upstream and downstream.

DETERMINATION OF LOW POINT OF FLOODWATER ENTRY

Each residence was field surveyed to determine the elevation of all openings into crawlspaces or basements, and ground at the house, first floor, and basement slab. A county engineer reviewed the survey data and determined what elevation the floodwater would have to reach before the residence would begin to flood. Many times this elevation was a vent, an entrance into a crawl space, a walkout from a basement, or the top of a stairwell into a basement.

DETERMINATION OF TYPE OF CONSTRUCTION

Each residence was reviewed by a team of engineers to determine the type of construction used in the residence. Three types of structures were identified: slab on grade, crawl-space, and full basement.

DETERMINATION OF STRUCTURAL COMPETENCE

The team of engineers reviewed the construction and condition of each residence to determine if the residence could be successfully retrofitted.

DETERMINATION OF RETROFITTING METHOD

Each residence was evaluated separately, but structures of similar construction were considered receptive to similar retrofitting methods.

DETERMINATION OF RETROFITTING COSTS

The county developed a database of current costs (1988) associated with the retrofitting of residential structures. Personal knowledge and contacts with other individuals involved in similar work in other jurisdictions as well as cost data from publications including *Engineering News-Record (ENR)* and *Mean's Guide* were used to develop the estimates.

DETERMINATION OF DESIGN CRITERIA

The structural analysis of the houses was performed in full accordance with the design requirements set forth in the following codes and regulations:

- *Prince George's County Building Code*, 1983
- *Building Officials and Code Administrators (BOCA) National Building Code*, Ninth ed., 1984
- *American Standard Building Code Requirements for Masonry* (ANSI A41.1-1953, Reaffirmed 1970)
- *Flood-Proofing Regulations* (EP 1165 2 314), U.S. Army Corps of Engineers, June 1972

In addition, the following references were used as guidelines in the structural computations:

- *Specification for the Design and Construction of Load-Bearing Concrete Masonry* (TR75B) National Concrete Masonry Association (NCMA) February, 1987
- *Basement and Foundation Walls* (TR68-A), NCMA, 1971
- *Nonreinforced Concrete Masonry Design Tables* (TR03), NCMA, 1971
- *Reinforced Concrete Masonry Design Tables* (TR84), NCMA, 1971
- *Design Manual for Retrofitting Flood-prone Residential Structures* (FEMA 114), Federal Emergency Management Agency, September 1986
- *Cost Report on Non-Structural Flood Damage Reduction Measures for Residential Building Within the Baltimore District,* U.S. Army Engineer Institute for Water Resources, IWR Pamphlet #4, July 1977

The following design values were used in the structural analysis of the foundation walls:

Soil:

Soil unit weight = 120 pcf
Internal friction angle = 30 degrees
Active pressure coefficient = 0.33

Masonry:

Allowable tension in flexure (normal to bed joints) Type M or S mortar

Hollow Units: 23 psi
Solid Units: 39 psi

Allowable Shear (Type M or S mortar)

All Units: 34 psi

Compressive Strength, f'm = 1,000 psi

Unit Weight (ASTM C-140) = 120 pcf

Allowable stress for grade 60 reinforcing steel, f_s = 24,000 psi

Dead Loads:

Floor and Roof: 15 psf

Foundation Walls: Density of masonry block = 120 pcf

Density of wood: 40 pcf

Live Loads:

Lateral Earth Pressures:

Saturated Soil: 40 psf
Water: 62 psf
Water plus buoyant soil: 82 psf

Wind Pressure: 16 psf

Engineering Analysis Summary

Site #1: The site is a one-story, brick veneer over wood-frame slab-on-grade house located south of Henson Creek (see Figure VII-5.1). The first floor elevation (FF) and low point of entry (LPE) is 198.4 and the 100-year water-surface elevation (WSEL) is 199.0 (see Figure VII-5.2).

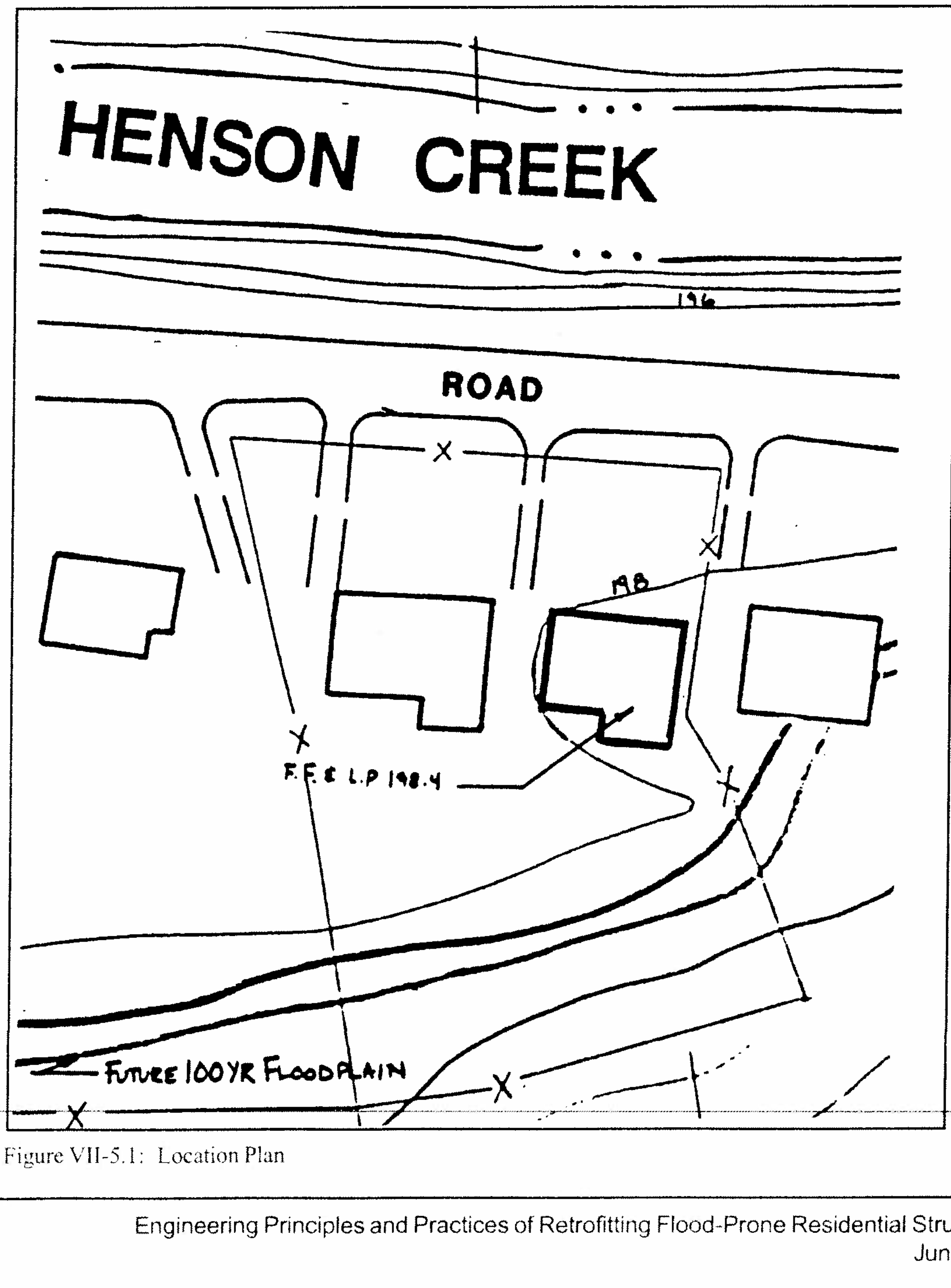


Figure VII-5.1: Location Plan

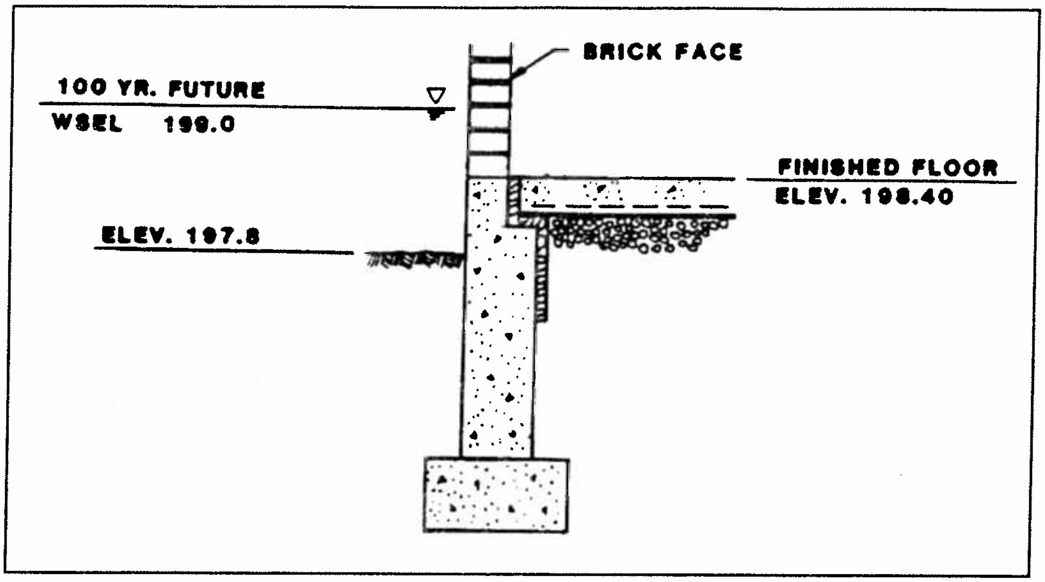


Figure VII-5.2: Preexisting Slab-on-Grade Construction Detail

Since the 100-year water surface elevation (WSEL) was only 0.6 feet above the finished floor, the construction of a floodwall around the perimeter of the house proved to be the best option in terms of overall cost (approximately $18,000) and risk to the building. This would allow the house to stand as is and be protected by a separate structural element. The owners were advised of the elevation and/or relocating problems associated with their house and that the county selected the floodwall alternative. The recommendations listed below were developed based on the engineering analysis:

- Construct a floodwall around the perimeter of the house. The wall must be at least one foot above the 100-year WSEL, or approximately 2.6 feet high to comply with the county code.

- Provide at least two step-up/step-down accesses over the wall to the entrances into the house.

- Rebuild the concrete patio located in back of the house inside the floodwall.

- Provide a gravity drainage system behind the floodwall to rid the ringed area of the trapped water.

- Tie down the tool sheds to resist flotation.

Proposed Work

The proposed work is keyed to Figure VII-5.3, Site Plan.

1. Construct floodwall (see Figure VII-5.4).

2. Construct concrete steps for access over the floodwall (see Figure VII-5.9 and VII-7.12).

3. Install steel pipe railing.

4. Construct a concrete slab on four-inch gravel base inside the floodwall. Provide positive drainage to sump pump.

5. Relocate telephone junction box vertically to elevation 200.5.

6. Limits of grading, seeding, and mulching.

7. Provide four-inch-high concrete equipment pad under air conditioner, (see Figure VII-5.10).

8. Apply waterproofing between existing wall and topsoil.

9. Install new downspout drain with new coupling through landing.

10. Plant new shrubs.

11. Remove existing concrete pad.

12. Install 6 x 6-inch treated timber retaining wall (see Figure VII-5.9).

13. Fill planter with topsoil.

14. Remove existing concrete slab in its entirety.

15. Furnish and install new sump pump and pit (see Figure VII-5.8).

16. Tie down existing shed.

17. Remove and dispose of fence.

18. Install one-inch round PVC schedule 40 conduit for sump pump electrical cables.

19. Install outside rated double receptacle in 6 x 6-inch exterior lockable box.

20. Verify location of gas line prior to excavation.

21. Limit of disturbance and sod.

22. Provide concrete encasement of three-inch diameter PVC sleeve around existing gas line.

Plans, Elevation, and Construction Details

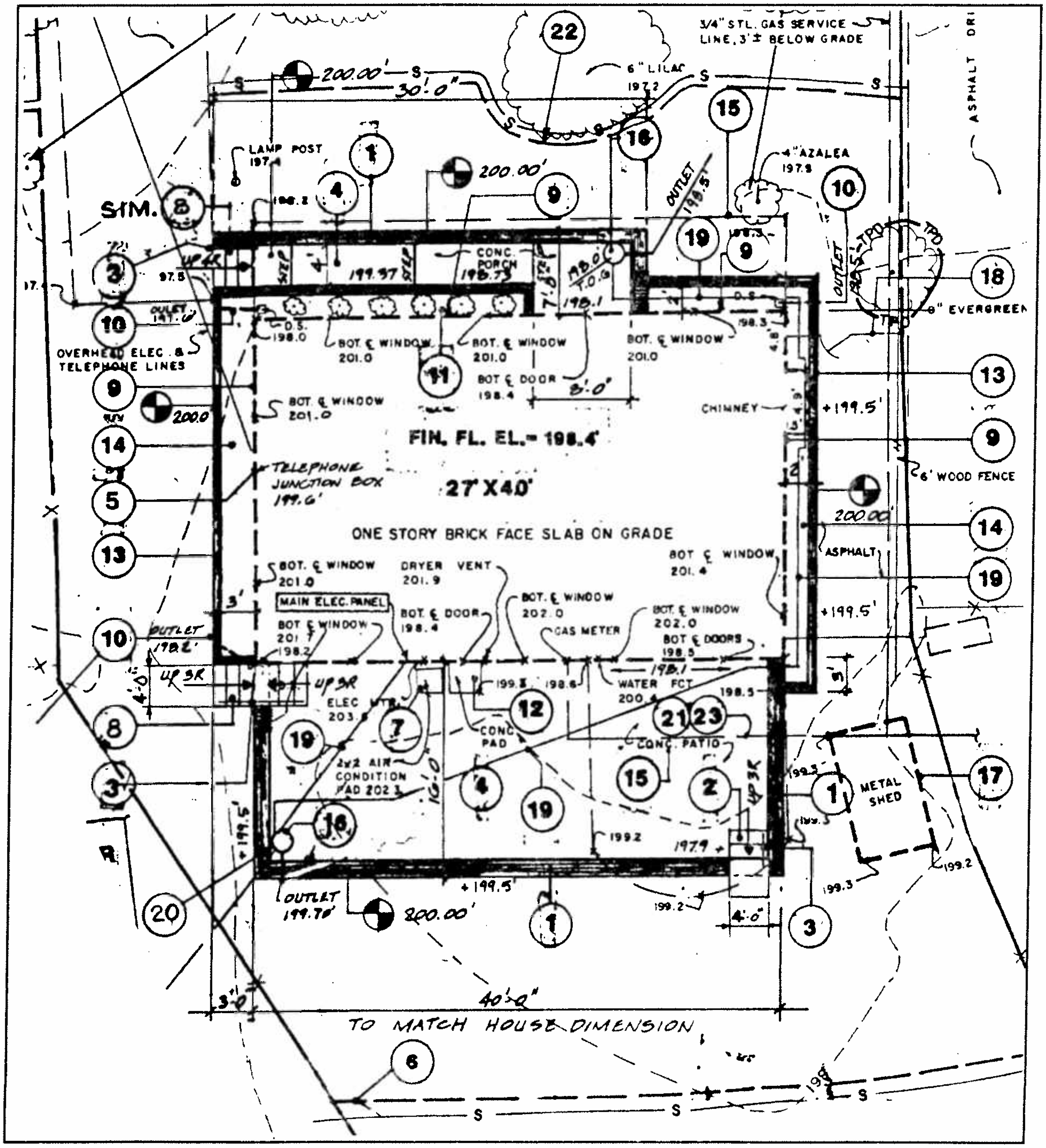


Figure VII-5.3: Site Plan

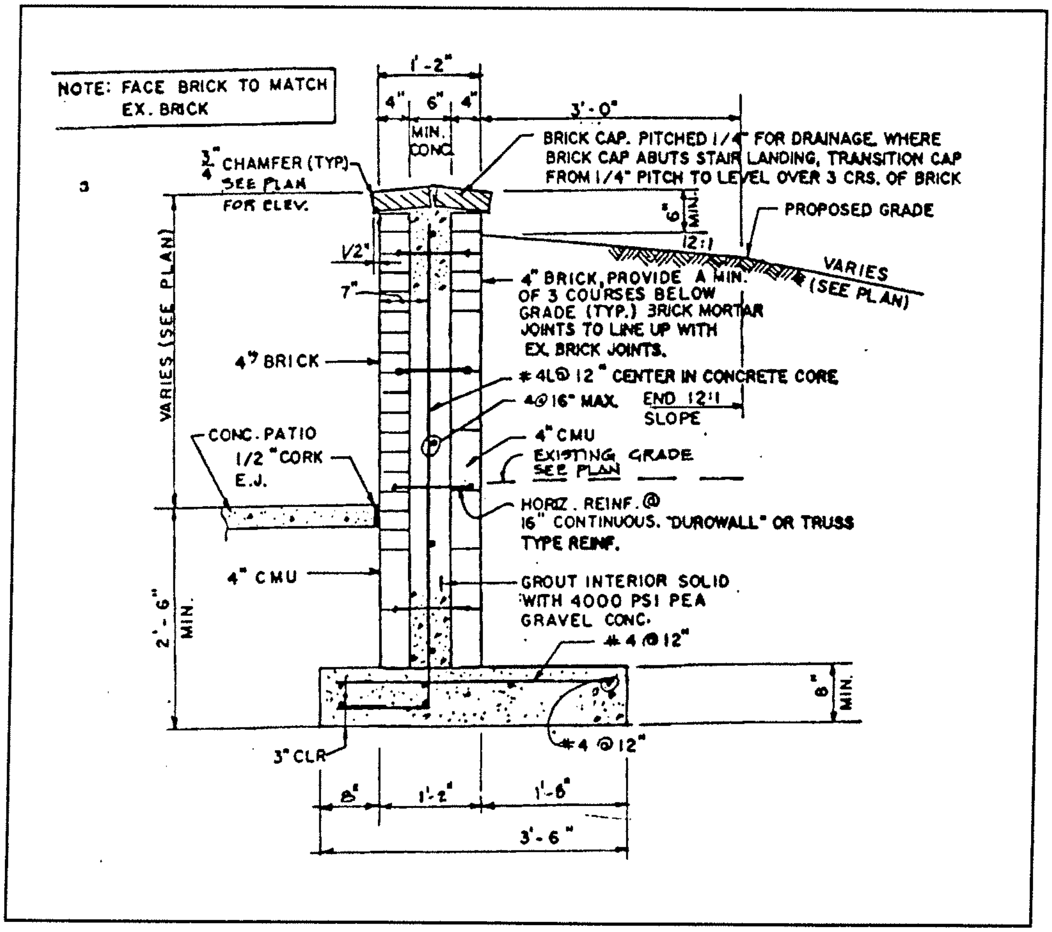


Figure VII-5.4: Typical Floodwall Detail Section

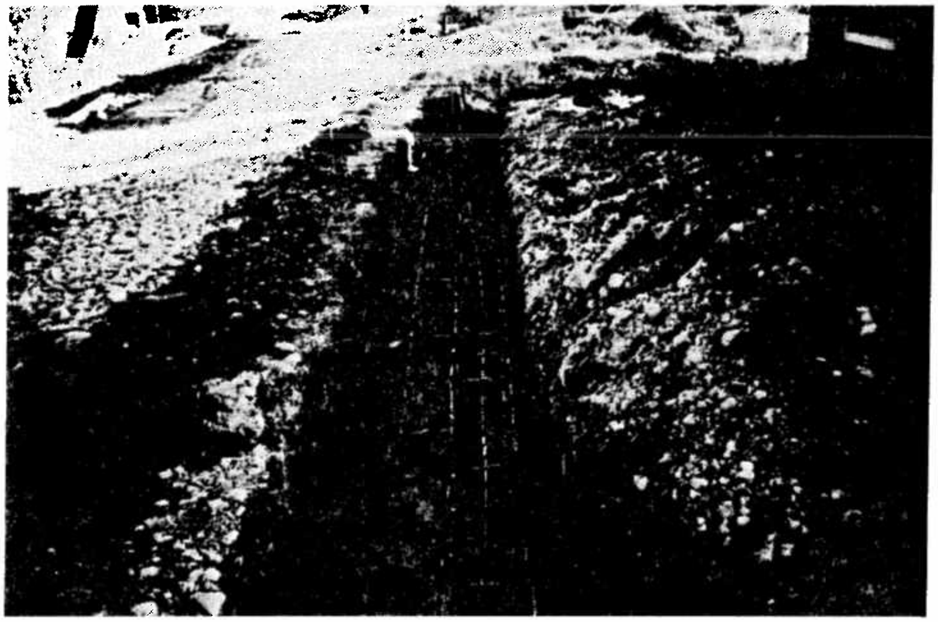


Figure VII-5.5: Footer Detail

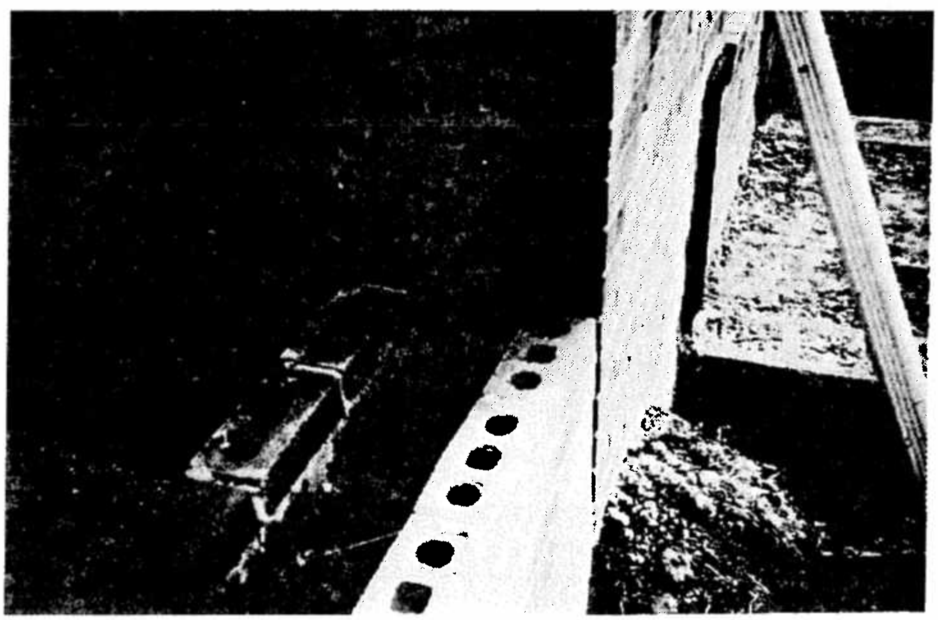


Figure VII-5.6: Wall-to-House Connection Detail

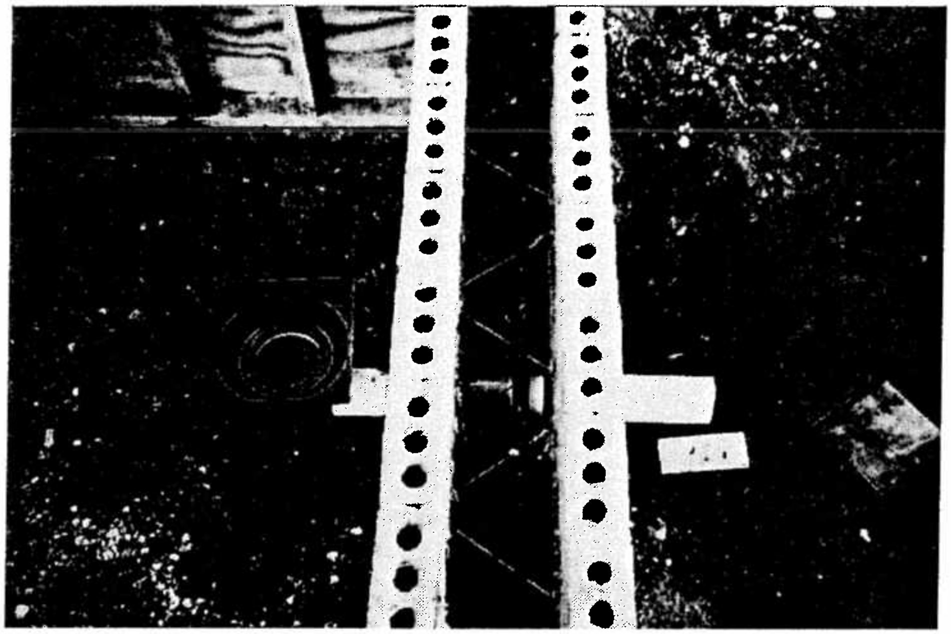


Figure VII-5.7: Drain Detail

Figure VII-5.8: Sump Detail

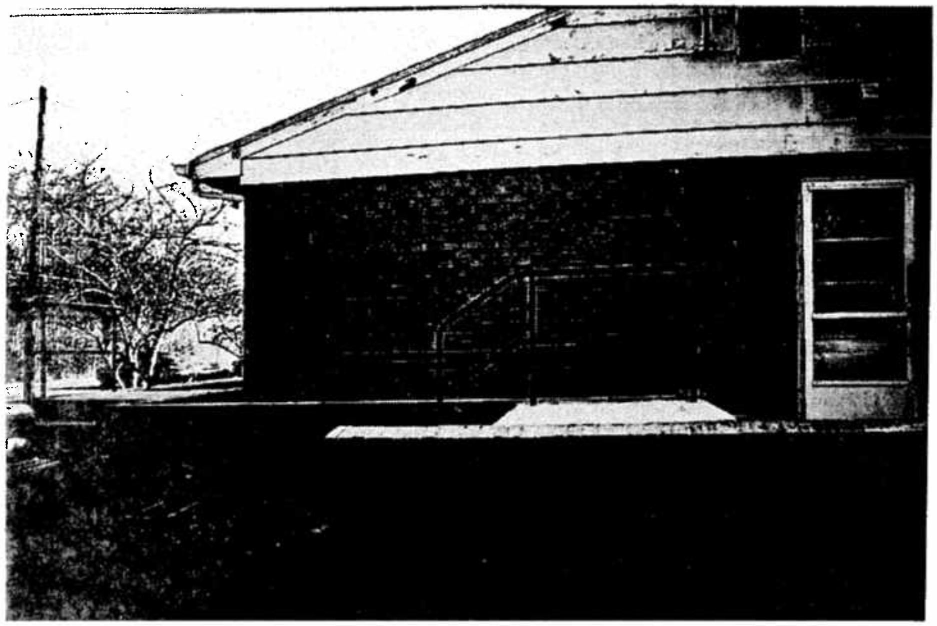


Figure VII-5.9: Floodwall Steps and Landscaping Timber

Figure VII-5.10: Sump Pump Outlet and Raised Air Conditioner Unit

Figure VII-5.11: Completed Project

WET FLOODPROOFING

This section presents a case study that identifies procedures, methodology, and design parameters used in wet floodproofing.

CASE STUDY #6
Wet Floodproofing a House on a Crawlspace
Henson Creek, Prince George's County, Maryland

This case study discusses wet floodproofing measures that were taken to protect houses located along Henson Creek in Prince George's County, Maryland. (See Chapter VII, Case #5 for complete background and retrofitting methodology.)

Engineering Analysis Summary

Site #2: The site is a two-story wood-frame house on a crawlspace with a first-floor (FF) elevation of 199.3 (see Figure VII-6.1). The bottom of the crawlspace vent is 197.5 and the bottom of the crawlspace access door or low point of entry (LPE) is 196.9. The 100-year water-surface elevation (WSEL) is 198.4 (see Figure VII-6.2).

The types of forces imposed by the floodwater will be lateral hydrostatic pressure on the exterior masonry walls and a buoyant force on the first floor timber framing. The house was analyzed under dry floodproofing and wet floodproofing conditions in order to investigate the feasibility of each condition. Figure VII-6.2 is the preexisting foundation wall section.

Dry Floodproofing Option

On the field inspection, the existing masonry walls appeared to be in good condition; therefore, the mortar joints were assumed to have a structural capacity equal to their capacity at construction. In addition, the calculations are based on the assumption that the bottom of the footing is exactly 30 inches below grade as required by code.

