Department of the Army
U.S. Army Corps of Engineers
Washington, DC

CECW-EC

Engineer Manual* 1110-2-2104

19 December 2024

Engineering and Design

Strength Design for Reinforced Concrete Hydraulic Structures

FOR THE COMMANDER:

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Purpose. This engineer manual provides guidance for designing reinforced concrete hydraulic structures by the strength design method. Plain concrete and prestressed concrete are not covered in this manual.

Applicability. This manual applies to all Headquarters, United States Army Corps of Engineers commands having civil works responsibilities. The user of this engineer manual is responsible for incorporating the environmental operating principles wherever possible.

Distribution statement. Approved for public release; distribution is unlimited.

Proponent and exception authority. The proponent of this manual is the Headquarters, United States Army Corps of Engineers, Engineering and Construction Division. The proponent has the authority to approve exceptions or waivers to this manual that are consistent with controlling law and regulations. Only the proponent of a publication or form may modify it by officially revising or rescinding it.

*This manual supersedes EM 1110-2-2104, dated 30 November 2016.

Summary of Change

EM 1110-2-2104
Strength Design for Reinforced Concrete Hydraulic Structures

This major revision, dated 01 November 2023:

- Updates main design references to ACI 318-19.
- Adds guidance for glass fiber reinforced polymer bars.
- Revises bar spacing requirements to be compatible with ACI 318-19.
- Clarifies requirements for mass concrete.
- Revises serviceability design requirements.
- Clarifies load descriptions.
- Revises and updates design load factors.
- Clarifies earthquake design guidance.
- Adds guidance for shear design of sections without shear reinforcement.
- Updates the examples in Appendix D.
- Updates example load combinations in Appendix E.
- Adds the Glossary of Terms

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Chapter 1
Introduction

1–1. Purpose

This engineer manual provides guidance for designing reinforced concrete hydraulic structures by the strength design method. Plain concrete and prestressed concrete are not covered in this manual.

1–2. Distribution statement

Approved for public release; distribution is unlimited.

1–3. References

See Appendix A.

1–4. Records management (recordkeeping) requirements

The records management requirement for all record numbers, associated forms, and reports required by this publication are addressed in the Army Records Retention Schedule. Detailed information for all related record numbers is located on the U.S. Army Corps of Engineers (USACE) Records Management Site https://usace.dps.mil/sites/INTRA-CIOG6/SitePages/Records-Management.aspx. If any record numbers, forms, and reports are not current, addressed, and/or published correctly, see DA Pam 25-403 for guidance.

1–5. Associated publications

This section contains no entries.

1–6. Discussion

This manual covers requirements for design of reinforced concrete hydraulic structures (RCHS) by the strength design method. It is applicable to all hydraulic structures. This manual also contains provisions for design of structures that are satisfactory for both serviceability and ultimate strength. Industry design and construction standards have been adopted as applicable. The users of this manual are also responsible for incorporating the environmental operating principles whenever possible. These principles are found at: http://www.usace.army.mil/Missions/Environmental/EnvironmentalOperatingPrinciples.aspx.

1–7. Background

a. Industry design and construction standards (American Concrete Institute [ACI], American Association of State Highway and Transportation Officials [AASHTO], etc.)

are adopted as applicable to provide safe, reliable, and cost-effective hydraulic structures for civil works projects. RCHS are directly subjected to submergence, wave action, spray, icing, or other severe climatic conditions, and sometimes to a chemically contaminated atmosphere.

b. Typical RCHS are stilling basin slabs and walls; concrete-lined channels; submerged features of powerhouses and pump stations; spillway piers; spray and training walls; floodwalls; submerged features of intake and outlet structures (towers, conduits, and culverts); locks and dams; guide and guard walls; submerged retaining walls, and other structures used for flood barriers, conveying or storing water, generating hydropower, water-borne transportation, and for restoring the ecosystem.

c. Satisfactory long-term service requires that the saturated concrete be highly resistant to deterioration from daily or seasonal weather cycles and tidal fluctuations at coastal sites. The often relatively massive members of RCHS must have adequate density and impermeability and must sustain minimal cracking for control of leakage and for control of corrosion of the reinforcement. Most RCHS are lightly reinforced structures (reinforcement ratios less than 1 percent) composed of thick walls and slabs that have limited ductility compared to the fully ductile behavior of reinforced concrete buildings (in which reinforcement ratios are typically 1 percent or greater).

d. This manual describes typical loads for the design of RCHS. Load factors are provided. The load factors resemble those shown in ACI 318-19 but are modified to account for unique loads on hydraulic structures, serviceability needs of hydraulic structures, and the higher reliability needed for critical structures.

e. RCHS typically have very long service lives. A service life of 100 years is the basis for the requirements of this manual.

1–8. General requirements

a. Reference. RCHS are designed with the strength design method according to ACI 318-19, except as specified hereinafter. The notations used are the same as those in the ACI 318-19, except as defined herein.

b. Performance. Design of civil works projects must ensure acceptable performance of all RCHS during and after each design event. Three levels of performance for stability, strength, and stiffness are used to satisfy the structural and operational requirements for load categories with three expected ranges of recurrence (usual, unusual, and extreme). Chapter 3 describes the strength and serviceability requirements for design.

c. Glass fiber reinforced polymer bars.