The dry floodproofing option was rejected because the analysis showed that the flexural stress in the mortar joints exceeds the allowable stress under 100-year flood conditions. Moreover, dry floodproofing would be difficult to achieve since the soil around the foundation was relatively permeable sandy soil and would allow water to seep into the crawlspace. This is due to the difference in water level between the inside and outside of the wall during flood conditions and the permeability of the soil. The dry floodproofing calculations are shown in Figure VII-6.3.

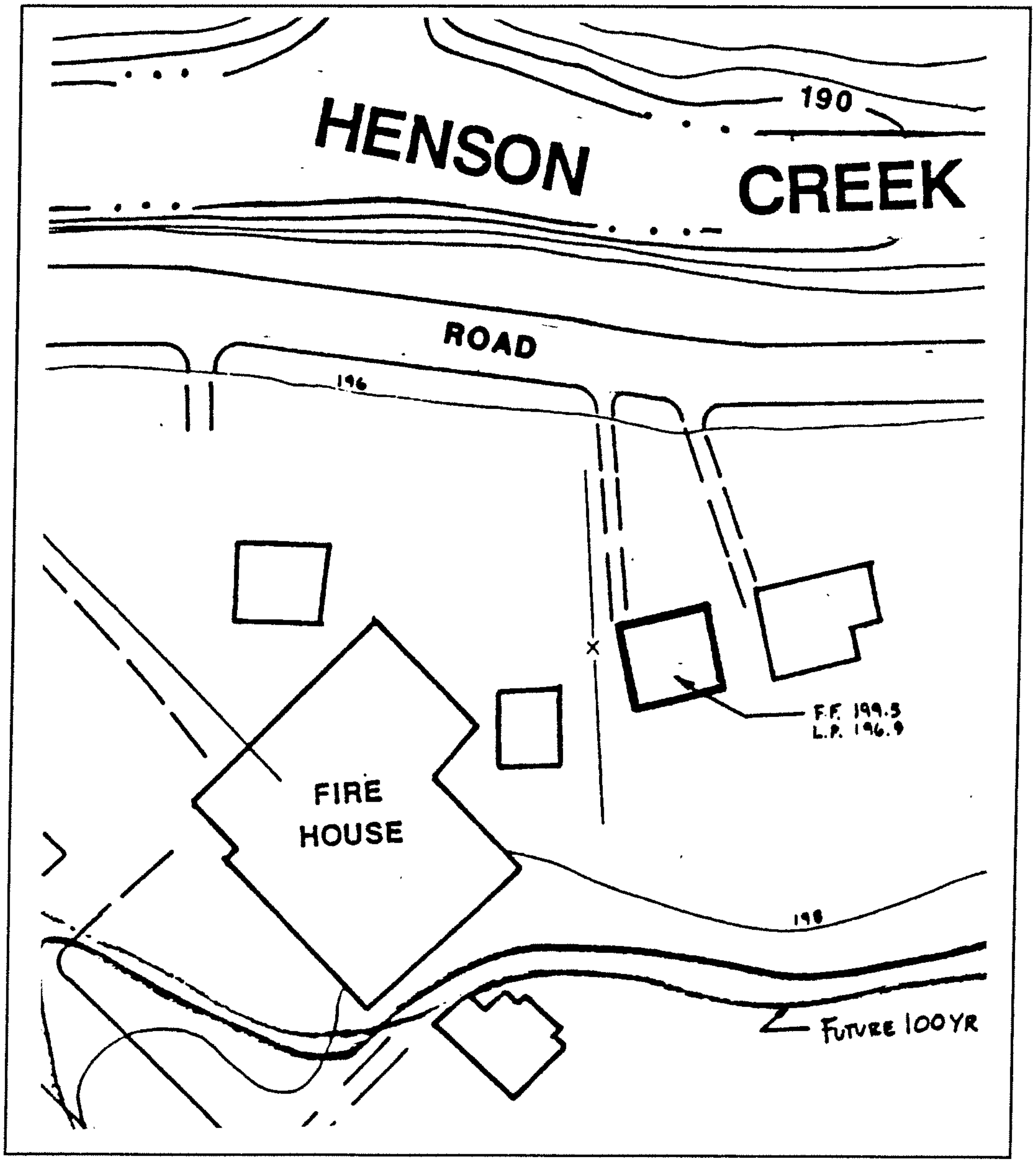


Figure VII-6.1: Location Plan

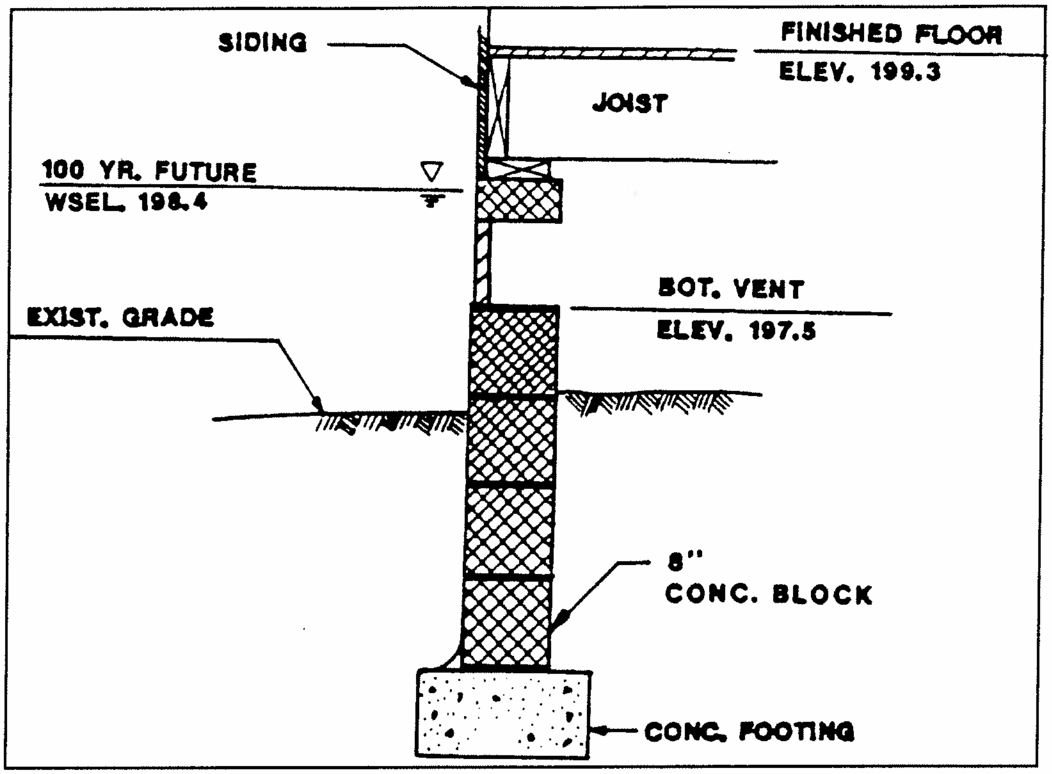
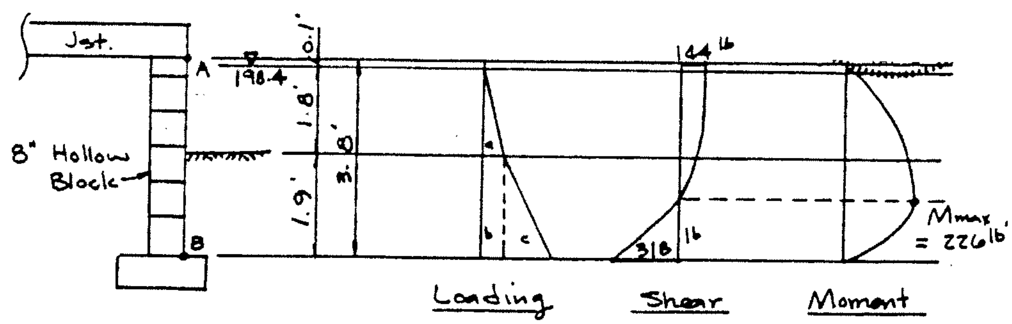


Figure VII-6.2: Preexisting Foundation Wall Section Detail

Dry Flood Proofing Option



$H_a = \frac{1}{2}(62.4)(1.8)^2 = 101^{lb}$ $\qquad \bar{y}_a = 2.50'$ (from 'B')

$H_b = (62.4)(1.8)(1.9) = 213^{lb}$ $\qquad \bar{y}_b = 0.95'$ "

$H_c = \frac{1}{2}(82)(1.9)^2 = 148^{lb}$ $\qquad \bar{y}_c = 0.63'$ "

$H_T = 462^{lb}$

$\Sigma M_B = (101)(2.50) + (213)(.95) + (148)(.63) - 3.8 R_a = 0$

$R_A = 144^{lb}$

$R_B = 318^{lb} < V_{allow} = (45 in^2)^{*}(34 psi)^{**} = 1{,}530^{lb}$ OK

Find R_A using γ_{eq}.

$$\gamma_{eq} = \frac{(462)(2)}{(3.7)^2} = 67.5 \text{ pcf}$$

$$R_A' = \frac{(67.5)(3.7)^3}{6} \Big/ 3.8 = 150^{lb} \sim 144^{lb}$$

Since $R_A \sim R_A'$ the following formula is appropriate for the bending moment in the wall:

$$M_{max} = \frac{(462)(3.7)}{3(3.8)}\left[0.1 + \frac{2}{3}(3.7)\sqrt{\frac{3.7}{3(3.8)}}\right] = 226 \text{ lb}'$$

$$f_t = \frac{226^{lb} \times 12''/'}{80 in^3 *} = 33.9 \text{ psi}$$

Figure VII-6.3: Dry Floodproofing Calculations

Compressive Stress due to DL of framing

$$f_c = \frac{250 \text{ plf}}{45 \text{ in}^2 *} = 5.6 \text{ psi}$$

Resultant tensile stress = 33.9 − 5.6 psi = 28.3 psi

> 23.0 psi **

NG

Since the applied stress exceeds the allowable limit, the ingress of water must be made at a lower level. Try the wet flood proofing option.

* Sectional properties from NCMA Nonreinforced Concrete Masonry Design Tables, p. 22

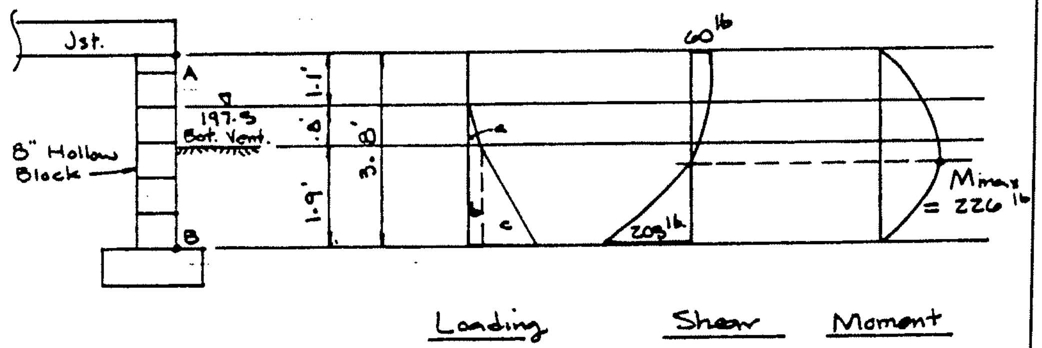
** See 'Design Criteria'

Figure VII-6.3: Dry Floodproofing Calculations (continued)

Wet Floodproofing Option

This option allows the water to enter the crawlspace through vents or the access doors. This results in a reduction in the mortar joint stress to below the allowable limit. It is imperative that the openings are free of debris to sufficiently allow the water to flow through. When the water reaches its peak elevation, the wood floor framing will be partially submerged and will cause an upward buoyant force on the first floor. A conservative approach was taken in the structural calculations, which checked the buoyant force with the entire floor joists submerged. The analysis showed that the dead load of the first floor alone is sufficient to resist the upward force caused by the water. The main floor beam and possibly the floor joists will be inundated by the water for a period of two to three hours, and structural damage could occur to the floor joists, beam, and possibly to the subflooring. Therefore, waterproofing should be applied to the floor joists to allow the implementation of the wet floodproofing option. The wet floodproofing calculations are shown in Figure VII-6.4.

Wet Flood Proofing Option – Ingress through vent



$H_a = \frac{1}{2}(62.4)(.8)^2 = 20^{lb}$ $\quad \bar{y}_a = 2.17'$ (from 'B')

$H_b = (62.4)(.8)(1.9) = 95^{lb}$ $\quad \bar{y}_b = 0.95'$ "

$H_c = \frac{1}{2}(82)(1.9)^2 = 148^{lb}$ $\quad \bar{y}_c = 0.63'$ "

$H_T = 263^{lb}$

Figure VII-6.4: Wet Floodproofing Calculations

$$\Sigma M_B = (20)(2.17) + (95)(0.95) + (148)(0.65) - 3.80\,R_a = 0$$

$R_A = 60^{\,lb.}$

$R_B = 203^{\,lb.} < 1{,}530^{\,lb}$ OK

Find R_A using γ_{eq}.

$$\gamma_{eq} = \frac{(263)(2)}{(2.7)^2} = 72.2 \text{ pcf}$$

$$R_A' = \frac{(72.2)(2.7)^3}{6} \Big/ 3.8 = 62^{\,lb} \sim 60^{\,lb}$$

Since $R_A' \sim R_A$ the following formula is appropriate for the bending moment in the wall.

$$M_{max} = \frac{(263)(2.7)}{3(3.8)}\left[1.1 + \frac{2}{3}(2.7)\sqrt{\frac{2.7}{3(3.8)}}\right] = 123^{\,lb'}$$

$$f_t = \frac{123 \times 12}{80} = 18.5 \text{ psi}$$

$f_c = 5.7$ (see p. 2 of 4)

Resultant tensile force $= 18.5 - 5.7 = 12.8$ psi

Wind Load: ht. of wall $\sim 20'$

$$F_{wind} = \frac{(16 \text{ psf})(20')}{2} = 160^{\,lb}$$

$$f_{wind} = 160^{\,lb} / 45 \text{ in}^2 = 3.6 \text{ psi} \rightarrow$$

$$\text{Resultant Stress} = \sqrt{(12.8)^2 + (3.6)^2} = 13.3 \text{ psi}$$

$$< 23\,(1.33) = 30.$$

∴ Ingress through the vents is allowed OK

Check bouyancy of first floor:

DL of floor = 15 psf. ↓

Additional Live Load will decrease the adverse affects of bouyancy.

$$\text{Bouyant force} = \frac{(1.5)(7.5)}{144} \times 62.4 = 4.9 \text{ plf} = 3.7 \text{ psf} \uparrow$$

Bouyant floor wt. $= 15 - 3.7 = 11.3$ psf ↓ OK

Figure VII-6.4: Wet Floodproofing Calculations (continued)

Engineering Analysis

The following recommendations were developed based upon the engineering analysis:

- Waterproof the floor joists, main beam, and the bottom of the subflooring to eliminate possible structural/water damage.
- Replace any electrical wiring that has any bare wire exposed due to deterioration, splices, or connections with a water-resistant romex cable.
- Outside oil and gas tanks need to be anchored to the ground.
- The fuse and junction box on the back of the house should be raised to at least 1.0 feet above the 100-year future WSEL.
- Replace, clean, or add any vent openings to meet the current building code requirements and water flow requirements.
- Provide drainage from the crawlspace interior.
- Provide a water permeable access door to the crawlspace.
- Tie down tool shed in back yard to resist flotation.

Cost Estimates

The following are cost estimates in 1988 dollars to wet floodproof the house:

Waterproof joists & subfloor	$ 300
Misc. electric and plumbing	$ 500
Oil and gas tank foundations	$ 1,200
New vents	$ 300
Water permeable access door	$ 150
Tie down tool shed	$ 200
TOTAL	$ 2,650

Proposed Work

The proposed work is keyed to Figure VII-6.5.

1. Remove existing block vent. Furnish and install new block vent into existing opening. Rework opening as required to accommodate new vent (see Figure VII-6.6).

2. Furnish and install new water-permeable access door (see Figure VII-6.7).

3. Remove existing concrete pad in its entirety.

4. Provide new concrete pad.

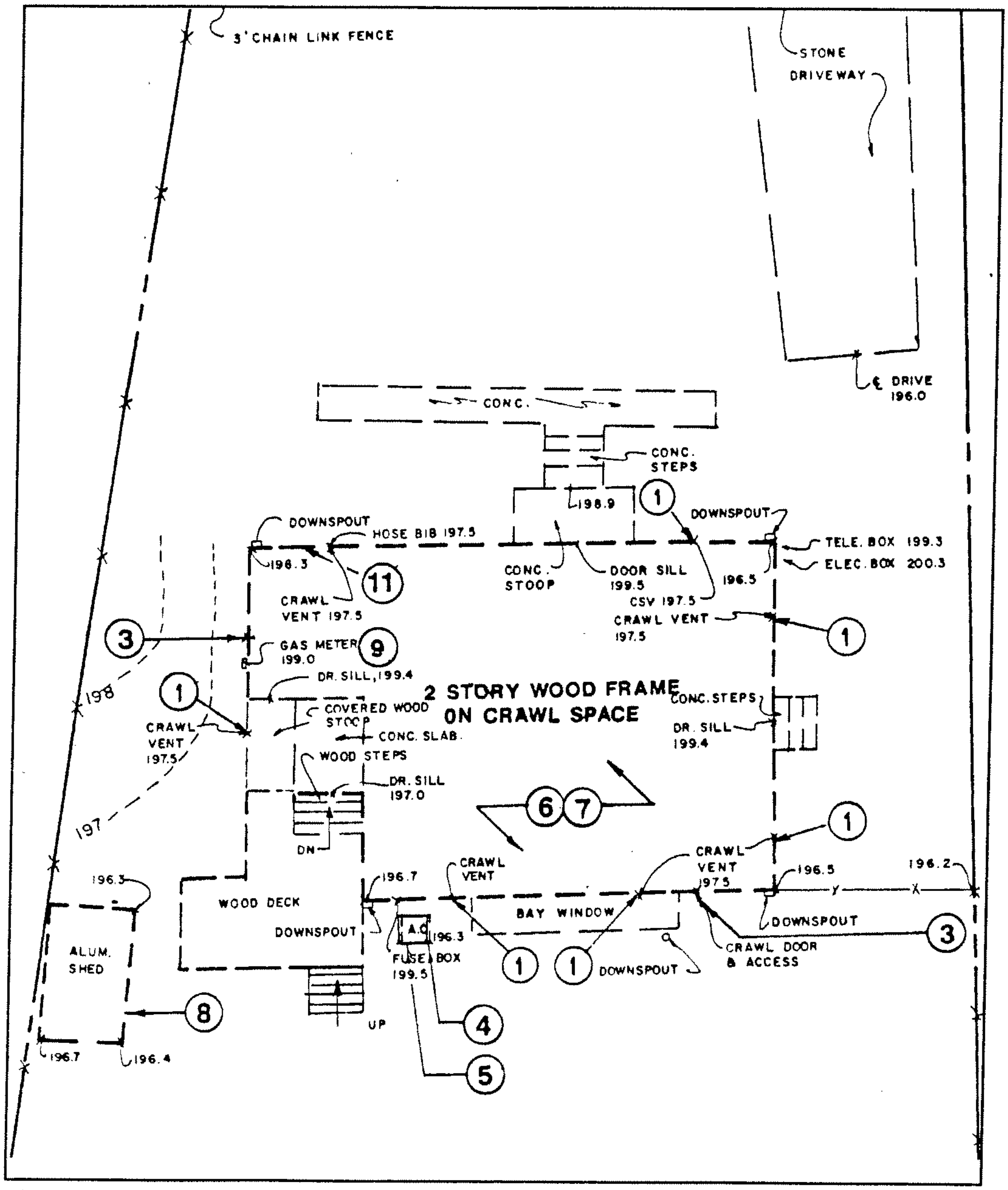
5. Final grade in crawlspace adjacent to all exterior walls shall not be lower than six inches below bottom of crawlspace access door. In addition the grade in the crawlspace shall not differ by more than one foot.

6. Waterproof floor joist and underside of subfloor in crawlspace.

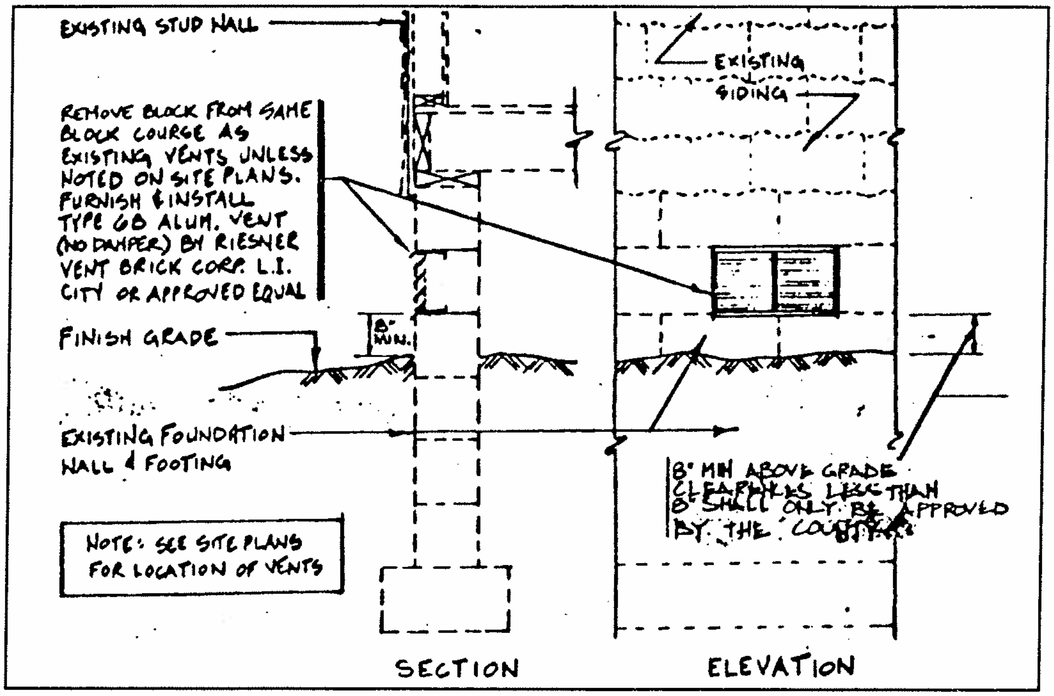
7. Tie down tool shed (see Figure VII-6.8).

8. Gas meter to be raised to elevation of 199.4.

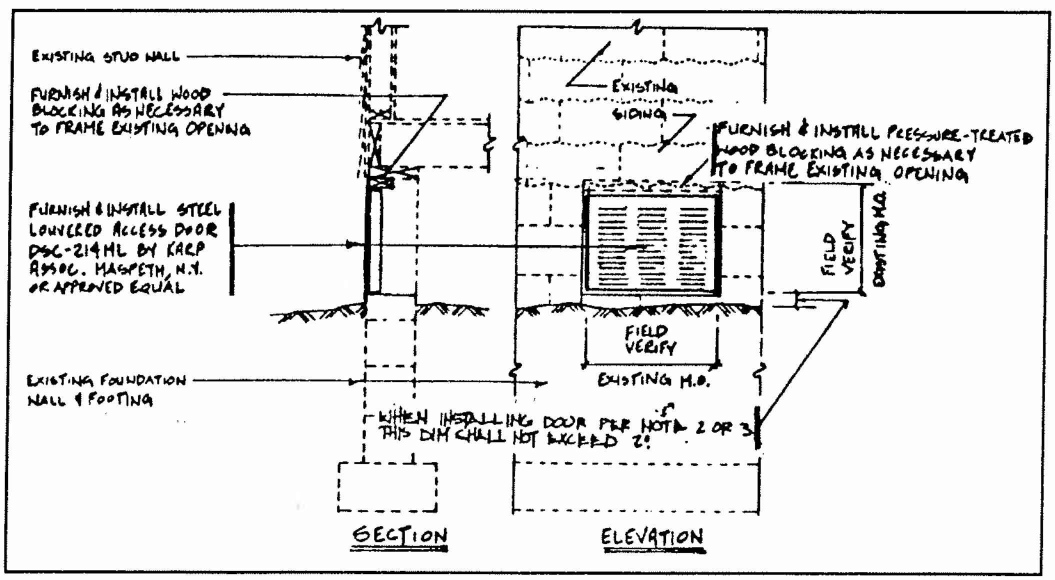
9. Furnish and install new block vent (see Figure VII-6.6). Remove existing concrete masonry block and locate vent within three feet of corner. Remove existing adjacent vent and replace with new concrete masonry block. Paint as necessary to match existing colors. Seal opening where hose bib penetrates the new concrete masonry block.



Figures VII-6.5: Site Plan



Figures VII-6.6: Block Vent Detail



Figures VII-6.7: Access Door Detail

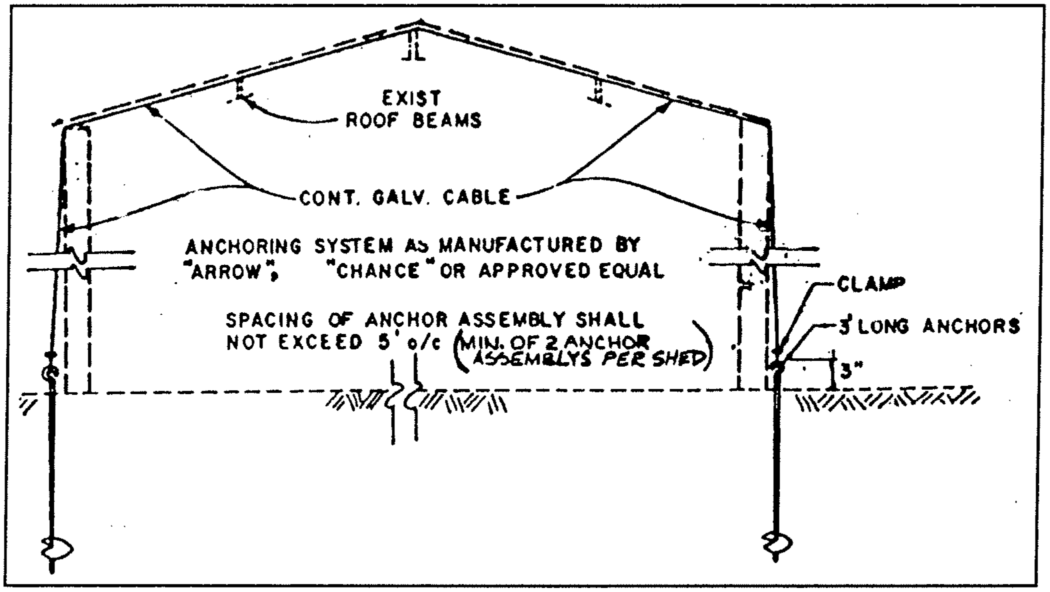


Figure VII-6.8: Anchorage Detail for Sheds

Sheds are anchored so they do not become floating debris.

DRY FLOODPROOFING

This section presents two case studies that identify procedures, methodology, and design parameters used to dry floodproof houses. Case Study #7 illustrates dry floodproofing of a house with a walkout basement in Prince George's County, Maryland. Case Study #8 illustrates dry floodproofing using a veneer wall in the Tug Fork Valley, West Virginia.

CASE STUDY #7
Dry Floodproofing a House with a Walk-out Basement
Henson Creek, Prince George's County, Maryland

The site is a two-story wood-frame house with a walk-out basement. The first floor elevation is 211.7 and the basement floor elevation is 204.0. The top of the existing retaining wall that encompassed the walkout is 207.0 (see Figure VII-7.3). The 100-year water-surface elevation (WSEL) is 206.0 based on future upstream land use conditions (see Figure VII-7.1). A flood protection elevation of 207.0 was utilized in this design.

The foundation consists of a full basement with a walkout on the Henson Creek side of the house. The existing grade varies in elevation along the foundation wall where the highest elevation occurs in the front and slopes down toward the walkout (see Figure VII-7.4).

Engineering Analysis Summary

The foundation walls were checked for structural adequacy against the lateral pressures exerted by the soil and the floodwater (see Figure VII-7.2). The worst case, which occurs along the front, was investigated in the structural calculations similar to Case #6. The existing walls prove to be structurally sound and able to resist the lateral forces imparted by the 100-year flood.

Since the house has a walk-out basement with a finished floor 2.0 feet below the 100-year WSEL, the proposed replacement floodwall that wraps around the back of the house will have to retain the floodwater in addition to the soil. The present condition of the existing wall is questionable due to the numerous cracks in the joints and the cracks around the grouted pockets at the wood columns and unknown wall foundation conditions. Furthermore, the wall was not designed to resist the relatively high lateral forces occurring during the flood. Therefore, it was recommended that the wall be replaced with a reinforced concrete floodwall. Temporary supports will be required for the first-floor overhang during the construction of the wall. The wood columns supporting the overhanging room should bear on top of the wall with a bearing plate to distribute the column load. A step-up/step-down entrance over the wall is required for ingress and egress to the basement.

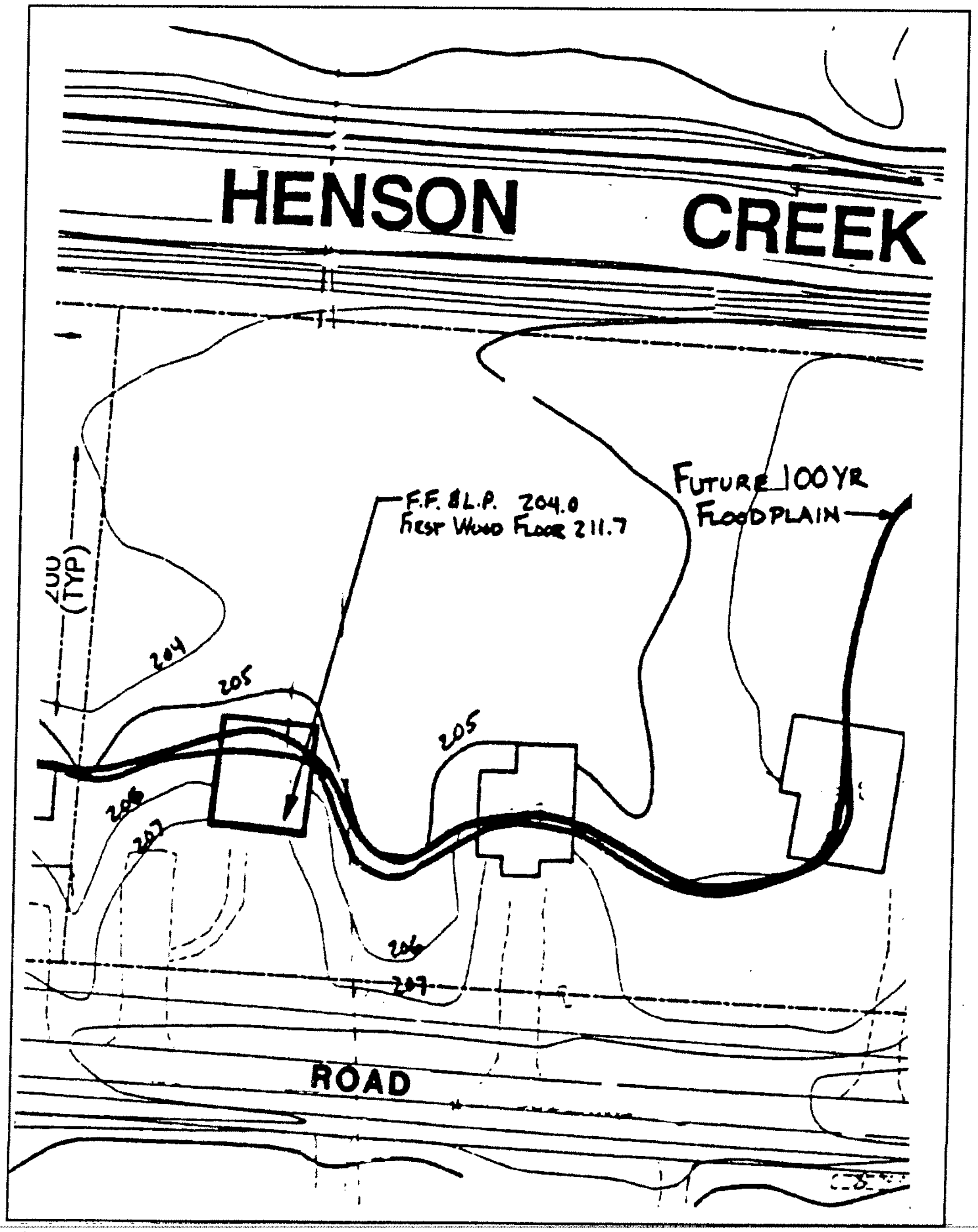


Figure VII-7.1: Location Plan

The following recommendations were developed based upon the engineering analysis:

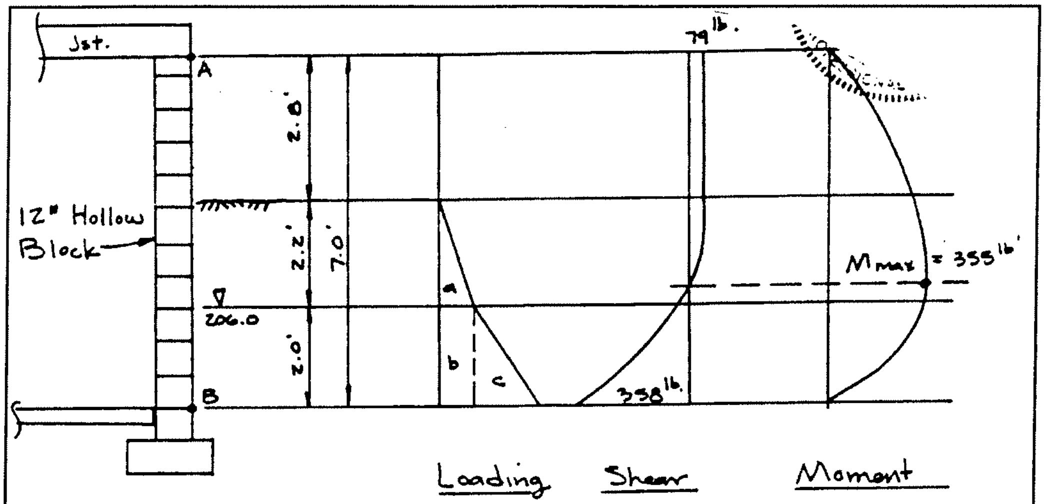
- Construct a new reinforced concrete wall to replace the existing wall. Top of wall must be at elevation 207.0 or higher.

- Apply waterproofing to the inside basement wall to prevent leakage into the living areas of the basement.

Engineering Calculations and Cost Data

The cost to dry floodproof the house was estimated in 1988 dollars at $4,800. The following calculations (see Figure VII-7.2) were applied to the existing foundation to determine if the house could be retrofitted using dry floodproofing techniques.

Demolish existing wall		$ 500
Waterproofing		$ 400
Rebuild wall		$3,900
	TOTAL	$4,800



$H_a = \frac{1}{2}(40)(2.2)^2 = 97^{lb}$ $\qquad \bar{y}_a = 2.73'$ (from 'B')

$H_b = (40)(2.2)(2.0) = 176^{lb}$ $\qquad \bar{y}_b = 1.00'$ "

$H_c = \frac{1}{2}(82)(2.0)^2 = 164^{lb}$ $\qquad \bar{y}_c = 0.67'$ "

$= 437^{lb}$

$\Sigma M_B = (97)(2.73) + (176)(1.00) + (164)(.67) - 7.0 R_A = 0$

$R_A = 79^{lb}$

$R_B = 358^{lb} < V_{allow} = (34\ psi)^{**}(70\ in^2)^{*} = 2{,}380^{lb}$ OK

Figure VII-7.2: Dry Floodproofing Calculations

Find R_A using γ_{eq}.

$$\gamma_{eq} = \frac{(437)\,2}{(4.2)^2} = 49.5 \text{ pcf.}$$

$$R_A' = \frac{(49.5)(4.2)^3}{6} \Big/ 7.0 = 87^{lb} > R_A = 79^{lb}$$

$\therefore$ the following bending moment equation is slightly conservative:

$$M_{max} = \frac{(437)(4.2)}{3(7.0)}\left[2.8 + \frac{2}{3}(4.2)\sqrt{\frac{4.2}{3(7.0)}}\right] = 355^{lb'}$$

$$f_t = \frac{355 \times 12''/'}{{}^{*}\ 190 \text{ in}^3} = 22.4 < 23.0 \text{ psi}^{**} \ \underline{\underline{OK}}$$

The addition of DL on the wall will further reduce the tension stress in the joint.

* Sectional properties from NCMA <u>Nonreinforced Concrete Masonry Design Tables</u>, p. 22

** See 'Design Criteria'

Figure VII-7.2: Dry Floodproofing Calculations (continued)

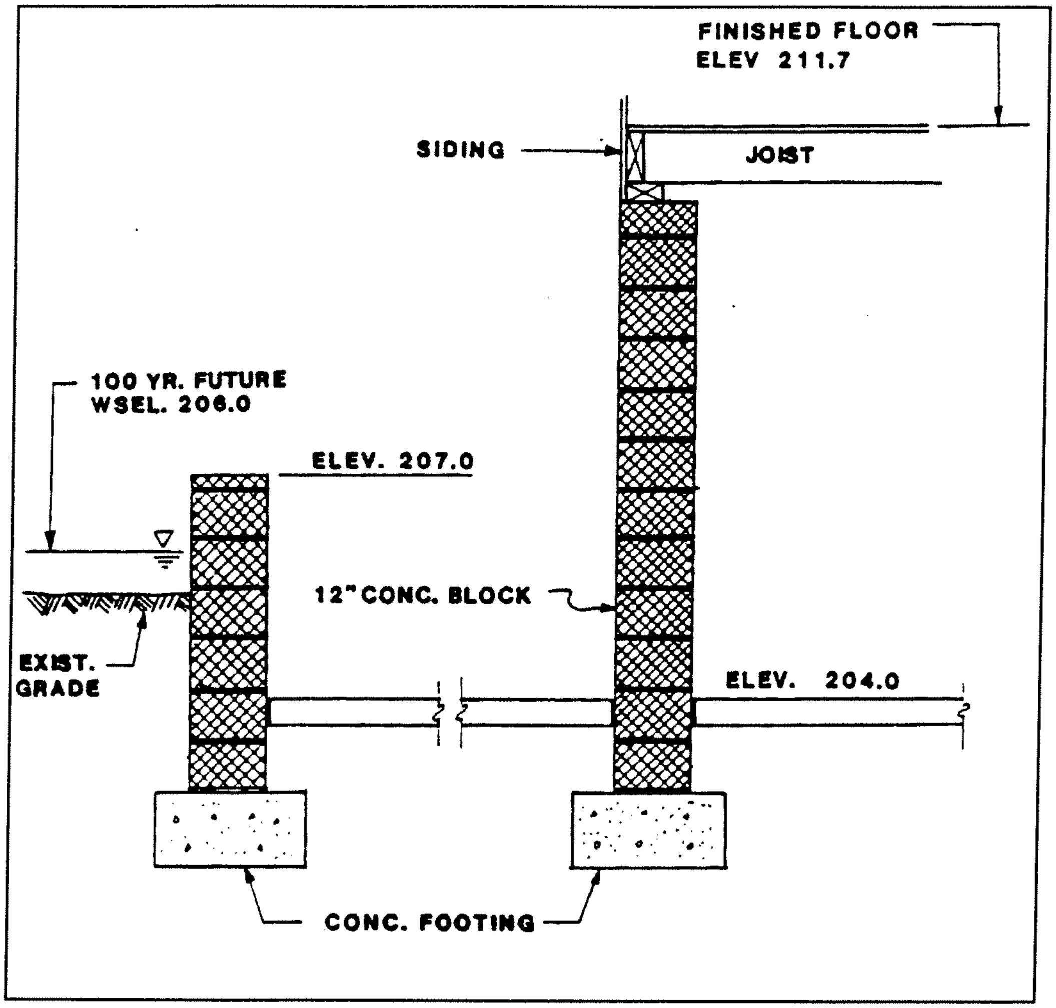


Figure VII-7.3: Preexisting Walk-out Basement Foundation Wall Detail Section

Proposed Work

See Figures VII-7.4 through VII-7.17.

Plans, Elevations, and Construction Details

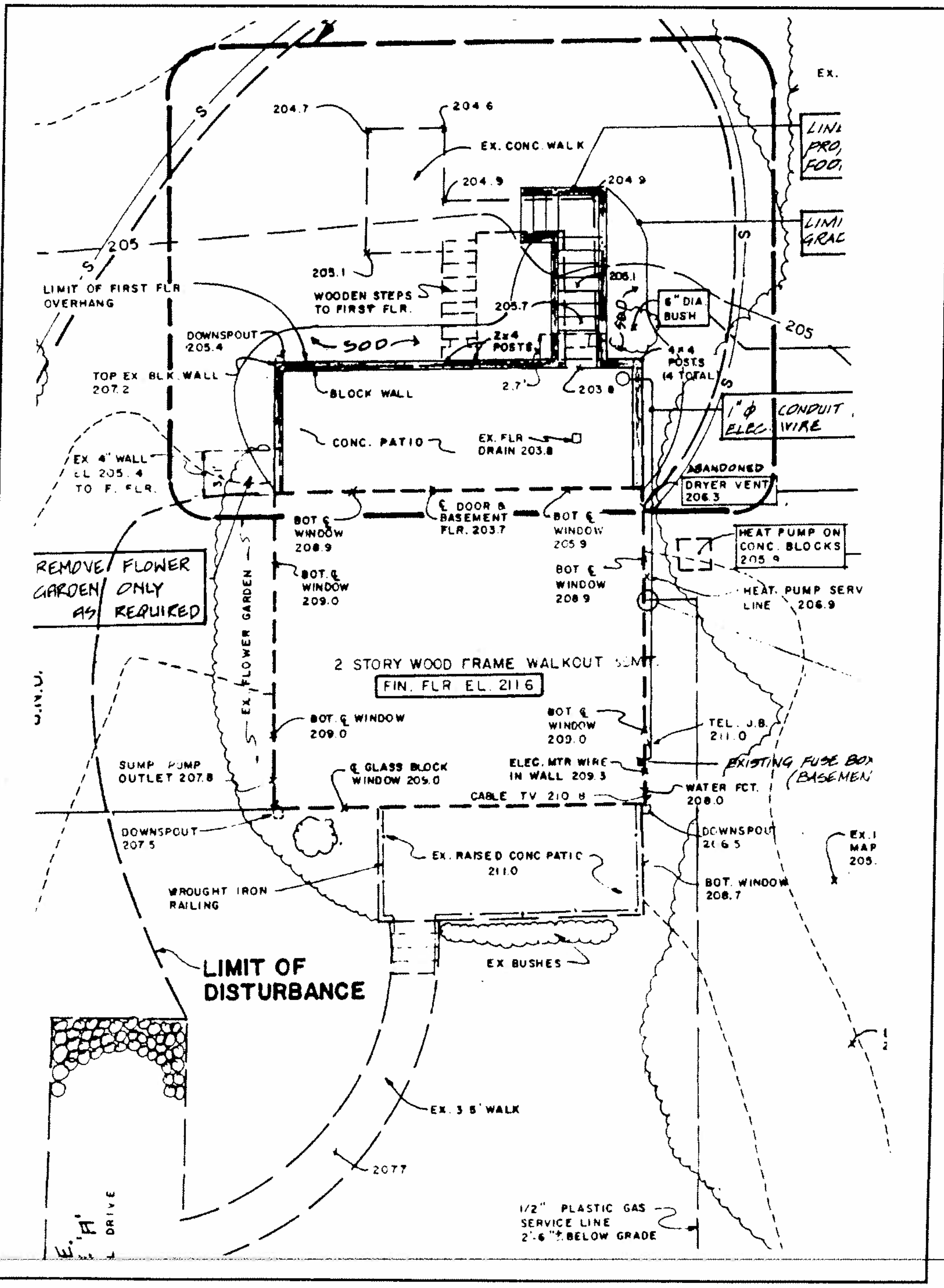


Figure VII-7.4: Site Plan

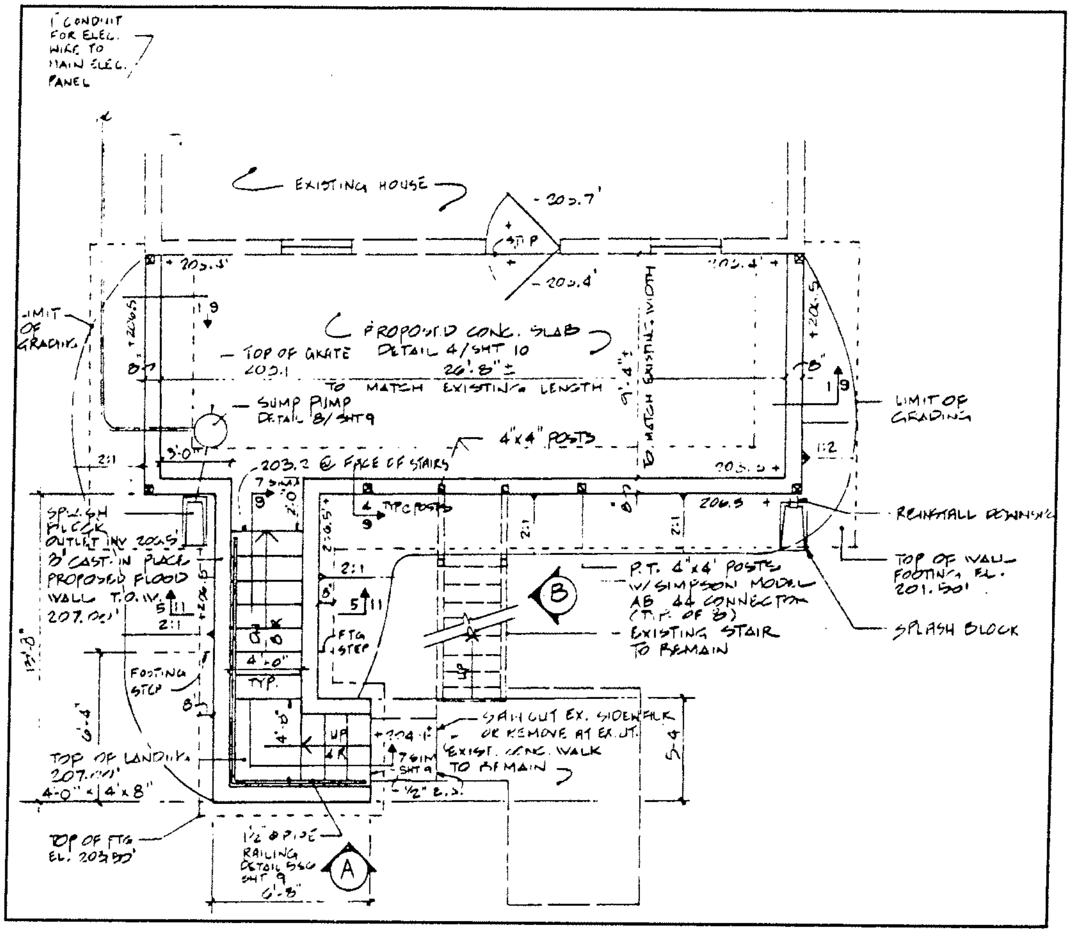


Figure VII-7.5: Concrete Patio, Replacement Floodwall, and New Access for Basement Detail