(1) RCHS designed with this engineer manual are expected to use steel reinforcing bars. Glass fiber-reinforced polymer (GFRP) bars are emerging as an alternative to steel bars because they are noncorrosive. They have been successfully used for a number of applications. However, GFRP bars are anisotropic, displaying high tensile

strength only in the direction of the glass fibers. Shear strength and compressive strength is less than for steel. In addition, GFRP materials do not yield. They are elastic until failure and are, therefore, nonductile. GFRP bars have a lower modulus than steel bars and are also subject to creep.

(2) Due to differences in the physical and mechanical behavior of GFRP materials compare to steel, ACI has unique guidance for using GFRP bars. GFRP bars have not been investigated for use in RCHS. Using GFRP must not be used for RCHS unless a study of strength, serviceability, and deflection is performed with consultation and approval by CECW-EC. The study must be specific to the intended application.

1–9. Scope

This manual is written with sufficient detail to provide the designer not only with design procedures, but also with examples of their application. Also, derivations of the combined flexural and axial load equations are given to increase the designer's confidence and understanding. Chapter 2 presents general detailing requirements. Chapter 3 gives strength and serviceability requirements, including load factors and limits on flexural reinforcement. Chapter 4 includes design equations for members subjected to flexural and/or axial loads (including biaxial bending). Chapter 5 presents guidance for design for shear, including provisions for curved members and special straight members. Appendixes include:

a. Appendix A: References.

b. Appendix B: Design Equations for Flexural and Axial Loads.

c. Appendix C: Investigation Examples.

d. Appendix D: Design Examples.

e. Appendix E: Load Combinations for Design of Typical Reinforced Concrete Hydraulic Structures.

f. Appendix F: Commentary on Chapter 3.

g. Appendix G: Commentary on Chapter 5.

h. Glossary of Terms (Acronyms, Notation, and Unit Conversion Factors)

1–10. Mandatory requirements

a. RCHS must be designed according to this manual.

b. GFRP must not be used for RCHS unless a study of strength, serviceability, and deflection is performed with consultation and approval by CECW-EC.

Chapter 2
Details of Reinforcement

2–1. General

This chapter presents guidance for furnishing and placing steel reinforcement in various concrete members of hydraulic structures.

2–2. Quality

The type and grade of reinforcing steel should generally be American Society for Testing and Materials (ASTM) A615, Grade 60. Reinforcement of other types and grades that comply with the requirements of ACI 318-19 and paragraph 3–4 may be used as needed.

2–3. Reinforcement

Reinforcement is categorized as either primary or secondary reinforcement. Primary reinforcement consists of the bars required for strength. Secondary reinforcement consists of bars that serve as confining reinforcement (ties, etc.), or as reinforcement to control shrinkage or changes resulting from variations in temperature. Unless the plans and specifications specify that the primary reinforcement is to be on the outside, the width of secondary reinforcement should be subtracted when calculating effective depth (d) of the section.

2–4. Anchorage and bar development

The anchorage, bar development, and splice requirements must conform to ACI 318-19 and to the requirements presented below. Since the development length depends on a number of factors such as concrete strength and bar position, function, size, type, spacing, and cover, the designer must indicate the length of embedment required for bar development on the contract drawings.

2–5. Hooks and bends

Hooks and bends must be according to ACI 318-19. Some RCHS members can require larger bars. Detailing of bends for larger bars must consider the width of the bars and the actual bend radii to assure proper clear spacing and concrete cover. Bends with larger bars at corners, block outs, nosing, or other changes in geometry may require additional reinforcement where large spaces outside of bend are left unreinforced.

2–6. Bar spacing

a. *Minimum spacing*. The clear distance between parallel bars must not be less than 1.5 times the nominal diameter of the bars, nor less than 1.5 times the maximum size of coarse aggregate. No. 14 and No. 18 bars should not be spaced closer than 6 in. (15 cm) and 8 in. (20 cm), respectively, center to center.

b. *Maximum spacing*. To control cracking, the maximum center-to-center spacing of both primary and secondary reinforcement must not exceed 12 in. (30 cm). In addition, the primary flexural reinforcement must meet serviceability requirements for maximum reinforcement spacing from ACI 318-19, with modifications as stated in the following paragraphs.

(1) *Clear cover*. The clear cover used in RCHS is typically much greater than ACI 318-19 requirements. Therefore, the maximum spacing may be calculated using a value for clear cover of the lesser of 2.5 inches or the actual clear cover.

(2) *Usual loads*. For usual loads, the maximum spacing requirements from ACI 318-19 will be met by 12 in. (30 cm) maximum spacing when the serviceability requirements specified in Chapter 3 are met.

(3) *Unusual loads*. ACI 318-19 allows a stress of 2/3 f_y to be used to compute maximum spacing requirements. Instead, either the reinforcement service stress limit in paragraph 3–4 or the actual service stress should be used to compute the maximum spacing.

(4) *Extreme loads*. For extreme loads, control of cracking is not a performance requirement according to paragraph 3–1e(4). Therefore, the spacing requirements of ACI 318-19 are not required to be met for extreme loads.

2–7. Concrete protection for reinforcement

The minimum cover for reinforcement must conform to the limits shown below for the various concrete sections. The dimensions indicate the clear distance from the edge of the reinforcement to the surface of the concrete (Table 2–1).

Table 2–1
Minimum clear distance from the edge of the reinforcement to the surface of the concrete

Concrete Section	Minimum Clear Cover of Reinforcement
Unformed surfaces in contact with foundation	4 in. (10 cm)
Formed or screeded surfaces subject to cavitation or abrasion erosion, such as baffle blocks and stilling basin slabs	6 in. (15 cm)
Formed and screeded surfaces not subject to cavitation or abrasion	–
Equal to or greater than 24 in. (61 cm) thick	4 in. (10 cm)
Greater than 12 in. (30 cm) and less than 24 in. (61 cm) thick	3 in. (7.5 cm)
Equal to or less than 12 in. (30 cm) thick	According to ACI 318-19

Note: In no case can the cover be less than 1.5 times the nominal maximum size of aggregate, or 2.5 times the maximum diameter of reinforcement.

2–8. Splicing

a. General. Bars must be spliced only as required, and splices must be indicated on contract drawings. Splices at points of maximum tensile stress should be avoided. Where such splices must be made, they should be staggered. Splices may be made by lapping of bars or butt splicing.

b. Lapped splices. Bars larger than No. 11 must not be lap spliced. Tension splices should be staggered longitudinally so that no more than half of the bars are lap spliced at any section within the required lap length. If staggering of splices is impractical, applicable provisions of ACI 318-19 must be followed.

c. Butt splices.

(1) *General*. Bars larger than No. 11 must be butt spliced. Bars No. 11 or smaller should not be butt spliced unless clearly justified by design details or economics. Due to the high costs associated with butt splicing of bars larger than No. 11, especially No. 18 bars, careful consideration should be given to alternative designs that use smaller bars.

(2) *Fabrication.* Butt splices must be made by either welding or an approved mechanical splicing method according to the provisions contained in the following paragraphs and in ACI 318-19. Tension butt splices should be staggered longitudinally as shown in Table 2–2.

Table 2–2
Longitudinal stagger of tension butt splices

Bar Size	Longitudinal Stagger
≤ No. 11	ACI 318-19 required lap length*
> No. 11	No less than 5 ft (1.5 m)*

Note:
*No more than half of bars are spliced at any one section.

(3) *Mechanical splicing*. Mechanical splicing must be made by an approved exothermic, threaded coupling, swaged sleeve, or other positive connecting type and designed according to ACI 318-19. The designer should be aware of the potential for slippage in mechanical splices and should insist that testing provisions be included in the contract documents and be used in the construction work.

2–9. Temperature and shrinkage reinforcement

a. In the design of structural members other than mass concrete for temperature and shrinkage stresses, the area of reinforcement must be a minimum of 0.003 times the gross cross-sectional area, half in each face, except as modified in the following paragraphs. However, past performance and/or analyses may indicate the need for an amount of reinforcement greater than this if the reinforcement is to be used for distribution of stresses as well as for temperature and shrinkage. Generally, for ease of placement, temperature and shrinkage reinforcement will be no less than No. 4 bars at 12 in. (30 cm) in each face.

b. The area of shrinkage and temperature reinforcement need not exceed the area equivalent to No. 9 bars at 12 in. (30 cm) in each face. Adding more reinforcement to thick sections for control of temperature and shrinkage cracking is generally not effective. Thick sections are designed as mass concrete described in paragraph 2–9h. Temperature and shrinkage reinforcement may not be required for mass concrete when properly designed.

c. Monolith length and control joint spacing may dictate the requirements for more shrinkage and temperature reinforcement than indicated in paragraph 2–9a. Good design practice can minimize cracking and control visibly wide cracks by minimizing restraint, using adequate reinforcing, and using control joints. Control joints generally include monolith joints, expansion joints, contraction joints, and construction joints.

d. Additional considerations should be addressed when longer monolith lengths are required (road closures, pump stations, gate monoliths, long walls, etc.) to provide a practical design. Table 2–3 lists minimum shrinkage and temperature reinforcement ratios for various joint spacings. Shrinkage and temperature reinforcement in the transverse (shorter) direction must be according to paragraph 2–9a.

Table 2–3
Minimum shrinkage and temperature reinforcement ratios for various joint spacings

Length Between Control Joints	Minimum Temperature and Shrinkage Reinforcement Ratio, Grade 60
Less than 30 ft (9 m)	0.003
30–40 ft (9–12 m)	0.004
Greater than 40 ft (12 m)	0.005

e. Within a monolith, using contraction joints is an effective method for crack control; consideration for more shrinkage and temperature reinforcement should be weighed against using contraction joints. A balance between additional shrinkage and temperature reinforcement and contraction joint spacing is left to the engineer's discretion. In general, using fewer contraction joints with slight increases in shrinkage and temperature reinforcement provides a more practical design with good service performance.

f. For longer monoliths or concrete features, contraction joints within the monolith should be considered, and if used, should be spaced no more than 1 to 3 times the height of the monolith or the feature's transverse (shorter) dimension. Typically, taller, or wider features tend toward the lower end of the stated range. Shorter features of 8 ft (2.4 m) and less tend toward the higher end of the stated range. For example, a 24 ft- (7.3 m-) high wall could have 24 ft (7.3 m) monoliths and no contraction joints requiring 0.3 percent reinforcement. The same 24 ft- (7.3 m-) high wall could have 48 ft (15 m) monoliths with a contraction joint in the center requiring 0.3 percent reinforcement. The same 48 ft (15 m) monolith without the center contraction joint would require 0.5 percent reinforcement. An 8 ft (2.4 m) wall could have 24 ft (7.3 m) monoliths with no contraction joints requiring the 0.3 percent reinforcement.

g. In general, additional reinforcement for temperature and shrinkage will not be needed in the direction and plane of the primary tensile reinforcement when restraint is accounted for in the analyses. However, the primary reinforcement must not be less than that required for shrinkage and temperature as determined above.

h. Mass concrete has special requirements for consideration of temperature and shrinkage.