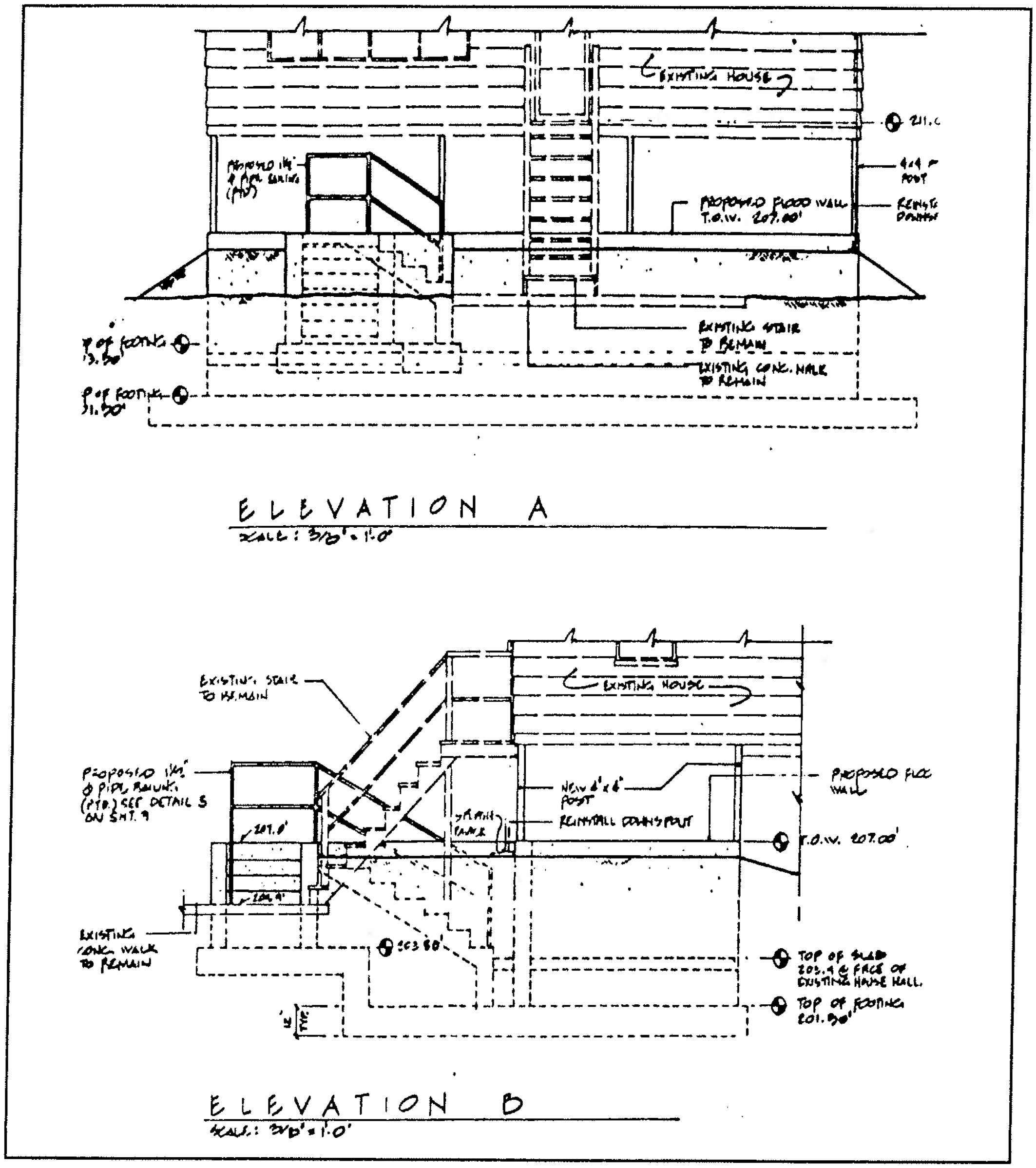


Figure VII-7.6: Step and Wall Detail Elevations

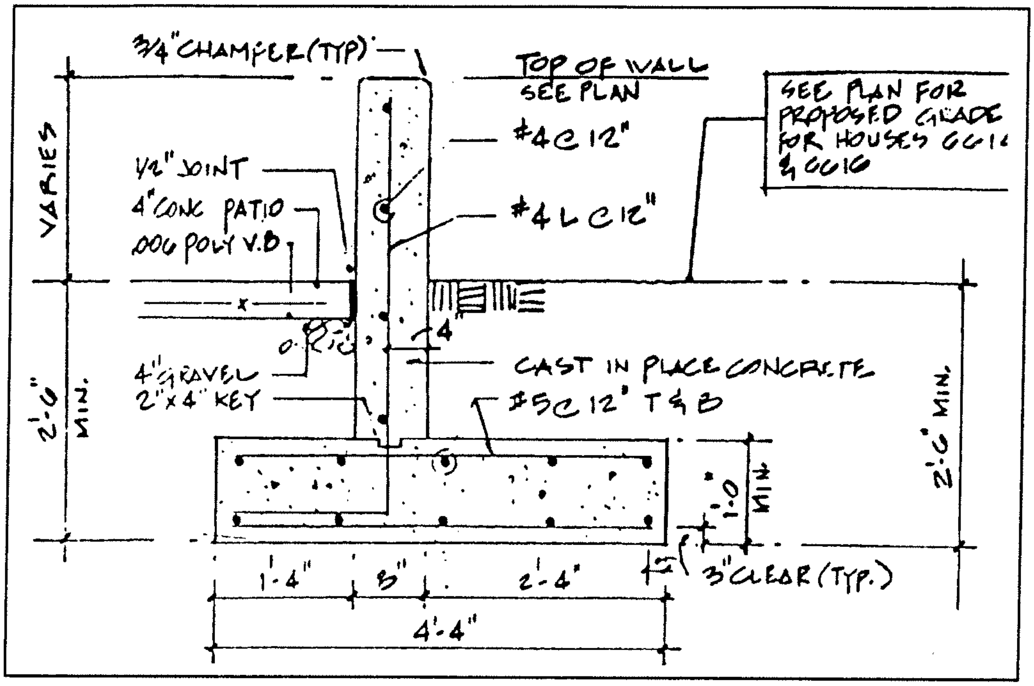


Figure VII-7.7: Concrete Floodwall Detail

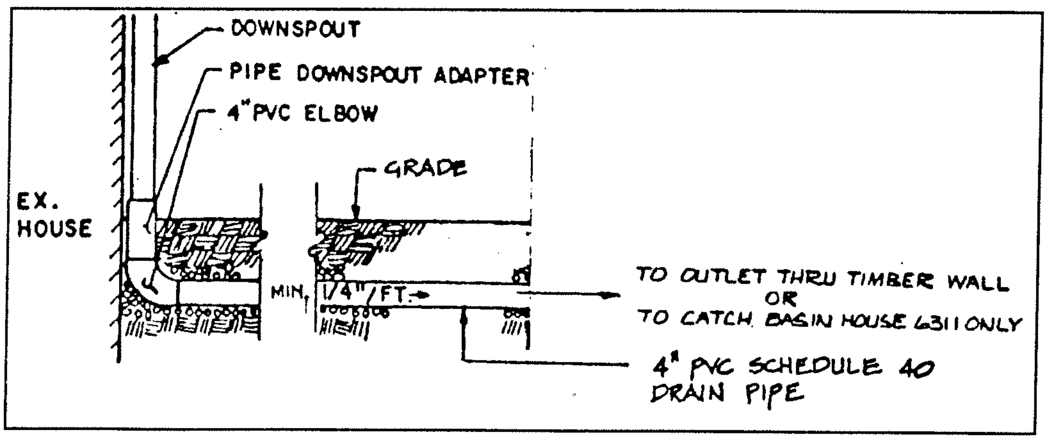


Figure VII-7.8: Downspout Connection to Drain Detail

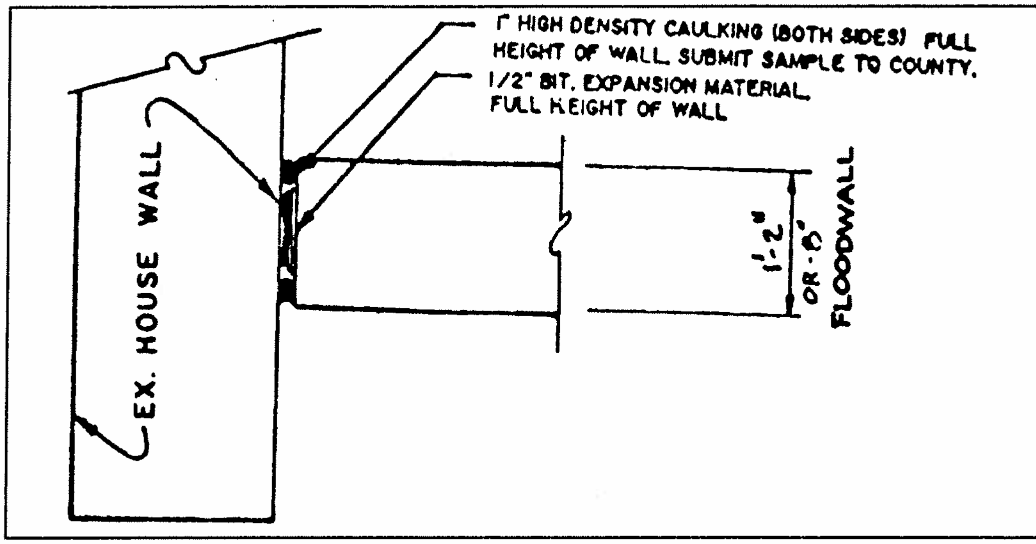


Figure VII-7.9: Floodwall Connection to House Detail

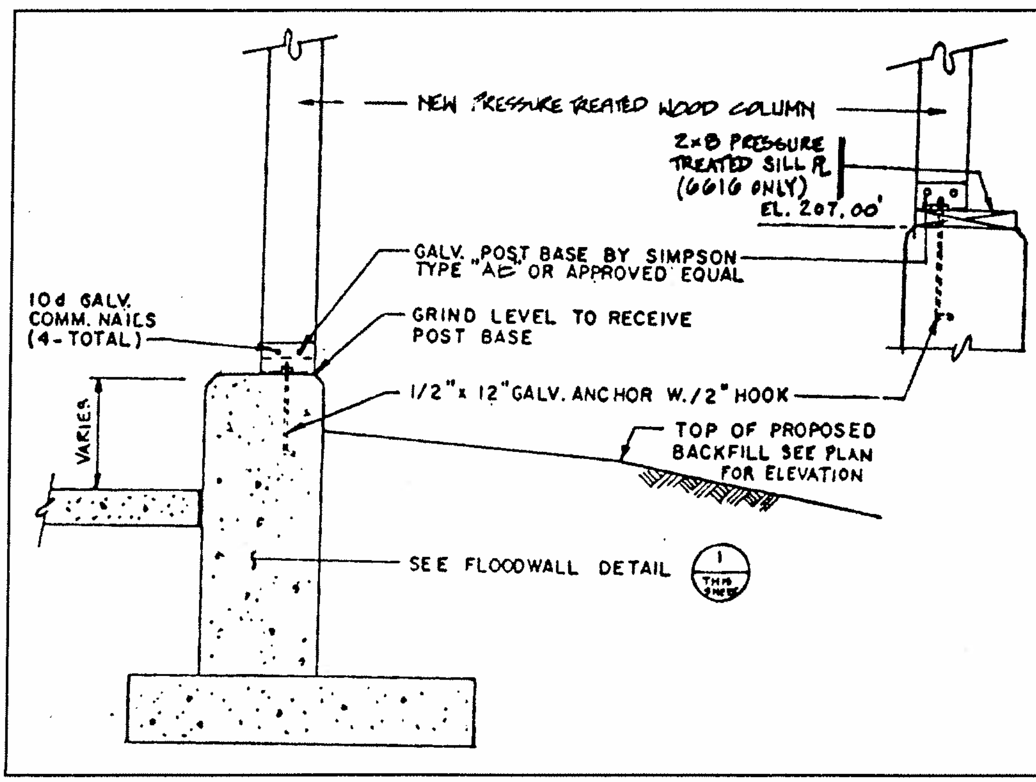


Figure VII-7.10: Floodwall Supporting Columns Detail

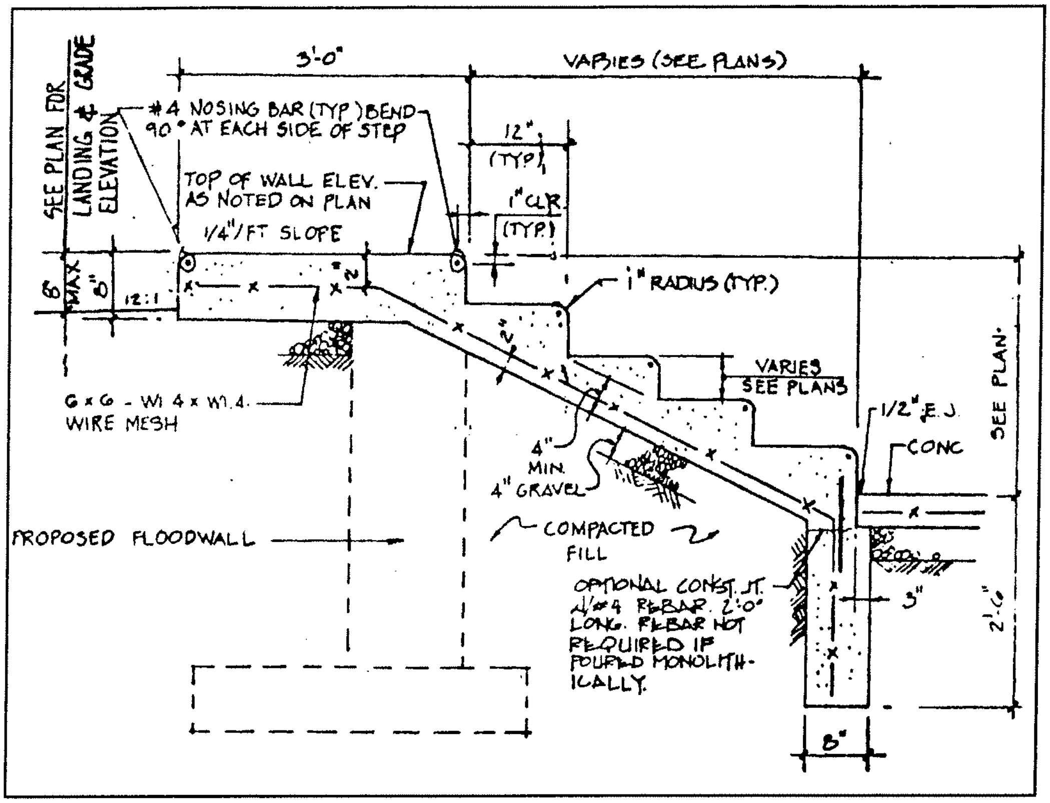


Figure VII-7.11: Step Detail

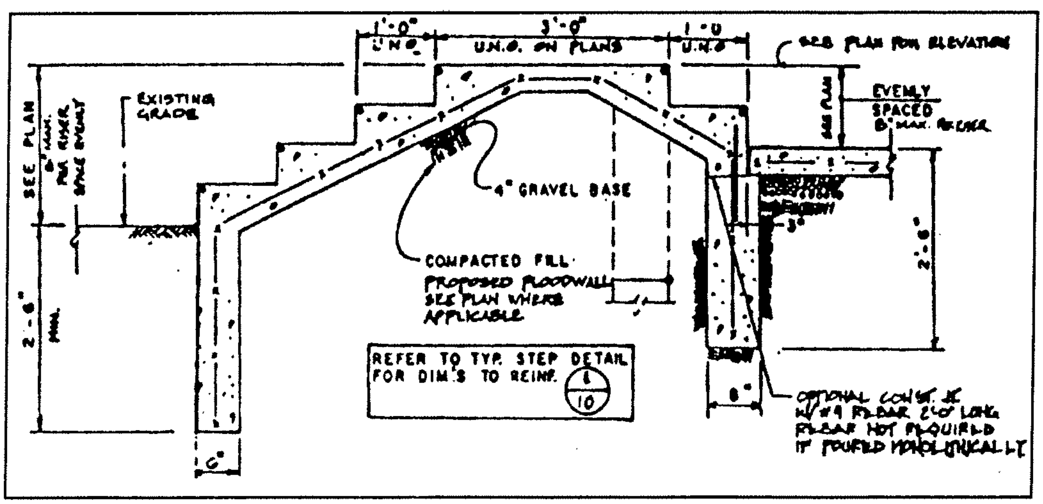


Figure VII-7.12: Step Detail

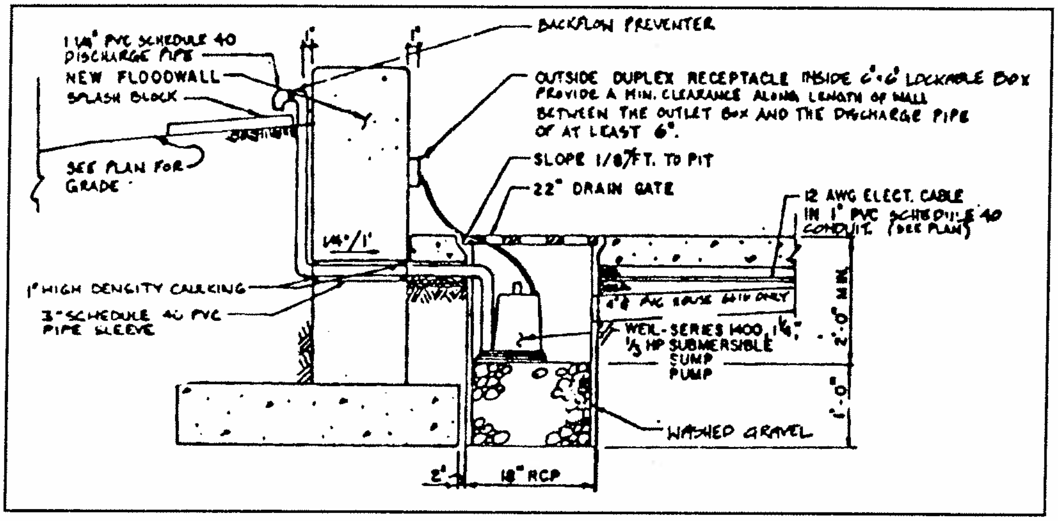


Figure VII-7.13: Sump Pump Detail

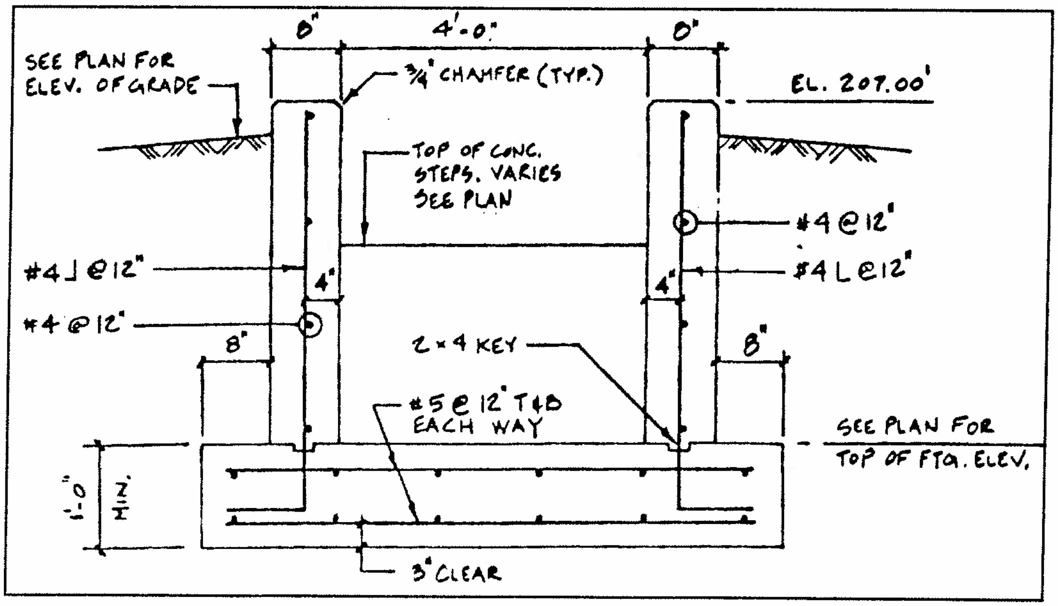


Figure VII-7.14: Stair Section

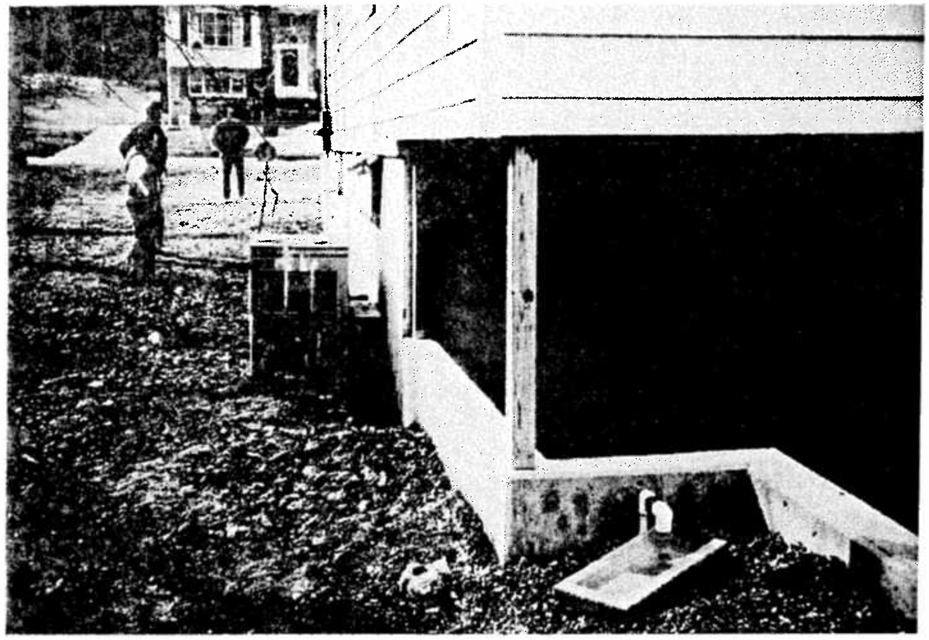


Figure VII-7.15: Air Conditioning Pad and Sump Pump

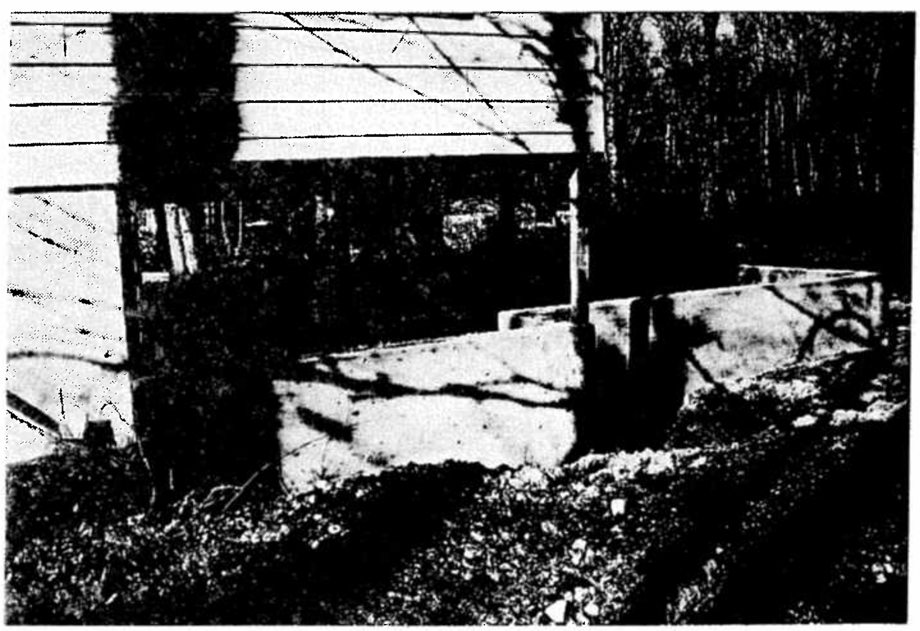
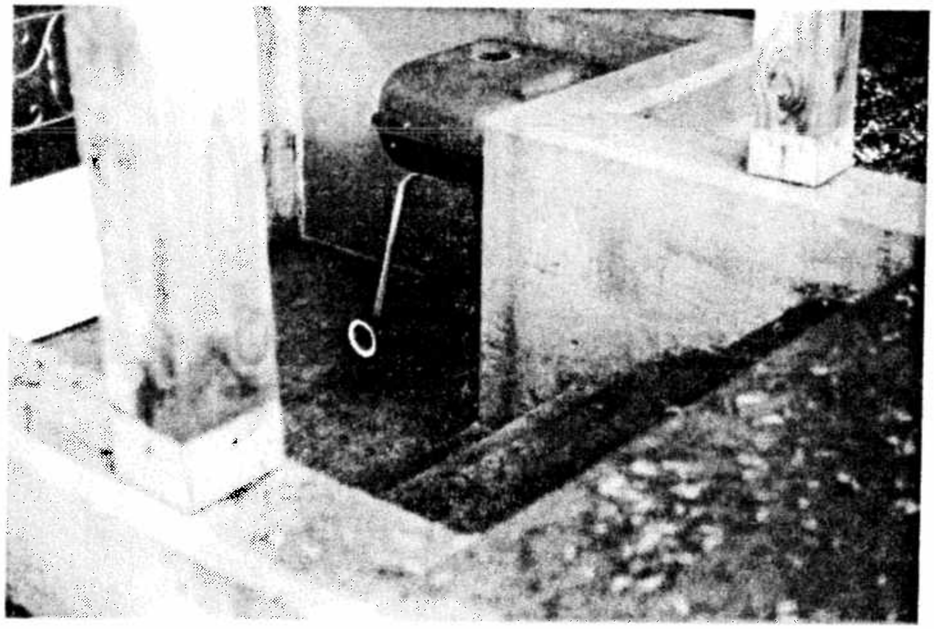


Figure VII-7.16: Floodwall and Supporting Columns



VII-7.17: Stairs and Supporting Columns

CASE STUDY #8
Veneer Wall, Dry Floodproofing
Tug Fork Valley, West Virginia

A two-story church of 1,920 square feet located within the floodway fringe experienced only 1.82 feet of flooding to the first floor area during the 1977 flood. The first floor of the church was constructed with masonry walls and the second story was wood-frame construction. The 100-year floodwater velocity at the church site was between two and three feet per second. This church was determined eligible for the retrofitting program since it met the criteria needed for construction of a veneer wall. This method has proven effective on residential structures, as well.

Veneer Wall

This type of perimeter wall is included under the category of dry floodproofing. In this category, water is prevented from entering the first floor of the structure by the use of veneers, closures, and sealants. Several factors limit the use of veneer walls for protecting structures, including:

- the inherent strength of the structure's existing perimeter walls,
- the depth of flooding at the structure,
- floodwater velocity and debris impact potential at the structure,
- size and number of closures needed to service the structure, and
- the structure owner's capability to operate and maintain the aspect of the retrofitting system that requires human intervention.

A detailed engineering analysis of the structure's walls, closures, and utilities determined that the structure could be dry floodproofed by constructing a veneer wall attached to the existing first-floor masonry wall. The owners of the church exhibited a willingness and capability to operate and maintain the veneer wall, closures, and utilities to prevent future flood damages to the structure.

The veneer wall was constructed of reinforced poured concrete. The wall was six inches thick and extended from the existing footing to an elevation one foot above the design flood (see Figure VII-8.1). The wall was attached to the existing masonry wall with metal anchors (see Figure VII-8.2), and formed rubber waterstops were installed between all concrete joints.

Aluminum flashing was installed along the top of the wall to prevent rainwater from seeping between the veneer wall and the existing masonry wall (see Figure VII-8.3).

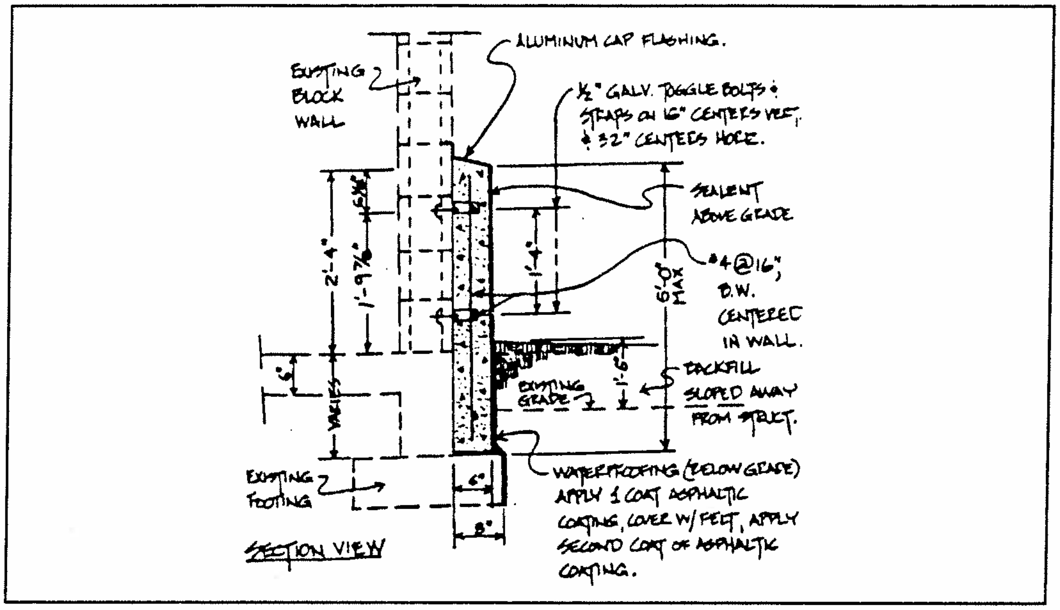


Figure VII-8.1: Veneer Wall Detail Section

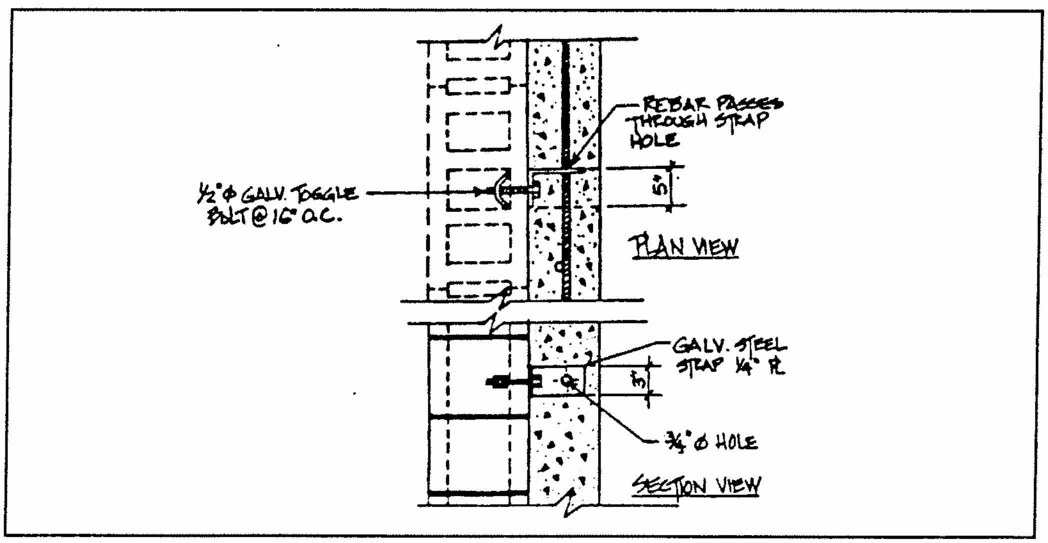


Figure VII-8.2: Veneer Wall Metal Anchor Detail Section

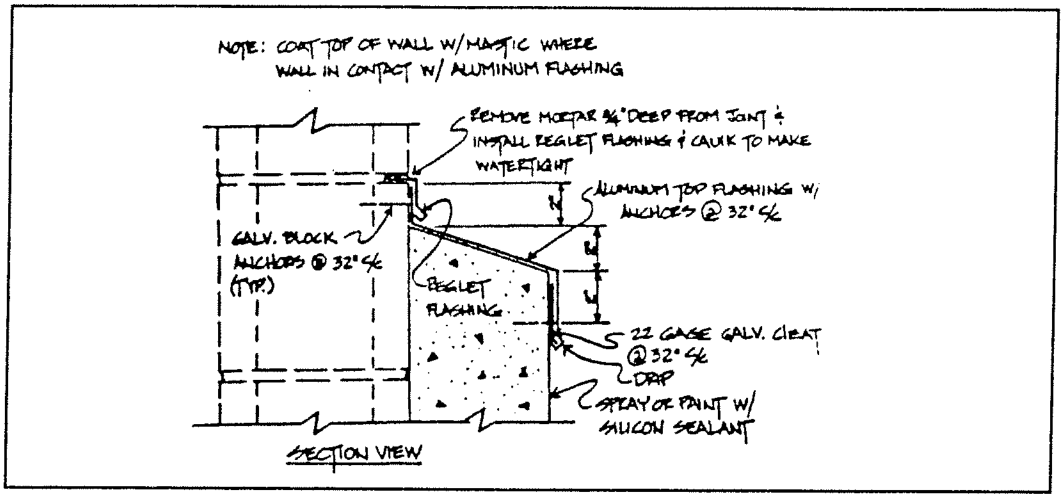


Figure VII-8.3: Aluminum Flashing Detail Section

Asphaltic waterproofing was applied to the veneer wall surface below ground and a waterproof silicone sealant was applied to the veneer wall surface above the exterior grade (see Figure VII-8.1).

Only one entrance to the first floor required a closure. The remaining door accessed an equipment room on the first floor and was shortened to avoid the need for a second closure in the veneer wall. A three-by-two-foot solid aluminum panel with perimeter seals and lock bolts was used to seal the closure (see Figure VII-8.4). The second floor was accessed by exterior concrete steps and interior steps.

An exterior air-conditioning unit was relocated onto a raised pressure-treated wood platform. A water line was relocated to avoid penetration of the veneer wall, and a valve box and gate valve were installed on the underground sewer line to prevent backflows into the first floor area.

Detailed instructions regarding the operation and maintenance of the veneer wall, closure, and utility valve were placed on wall placards both on the exterior wall next to the closure and inside the church. These items were included in the agreement executed between the church owners and the Corps of Engineers.

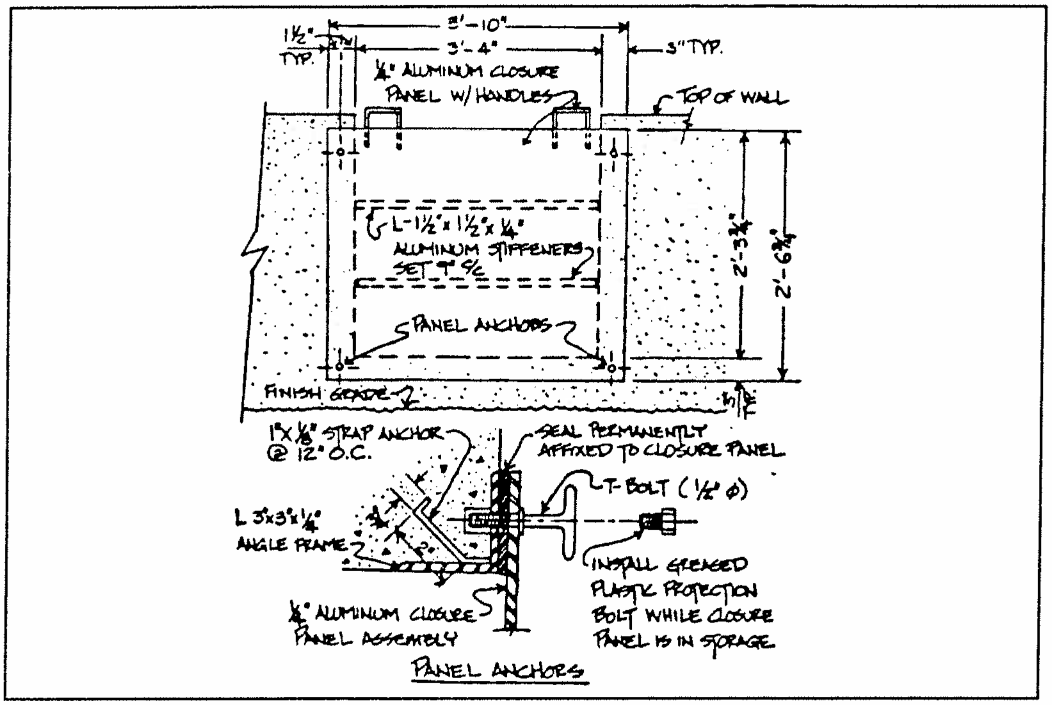


Figure VII-8.4: Watertight Closure for Opening

Construction Cost

Key factors that influence the construction cost of veneer walls include:

- height of design flood at the structure;
- type and condition of the structure walls;
- type, extent, and condition of structure footing;
- number and size of structure access closures needed;
- number, size, and location of underground utilities entering the structure; and
- permeability and bearing capacity of soils at the structure.