(1) Many RCHS are large and meet the definition of mass concrete. Mass concrete is defined as any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cementitious materials and attendant volume change to minimize cracking. Since mass concrete generates and gradually dissipates a significant amount of heat of hydration, it goes through a series of volumetric changes due to thermal expansion and contraction as well as shrinkage. The volumetric changes combined with restraint can create sufficient stresses to create cracking in the concrete.

(2) Control of cracking in mass concrete is managed by concrete mix design, placement, curing, temperature control, joints, and construction sequencing. Design of RCHS with mass concrete requires close coordination with the materials engineer. Reinforcing steel is sometimes also used to control cracking. The potential for cracking is determined by thermal studies, which must be performed for all structures meeting the definition of mass concrete. Requirements for temperature and shrinkage reinforcement is determined through thermal analysis. Nonlinear incremental structural analysis may also be required for large complex structures. In addition to shrinkage crack control, the engineer should consider the combination of self-straining stresses from thermal affects with forces from external loads.

i. Concrete materials also affect the temperature and shrinkage performance of concrete.

(1) Additional reduction of drying shrinkage cracking can be achieved by considerations made to the concrete mixture design. There are two main considerations within a concrete mixture design that influence the potential of drying shrinkage cracking. The first is to minimize the paste content of the mixture. This can be achieved by minimizing the total water content used within the concrete mixture, and by keeping the total coarse aggregate content of the concrete as high as possible.

(2) The second consideration in concrete mix design is the type of aggregates used within the concrete mixture. The designer should avoid using aggregates that have high drying shrinkage properties. Materials with these properties include sandstone and greywacke, and aggregates containing excessive amounts of clay. Aggregates that

generally produce concrete with lower drying shrinkage effects are quartz, granite, feldspar, limestone, and dolomite.

(3) Other factors contributing to the amount of shrinkage include the size and shape of the concrete element, relative humidity and temperature of the ambient air, method of curing, degree of hydration, and time. If conducted properly, shrinkage can also be reduced during the curing period after the placement has been completed. Studies have shown that the amount of shrinkage can be reduced by 10 to 20 percent in a concrete mixture if the period of moist curing is extended beyond 7 days, 14 days if cements blended with pozzolans are used. Note that for varying the water/cementitious materials ratio (w/cm) the reduction in shrinkage will also be varied.

(4) In efforts to reduce the amount of drying shrinkage within the concrete mixture design, the designer should consult with a concrete materials engineer and/or EM 1110-2-2000, to verify that the strength and durability requirements of the structure have been satisfied.

2–10. Reinforcement detailing

a. Detailing at joints and corners where bending moment is transferred. This section pertains to situations where a wall or beam connects to another member to form a T- or L-shape, such as a wall stem connecting to the footing of a T-wall. Full moment transfer is attained by turning the hooks from the beam as shown on the left side of Figure 2–1. (Transverse reinforcement and other required reinforcement in the vertical member are not shown.) See retaining wall guidance in ACI 318-19 for additional information.

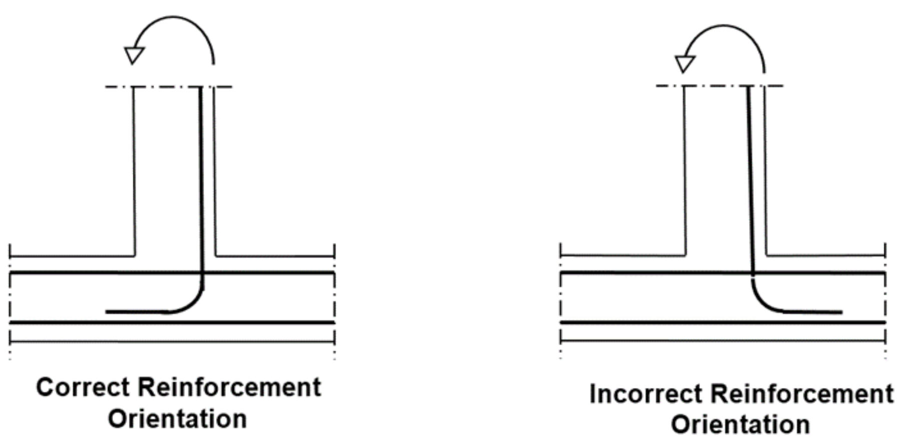

Figure 2–1. Reinforcement detailing at moment connections

b. Detailing for seismic loads.

(1) *General*. Detailing of the reinforcement is very important to the performance of a structure in a seismic event. It is essential to provide confinement in the concrete in those areas where inelastic action (hinges) is anticipated. These areas are typically in

locations of high moments such as the base of culvert walls, the base of the chamber walls, or the chamber walls at points of cross-sectional discontinuities.

(2) *Confinement*. Confinement is necessary in those areas reaching the ultimate compressive stress since these areas will be susceptible to spalling and cracking of the concrete. The confinement keeps the concrete in place, forcing it to continue to carry load, even though it is severely cracked and no longer a continuum. It is important to reinforce corners in a manner that will arrest the anticipated cracks, to maintain the bond and embedment of tension and fixed piles, and to provide adequate anchorage of the reinforcement. The following paragraphs provide a general discussion of adequate details for the reinforcement.

(3) *Example of detailing*. Figure 2–2 shows an example of a pile-founded lock chamber monolith at Olmsted Locks and Dam (L&D)[1] (pile foundation is not shown), which highlights the importance of seismic reinforcement details. The Olmsted L&D is located in a high-seismicity region. More information on seismic details is provided in EM 1110-2-6053 and ACI 318-19. References to design aids that include detailing are also provided in the introduction to ACI 318-19.

(a) *Openings*. When possible, typical reinforcement of openings should include bars inclined at 45 degrees at the corners. This typical reinforcement must be no less than that required for temperature effects. The openings must also have adequate vertical and horizontal steel to resist the internal forces and to confine the concrete in the walls. See Figure 2–2.

(b) *Lock walls*. The base of the lock wall and the thin portions of the wall must have enough reinforcement to provide ductile behavior. The column (wall) between the chamber and the culvert could be highly susceptible to the formation of plastic hinges and should, therefore, be adequately reinforced. This requires, first, that there be adequate steel to form a plastic hinge, and second, that this steel be properly anchored. The anchorage may be achieved by straight embedment or by bending the bars and running them parallel to the base slab moment reinforcement. See Figure 2–2.

(c) *Base slab*. The base slab moment reinforcement must be tied with stirrups in regions of high moment to confine the concrete and provide a region for inelastic deformation to occur. These stirrups must be no smaller or fewer than the reinforcement used for temperature effects.

2–11. Mandatory requirements

a. Anchorage and reinforcing bar development must meet the requirements of paragraph 2–4.

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[1] The Olmsted L&D is located on the Ohio River, on the border between Illinois and Kentucky, just east of Olmsted, IL.

b. Hooks and bends must meet the requirements of paragraph 2–5.

c. Reinforcing bar spacing must meet the requirements of paragraph 2–6.

d. Cover for reinforcing bars must meet the requirements of paragraph 2–7.

e. Splicing of reinforcing bars must meet the requirements of paragraph 2–8.

f. Temperature shrinkage reinforcement must meet the requirements of paragraph 2–9. Primary reinforcement must be no less than that required for temperature and shrinkage.

g. Thermal studies must be performed to determine temperature and shrinkage requirements for mass concrete.

h. When designing for seismic loads, openings must have adequate vertical and horizontal steel to resist the internal forces and confine the concrete. Base slab reinforcement must be tied with stirrups in regions of high moment.

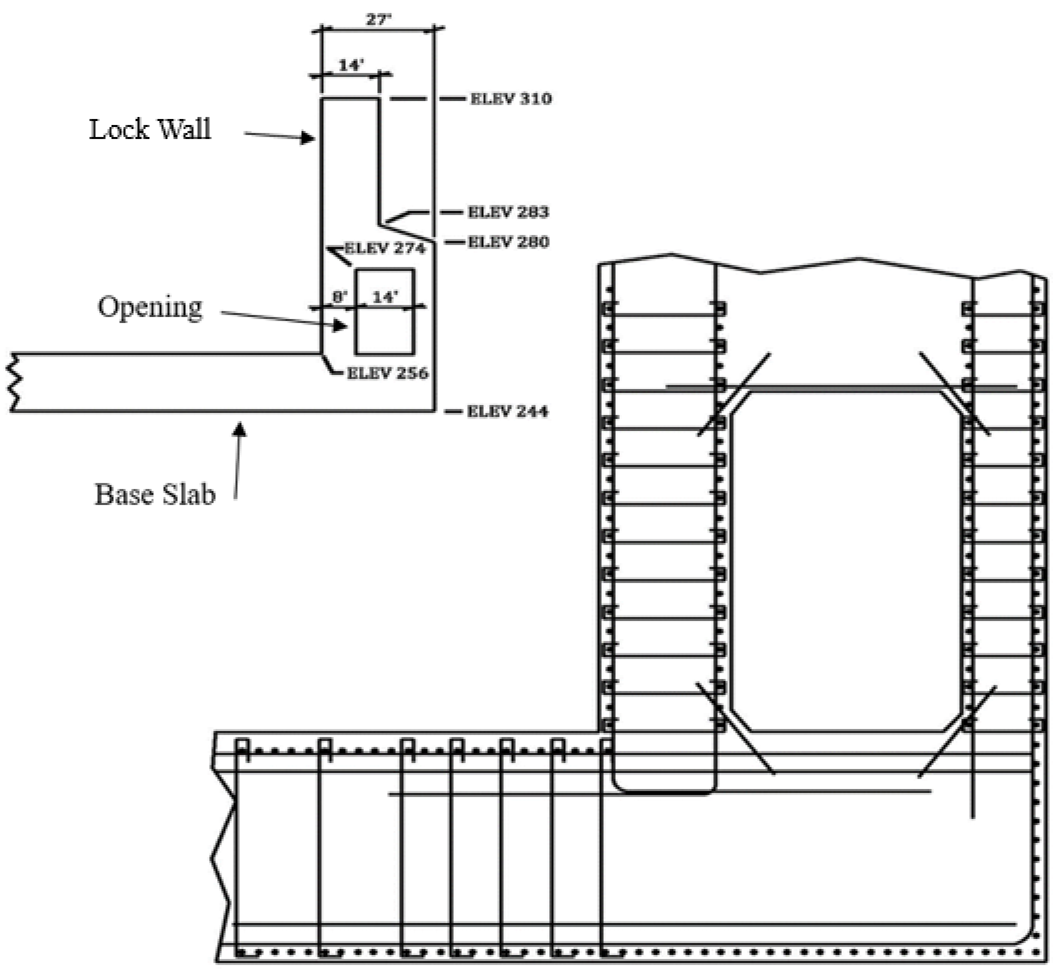


Figure 2–2. Typical seismic reinforcement details – Olmsted Lock and Dam

Chapter 3
Strength and Serviceability Requirements

3–1. Overview

a. General. RCHS must be designed to satisfy all serviceability, strength, and stability requirements according to ACI 318-19 with the exceptions to the provisions of ACI 318-19 for load factors, load cases, and reinforcement limits contained herein.