Additional factors that influence the cost of floodproofing include the availability of skilled contractors and competitively priced building materials. Table VII-8.1 below shows the percentage contribution to construct a veneer wall against the structure.

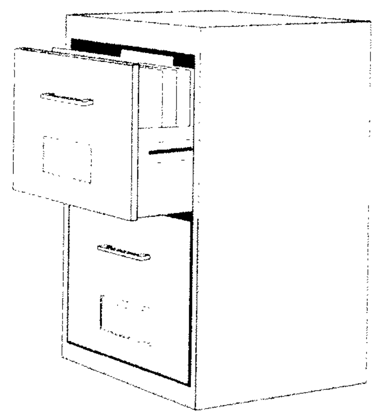
Table VII-8.1 **Floodproofing Cost for a Veneer Wall**

Construction Items	Percent of Total Construction
Site Work, Mobilization, and Cleanup	40
Concrete and Masonry	24
Metals	26
Carpentry and Finishes	7
Mechanical and Electrical	3
	100

Flood Proofing Technology in the Tug Fork Valley, U.S. Army Corps of Engineers, National Flood Proofing Committee, April 1994.

APPENDIX A

THE NATIONAL FLOOD INSURANCE PROGRAM



THE NATIONAL FLOOD INSURANCE PROGRAM (NFIP)

Flood insurance coverage is available from the NFIP to all owners and occupants of insurable property (buildings and certain contents) in participating communities. Walled and roofed structures that are principally above ground and not entirely over water may be insured. Flood insurance is available for all buildings in a participating community whether the buildings are located inside or outside of the floodplain. This coverage is available for manufactured homes that are anchored to permanent foundations. Up to 10 percent of the policy value for building coverage may apply to a detached garage or carport on the same lot. Contents within insured buildings also may be insured under separate coverage.

The purchase of flood insurance is required for buildings located in the 100-year floodplain as a condition of obtaining a federally regulated or insured mortgage or home improvement loan. NFIP flood insurance is available through private insurance companies and agents, as well as directly from the federal government. All companies offer identical coverage and rates as prescribed by the NFIP.

PRE-FLOOD INSURANCE RATE MAP (PRE-FIRM) CONSTRUCTION VERSUS POST-FLOOD INSURANCE RATE MAP (POST-FIRM) CONSTRUCTION

For flood insurance rating purposes, buildings are classified as being either pre-FIRM or post-FIRM.

- Pre-FIRM construction means construction or substantial improvement started on or before December 31, 1974, or before the effective date of the community's initial FIRM, whichever is later.

- Post-FIRM construction means construction or substantial improvement started after December 31, 1974, or on or after the effective date of the community's initial FIRM, whichever is later.

Insurance rates for pre-FIRM buildings located in Special Flood Hazard Areas are set on a subsidized basis, while insurance rates for post-FIRM buildings located in Special Flood Hazard Areas are set actuarially on the basis of designated flood hazard zones on the community's NFIP maps (FIRMs) and the elevation of the first floor of the building in relation to the expected 100-year flood level. For both pre-FIRM and post-FIRM buildings located outside a Special Flood Hazard Area, insurance rates are set actuarially, as well. This rate structure provides an incentive to property owners to elevate buildings in exchange for receiving the financial benefits of lower insurance rates. Subsequent to substantial improvements, a pre-FIRM building may become a post-FIRM building for flood insurance rating purposes. The enclosed Flood Insurance Rate Tables (Figures A-1 and A-2) provide information on costs of coverage for different buildings subject to various flooding scenarios.

HYPOTHETICAL CASE STUDY

To illustrate the impact of elevating a building on flood insurance premium rates and how the Flood Insurance Rate Tables are used, the following hypothetical example is provided:

> The limits of coverage used in these examples became effective on May 1, 2000

A family purchased a home located within a Special Flood Hazard Area (Zone AE) identified on their community's FIRM. The home was a one-floor single-family dwelling with no basement. As a condition of receiving a federally-backed mortgage, a flood insurance policy was required by the lending institution. Because the home was constructed before the initial FIRM for this community became effective, this home was rated as pre-FIRM construction. The homeowners chose to purchase the maximum amount of coverage available for the building and its contents: \$50,000 of basic coverage plus an additional \$200,000 of coverage for the building. For contents, they purchased the basic \$20,000 of coverage plus an additional \$80,000 of coverage. Thus, the total flood insurance coverage for the building and contents was \$350,000.

To determine the annual rate to purchase this coverage, the Flood Insurance Rate Tables were utilized. Because their community participated in the regular program of the NFIP and the building is pre-FIRM construction, the *Regular Program - Pre-FIRM Construction Rate Table* was utilized. In this table, the flood insurance rates for buildings located in a Special Flood Hazard Area are subsidized in that (1) they are independent of the relationship between the first floor elevation and the BFE (2) the rates are below actuarial rates. In this table, a single-family home with no basement located in Zone AE has a rate of .68/.20 listed for building coverage. This means that for every $100 of basic building coverage, the annual premium would be $0.68. For every $100 of additional building coverage, the annual premium would be $0.20. There is a separate column in the table to determine premiums for basic and additional contents coverage, in this case $0.79 and $0.36 for every $100 of coverage, respectively. Figure A-1 shows the computations of the annual flood insurance premium for this example home providing maximum coverage allowable under the NFIP.

	Coverage (Hundreds of Dollars)	Rate[1]	Annual Premium
Basic Coverage			
Building	500	$0.68	340.00
Contents	150	$0.79	118.50
			$458.50
Additional Coverage			
Building	2,000	$0.20	400.00
Contents	850	$0.36	306.00
			$706.00
Federal Policy Fee			30.00
Expense Constant			45.00
Total Premium For This Policy =			$1,239.50

[1]Annual rate per $100 of coverage. Values taken from pre-FIRM insurance rate tables dated May 1, 2000

Figure A-1: Annual Flood Insurance Premium for Sample Home (Pre-FIRM) Before Elevating

Subsequently, the home was substantially damaged in a flood. When repairing the building, the owner elevated the first floor to the BFE shown on the community's FIRM in order to comply with the community's floodplain management ordinance. Because the building was substantially improved, it was now considered post-FIRM for flood insurance rating purposes. Thus, the flood insurance premium was adjusted accordingly. Because the building is now considered post-FIRM construction, the new premium is determined actuarially based on the elevation of the first floor relative to the BFE. The computations for the new premium are made in a manner similar to that used for pre-FIRM, except that the *Regular Program Post-FIRM-Construction Rate Tables* were used. Note that there are separate tables for building and contents coverage for post-FIRM construction located in Zone AE. Because the first floor of this home was elevated to the BFE, the computations for the new premium are shown in Figure A02, assuming the same maximum level of coverage that was previously purchased.

	Coverage (Hundreds of Dollars)	Rate[1]	Annual Premium
Basic Coverage			
Building	500	$0.61	305.00
Contents	150	$0.89	133.50
			$438.50
Additional Coverage			
Building	2,000	$0.08	160.00
Contents	850	$0.12	102.00
			$262.00
Federal Policy Fee			30.00
Expense Constant			45.00
Total Premium For This Policy =			$775.50

[1]Annual rate per $100 of coverage. Values taken from pre-FIRM insurance rate tables dated May 1, 2000

Figure A-2: Annual Flood Insurance Premium for Sample Home (Pre-FIRM) Before Elevating

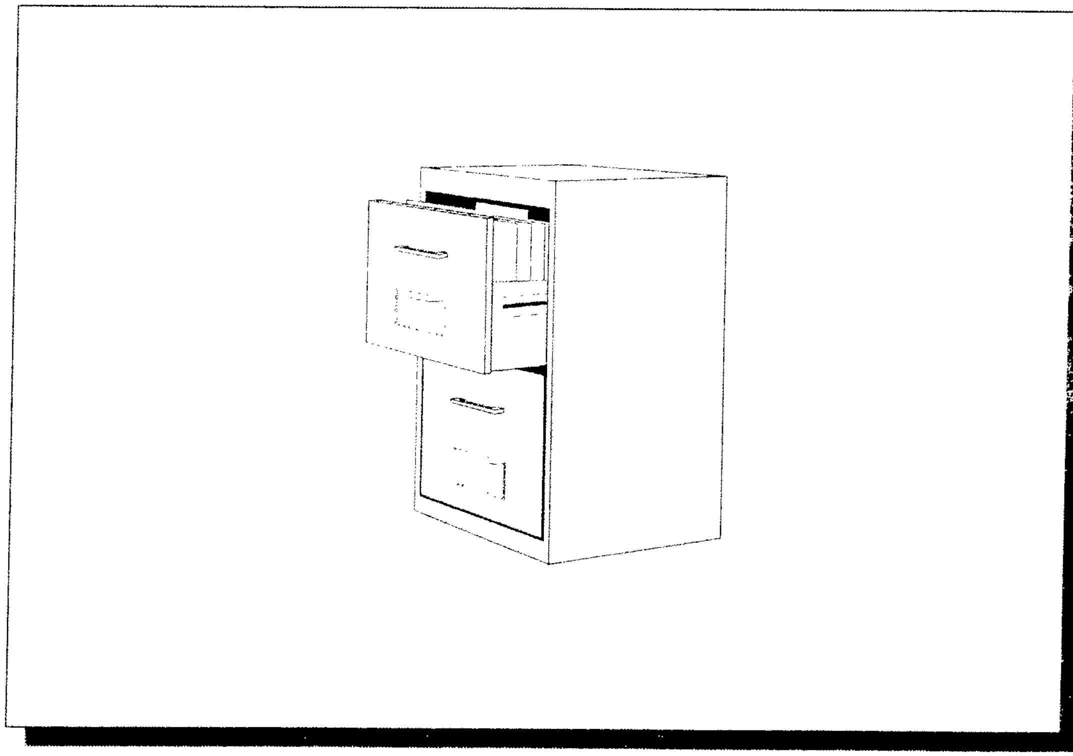
By elevating the home to meet NFIP requirements, the property owners were able to reduce the annual flood insurance premium by $464.00. Over the life of a mortgage, this can be a significant savings. Elevating the structure higher would have resulted in an additional reduction in the annual flood insurance premium.

Elevating a building above the BFE does not eliminate the requirement to purchaase flood insurance but will reduce the insurance rate. Even though a building is elevated, the potential exists for damage to the foundation system which, in turn, could result in structural damage to the home. This is one reason why continued flood insurance is required.

Many flood-prone homes were built prior to their community's adoption of NFIP regulations. Therefore, those flood-prone homes do not meet current floodplain management standards. Consequently, owners wanting to substantially modify, improve, repair, or retrofit their home as a result of preference or damage are subject to the NFIP substantial improvement (substantial damage) requirements discussed previously.

APPENDIX B

GLOSSARY OF TERMS



GLOSSARY OF TERMS

A Zone	See **Special Flood Hazard Area**.
Alluvial Fan	Area of deposition where steep mountain drainages empty into valley floors, usually in arid regions. Flooding in these areas often includes characteristics that differ from those in riverine or coastal areas.
Anchor	A series of methods used to secure a structure to its footings or foundation walls so that it will not be displaced by forces acting on the structure.
Armor	To protect fill slopes from erosion or scouring by floodwaters. Techniques of armoring include the use of riprap, vegetation, gabions, or concrete mats.
Backflow Valve	See **Check Valve**.
Base Flood Elevation (BFE)	The flood elevation having a one-percent chance of being equaled or exceeded in any given year. The BFE is determined by statistical analysis for each local area and designated on the Flood Insurance Rate Maps. The BFE is **also known as the 100-Year Flood Elevation**.
Berm	A bank or mound of earth, usually placed against a foundation wall.
Borrow Area	An area where material has been excavated for use as fill at another location.
Building Code	Regulations adopted by local governments that establish standards for construction, modification, and repair of buildings and other structures.
Caulking	Flexible material used to fill joints in a structure, such as around windows or doors, which is able to resist the passage of moisture.

Check Valve	A type of valve that allows water to flow one way, but automatically closes when water attempts to flow in the opposite direction.
Closure	A shield made of strong material, such as steel, aluminum, or plywood, used to temporarily fill gaps in floodwalls, levees, or sealed structures and protect against water entrance through areas that have been left open for day-to-day convenience at entrances such as doors and driveways.
Coastal High-Hazard Area	Designated as **V Zone** on Flood Insurance Rate Maps, this is the portion of the coastal floodplain subject to storm-driven velocity waves of three feet or more in height.
Column	Upright support units for a building, set in pre-dug holes and backfilled with compacted material. They are also known as posts, although columns are usually of concrete or masonry construction.
Concrete Masonry Unit (CMU)	Block of concrete used in construction.
Crawl Space	Low space below the first floor of a house, where there has not been excavation deep enough for a basement, but where there is often access for pipes, ducts, and utilities.
Debris Impact Loads	Sudden loads induced on a structure by debris carried by flood-water. Though difficult to predict, impact loads must be considered when floodproofing a structure.
Design Flood Elevation (DFE)	The height, in feet, above NGVD or NAVD to which floodproofing measures are designed. It is normally the sum of the expected flood elevation plus freeboard.
Dry Floodproofing	A retrofitting method used in areas of low-level flooding to completely seal a home against water, by making the walls substantially impermeable to the passage of water. Also referred to as sealing in this manual.

Elevation	The raising of a structure to place the lowest floor at or above the flood protection elevation on an extended support structure.
Existing Construction	For floodplain management purposes; a structure already existing or under construction prior to the effective date of a community's floodplain management regulations. For flood insurance purposes, a structure for which the "start of construction" commenced before the effective date of the FIRM or before January 1, 1975, for FIRMs effective before that date.
Extended Foundation	The construction of additional height of foundation wall above existing foundation walls in order to elevate a structure to or above the design flood elevation.
Federal Emergency Management Agency (FEMA)	Agency created in 1978 to provide a single point of accountability for all federal activities related to disaster mitigation and emergency preparedness, response, and recovery.
Federal Insurance Administration (FIA)	The governmental unit, a part of FEMA, that administers the flood insurance aspects of the National Flood Insurance Program.
Fill	Material such as earth, clay, or crushed stone that is placed in an area and compacted to increase ground elevation.
Flash Flood	A flood that crests in a short length of time and is often characterized by high velocity flow. It is often the result of heavy rainfall in a localized area.
Flood (For NFIP flood insurance policies)	1) A partial or complete inundation of normally dry land areas from the overland flood of a lake, river, stream, ditch, etc.; 2) the unusual and rapid accumulation or runoff of surface waters; and 3) mudflows or the sudden collapse of shoreline land.
Flood Depth	The height difference between the flood elevation and the lowest grade adjacent to the structure.

Flood Fringe	That portion of the floodplain that lies beyond the floodway and serves as a temporary storage area for floodwaters during a flood. This section receives waters that are generally shallower and of lower velocities than those of the floodway.
Flood Hazard Boundary Map (FHBM)	The official map of a community, issued by FEMA, where the boundaries of the flood, mudslide, and related erosion areas having special hazards have been designated as Zones A, M, and/or E.
Flood Insurance Rate Map (FIRM)	The official map of a community issued by FEMA that shows the BFE, along with the special hazard areas and the risk premium zones applicable to the community.
Flood Insurance Study (FIS)	A study performed by any of a variety of agencies and consultants to delineate the special flood hazard areas, base flood elevations, and risk premium zones. The study is funded by FEMA and is based on detailed site surveys and analysis of the site-specific hydrologic and hydraulic characteristics.
Floodplain	Normally dry land adjacent to a body of water, such as a river, stream, lake, or ocean, that is susceptible to inundation by floodwaters.
Floodplain Management	A program of corrective and preventive measures for reducing flood damage, including but not limited to flood control projects, floodplain land-use regulations, retrofitting (or floodproofing) of buildings, and emergency preparedness plans.
Floodproofing Design Depth	The height difference between the DFE and the lowest grade adjacent to the structure.
Floodproofing	Any combination of measures taken on a new or existing structure for reducing or eliminating flood damage to a structure. For existing structures, it is **also known as retrofitting**.

Floodwall	A constructed barrier of resistant material, such as concrete or masonry block, designed to keep water away from a structure.
Floodway	The central portion of the floodplain that carries the greatest portion of the waterflow in a flood. Obstructions in the floodway will result in increased flood levels upstream.
Footing	The enlarged base of a foundation wall, pier, or column designed to spread the load of the structure so that it does not exceed the soil bearing capacity.
Foundation Walls	A support structure that connects the foundation, or the building substructure, to the main portion of the building, or the building superstructure.
Freeboard	An additional amount of height used as a factor of safety in determining the design height of a flood protection measure to compensate for unknown factors, such as wave action and the hydrologic effect of urbanization. Certain guidelines and restrictions apply for establishing freeboard on levees and floodwalls in Special Flood Hazard Areas.
Human Intervention	The required presence and active involvement of people to enact any type of flood protection measure prior to flooding.
Hydrodynamic Loads	Forces imposed on an object, such as a structure, by water moving around it. Among these loads are positive frontal pressure against the structure, drag effect along the sides, and negative pressure on the downstream side.
Hydrostatic Loads	Forces imposed on a surface, such as a wall or floor slab, by a standing mass of water. The water pressure increases with the square of the water depth.
Interior Grade Beam	A section of a floor slab that has a thicker section of concrete to act as a footing to provide stability under load-bearing or critical structural walls.

Levee	A barrier of compacted soil designed to keep floodwater away from a structure.
Lift	A layer of soil that is compacted before the next layer is added in the construction of a fill pad or levee.
Mean Sea Level	The average height of the sea for all stages of the tide, usually determined from hourly height observations over a 19-year period on an open coast or in adjacent waters having free access to the sea.
Mitigation Directorate	The governmental unit, a part of FEMA, that administers the floodplain management aspects of the National Flood Insurance Program.
National Flood Insurance Program (NFIP)	The federal program created by an act of Congress in 1968 that makes flood insurance available in communities that enact satisfactory floodplain management regulations.
One Hundred (100)-Year Flood	The flood elevation that has a one-percent chance of being equaled or exceeded in any given year. It is also known as the BFE.
Openings	See **Venting**.
Permeability	The property of soil or rock that allows water to pass through it.
Phreatic Surface	The upper boundary of a subsurface area which contains saturated soil.
Pier	An upright support member of a building with a height limited to a maximum of three times its least lateral dimension. It is designed and constructed to function as an independent structural element in supporting and transmitting building and environmental loads to the ground.

Pile An upright support member of a building usually long and slender in shape, driven or jetted into the ground by mechanical means and primarily supported by friction between the pile and the surrounding earth.

Post Long upright support units for a building, set in pre-dug holes and backfilled with compacted material. Each post usually requires bracing to other units. They are also known as **columns**, although posts are usually made of wood.

Regulatory Floodway As referenced in a floodplain management ordinance, this is the portion of the floodplain needed to discharge the 100-year flood without increasing the flood elevation by more than a designated height; under the NFIP, this is one foot. Severe restrictions apply to development within regulatory floodways.

Relocation Moving a structure from a flood-prone area to a new location, normally to one where there is no threat of flooding.

Retrofitting Floodproofing measures taken on an existing structure.

Riprap Broken stone, cut stone blocks, or rubble that is placed on slopes to protect the slopes from erosion or scouring caused by floodwaters or wave action.

Scouring The localized erosion around flow obstructions caused by the entrainment of soil or sediment.

Slab-on-Grade A structural design where the first floor sits directly on a poured concrete slab, which sits directly on the ground.

Special Flood Hazard Area An area having a special flood hazard and shown on an FHBM or FIRM as Zones A, A0, A1-30, AE, AR, A99, V0, V1-30, VE, V, M, or E.

Stile A set of stairs to allow access over an obstruction, such as a floodwall.

Structural Mat Slab The concrete slab of a building that includes structural reinforcement to help support the building's structure.

Substantial Damage Damage of any origin sustained by a structure whereby the cost of restoring the structure to its before-damaged condition would equal or exceed 50 percent of the value of the structure before the damage occurred.

Substantial Improvement Any reconstruction, rehabilitation, addition, or other improvement of a structure, the cost of which equals or exceeds 50 percent of the value of the structure before the "start of construction" of the improvement. This term includes structures which have incurred "substantial damage," regardless of the actual repair work performed. The term does not, however, include either:

1.) Any project for improvement of a structure to correct existing violations of state or local health, sanitary, or safety code specifications which have been identified by the local code enforcement official and which are the minimum necessary to assure safe living conditions, or

2.) Any alteration of a "historic structure" provided that the alteration will not preclude the structure's continued designation as a "historic structure."

Venting A system designed to allow floodwaters to enter an enclosure, usually the interior of foundation walls, so that the rising water does not create a dangerous differential in hydrostatic pressure. This is usually achieved through small openings in the wall, such as a missing or rotated brick or concrete block, or a small pipe. Also known as **openings**.

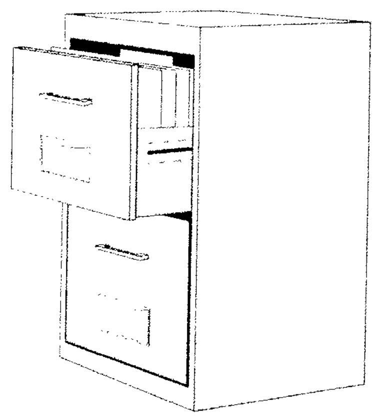
V Zone See **Coastal High-Hazard Area**.

Watershed An area that drains to a single point. In a natural basin, this is the area contributing flow to a given place or stream.

Zero Flood Depth The elevation of the lowest finished floor of a structure.

APPENDIX C

GLOSSARY OF RESOURCES



GLOSSARY OF RESOURCES

This appendix presents information on resources available to the engineer, code official, or architect interested in floodproofing. Recommendations for establishing a basic retrofitting library, information on programs and organizations that can provide assistance, and a bibliography of references utilized in this manual are included. Much of this information was taken from *Flood Proofing: Techniques, Programs and References*, prepared by the U.S. Army Corps of Engineers National Flood Proofing Committee in February 1991.

THE BASIC RETROFITTING LIBRARY

This section lists readily available references that form a basic floodproofing library. People interested in more detailed information on this subject are encouraged to obtain copies of these publications, as they cover most of the technical and programmatic aspects of retrofitting. They are listed below by agency source. Single copies of USACE and FEMA publications are free.

The next section discusses how to obtain more references on specific topics. State floodplain management coordinators usually know of any additional publications that may be available from state and local offices.

PUBLICATIONS AND SOURCES

Order the following publications from the **U.S. Army Corps of Engineers, Attn: ECW-PF20 Massachusetts Avenue, NW, Washington, D.C. 20314**

Flood Proofing Regulations, U.S. Army Corps of Engineers, Pittsburgh District, 1992, 96 pages (Corps publication EP 1165 2-314). The definitive work by the Corps of Engineers that provides construction specifications for retrofitting new buildings. It includes detailed lists of materials for areas to be wet floodproofed. The manual is organized to facilitate easy adoption by reference to a building code and provides both technical data and guidelines for ordinance administration. Illustrated with line drawings. Note: This document supersedes EP 1165-2-314 dated June 1972.

Flood Proofing Systems & Techniques, U.S. Army Corps of Engineers, L.N. Flanagan, editor, 1984, 100 pages. An illustrated, easy-to-read review of 40 different buildings that have been elevated, dry and wet floodproofed, leveed, or otherwise protected. Buildings include new construction and retrofitted houses, businesses, schools, office buildings, and factories. Narrative includes costs. Many examples include photos of flooding.

Flood Proofing: Techniques, Programs and References, U.S. Army Corps of Engineers, National Flood Proofing Committee, with French and Associates, Ltd., February 1991, 22 pages. This report addresses retrofitting techniques and government retrofitting programs, references, and terminology. It presents a general overview of retrofitting measures and provides the reader with information on government agencies that offer more specific assistance and detailed retrofitting information.

Flood Proofing: How to Evaluate Your Options, U.S. Army Corps of Engineers and National Flood Proofing Committee, July 1993, 55 pages. This document was prepared to help answer the question "should floodproofing be used?" It is intended as a tool to assist in the preliminary evaluation of whether floodproofing is appropriate and what may be the best floodproofing measure to consider. It includes an introduction to floodproofing, the various measures, factors to consider, flooding characteristics, and the thought process for evaluating physical, economic, and other factors influencing the floodproofing decision. Finally, an appendix provides a detailed explanation on how to perform an economic analysis comparing flood proofing benefits with floodproofing costs.

Raising and Moving the Slab-on-Grade House with Slab Attached, U.S. Army Corps of Engineers and National Flood Proofing Committee, 1990, 28 pages. This report presents an overview of the raising and relocation process including advantages, methods and techniques, the steps involved, foundation design consideration, and costs. A photographic study of jobs in process is also included.

Local Flood Proofing Programs, U.S. Army Corps of Engineers and National, Flood Proofing Committee, June 1994, 54 pages.

A Flood Proofing Success Story Along Dry Creek at Goodlettsville, Tennessee, U.S. Army Corps of Engineers and National Flood Proofing Committee, September 1993, 20 pages. This report documents a successful floodproofing project where 19 homes were raised in place. Included are detailed descriptions of the homes involved, implementation procedures, and project costs.

Flood Proofing Technology in Tug Fork Valley, West Virginia and Kentucky, U.S. Army Corps of Engineers and National Flood Proofing Committee, August 1993, 32 pages. This report documents elevation and dry floodproofing actions taken to reduce flooding in the Tug Fork Valley. Included are design details, cost information, and examples from the 136 homes that were floodproofed.

Order the following publications from the **Federal Emergency Management Agency, Attn: Publications, P.O. Box 2012, Jessup, Maryland, 20794-2012.**

Design Manual for Retrofitting Flood-Prone Residential Structures, Federal Emergency Management Agency, 1986, 265 pages (FEMA 114). An extensive review that discusses all aspects of protecting an existing house from flood damage. The book has many drawings and photographs. Each chapter covers a different technique with an introduction and sections on considerations (e.g., flood hazard, building type, regulatory restrictions), cost, and technical design criteria.

FEMA Technical Bulletins, Federal Emergency Management Agency, 1993. FEMA has developed seven technical bulletins providing guidance on *Openings in Foundation Walls* (TB #1-93), *Flood Resistant Material Requirements* (TB #2-93), *Non-Residential Floodproofing Requirements* (TB #3-93), *Elevator Installation* (TB #4-93), *Free-of-Obstruction Requirements* (TB #5-93), *Below-Grade Parking Requirements* (TB #6-93) and, *Wet Floodproofing Requirements* (TB #7-93). Refer to the bibliography (page C-22) for a complete reference on each Technical Bulletin.

Elevated Residential Structures, Federal Emergency Management Agency, 1984, 135 pages (FEMA 54). A review of how to build an elevated building. Concepts, examples, and performance criteria are given, but technical specifications are not. Numerous examples are discussed with architectural drawings and photographs. Cost analyses are covered and calculation forms are included. Sources of information and assistance are listed.

Flood Proofing Non-Residential Structures, Federal Emergency Management Agency, 1986, 200 pages (FEMA 102). An overview of retrofitting new and existing buildings designed to familiarize the reader with a variety of techniques. Retrofitting is divided into two parts: permanent (elevation, dry floodproofing, and levees and floodwalls) and emergency wet floodproofing. There are many drawings and photos to illustrate key points. Selection processes, case studies, sources of assistance, and performance criteria are also covered.

Homeowner's Guide to Retrofitting: Six Ways to Protect Your House from Flooding, Federal Emergency Management Agency, 1986, 117 pages (FEMA 312). A guide prepared specifically for homeowners who want to know how to protect their homes from flood damage. The book provides an overview, basic techniques, and cost information on various methods of retrofitting new or existing residential buildings. Assists homeowners in selecting design professionals to perform the appropriate retrofitting work.