b. Serviceability limit states. The serviceability limit state ensures that the RCHS will meet all operational requirements by imposing limits on stress, deformation, and cracking. Requirements for allowable stress and guidance on deflection is provided in paragraph 3–4. That section, along with other requirements in this manual and in ACI 318-19, are intended to provide adequate serviceability performance for RCHS. These design service stresses in the concrete and reinforcing steel have been shown in past U.S. Army Corps of Engineers (USACE) RCHS designs to limit cracking and provide durability for long service life.

c. Strength limit states. The strength limit state ensures safety against yield and fracture of the reinforced concrete section during the intended life of the structure. Strength limit states are evaluated using Load and Resistance Factor Design (LRFD). Using LRFD, nominal loads and load factors are intended to provide adequate reliability against exceeding strength limit states in ACI 318-19 and Chapters 4 and 5. Loads to be used are described in paragraph 3–2 and load factors are defined in paragraph 3–3.

d. Critical and normal structures. RCHS, for the purpose of establishing return periods that delineate the strength load category, are designated as either critical or normal.

(1) *Critical structures*. Critical structures are those where failure could result in the potential for loss of life. Loss of life could result directly from breach (uncontrolled release of water) or indirectly from flooding damage to a lifeline facility. A risk assessment for a dam or levee can help inform the potential for loss of life determination. Hazard potential classification for dams can also be used. Guidance on the determination of hazard potential is provided in ER 1110-2-1806 and ER 1110-2-1156. Examples of classification of structures according to probable loss of life are provided in Appendix H of EM 1110-2-2100.

(2) *Economic or environmental consequences*. In some cases, potential for extreme economic or environmental loss, as determined by the engineer, may be justification for the designation of a structure as critical.

(3) *Normal structures*. All RCHS not meeting the definition of critical in the previous paragraphs are normal structures.

e. *Performance requirements*.

(1) *Introduction.* Performance requirements are defined to establish design criteria. Performance requirements for RCHS are described in the following paragraphs for the usual, unusual, and extreme load categories. The load categories are based on the probability of loading as defined in paragraph 3–2d.

(2) *Usual*.

(a) For usual loads, the structural behavior of the RCHS is expected to be in the linear elastic range with minimal cracking under service stresses. The maximum strain in the concrete does not exceed the crushing strain, and the maximum strain in the reinforcement does not exceed the yield strain. Only tight, hairline cracking of concrete surfaces is barely visible, and no leakage is visible.

(b) To meet these requirements, usual loads are designed using serviceability requirements of paragraph 3–4. In addition, reinforcing steel must meet the maximum spacing requirements of paragraph 2–6b.

(3) *Unusual*.

(a) For unusual loads, the structural behavior of the RCHS is expected to be essentially elastic. The maximum strain in the concrete does not exceed the crushing strain. The maximum strain in the reinforcement remains elastic. The loading will create tensile cracking in areas of large bending moment. These cracks are expected to reduce or close after the temporary load has receded.

(b) To meet these requirements, unusual loads are designed using the serviceability requirements of paragraph 3–4. In addition, reinforcing steel must meet the maximum spacing requirements of paragraph 2–6b.

(4) *Extreme*.

(a) The structural behavior of the RCHS under extreme loads may be nonlinear. It may sustain damage but should not collapse and cause uncontrollable flooding. Significant structural repairs may be required after one or more extreme loading to repair local damage or to ensure that degradation of strength or stiffness does not result in collapse of the RCHS.

(b) To meet these requirements, extreme loads are designed using strength limit states with LRFD by using ACI 318-19 capacity limit states and the load factors in Table 3–1.

f. *Structural stability analysis*. In addition to strength and serviceability requirements, RCHS must also satisfy stability requirements under various loading and foundation conditions. Design forces in RCHS are obtained from these stability and pile group analyses. For strength limit state design using LRFD, these analyses must be performed using factored loads to obtain factored moments, shears, and thrusts at critical sections of the RCHS. However, for design of RCHS using a single load factor, analysis can be performed with unfactored loads. The resulting moments, shears, and thrusts may then be multiplied by the single load factor to determine the factored design forces.

g. Hydraulic structures supporting vehicles.

(1) Hydraulic structures may carry vehicle loads (including railroads and cranes) either by supporting vehicle bridge structures or by supporting vehicle loads directly, such as in a culvert or the foundation of a flood closure. These structures are designed according to both industry guidance (AASHTO for roads or American Railway Engineering and Maintenance-of-Way Association (AREMA) for railways) and the RCHS requirements of this manual.

(2) Design according to industry guidance when vehicle loads are principal loads. Water and other companion loads are applied according to industry criteria. When design is performed with RCHS loads as the principal loads, vehicle live loads are applied as companion loads according to paragraph 3–2f(11).