Protecting Building Utiities from Flood Damage, Federal Emergency Agency, 1999 (FEMA 348). This manual presents the principles and practices for the design and construction of flood resistant building utility systems. The document provides guidance on methods of elevating, relocating, and protecting the components of HVAC, fuel, electrical, sewage, and potable water systems to prevent flood damage. New construction and retrofitting of existing structures are discussed. Photographs and figures are used throughout.

Above the Flood: Elevating Your Floodprone House, Federal Emergency Management Agency, 2000 (FEMA 347). This manual presents the principles and practices of elevating flood-prone residential structures. The document provides guidance on three techniques for elevating houses to prevent flood damage. Detailed figures, multiple case studies for each technique, and sources of additional information are also included.

Coastal Construction Manual, Federal Emergency Management Agency, 2000 (FEMA 55, Third Edition). A three-volume technical document that presents the principles and practices of planning, siting, designing, constructing, and maintaining residential buildings in coastal areas. Volume 1 and 2 provide guidance on the design and construction of coastal residential structures (single-family, townhouses, and low-rise multi-family) able to resist flood, wind, erosion, and earthquake damage. New construction and retrofitting of existing structures are discussed. Photographs and figures are used throughout. Detailed appendices (Volume 3) include FEMA technical bulletins, design details, and additional sources of information.

Order the following publications from the **Association of State Floodplain Managers, Attn: Publications, P.O. Box 2051, Madison, WI 53701-2051.**

Floodplain Management 1995: State and Local Programs, Association of State Floodplain Managers, 1995, 100 pages, $15 for Association members, $20 for nonmembers. This publication discusses what the states are doing in floodplain management. There are numerous tables that identify what is being done by all 50 states and the District of Columbia, including state retrofitting activities. Each state's programs and selected local programs are reviewed.

National Directory of Floodplain Managers, Association of State Floodplain Managers, 1994, 157 pages, free to Association members, $20 for nonmembers. A directory of all members of the Association that includes sections on federal agencies, summaries of their programs, publications, committee progress reports, and cross references of members by area of interest and state. This is the only national directory of state floodplain management staff. Note: revised versions of this document are published every year.

GOVERNMENT RETROFITTING PROGRAMS

Local, state, and federal government agencies perform a variety of activities that implement or support retrofitting. This chapter groups the activities into six categories: general information, technical assistance, regulations, financial assistance, projects, and research and technology transfer.

GENERAL INFORMATION

The most common way government agencies support retrofitting is by providing publications and general information to interested persons. Several federal and state agencies have published manuals on the topic that are available to individuals and local governments for free distribution. Some of the publications are listed in the previous section of this Appendix.

Many local governments have prepared their own brochures that address local flooding and building conditions. Often these are distributed free to all residents of the floodplain or, particularly in the case of basement flooding, to all residents of the community. These federal, state, and local publications usually discuss retrofitting in general terms and provide property owners with an idea of what techniques would work for their situation.

Agencies also answer general questions about retrofitting and related topics. Local building, housing, and community development departments refer callers to the publications or state and federal agencies that provide assistance. Some maintain lists of retrofitting contractors or consultants.

TECHNICAL ASSISTANCE

While many agencies provide general information, a few provide more specific information to advise property owners about retrofitting individual buildings. This can include a range of services such as providing flood and building elevations, discussing options for protecting a building, recommending specific techniques, and reviewing the owner's building plans.

Several agencies have developed flood audit programs. These include a site investigation, discussions with the owner, and a written report that recommends specific retrofitting and other preparedness steps, such as purchasing flood insurance. Flood audits have been conducted for residences as well as large commercial or industrial complexes.

Technical assistance is specific and usually provides more help to a property owner than general information, such as that found in a brochure or other publication. However, few governmental agencies provide technical assistance for individual buildings due to the staff time necessary. In addition, free technical assistance service may not be based on careful examination of a building's structural condition, tests of wall strength, etc. Government agencies are hesitant to make specific recommendations based on what can only be a relatively cursory inspection.

REGULATIONS

Most regulations for retrofitting are based on the minimum standards of the National Flood Insurance Program. The NFIP sets minimum regulatory standards for constructing, modifying, or repairing buildings located in the floodplain to keep flood losses to a minimum. Over 18,000 flood-prone communities have adopted and enforce the minimum standards, and many have more restrictive requirements. The NFIP limits some retrofitting: it prohibits obstructions, such as berms or levees in floodways.

The NFIP requires that a building that is substantially improved or substantially damaged be elevated so its lowest floor is at or above the BFE. Substantial damage is defined as "damage of any origin sustained by a structure whereby the cost of restoring the structure to its before-damaged condition would equal or exceed 50 percent of the value of the structure before the damage occurred." Houses that have been substantially damaged or are being substantially improved (renovated) must be elevated to or above the 100-year flood level.

Many states and communities have more restrictive standards than the NFIP. The most common is freeboard, requiring an extra margin of safety in the design and construction of flood protection measures to account for waves, debris, hydraulic surge, or lack of flooding data. Some prohibit buildings or residences in certain areas, such as a floodplain or conservation zone. In these communities, substantially damaged buildings may not be allowed to be rebuilt unless they are relocated.

FINANCIAL ASSISTANCE

It is clear that homeowners' decisions to retrofit and the retrofitting measures they choose are directly related to their financial condition. This is particularly true after a flood, when opportunities for retrofitting are most evident and the homeowners' interest levels are high, but they are in a difficult financial position to take action. In many cases, availability of financial assistance is the determining factor in whether or not a property will be retrofitted.

Financial assistance can come in a variety of forms. For example, local governments could use property tax incentives to encourage retrofitting. Most financial assistance programs provide low-interest loans and grants. Generally, grants are limited to lower income families.

There are several federal, state, and local financial assistance programs for which retrofitting is a secondary objective. Usually, the owner must show that retrofitting is related to the program's primary concerns of rebuilding after a disaster, improving housing, or preserving or increasing employment opportunities.

PROJECTS

The greatest degree of government involvement is in the construction of public retrofitting projects. The agency prepares the construction plans, gets the owner's agreement, hires the contractor, and inspects the work. The more common projects include public buildings such as schools and waterfront park buildings.

There are a few examples of government-built retrofitting projects on private property. Some of these start as financial assistance programs but evolve into projects because the homeowners are unable to handle the technical aspects of managing a construction project. Others begin when flood control project plans find that retrofitting is the most cost-effective approach to reduce flood damages.

RESEARCH AND TECHNOLOGY TRANSFER

Several federal and some state agencies have conducted or sponsored research into retrofitting materials and measures, as well as ways to assist property owners with retrofitting. Two of the largest research programs are sponsored by the U.S. Army Corps of Engineers' National Flood Proofing Committee and FEMA. The USACE has conducted studies and tests of the ability of structure walls to withstand flooding, waterproofing compounds and materials, raising and moving structures (including slab-on-grade houses), and other miscellaneous retrofitting measures, including the use of a flexible, waterproof membrane to wrap a house.

Other agencies have investigated retrofitting measures, ways to motivate owners, alternative assistance arrangements, and methods for disseminating technical information.

While research itself is important, it is equally important to disseminate both the findings from research and lessons learned from practical experience. For example, FEMA and the USACE often inspect buildings after a flood to determine how well retrofitting measures have performed. The findings are published in papers and books and explained at conferences and workshops.

There have been a few retrofitting training programs, most of them for disaster assistance workers or local officials who implement state or federal technical or financial assistance programs. The USACE and the Model Building Code Groups conduct training programs under contract to FEMA. Some agencies also hold or sponsor public meetings or workshops for property owners.

ORGANIZATIONS THAT SUPPORT RETROFITTING ACTIVITIES

This section reviews the retrofitting programs conducted by six federal agencies. Programs that are usually undertaken by state and local agencies are also covered.

U.S. ARMY CORPS OF ENGINEERS (USACE)

The U.S. Army Corps of Engineers is the nation's oldest and largest water resources organization. Through its flood control program, the Corps conducts feasibility studies and builds flood control projects. Where it is shown to be economically feasible, these projects can include retrofitting. Major projects require specific authorization and funding by Congress, while small projects can be implemented with agency authority.

The Corps Floodplain Management Services Program provides flood hazard determinations, technical data on flood hazards, and guidance on retrofitting, floodplain regulations, flood warning, emergency preparedness, and evacuation planning. It also staffs the National Flood Proofing Committee, which supervises research and provides technology transfer on relocation, elevation, and other retrofitting measures. The Committee also coordinates with other agencies and associations involved in floodproofing.

Point of Contact: The Corps' civil works programs are organized in divisions and districts that cover the entire country. The main point of contact at these divisions and districts is the Floodplain Management Services office, whose telephone numbers and addresses are presented below.

U.S. ARMY CORPS OF ENGINEERS OFFICES

Alabama

Mobile District
P.O. Box 2288
Mobile, AL 36628-0001
Attn: CESAM-PD-P
205/694-3879

Alaska

Alaska District
P.O. Box 898
Anchorage, AK 99506-0898
Attn: CENPA-EN-PL-FP
907/753-2610

Arkansas

Little Rock District
P.O. Box 867
Little Rock, AR 72203-0867
Attn: CESWL-PL-F
501/378-5611

California

Los Angeles District
P.O. Box 2711
Los Angeles, CA 90053-2325
Attn: CESPL-PD-WF
213/894-5375

Sacramento District
650 Capitol Mall
Sacramento, CA 95814-4794
Attn: CESPK-PD-F
916/551-1881

San Francisco District
211 Main Street
San Francisco, CA 94105-1905
Attn: CESPN-PE-W
415/974-0460

South Pacific Division
Room 720
630 Sansome Street
San Francisco, CA 94111-2206
Attn: CESPD-PD-P
415/705-1637

District of Columbia

Headquarters
20 Massachusetts Ave., NW
Washington, DC 20314-1000
Attn: CECW-PF
202/272-0169

Florida

Jacksonville District
P.O. Box 4970
Jacksonville, FL 32232-0019
Attn: CESAJ-PD-FP
904/791-1102

Georgia

Savannah District
P.O. Box 889
Savannah, GA 31402-0889
Attn: CESAS-PD-F
912/944-5339

South Atlantic Division
Room 313
77 Forsyth Street, SW
Atlanta, GA 30335-6801
Attn: CESAD-PD-A
404/331-4441

Hawaii

Pacific Ocean Division
Ft. Shafter, HI 96585-5440
Attn: CEPOD-ED-PH
808/438-7009

Illinois

Chicago District
219 S. Dearborn Street
Chicago, IL 60604-1797
Attn: CENCC-PD-R
312/353-4078

North Central Division
536 S. Clark Street
Chicago, IL 60605-1592
Attn: CENCD-PD-FP
312/353-6531

Rock Island District
P.O. Box 2004
Clock Tower Building
Rock Island, IL 61204-2004
Attn: CENCR-PD-F
309/788-6361

Kentucky

Louisville District
P.O. Box 59
Louisville, KY 40201-00591135
Attn: CEORL-PD-S
502/582-5742

Louisiana

New Orleans District
P.O. Box 60267
New Orleans, LA 70160-0267
Attn: CELMN-PD-FG
504/862-2507

Maryland

Baltimore District
Supervisor of Baltimore Harbor
P.O. Box 1715
Baltimore, MD 21203-1715
Attn: CENAB-PL-B
301/962-3235

Massachusetts

New England Division
424 Trapelo Road
Waltham, MA 02254-9149
Attn: CENED-PL-B
617/647-8255

Michigan

Detroit District
P.O. Box 1027
Detroit, MI 48231-1027
Attn: CENCE-PD-PF
313/226-6773

Minnesota

St. Paul District
USPO & Custom House
St. Paul, MN 55101-1479
Attn: CENCS-PD-FS
612/220-0280

Mississippi

Lower Miss. Valley Division
P.O. Box 80
Vicksburg, MS 39181-0080
Attn: CELMV-PE-F
601/634-5827

Vicksburg District
P.O. Box 60
Vicksburg, MS 39181-0060
Attn: CELMK-PD-F
601/631-5416

Missouri

Kansas City District
700 Federal Building
Kansas City, MO 64106-2896
Attn: CEMRK-PD-P
816/426-3674

St. Louis District
1222 Spruce Street
St. Louis, MO 63103-2833
Attn: CELMS-PD-M
314/331-8480

Nebraska

Missouri River Division
P.O. Box 103, Downtown Station
Omaha, NE 68101-0103
Attn: CEMRD-PD-F
402/221-7273

Omaha District
Room 6014 USPO & Courthouse
Omaha, NE 68102-4978
Attn: CEMRO-PD-F
402/221-4596

New Mexico

Albuquerque District
P.O. Box 1580
Albuquerque, NM 87103-1580
Attn: CESWA-ED-PH
505/766-2635

New York

Buffalo District
1776 Niagara Street
Buffalo, NY 14207-3199
Attn: CENCB-PD-FP
716/879-4143

New York District
Supervisor of New York Harbor
26 Federal Plaza
New York, NY 10278-0090
Attn: CENAN-PL-FP
212/264-8870

North Atlantic Division
90 Church Street
New York, NY 10007-9998
Attn: CENAD-PL-FP
212/264-7482

North Carolina

Wilmington District
P.O. Box 1890
Wilmington, NC 28402-1890
Attn: CESAW-PD-F
919/251-4720

Ohio

Ohio River Division
P.O. Box 1159
Cincinnati, OH 45201-1159
Attn: CEORD-PD-J
513/684-3012

Oklahoma

Tulsa District
P.O. Box 61
Tulsa, OK 74121-0061
Attn: CESWT-PL-GF
918/581-7896

Oregon

North Pacific Division
P.O. Box 2946
Portland, OR 97208-2870
Attn: CENPD-PL-FS
503/326-3823

Portland District
P.O. Box 2946
Portland, OR 97208-2946
Attn: CENPP-PL-CF
503/326-6411

Pennsylvania

Philadelphia District
U.S. Customs House
2nd & Chestnut Streets
Philadelphia, PA 19106-2991
Attn: CENAP-PL-F
215/597-4808

Pittsburgh District
William S. Moorehead Fed. Bldg.
1000 Liberty Avenue
Pittsburgh, PA 15222-4186
Attn: CEORP-PD-J
412/644-4180

South Carolina

Charleston District
P.O. Box 919
Charleston, SC 19402-0919
Attn: CESAC-EN-PH
803/727-4682

Tennessee

Nashville District
P.O. Box 1070
Nashville, TN 37202-1070
Attn: CEORN-ED-P
615/736-5055

Memphis District
B-202 Clifford Davis Bldg.
Bldg. 167 North Main Street
Memphis, TN 38103-1894
Attn: CELMM-PD-M
901/544-3968

Texas

Galveston District
P.O. Box 1229
Galveston, TX 77553-1229
Attn: CESWG-PL-P
409/766-3023

Fort Worth District
P.O. Box 17300
Fort Worth, TX 76102-0300
Attn: CESWF-PL-F
818/334-3207

Southwestern Division
1114 Commerce Street
Dallas, TX 75242-0216
Attn: CESWD-PL-M
214/767-2310

Virginia

Norfolk District
Supervisor of Norfolk Harbor
803 Front Street
Norfolk, VA 23510-1096
Attn: CENAO-PL-FP
804/441-7779

Washington

Seattle District
P.O. Box C-3755
Seattle, WA 98124-2255
Attn: CENPS-EN-HH
206/764-3660

Walla Walla District
Bldg. 602 City-County Airport
Walla Walla, WA 99362-9265
Attn: CENPW-PL-FP
509/522-6589

West Virginia

Huntington District
502 8th Street
Huntington, WV 25701-2070
Attn: CEORH-PD-S
304/529-5644

FEDERAL EMERGENCY MANAGEMENT AGENCY (FEMA)

Created by the Congress in 1968, the National Flood Insurance Program aims to reduce future damage to existing and new construction through prudent floodplain development and to transfer the risk of that development from the public to the private sector through an insurance mechanism that protects the financial interest of the property owner while requiring a premium to be paid for that protection.

The Federal Emergency Management Agency identifies and maps flood hazards nationwide. Flood Insurance Rate Maps distinguish several flood hazard zones, including the 100-year floodplain, which is defined as an area inundated by a flood that has a one-percent chance of being equalled or exceeded in any year (i.e., the 100-year flood, also called the Base Flood Elevation). In riverine areas and tidal areas subject to waves of less than three feet in height, the 100-year floodplain is referred to as the Special Flood Hazard Area and is designated Zone A. In coastal areas where wave heights equal or exceed three feet, the 100-year floodplain is referred to as the Coastal High Hazard Area and is designated Zone V.

In communities that participate in the program, construction is allowed within the Special Flood Hazard Area if it complies with local floodplain ordinances that meet National Flood Insurance Program requirements. A fundamental requirement is that any new or substantially improved residential building must have its lowest floor elevated to or above the Base Flood Elevation. A building is considered substantially improved when the cost of any rehabilitation, addition, or other improvement, or repair or reconstruction after damage, equals or exceeds 50 percent of the pre-improvement/pre-damage value of the building. In A Zones, the lowest residential floor must be elevated either on earthen fill or solid or open foundations to or above the Base Flood Elevation. In V Zones, the lowest horizontal structural member must be elevated to or above the Base Flood Elevation on an open foundation.

The foundation of the NFIP is a *quid pro quo*: if a community will adopt and enforce ordinances to reduce future flood risks, the federal government will make flood insurance available to property owners in the community.

Lending institutions require the purchase of flood insurance for buildings located in the Special Flood Hazard Area as a condition of obtaining a federally sponsored or insured mortgage or home improvement loan. Flood insurance policies are available through both private insurance agents and the federal government.

Point of contact: FEMA's work is conducted through ten regional offices as shown on the following page.

FEMA OFFICES

FEMA Headquarters
500 C Street, SW
Washington, DC 20472
202/646-4622

REGION I - CT, MA, ME, NH, RI, VT
442 J.W. McCormick POCH
Boston, MA 02109-4595
617/223-9561

REGION II - NJ, NY, PR, VI
26 Federal Plaza, Room 1337
New York, NY 10278-0002
212/225-7200

REGION III - DE, DC, MD, PA, VA, WV
615 Chestnut Street, Sixth Floor
Philadelphia, PA 19106
215/931-5502

REGION IV - AL, FL, GA, KY, MS, NC, SC, TN
3003 Chamblee-Tucker Road, Room 270
Atlanta, GA 30341
770/220-5400

REGION V - IL, IN, MI, MN, OH, WI
536 S. Clark Street, 6th Floor
Chicago, IL 60605-1521
312/408-5200

REGION VI - AR, LA, NM, OK, TX
Federal Regional Center
800 North Loop 288
Denton, TX 76201-3698
940/898-5165

REGION VII - IA, KS, MO, NE
2323 Grand Boulevard
Kansas City, MO 64108-2760
816/283-7002

REGION VIII - CO, MT, ND, SD, UT, WY
Denver Federal Center
Building 710, Box 25267
Denver, CO 80225-0267
303/235-4830

REGION IX - AZ, CA, GU, HI, NV
Presidio of San Francisco
Building 105
San Francisco, CA 94129-1250
415/923-7175

REGION X - AK, ID, OR, WA
Federal Regional Center
130 - 228th Street, SW
Bothell, WA 98021-9796
425/487-4678

INFORMATION SOURCES

The following are sources of information about topics related to coastal construction, including natural hazards, coastal science, building science, and builidng codes and standards.

FEDERAL, STATE, AND LOCAL GOVERNMENT AGENCIES

Federal Emergency Management Agency (FEMA)
http://www.fema.gov

Metro-Dade Building Code Compliance
http://www.buildingcodeonline.com

Office of Ocean & Coastal Resource Management (OCRM)
http://wave.nos.noaa.gov/ocrm/czm

United States Army Corps of Engineers (USACE)
http://www.usace.army.mil

PROFESSIONAL ORGANIZATIONS

American Institute of Architects (AIA)
http://www.aia.org

American Society of Civil Engineers (ASCE)
http://www.asce.org

National Society of Professional Engineers (NSPE)
http://www.nspe.org

TRADE ORGANIZATIONS

American Forest & Paper Association (AF&PA)
http://www.awc.org

American Iron and Steel Institute (AISI)
http://www.steel.org

APA - The Engineered Wood Association
http://www.apawood.org/home.html

Institute for Business and Home Safety (IBHS)
http://www.iiplr.org

Office National Association of Home Builders (NAHB)
http://www.nahb.com

National Association of Home Builders Research Center
http://www.nahbrc.org

National Concrete Masonry Association (NCMA)
http://www.ncma.org

National Roofing Contractors Association
National Roofing Contractors Association (NRCA)
http://www.roofonline.org

Pile Driving Contractors Association
http://www.piledrivers.org

CODE AND STANDARDS ORGANIZATIONS

American National Standards Institute (ANSI)
http://web.ansi.org/default.htm

Building Officials Code Administrators (BOCA)
http://www.bocai.org

International Code Council (ICC)
http://www.intlcode.org

International Conference of Building Officials (ICBO)
http://www.icbo.org

National Fire Protection Association (NFP)
htp://www.nfpa.org

Southern Building Code Conference International (SBCCI)
http://www.sbcci.org

NATURAL HAZARDS AND COASTAL SCIENCE ORGANIZATIONS

Association of State Flood Plain Managers (ASFPM)
http://www.floods.org

National Hazards Center at the University of Colorado
http://www.colorado.edu/IBS/hazards/index.html

National Sea Grant Office
http://www.mdsg.umd.edu/NSGO/NationalSeaGrant.html

TENNESSEE VALLEY AUTHORITY (TVA)

Since 1953, the TVA has assisted state and local officials and property owners in planning and implementing sound floodplain management practices within the Tennessee River watershed. Since October 1994, floodplain management assistance has been provided by the USACE and local/state governments. Information on TVA reservoirs is presently available from TVA.

Point of contact: Tennessee Valley Authority
524 Union Avenue
Evans Building, Room 1A
Knoxville, TN 37902-1499
615/632-2101

NATURAL RESOURCES CONSERVATION SERVICE (NRCS)

As part of the U.S. Department of Agriculture, the NRCS, formerly known as the Soil Conservation Service (SCS), primarily serves rural areas. NRCS staff provides information on land-use planning, conservation planning, resource development, water management, and flood prevention to farmers, community officials, and land developers. While mostly a general information and technical assistance operation, NRCS also funds flood protection projects that can include retrofitting elements.

Point of contact: NRCS work is conducted through local soil and water conservation districts. The point of contact is the district conservationist. (Check the local telephone directory.)

SMALL BUSINESS ADMINISTRATION (SBA)

The SBA administers the federal government's major disaster loan program. In spite of its name, SBA disaster loans are available for any privately owned property, including businesses and residences. The low-interest loans are provided to rebuild a damaged building, including the cost of bringing a building up to the current building code standards. The loans can pay for code-required retrofitting of substantially damaged buildings and some smaller projects.

Point of contact: SBA loans are only available following either an SBA or Presidentially declared disaster. Disaster Application Centers are established to process applications. The location and hours of these centers are well publicized.

DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT (HUD)

HUD programs are designed to improve housing conditions, local economies, and neighborhoods. As the nation's housing agency, HUD has been active in protecting both public and privately owned houses from flood damage. HUD's major retrofitting program is the Community Development Block Grant (CDBG), which provides funds directly to larger cities and counties. States handle CDBG funds for smaller communities.

The block grant concept allows states and communities to set their funding priorities as long as the local projects relate to program objectives, i.e., they must benefit low and moderate income people, prevent or eliminate slums and blight, or meet other urgent community development needs. Many communities have used CDBG funds to retrofit buildings as a way to provide low-income residents with safe and sanitary housing. Some states have reserved block grant funds for special post-disaster projects that have included retrofitting.

Point of contact: Each state has a HUD Area Office, located in its capital or largest city. State departments of community affairs are also points of contact on the Community Development Block Grant. (Check the local telephone directory.)

ASSOCIATION OF STATE FLOODPLAIN MANAGERS (ASFPM)

While not a government agency, the ASFPM supports many government retrofitting programs. Its Floodproofing/Retrofitting committee works on coordinating and publicizing federal, state, and local retrofitting activities. The Mitigation Committee focuses on post-disaster activities, especially programs that can provide funding help to property owners.

The Association is a provider of general information and has published several reports on retrofitting activities. Its conferences are the largest in the nation on floodplain management and usually include many sessions on retrofitting. The Association is also a good source of information on state and local floodplain management programs and contacts.

Point of contact: Executive Director
Association of State Floodplain Managers
P.O. Box 2051
Madison, WI 53701-2051
608/266-1926

STATE HOUSING AND COMMUNITY AFFAIRS AGENCIES

Most states have a department of community affairs or similar office that is responsible for managing the Community Development Block Grant (see HUD). Some states have their own funding programs that operate similar to the block grant program. They fund housing or economic improvement projects, including protecting buildings from floods. Some agencies provide technical assistance to communities undertaking floodplain management planning or establishing programs to help property owners.

Point of contact: The title and duties will vary from state to state, but most will have a community affairs agency located in the state capital.

STATE FLOODPLAIN MANAGEMENT COORDINATORS

Most states have a floodplain management coordinator whose duties include advising and assisting local officials and property owners about the National Flood Insurance Program (NFIP), particularly its regulatory aspects. These offices are also the best sources of information about related floodplain management issues, including programs that affect or support retrofitting. A few state coordinating offices provide technical assistance or manage financial assistance programs.

Point of contact: State coordinators can be located by contacting the appropriate FEMA Regional Office, the Association of State Floodplain Managers or local floodplain administrators.

LOCAL BUILDING AND FLOODPLAIN MANAGEMENT AGENCIES

Regulations that affect retrofitting are implemented by local building, zoning, floodplain, or housing code departments. These offices sometimes provide general information and technical assistance to property owners. Several have developed handbooks on retrofitting for their residents.

Point of contact: Generally, county regulatory departments operate only in unincorporated areas. Municipal departments have jurisdiction in incorporated cities, towns, and villages (check the local telephone directory). State NFIP coordinators and FEMA Regional Offices may know of local departments particularly active in retrofitting.

LOCAL HOUSING, COMMUNITY DEVELOPMENT, AND PLANNING AGENCIES

There are many different kinds of city, county, and regional agencies involved in housing, planning, urban renewal, and community development. Community development departments and housing authorities work to improve local housing conditions through both public housing and programs to help low and moderate income residents. This work can be in the form of building inspections, technical assistance, and financial assistance. Other local and regional agencies include regional planning commissions, sanitary districts, and water management districts.

Most provide general information to residents and technical assistance to local officials. Some sanitary districts have regulatory authority based on the need to keep floodwater out of sewer lines. Some of these agencies have active technical and financial assistance programs to help property owners in retrofitting projects.

Point of contact: These agencies may be listed in the local telephone directory. State NFIP coordinators, FEMA Regional Offices, and local floodplain administrators may know of agencies particularly active in retrofitting.

VIDEOTAPES

Valuable retrofitting information and training are available on video cassette. Floodproofing information videos have been prepared for general distribution by the following entities.

FEMA and the National Association of Home Builders *Best Build Series*, which may be purchased from the NFIP at a cost of $10, includes these titles:

- *Constructing a Sound Coastal Home* (20 minutes)
- *Construction in a Riverine Floodplain* (24 minutes)
- *Protecting a Flood-Prone Home* (30 minutes)

FEMA Mitigation Division

- *Above the Flood: Elevating Your Floodprone House* (FEMA 347 VT, 30 minutes)

(The regional offices of FEMA are listed on page C-16.)

USACE National Flood Proofing Committee

- *House Raising with Slab Attached* (7 minutes)

(The USACE address nearest you can be found on pages C-10 through C-13.)

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Federal Emergency Management Agency, TB #4-93, *Elevator Installation for Buildings Located in Special Flood Hazard Areas in Accordance with the NFIP*, April 1993.

Federal Emergency Management Agency, TB #5-93, *Free-of-Obstruction Requirements for Buildings Located in Coastal High Hazard Areas in Accordance with the NFIP*, April 1993.

Federal Emergency Management Agency, TB #6-93, *Below-Grade Parking Requirements for Buildings Located in Special Flood Hazard Areas in Accordance with the NFIP*, April 1993.