(3) Vehicle loads may include crane and other special loads for bridges that are used to service hydraulic structures. Crane loads can be significantly greater than AASHTO design vehicle loads and may require design for specific equipment.

h. Reactions from hydraulic steel structures. Loads on RCHS may originate as reactions from hydraulic steel structures (HSS). In those cases, factored HSS reaction loads derived from EM 1110-2-2107 will be applied to corresponding RCHS LRFD load combinations without the addition of load factors described here. For serviceability checks, unfactored HSS reaction loads are added to unfactored RCHS load combinations.

3–2. Loads

a. General. All loads to which an RCHS is subjected will be considered in the design according to paragraph 3–3. Loads with a negligible impact on the design may not require full design analysis.

b. Load combinations. Loads are combined to produce maximum effects for a given limit state under the varying load frequencies. Load combinations are identified, and varying load factors are applied to achieve a consistent level of reliability. Loads are combined using principal and companion action loads as described in the following paragraphs.

c. Principal and companion action loads. A load used in combination with other loads can be defined as a principal load or companion load. The maximum of combined load occurs when one load, the principal action, is at its extreme value; while the other loads, the companion actions, at the values that would be expected while the principal action is at its extreme value. Definitions are:

(1) *Principal load*. A principal load is a specified variable load or rare load that dominates in a given load combination. For LRFD, a principal load factor is applied to the principal load in a load combination to account for the variability of the load and the load pattern for the analysis of its effects. Principal loads and load factors are selected as described in paragraph 3–3e.

(2) *Companion load*. A companion loads is a specified variable load that accompanies the principal load in a given load combination. For LRFD, a companion load factor is applied to a companion load in a load combination to account for uncertainty in the magnitude of the companion load acting simultaneously with the factored principal load. Companion loads are typically usual loads. For strength load combinations, unless otherwise specified in paragraph 3–3e, temporary and dynamic companion loads must have a minimum 10-year return period. This is the maximum return period of the usual load category defined in paragraph 3–2d. For hydrostatic loadings, this is the normal operating condition (pool).

d. *Probability of loading*. Loads can be separated into categories based on their probability of occurrence. Loads with less probability of occurrence can have different design requirements to achieve the same reliability. Loads are categorized as usual, unusual, and extreme, based on average annual return periods (t_r) or annual exceedance probability (AEP) (equal to $1/t_r$). The probability of loading associated with the usual, unusual, and extreme load categories are described below and are illustrated in Figure 3–1.

(1) *Usual*. The usual load category represents daily or frequent operational conditions for which highly reliable performance is required. The design criteria for the usual load category applies to load cases with the predominant load (or combined loads) having a mean return period (t_r) less than or equal to 10 years (AEP of 0.10).

(2) *Unusual*. The unusual load category represents infrequent operational conditions. These conditions can be reasonably expected to occur within the service life of the project for which a defined level of performance is required. The design criteria for the unusual load category applies to load cases with predominant loads expected to have a return period (t_r) of greater than 10 years (AEP of 0.10) and less than or equal to 750 years (AEP of 0.0013) for critical structures, and less than or equal to 300 years (AEP of 0.0033) for normal structures.

(3) *Extreme*. The extreme load category represents possible conditions that are not likely to occur within the service life of the project. The design criteria for extreme load cases are applicable if the predominant load (or combined loads) has a return period (t_r) of greater than 750 years (AEP of 0.0013) for critical structures and greater than 300 years (AEP of 0.0033) for normal structures.