Federal Emergency Management Agency, TB#7-93, *Wet Floodproofing for Requirements for Structures Located in Special Flood Hazard Areas in Accordance with the NFIP*, December 1993.

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THE RESOURCE CENTER

This appendix has introduced 14 publications that are readily available and that provide overviews of retrofitting, as well as our bibliography for this publication. There are many more references on various technical aspects of retrofitting. Most of them have been collected and cataloged at the Floodplain Management Resource Center. This chapter explains how to locate these additional publications.

Any person may use the Resource Center. It is a public service established by the Association of State Floodplain Managers with financial support from the Corps of Engineers, the Federal Insurance Administration, the Tennessee Valley Authority, and other public and private organizations.

OPERATION

The Floodplain Management Resource Center is located at the Natural Hazards Center in Boulder, Colorado. It houses the nation's largest collection of documents on retrofitting. Each document has been categorized and summarized. The summaries have been entered into a computer data base that enables Center staff to quickly identify those documents most appropriate for an inquirer's needs.

Contact the Center by calling 303/492-6818 between 9:00 and 4:00 Mountain Time, Monday through Friday, or by writing to the Natural Hazards Center, IBS No.6, Campus Box 482, Boulder, Colorado, 80309-0482. Upon receiving an inquiry, a Center staff person will review the database and retrieve summaries of those documents that appear most useful.

The Center staff person may read excerpts from the document summaries over the telephone or mail printed document summaries to the inquirer. The Resource Center does not send a document to the inquirer; it only tells the inquirer how to obtain a copy. The staff may copy all or portions of a document that are in the public domain (especially those that are out of print).

The cost of answering inquiries, including printing and mailing up to ten document summaries, is borne by the Resource Center. There is no cost for these services to any caller. The Center may charge a fee for copying a document or providing additional services. The fee is based on the actual cost of duplicating or performing the service.

DOCUMENT SUMMARIES

All records on the Center's retrofitting publications are kept on document summaries. The summaries follow the adjacent format. This format provides all necessary data about a document on one page so the Center staff and the inquirer can quickly and easily identify that the document is appropriate. While no document takes more than one page, a different summary page may be used for each article in publications such as conference proceedings and edited collections of articles by different authors on different topics.

KEYWORDS

The Resource Center's computer program can search for any word. Three sections of the document summary list selected keywords that help the Center and the inquirer locate the documents they need. The Topic Keywords identify the floodplain management activity, the Focus Keywords explain how the topic is addressed, and the Audience Keywords list the type of reader the publication is directed to.

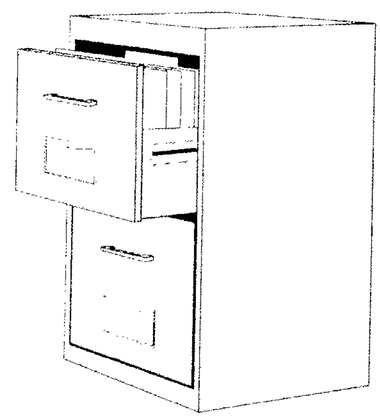
Using the keywords can greatly assist in the document search. For example, a request for a book on retrofitting basements will yield more than 25 publications. In most cases, the inquirer has a more specific interest. For example, if a caller wants a book that explains protecting basements from hydrostatic pressure to homeowners, the Center staff's search would be:

Topic="pressures" and "basement"
Focus="techniques"
Audience="lay persons"

This particular search will locate two books (more will probably be added over time). The inquirer will be told about the books and how to obtain them and will also be sent the document summaries.

APPENDIX D

ALLUVIAL FAN FLOODING



ALLUVIAL FAN FLOODING

> In mountainous regions in the west, floodwaters may spread out in a fan shape as they flow from the mouth of a watershed to the valley below. The floodwaters erode the steep slopes of the watershed and deposit sediment in a cone or fan shape over the flatter land. Over time, this process creates a land form known as an **alluvial fan**.
>
> Fan flood flows are characterized by surging, erosion, scour, channel avulsion, mud and debris flows, and sheet flows on the lower portions of the fan surface. Each fan flood event and each fan can exhibit different flood characteristics.

> Water flowing from the narrow mouth of a basin and spreading out as it leaves the opening is said to **debouch**.

Alluvial fan flooding is a hazard to communities in the mountainous regions of the western United States. Alluvial fan flooding is characterized by a sudden torrent of water capable of carrying rocks, mud, and debris that debouches from the steep valleys and canyons and spreads over the fan surface. The type of detailed flood damage mitigation information available for other flood-prone areas is limited for alluvial fan situations, but a profile of this type of flooding and general measures to mitigate its impact are beginning to emerge.

Across the western United States alluvial fans are appealing to residential developers for their vistas, and pressure to construct on fans is increasing as the valley floors become populated. Development over the last several decades has proceeded with little cognizance of the potential for flood hazards. On most fans, there is evidence of past floods, but the history of development is relatively short and the consequences of a 100-year flood have not been confronted. Many fan communities are now preparing flood management and mitigation plans, but existing structures may have to rely on floodproofing measures to reduce flood damage.

Contained in this appendix is a discussion of:

- alluvial fan physical processes and how fan flooding differs from riverine flooding;
- an overview of the regulatory framework and building code issues unique to fan areas;
- techniques for integrating floodproofing/retrofitting with fan-wide mitigation and master drainage plans; and
- guidance on retrofitting design criteria.

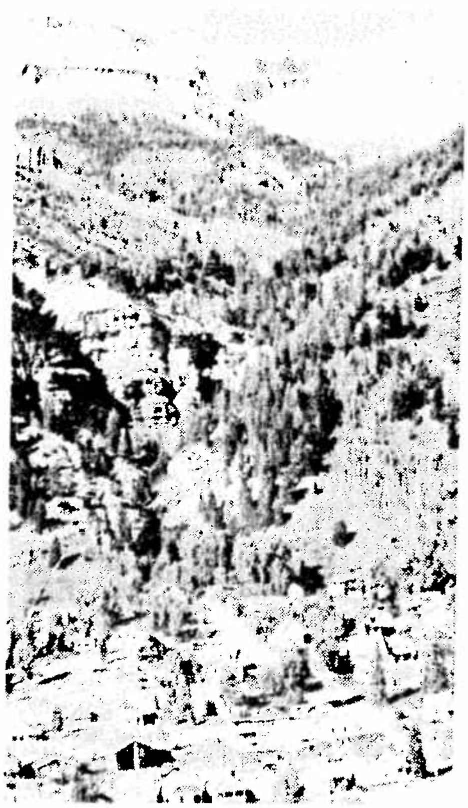


Figure D-1: Telluride, Colorado Fan

It is recognized that development on alluvial fans may vary in density and may include large commercial, single- and multifamily residential, and/or municipal structures that can significantly affect local hydraulic conditions. Where high density development exists or where there are major structures oriented across potential flow paths, upfan channel-related mitigation measures such as channelization, flow diversion, and debris basins are the most feasible approach for hazard avoidance. Fan-wide master plans for zoning and fan-wide mitigation measures are crucial for successful protection of the community as a whole. Where master plans or mitigation schemes are inadequate or nonexistent, floodproofing and retrofitting of residences may provide the only reasonable methods for flood loss reduction. Retrofitting can reduce future flood damage but is seldom recognized by the NFIP, particularly with respect to insurance premium rates.

In the desert Southwest, alluvial fans are subject to clear water flooding and debris-laden frontal waves. In parts of the mountainous West, mudflows dominate fan evolution. Fans in the Pacific Northwest are prone to flood hazards related to sediment transport from less common sources, such as volcanic activity and logging practices. The following sections provide some general concepts and definitions of terms related to alluvial fans and floodproofing design.


Figure D-2: Alluvial Fan Flooding Damage, Telluride, Colorado

INTRODUCTION TO ALLUVIAL FANS

FAN MORPHOLOGY

Both the hydraulic and hydrologic flood characteristics of alluvial fans are highly variable from fan to fan, which may be in different stages of episodic growth. A geologist, geomorphologist, hydrologist, or hydraulic engineer experienced in alluvial fan technology should be consulted to identify alluvial fan characteristics and the possible response to flooding.

Figure D-3: Rancho Mirage, California Fan Damage (1979)

An **alluvial fan** is a conical- or fan-shaped land form located at the mouth of a watershed, where floodwaters debouch from the basin and spread over the valley floor. Alluvial fans evolve over geologic time as sediments (boulders, gravel, sand, and fines), erode from the steep watershed slopes and are transported by flood flows to the flatter fan surface. Sediments accumulate on the fan as the slope decreases, flows spread out, and the flow loses its ability to transport sediment. The alluvial fan surface may be punctuated by deep channels or irregularly-shaped deposits formed by infrequent, often large flash flooding events.

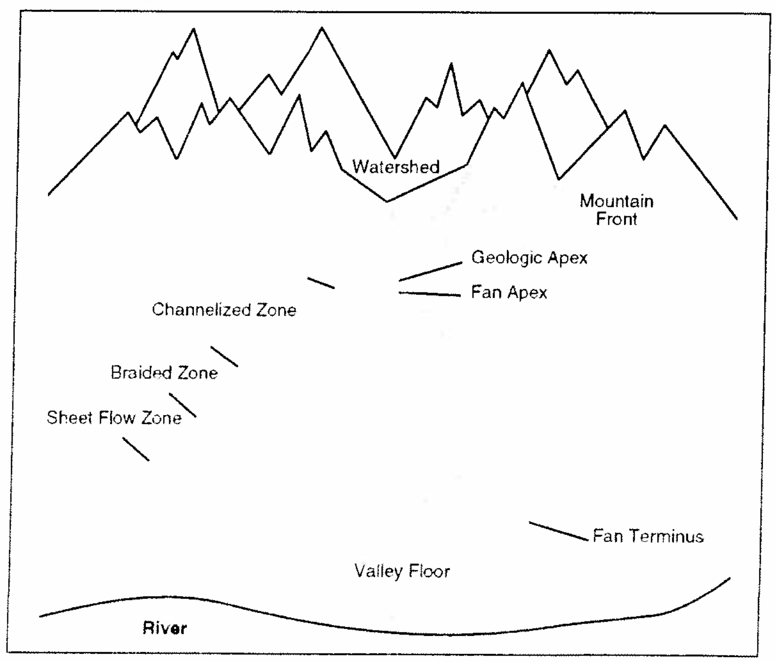


Figure D-4: Oblique View of an Alluvial Fan

The fan **apex**, usually located near the intersection of the mountain watershed and the top of the fan, is the point where storm runoff emerging from the confined mountain channel onto the alluvial fan diverges into either multiple channels or unconfined flow.

The fan **terminus**, or **toe**, is the intersection of the alluvial fan and the valley floor. Fan slope may become milder approaching the fan terminus, resulting in a concave profile.

Alluvial fans emerging from adjacent mountain watersheds may coalesce and form an apron of alluvial material along the mountain front, disguising the presence of the fan. This apron is called a **bajada**.

Three zones may be identified on the surface of an alluvial fan, reflecting the hydraulic and sediment-transport processes during a flooding event:

- the channelized zone (not always noticeable below the apex of an active fan);
- the braided zone; and
- the sheet flow zone.

The exact location of each zone on a given fan is dependent on flooding characteristics, but usually can be identified on the fan surface after a recent flood event. These zones are discussed throughout the text in relationship to feasible retrofitting alternatives.

The **channelized zone** is generally located at and below (downstream of) the fan apex. Flow within this zone is confined to well-defined channels, although channels may split or abruptly change direction. This zone is associated with hazardous flooding conditions related to high flow velocities, boulder and debris impact, and channel scour. If channels are deeply incised, this zone may extend further down the fan.

As channels progress over an alluvial fan, they may become shallower and wider, and split into a system of multiple channels in an area of the fan defined as the **braided zone**. Flow in the braided zone has an unstable pattern of numerous interlacing shallow channels. Flood hazards in this zone are related to flood inundation and sediment deposition, rather than high flow velocity or debris impact. Large boulder transport is generally absent in this zone.

Flow depths normally decrease in the downfan direction. Smaller channels may aggrade while other areas are subject to erosion or scour. Flow may continue to spread laterally until sheet flow is predominant. Sheet flow generally refers to flow depth less than 0.5 ft. Flood hazards in this **sheet flow zone** are usually limited to inundation by low velocity floodwater.

Streets and buildings can change the composition of a fan zone by redistributing floodwaters over the fan surface. The altered flood response can impact areas on the fan that may have been considered outside the originally delineated flood hazard zone. As a fan is developed, delineation of flood hazards may change.

TYPES OF FAN FLOODING

Water flooding dominates alluvial fan flows in the desert Southwest. The fan flows are generally characterized by relatively stable channels near the apex of the fan, with sheet flow and sediment deposition on lower portions of the fan surface. Flood damage occurs from water inundation, scour around structures, and sediment deposition requiring cleanup. In contrast, the alluvial fans of the Pacific Northwest, Rocky Mountains, and the West Coast ranges can experience severe mud and debris flows whose surges can engulf entire buildings, resulting in structural damage, movement, or complete collapse.

Alluvial fan processes and the resultant fan morphology are dependent upon hydrologic conditions of the upstream watershed. Factors contributing to devastating fan flooding include:

- high intensity rainfall events on sparsely vegetated steep slopes;
- steep watershed slopes with highly erosive soils or unstable geologic formations;
- sediment buildup and storage in watershed channels;
- saturated soil conditions from antecedent rain and snowmelt;
- recent forest fires, logging, or other soil-destabilizing activities in the watershed;
- intensity and configuration of development on the fan; and
- failure of flood mitigation measures.

Fan flooding can occur through the continuum of sediment transport processes from clear water flows to hyperconcentrated sediment flows such as mud floods and debris flows.

A **water flood** is the inundation of the fan surface from overbank discharge or rainfall/snowpack runoff. Fan water floods are common in the desert Southwest. Water flooding can cause damage by inundating the lower floor, scouring and undermining structures, displacing buildings from foundations, physically ripping or tearing apart structures, or depositing sediment in basements and yards. Sediment loads are less than 20 percent of the total flow and do not significantly affect fluid flow properties.

When the concentration of sediment in the flow reaches 20 to 40 percent by volume, the flow is considered to be "hyperconcentrated" and can be defined as **mud flow**. Mud flows with 20 to 40 percent volume are more common in the Rocky Mountains and along the West Coast. These concentrations of sediment cause an increase in viscosity of the flow matrix and a corresponding increase in the flow competence (ability to transport large boulders). Mud flows can be destructive to buildings because they are usually associated with high velocity flows. In addition to the property damage cited above for a water flood, mud flows can cause severe property damage related to sediment deposition. Cleanup costs can be significantly higher for a mud flood than a water flood.

The July 24, 1977, mud flows in Glenwood Springs, Colorado, resulted in approximately \$500,000 (1977 dollars) in damage, most of which involved mud removal.

Mud flows having a flow matrix with a sediment concentration ranging from 40 to 55 percent by volume are common in the alluvial fans of the Pacific Northwest and also occur in the Rocky Mountains. Damage results from inundation by mud, impact of mud frontal waves, and high lateral loading, which can result in structure collapse. Mud flows can raft large boulders and debris on their flow surfaces, causing substantial impact damage. Cleanup costs after a mudflow event can be severe.

Debris flows are hyperconcentrated flows with a sediment concentration that may be greater than 55 percent by volume. They consist primarily of rolling and tumbling boulders and debris and only a limited amount of fluid for lubrication. Fifty percent or more of the particles in a debris flow are generally larger than sand.

> Debris flows are less likely than water or mud floods to occur, but can cause more damage due to the impact of high velocity boulders or debris waves, which crash through building walls or knock structures off foundations.

Channel avulsion is the episodic, and often erratic, shift of a channel's path. Channel avulsion may be initiated by sediment deposition that can fill or block the channel, forcing the flow to create a new path, or by bank erosion, through which the flow will be diverted. The new flow path will often follow a steeper course. Structures located in the path of a newly forming channel are often undermined and destroyed.

FAN TYPES

Three types of alluvial fans are discussed in this manual; they are differentiated based on hydraulic and sediment transport processes: active alluvial fans, distributary flow systems, and inactive alluvial fans (French et al, 1993). Alluvial fans are also differentiated on the basis of flow conditions present on the fan between flooding events. **Dry fans** are associated with ephemeral streams; **wet fans** are associated with perennial streams. Virtually all alluvial fans in the southwestern states are dry fans.

> A key characteristic of active fans is the presence of evidence of relatively recent (in geologic terms) flood flows.

Active alluvial fans are generally associated with steep-sloped watersheds with high sediment yields. Active fans aggrade over time and are subject to debris flows, hyperconcentrated sediment flows, flash flooding, and aggradation and degradation related to sediment transport processes. Channels near the apex avulse episodically in response to the high sediment supply. Fan growth is relatively uniform. Active fans are generally regarded as high flood hazard fans. Portions of active alluvial fans may have inactive surfaces.

Distributary flow fan systems exhibit divergent or braided flow patterns. The channel proceeding downfan will split into one or more channels that may possibly recombine further downfan. These fan types are associated with watersheds where the sediment supply is in approximate equilibrium with the sediment transport conveyance through the system.

Debris flow activity on the fan surface is limited to frontal waves. The flood hazard associated with distributary fans is generally water inundation, sediment deposition, and scour, resulting in a moderate or low flood hazard.

Inactive alluvial fans are associated with watersheds in more geologically stable regions where the sediment transport processes on the fan exceed the sediment supply from the watershed. Inactive fans degrade over geologic time and channels are generally stable, creating a convergent pattern over the surface of the fan. The fan may actually be developing its own small watershed or drainage system. Recent sediment deposition on the fan surface, channel avulsion, and debris flows are absent. The flood hazard on inactive alluvial fans is usually moderate or low, although the steep fan slopes still have potential for severe erosion or sediment deposition if drainage conditions are altered.

ALLUVIAL FAN FLOOD HAZARDS

While alluvial fans present flood hazards found in riverine flooding such as inundation and differential hydrostatic loading, they are often compounded by high velocities, hyperconcentrated sediment flows, severe erosion, and extensive sediment deposition. Structures on alluvial fans may be susceptible to damage caused by high velocity water; lateral loading that forces structures off foundations or induces wall collapse; water inundation; scour and undermining of buildings; impact of mud, debris, and boulders; sediment burial; and landscape erosion.

Most alluvial fan floods are caused by high-intensity, short-duration summer thunderstorms. This is particularly true in the desert Southwest and Rocky Mountain region. Fan flooding on the western slopes of the West Coast mountains is often caused by longer duration rainstorms (e.g., West Coast frontal weather systems). Less common causes of fan flooding include spring snowpack melt, volcanically-induced flooding, and failure of water storage facilities. The flooding is often characterized by a frontal wave or "wall of water" that may carry boulders, trees, and debris; scour large channels; and carry off cars and property. The peak discharge in the flood wave may even overtake and become the frontal wave. If there is no rainfall in the valley or on the fan, the flood may arrive without warning.

Floods debouching from the watershed onto the alluvial fan are initially confined to a channel or between canyon walls. Structures located near the fan apex can be subjected to high velocities (greater than 10 fps), deep flow depths (greater than three feet), and debris. Flows cutting new channels or eroding existing channel beds may scour around buildings, tilting foundations and leaving unstable structures and large scour holes. After the flood event, layers of sediment deposition must be removed from yards, basements, or even first floors.

The frontal wave may collapse walls, topple structures, and rip buildings from their foundations.

On fans with a history of mud and debris flows, residents can experience a more devastating level of flood hazard.

On desert fans, the flow distributing itself between buildings and down streets can cause shallow flooding damage associated with high velocity flow in the streets include the inundation and transport of vehicles, filling of lowest floors with water and sediment, structural damage the upstream side of buildings from flow and debris impact, landscape erosion, local scour at building corners, and shallow sediment deposition. The blockage of flood conveyance facilities, such as bridges and culverts, or the failure of the storm sewer system can be exacerbate local flooding on lower portions of the fan.

A very large, high velocity mud flood can be devastating, resulting in the collapse of buildings and/or loss of life. Mud and debris flows can have frontal waves up to 15 feet high and have been known to sweep houses off their foundations, as in the Lake Whatcomb, Whatcomb County, Washington, 1983 torrent debris flows, which deposited two houses into the lake below. Mud flows have been found to travel at a rate of three to 20 feet per second with flow depths of up to 15 feet.

Similar to frontal waves, surging will increase the flood hazard by subjecting structures to significantly higher flow depths and velocities. Surges have been observed at eight feet high, more than double the flow depth.

Some watersheds are more prone to surges during flooding events due to channel geometry or sediment supply. Surges may entrain large boulders and other debris, increasing damage due to impact. In some cases, surging may also be due to the development of roll waves, a flow instability phenomenon often observed in open channels.

FEMA's *Alluvial Fans: Hazards and Management* (1989) provides an overview of alluvial fans and related management issues, and briefly discusses retrofitting of individual residential structures. Another FEMA publication entitled *Reducing Losses in High Risk Flood Hazard Areas: A Guidebook for Local Officials* specifically addresses alluvial fan flooding as a regulatory problem and provides outlines for the development of regulations and master plans for communities. This guidebook also summarizes the Dawdy Method for estimating flood frequency on alluvial fans and presents the Colorado Statute HB-1041 as a model geologic hazard ordinance that includes alluvial fan flooding hazards.

The hydrostatic pressure exerted on structural walls by sediment deposits can also be a significant flood hazard. Once the mud or debris flow has ceased, the resulting deposition against a building can exert large lateral pressures that may be nonuniform across the face of the wall. In addition to the impact and differential hydrodynamic loading related to mud flows, the high specific weight of the deposited mud and the resulting differential loads can cause structural damage to buildings designed to withstand predicted water hydrostatic and hydrodynamic loads. Often large boulders, trees, or other debris will come to rest against the upfan side of a building, contributing to the nonuniform lateral load on a wall.

REGULATORY FRAMEWORK

THE FEMA/NFIP FRAMEWORK AND ALLUVIAL FAN CONSIDERATIONS

A detailed description of the National Flood Insurance Program (NFIP) and its minimum regulations for floodplain management, as well as a discussion of building codes are provided in earlier chapters. Within this regulatory context, alluvial fan flooding poses special problems for individuals and agencies trying to interpret guidelines that were prepared specifically for riverine flooding conditions. Although FEMA recognizes alluvial fan flooding hazards, guidelines do not specifically address mud and debris flow hazards or sheet flow inundation on urbanized alluvial fans.

Unmapped urbanized fans are not subject to FEMA/NFIP insurance or mitigation criteria. In response to increased exposure to fan flooding, some communities have undertaken flood hazard delineation and have instituted local ordinances and regulations for fan development. In most states, there are no guidelines or regulations governing hazard delineation, zoning regulations, or mitigation for new construction.

Mapping of alluvial fans for the NFIP is conducted by a statistically based computer model called "FAN." FEMA provides a user's manual and program disk for those interested in performing the computations. The computations are based on certain assumptions regarding typical behavior of flow as it passes from the apex across the fan. The computations are not based on actual routing of flood hydrographs as applied in the normal riverine community flood insurance maps. Engineering firms with specialized alluvial fan analysis expertise have the capability to perform reasonable estimates of the physical processes that take place as the alluvial floods aggrade, degrade, change direction, and change concentration of sediment and debris loads. Such computations are generally not attempted for active fans because of their propensity to change physical configuration of the fan during floods. However, inactive alluvial fans with stabilized channels can often be successfully modeled. In view of the above concerns, retrofitting of buildings in the floodplains must be based on estimated design parameters (velocity, scour, depth, sediment, debris, etc.) in order to reduce future flood damages. However, future damage must be expected when the parameters are exceeded. Homeowners can expect some relief from the more frequent flood events with retrofitting, but this type of mitigation is recognized by FEMA only in its community rating system.

LOCAL FLOODPLAIN ADMINISTRATION

In communities that have not adopted specific alluvial fan flood hazard regulations and ordinances, it is left to the developers and homeowners to mitigate flood hazards or implement floodproofing. Progressive communities have conducted geologic/geomorphic surveys and hydrologic studies to more effectively determine the extent of flood hazards. Once the potential for the flood hazard is understood, a permitting and review body can draft ordinances and regulations governing development on alluvial fans.

> The depth of flooding shown on the FIRMs for alluvial fans should be considered an <u>estimate</u> for the entire fan area, not an absolute value. Alluvial fan flood depths may vary from the given flood depth by several feet, depending upon local conditions. For that reason, site-specific analysis should be undertaken to accurately determine flood depth for a retrofitting project.

In the communities investigated for this manual, flood hazard delineation methods and flood hazard zoning regulations varied widely. Some existing alluvial fan flood hazard ordinances and zoning regulations establish "no build" zones and zones where development is allowed contingent on mitigation or retrofitting. A few ordinances address impact loading, downfan flood impacts, and freeboard. Tasks, such as mitigating fan flood hazards, recommending floodproofing techniques, and providing comprehensive fan flood protection are being accomplished in different ways locally from community to community.

INTEGRATION WITH COMMUNITY PLANNING

> A well-integrated approach to floodproofing and fan flood mitigation can reduce flood losses and possibly lower flood insurance costs where the measures are approved by FEMA.

Residential retrofitting methods should be compatible with comprehensive alluvial fan flood hazard mitigation and master drainage plans. Integration of the retrofitting method with existing drainage and mitigation measures (such as streets designed as conveyance channels) can reduce flood damage in densely populated neighborhoods.

Flood damage incurred on densely urbanized fans may invoke disagreement regarding historical flooding conditions or newly created flooding from flow deflection.

There are areas on virtually every fan of such extreme flood hazard that hazard avoidance is essential. If "no build" zones have been designated, building permits should be denied within these zones (often in the fan channelized and braided zone). Residences constructed in these high hazard zones prior to the berms, floodwalls, reinforced walls, or landscaping may result in deflection of the flow. Buildings oriented to reduce hydrodynamic loading may also redirect flows. Flow deflection may result in increased flood hazards to residences that historically were subject to little or no flood hazard. Elevation of the structure on supporting members or the conveyance of flow between buildings potentially exposes a downfan property to increased flooding. Thus all proposed retrofitting measures should be designed to avoid increasing flood hazards to other properties. Local ordinances may specify that the proposed retrofitting must be able to pass the flood through the property or development without increased damage to others. NFIP regulations concerning conveyance around a new structure in AO Zones may also be applied to retrofitting situations.

Integration of floodproofing methods with master drainage and fan-wide comprehensive mitigation plans will have the benefit of reducing downfan flood damage. Floodproofing should direct flows into desirable paths such as streets or dedicated flow-through areas. Regulations may require setbacks from existing channels. Floodproofing should not encroach on setback distances.

Finally, structural flood mitigation and floodproofing measures should also be integrated into the community master emergency plan to avoid impeding emergency services during a flood event. The diversion of flow by a floodwall into a designated emergency route may eliminate access to areas of the fan by emergency equipment. Community emergency planning information is available through the community planning department.

STRUCTURAL FLOOD PROTECTION SOLUTIONS AND PLANNING

Ideally, communities will have implemented master drainage and flood mitigation plans prior to development on alluvial fans. Master plans can address hazard avoidance alternatives that set aside areas with high flood hazard potential as open space or parklands. In addition, master drainage plans can include structural mitigation aimed at protection of developed portions of the fan, such as flow diversion channels and debris basins. These mitigation measures may eliminate the need to retrofit residences and may be more technically and economically feasible for the community.

Master development or drainage plans can prohibit development in high flood hazard areas (zone near the fan apex) where the potential for catastrophic flooding, particularly related to mud and debris flows, exists. Most master plans, however, permit development in moderate to low flood hazard areas. Within the context of the master plans for drainage or development, regulation of unit layout and density can enhance hazard avoidance by designing for passage of floodwaters, dedicating areas for sediment deposition and ponding, and assigning emergency access routes. Approval of residential retrofitting measures should be contingent on compatibility with the master plan components. Retrofitting can negatively impact the downfan flood hazards when not considered in the context of a master drainage plan.

Residential retrofitting measures may include elevation, floodwalls, levees, site grading, dry floodproofing, wet floodproofing, landscaping, or building reinforcement. Retrofitting measures can be either permanent, contingent, or emergency. In general, fan flooding occurs with very little warning, limiting the effectiveness of contingency or emergency measures that require human intervention.

> San Diego County, California, amended its flood damage prevention ordinance (Ordinance No. 7534) to require that any development on alluvial fans must not disrupt natural alluvial fan processes. The intent of this ordinance is that unhindered flow conditions will cause less damage than if the flow is disturbed in a haphazard way. At an enforcement level, the ordinance requires that flood flows must be returned to natural conditions upon exiting a property. This approach is more feasible where development density is low and engineered obstructions cause only limited disruption to fan hydraulics. Where development density is moderate to high, this approach is not feasible because natural fluvial conditions no longer exist.

POTENTIAL DOWNFAN IMPACTS OF FLOODPROOFING

Homeowners, community officials, and design professionals must consider the hydraulic effects of proposed retrofitting measures on downfan properties. Flood protection must not create additional damage and liability during a flood event. The potential impacts of retrofitting measures fall into two categories: 1) damage resulting from the diversion of flow from one property onto another; and 2) constriction of flow upstream resulting in higher flow depth and velocities. Three scenarios are presented to illustrate the potentially damaging impacts related to retrofitting.

Scenario I: Flow diversion to contiguous properties as a result of retrofitting.

Retrofitting measures such as a floodwall, levee, or fill embankment divert the flow to an adjoining property or property across the street that has not been delineated within the flood hazard zone or has not been historically inundated. Potential flood damage to the unprotected property may be avoided by redirecting the flows back to natural drainage ways or open spaces or insuring sufficient street and stormwater system capacity.

Scenario II: Altered upfan flow depths and velocities as a result of retrofitting.

An existing residential structure located with no development upfan has been retrofitted to protect against shallow sheet flow. In proposed further development of the subdivision, two houses would be built directly upfan of this house. The two new houses would constrict the flow between them, subjecting the original house to a greater inundation flow depth, velocity impact, and scour than predicted. The retrofitting measures against shallow flows are then inadequate to protect the original structure against the new flooding conditions.

This flooding scenario may be avoided by diverting the flow upfan of the new houses to existing drainage ways or streets. A community should place performance requirements on developments in fan areas to avoid this situation, requiring the construction of a diversion facility.

Scenario III: Increased flow volumes to specific downfan areas as a result of retrofitting.

Streets may be designed on alluvial fans to convey floodwaters as a mitigation measure. The capacity of these streets to convey upfan floodwaters may be exceeded if upfan urbanization or diversion measures are allowed that increase runoff into the streets. The volume of water reaching the lower developed portion of the fan will increase, thus subjecting potential buildings to greater flow and potential damage. To avoid increased damage to the downfan properties, upfan storage or flow diversion to an undeveloped location would have to be designed. Increasing the street conveyance capacity is generally not cost effective. This scenario illustrates the need for wise community planning prior to new development.

DETAILED DESIGN PRACTICES

OPEN SPACE AND STRUCTURE RELOCATION

There is potential for flooding over the entire surface of active alluvial fans. The channelized zone experiences the greatest depths, velocities, and sediment transport capability and is particularly prone to severe flood hazards. NFIP regulations may not adequately address all the hazards presented on the fan. Existing pre-FIRM structures are subject to substantial damage/substantial improvement criteria.

As part of a master drainage or development plan on the fan, some communities may consider purchasing areas of high flood hazard and relocating existing homes. Public ownership of these lands allows the greatest flexibility in comprehensive fan flood hazard management. The floodplain administrator can use the channelized zone to build flood mitigation structures such as debris basins and channels and dedicate open space for sediment deposition. Removing development from areas of highest hazard relieves the community of all or part of the costs related to flood mitigation studies, regulation of building improvements, and cleanup costs following a flood event. Open space also enhances the aesthetics of the fan.

Master drainage plans, hazard zone delineation, building codes, public purchase of land, open space dedication, and land trades are all considerations for structure relocation. (Often, however, properties in the fan are quite expensive, which would preclude a buyout.) Although relocation is a significant undertaking, it may be economically feasible considering the potential threat to lives and property on the upper reaches of the fan.

STRUCTURAL DESIGN - BUILDING CODES

Minimum structural design requirements for buildings in flood-prone areas have been established by the NFIP and the International Conference of Building Officials Uniform Building Code (UBC). The UBC, generally adopted in most western states, addresses building requirements for structures located in riverine flood hazard areas designated by approved flood insurance maps or the local floodplain management ordinance. Although the UBC does not specify building requirements related to alluvial fan flood hazards, many of the floodproofing concepts discussed can be applied.

Under the UBC, building design is required to withstand the forces associated with the base flood level of the 100-year flood event. The UBC requires the use of well-established engineering principles in the design of structural members to resist flotation, stress increases, overturning, collapse, or permanent lateral movement due to flood-induced loads (hydrostatic, hydrodynamic, and impact loads). Reconciliation of discrepancies between the different codes can be made by referring to the Code Compatibility Report, Appendices A through F, (FEMA, Oct. 1992).

Within designated A zones (equivalent to FEMA FIRM Zone A), the UBC requires that the lowest floor of new or substantially improved residential buildings be situated at or above the base flood elevation. The Code makes an exception for enclosed spaces below the base flood elevation, provided that the space is used only as "building access, exits, foyers, storage, or parking garages."

ELEVATION TECHNIQUES

New or substantially improved/damaged structures must be elevated at least to the flow depth indicated on the FIRM, or at least two feet if no depth is given. Elevation can effectively remove the habitable positions of a structure from contact with floodwaters and in most instances mud and debris flows. The NFIP and UBC require that residential structures be elevated to the height of the base flood elevation (or flow depth in the case of alluvial fans). Local regulations may also require additional freeboard. In areas of potential mud, debris, and high-velocity flows, additional freeboard should be considered. Although elevating structures may be an expensive flood protection technique for retrofitting homes, it may still present a viable retrofitting option and should be evaluated for feasibility.

Elevation on posts or piles permits floodwaters to pass underneath the structure, causing little obstruction to flow. A properly designed pile will carry all inherent structural loads and lateral loads (hydrodynamic and impact) expected during the design flood. In addition to normal geotechnical concerns, the most important design consideration for piles is potential scour (refer to discussion of scour in Chapter IV). Spacing of posts and piles should be relatively wide to minimize flow constriction or the collection of debris found in the watershed or on the fan. The failure of supporting members could potentially cause more damage than inundation of a non-elevated structure.

Elevation of a residence on fill is a design practice for new homes on alluvial fans in the southwestern United States and is regulated by local ordinances. This floodproofing technique is most viable on fans regulated by a master drainage plan that specifies flood conveyance facilities and drainage ways. Fill slopes can be oriented to divert flow in a desired direction. Elevation on fill, in contrast to piles and posts, may impose a significant obstruction to the flood path; therefore, constriction and diversion of flow onto adjacent properties is a concern. Fill should consist of easily compactible sand or gravel. Application and compaction should follow standard engineering practices. The toe of the fill slope must be protected from scour. This slope protection should be extended at least two feet below ground surface. The fill slope above the ground surface should be protected by rock riprap or vegetation to at least the base flood level.

DRY FLOODPROOFING

Dry floodproofing consists of the application of an impermeable membrane to the walls of a structure to the flood protection elevation. Dry floodproofing is appropriate for shallow flooding zones where the base flood elevation is not determined. This technique can be used for brick veneer and masonry structures where floor slabs are rigidly connected to walls.

> Dry floodproofing is not allowed by FEMA for new or substantially improved or damaged residential structures located in an SFHA.

External dry floodproofing consists of an impervious layered sheet material such as tar or asphalt bitumen applied to the exterior of the building. Excavation around the foundation may be required to externally floodproof building material below the ground surface subject to soil saturation during the flooding event. Membrane materials should be designed to resist all expected flooding conditions including scour, abrasion, impact, and hydrostatic and hydrodynamic pressures. On alluvial fans subject to mud and debris flows, the external membrane cannot be exposed to the flow. External membranes may not be required on the downfan side of a building.

Internal membranes may also be used but, in general, are more prone to leaks than external membranes, which are held tightly against the structure by hydrostatic pressure. As with external membranes, any points of discontinuity may leak and require additional floodproofing during installation. Leaks are most likely to occur at membrane seams, construction joints and corners, and where pipes and ducts penetrate the membrane.

Waterproofing materials that may be considered include polyethylene, PVC, polyurethane, and polyisobutylene. This method also requires rigid connections between floor slabs and walls to prevent leaking. The foundation and walls should be protected against scour, decay, and cracking with the use of treated building materials and armored backfill. For existing structures being considered for remodeling or rehabilitation, this will require the application of additional foundation materials to standing walls.

BUILDING REINFORCEMENT

Structures located in areas subject to hydrodynamic and impact forces from water, mud, and debris flows can be protected against damage and collapse through structural reinforcement of upfan walls. Reinforcement may include the addition of structural supporting members or an exterior facade, or the removal and replacement of existing walls.

In conjunction with the reinforcement of upfan walls, removal of openings in the upfan wall should be investigated. If these openings are removed, they may need to be replaced with openings on other walls. Weak points in the bearing wall, such as windows, doors, and utility connections, may leak or fail under flooding conditions and should be reinforced and floodproofed or eliminated. Window wells should be retrofitted with reinforced waterproof coverings and backfilled. Doors and windows located wholly or partially below the expected base flood level should likewise be eliminated or replaced with reinforced water-tight coverings up to the level of the base flood plus freeboard. Reinforcement of upfan walls should be designed for impact pressures and hydrodynamic loading related to mud and debris flows.

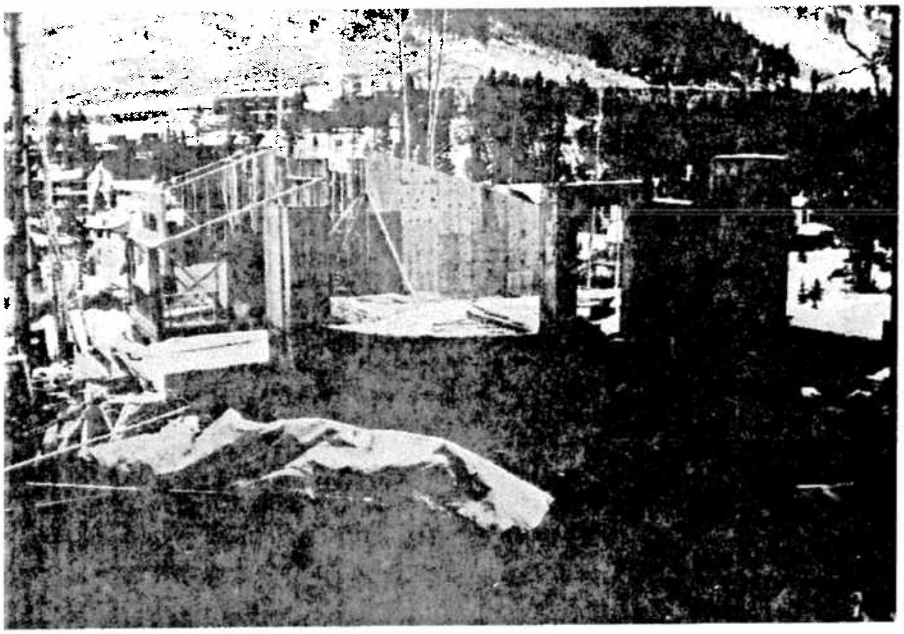


Figure D-5: Reinforced Upfan Walls

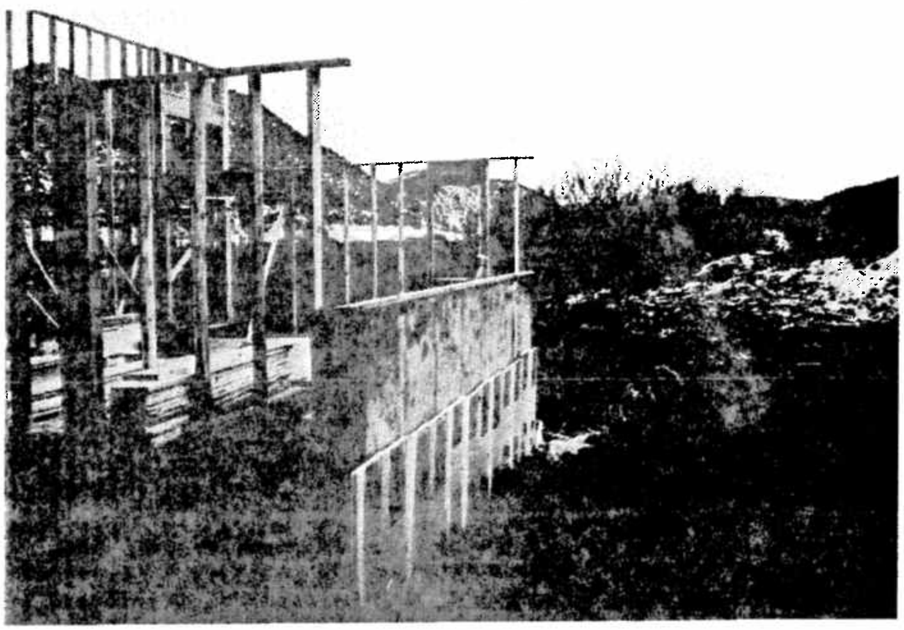


Figure D-6: Reinforced Upfan Walls

FLOODWALLS AND LEVEES

Floodwalls and levees may be constructed on the upfan portion of a building to protect it from the forces of moving water and inundation. This method of floodproofing may consist of blocks (brick or cinder), concrete, railroad ties, and other construction materials that would withstand the design hydrostatic, hydrodynamic and impact loads. The height of floodwalls should be based on a specified design maximum flow depth plus freeboard. The estimated freeboard should include velocity head, wave height, potential flow runup, potential for sediment deposition against the wall, and surging. Floodwalls should be constructed below grade to provide protection from scour. Stability design should take into account material removed by scour.

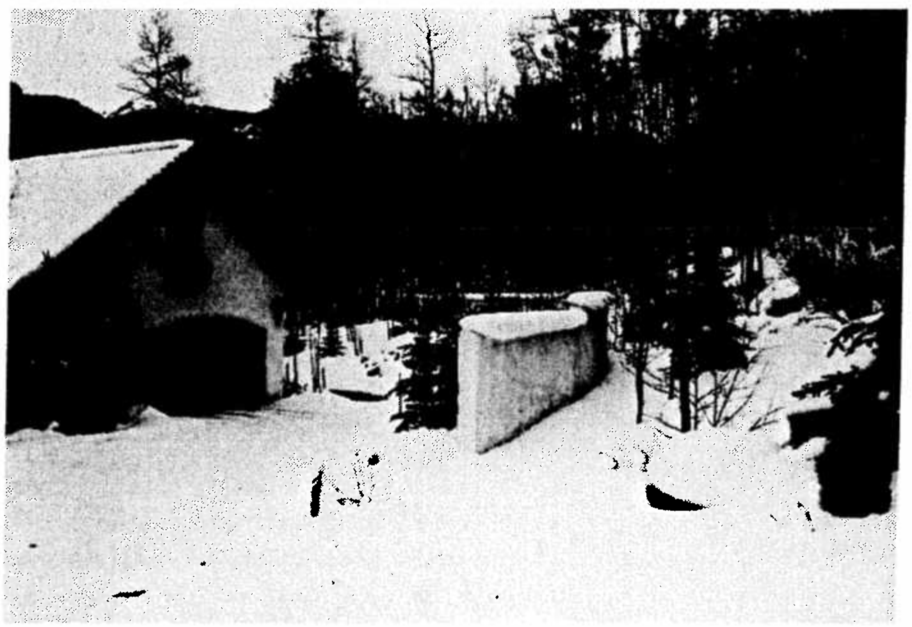


Figure D-7: Floodwall Protecting Residence in Colorado

Levees are raised fill embankments along an existing or planned conveyance channel designed to confine or prevent inundation of the floodplain. Frequently utilized on riverine floodplains, leeves may require some modification when applied on alluvial fans. On alluvial fans, levees can divert flow around a subdivision or residence, or they may provide protection along a natural or engineered channel through a developed area. Levees should be designed to protect against scour and levee slope erosion.

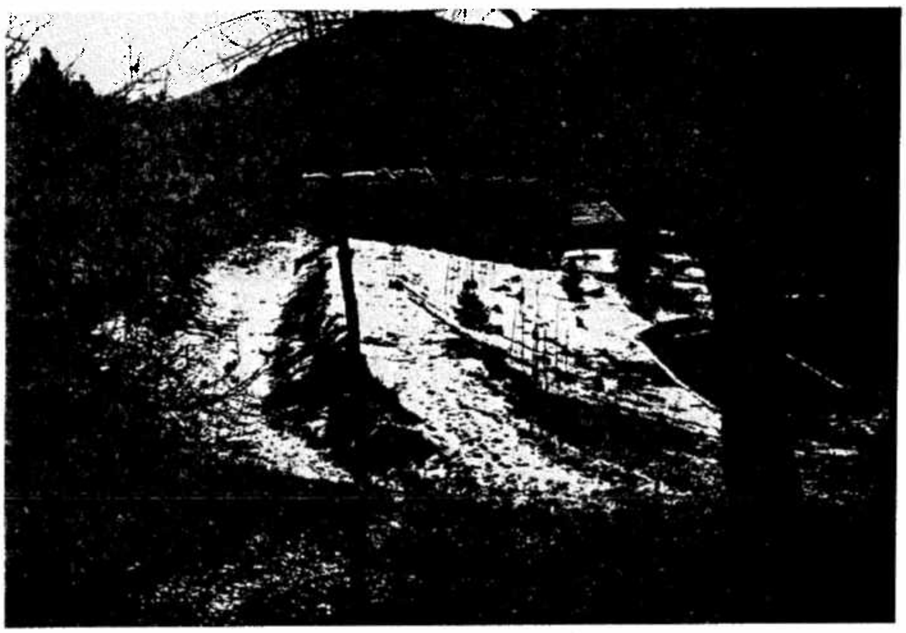


Figure D-8: Debris Flow Levee

On steep alluvial fan slopes, the complete enclosure of a structure by a floodwall and levee is not usually necessary. The downfan side of the property does not require a floodwall when the ground slopes significantly. On the other three sides, the retrofitting design should consider access to the building and grounds. Closures should not be included in the protective structure because failure of the closure may cause complete failure of the floodwall/levee. In some instances, floodwalls have been used primarily for protection against mud and debris flows, without restricting seepage but assuring structural stability.

The U.S. Army Corps of Engineers (draft report, undated) recommends avoiding this retrofitting alternative for mud and debris flows where the overtopping or failure of levees and floodwalls can cause catastrophic damage in excess of the damage that would have occurred in an area devoid of protection. In addition, mud and debris flow deposition on the upfan side of the wall or levee may increase the potential for overtopping or runup.

Floodwalls and levees are an excellent method of flood protection for an existing structure. Their use is most appropriate in fan flood hazard zones characterized by low and moderate velocity flows or mud flows in low density development. Design height for floodwalls and levees should be limited to three to four feet. This restricts their use where scour and debris conditions are prevalent.

Figure D-9: Diversion Levee in Colorado

SITE GRADING

Site grading on alluvial fans is constrained by the fan slope, street and driveway cuts, and drainage. Site grading can be effective as a flood protection method for existing homes if the predicted flooding is relatively shallow and the runoff from the property can be incorporated into and handled by the stormwater facilities designed as part of a drainage plan. Site grading should be considered for the sheet flow zone of alluvial fans (< 1 foot in depth). Grading a lot for flood protection may consist of grading the lawn away from the house at 1:12 slope for a minimum distance of six feet perpendicular to the house (UBC, 1991), creating a swale around the house, sloping the lawn or yard to the street or driveway, or establishing grading to work in conjunction with other damage reduction measures such as levees. It is important to determine if waters concentrated by a grading plan will cause unnecessary erosion or flow on adjacent properties or overload existing storm facilities or streets.

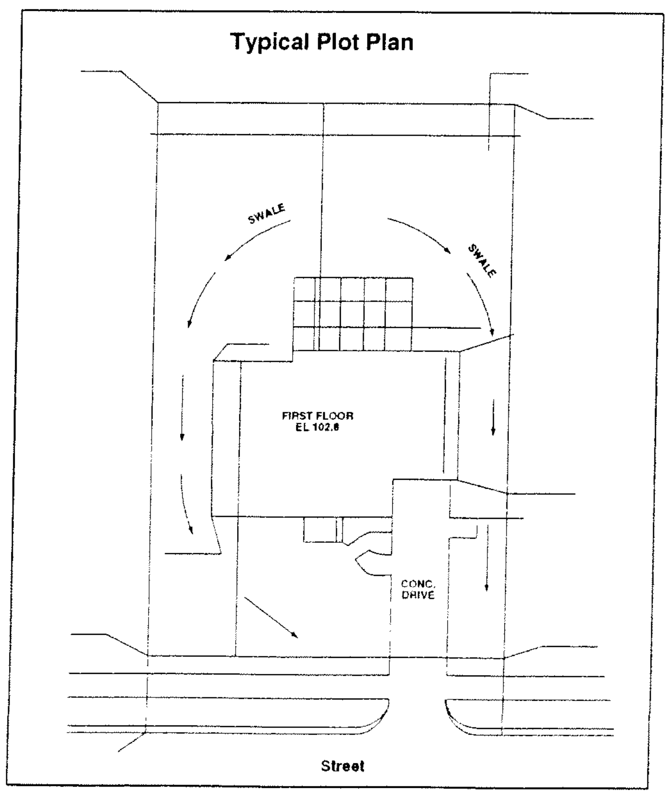


Figure D-10: Typical Subdivision Plot Plan

Care should be taken to avoid the risk of flood damage through negative site grading. Drainage ways and depressions should be located to minimize ponding and diversion of floodwaters near the structure. Excavation of the fan slope for a lawn may direct floodwaters toward the structure, causing more damage than if the yard were left at grade. Finally, any natural drainages or levees should remain undisturbed.

A ditch or shallow trough excavated around a structure or the property perimeter will collect and convey floodwater. The site may be graded to convey floodwater to the ditch. The disposal of ditch water should be considered with respect to the fan-wide master drainage plan. It may be possible to pass the water around the protected structure, then disperse the flow before leaving the property. Even in fan areas where the sediment loads are not important, ditches or troughs will require frequent maintenance for maximum effectiveness when a flood event occurs.

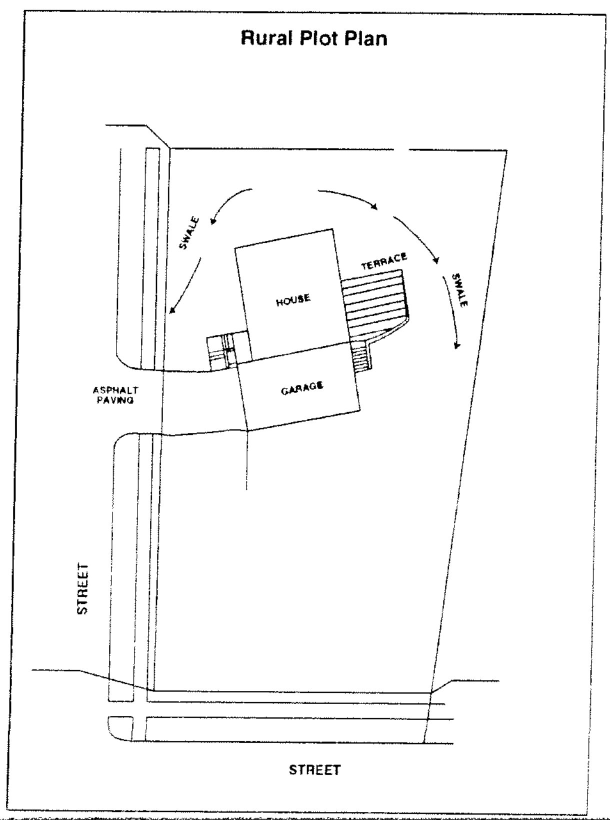


Figure D-11: Typical Rural Plot Plan

LANDSCAPING

Standard landscaping designs may be applied for floodproofing measures in fan zones of shallow flooding (less than one foot). Flood flows may be dispersed with landscaping that splits the flow with wedged flow barriers or spreads the flow through vegetated areas. Landscaped low mounds may be oriented to divert flows to an on-site drainage path or off-site flow conveyance area, such as the street or dedicated flow-through area. Mounds may be vegetated or armored to withstand erosion from low-velocity water flow and raindrop impact. Landscaping may not be compatible with flows having high sediment loads. Sediment deposition may render the landscaping design ineffective.

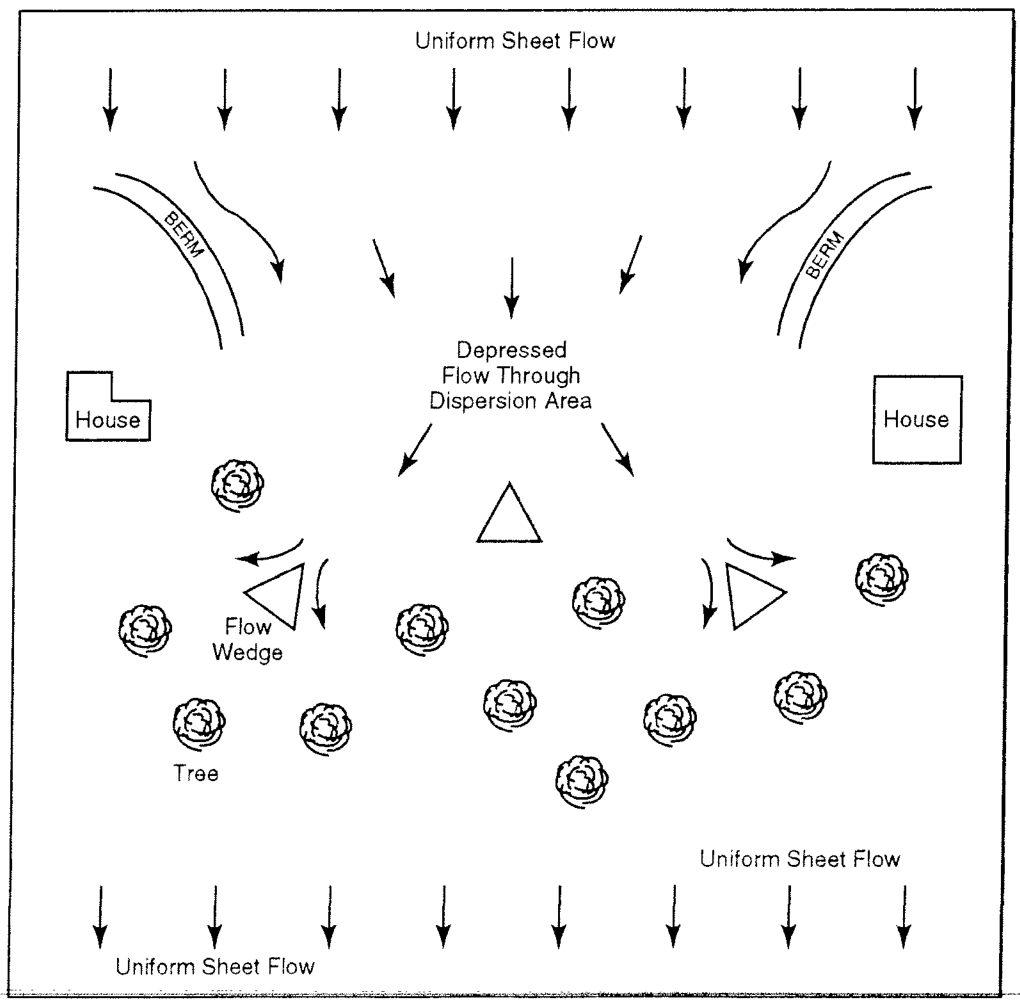


Figure D-12: Typical Dispersion Design

APPENDIX E

BENEFIT/COST ANALYSIS SOFTWARE

A new CD that includes the software "Benefit/Cost Analysis for Flood" and the accompanying user's guide is scheduled for release by fall 2001. The new software has been upgraded to operate in a runtime version of Microsoft Access® included with the program.

Users who need help with the new software or have questions about benefit/cost analysis can contact the FEMA Benefit-Cost Analysis Hotline, by phone at (301) 670-3399, ext. 710, or by email at bchotline@urscorp.com.