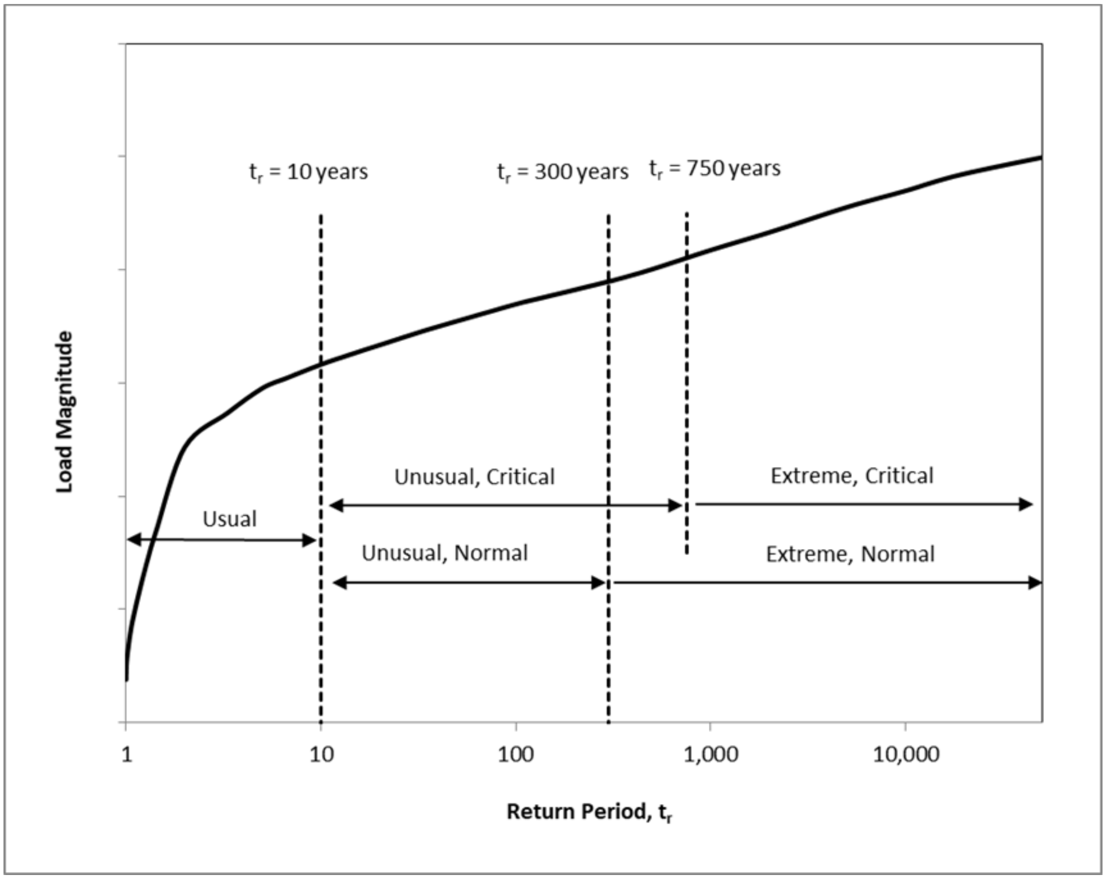


Figure 3–1. Load category versus return period

e. Load duration. Loads on structures vary with time. Loads can be grouped into the categories based on duration. These categories are important for combining loads.

(1) Permanent loads (Lp) are continuous loads, such as dead load or lateral earth pressure.

(2) Temporary (intermittent static) loads (Lt) are loads with durations from several minutes to several weeks, such as flood loads, maintenance dewatering, and operation live loads.

(3) Dynamic (impulse) loads (Ld) are loads with durations of seconds or less, such as vessel and ice impact, earthquake, wave, and turbulent water flow. Response of RCHS to these loads may be dynamic, but usually is designed as static for most RCHS. Because of the short duration, it is extremely unlikely that more than one dynamic load exists at any given time.

f. Load definitions. Loads used for design of RCHS are described in the following paragraphs. The loads are summarized in Table 3–1.

Table 3–1
Loads on hydraulic structures

Permanent Loads, Lp	**Variable**
Dead	D
Vertical Earth	EV
Lateral Earth	EH
Gravity	G
Temporary Loads, Lt	**Variable**
Hydrostatic	Hs
Thermal Expansion of Ice	IX
Soil Surcharge	ES
Operating Equipment	Q
Live Load	L
Self-Straining	T
Vehicle Live Loads	V
Dynamic Loads, Ld	**Variable**
Hydrodynamic (except earthquake)	Hd
Wave	Hw
Debris/Floating Ice Impact	IM
Barge/Boat Impact	BI
Wind	W
Earthquake	EQ
Hawser	HA

(1) *D, Dead load*. Dead load is the expected weight of permanent structural features. Dead load is a permanent load.

(2) *EH, Lateral earth*. This is moist or effective lateral earth pressures at the site from in situ conditions, engineered backfills, or deposition of silt during a minimum service life of 100 years. The earth pressures assumed to act on structures should be consistent with the expected movements of the structure system. Lateral earth pressures are at rest when little or no structural movement occurs. Where movement occurs, lateral earth pressures approach the active state on the driving side and the passive state on the resisting side. See EM 1110-2-2502 and EM 1110-2-2100 for additional information on determining lateral earth pressure. To provide the target reliability with the load factors in this manual, lateral earth pressures must be computed using soil parameters with an inherent conservative bias according to EM 1110-2-2502. Lateral earth load is a permanent load.

(3) *EV, Vertical earth*. This is expected weight of moist or buoyant soil on a structure. Water in the soil is included under hydrostatic load, Hs. The nominal earth load should be estimated with expected values of depth and unit weight. The effects of scour or sediment deposition should be included where applicable. Vertical earth load is a permanent load.

(4) *G, Gravity*. Gravity loads consist of nonpermanent loads such as mud (including silt), debris, and atmospheric ice. All of these loads are highly site-dependent and must be based on knowledge and past experience at a site. Atmospheric ice loads are determined using guidance of American Society of Civil Engineers (ASCE) 7-22. Other ice loads from spray, leakage, wave overwash, etc., are determined based on site conditions. Gravity loads are considered permanent loads.

(5) *Hs, Hydrostatic*.

*(a) Genera*l. Hydrostatic includes water pressure above and below the ground line. It also includes water forces above or in a structure, including weight of water within soil on a structure, and uplift. The hydrostatic loads should be selected in coordination with the hydraulic engineer. Hydrostatic loads are based on expected values (50 percent confidence level) at the design return periods. Additional uncertainty is accounted for in the load factors. For load combinations, hydrostatic loads are considered temporary loads because the level is usually variable. Water forces below the ground line may include seepage pressures.

(b) Usual. The usual load case is a serviceability case. The normal operating condition (pool) with 10-year return period should be used unless others are of interest for serviceability. See Appendix E for examples. In some cases, the maximum expected head differential on a particular structure may fall in this range.

(c) Principal hydrostatic loads for strength design. For design of hydrostatic loads as the principal loads in a load combination (Hs_{pr}), the design water levels must create the maximum hydrostatic loading caused by a differential head that is geometrically and hydrologically possible. Some considerations are:

1. The maximum hydrostatic loading may occur at water levels that are not necessarily the largest possible differential head.

2. Usually, the maximum hydrostatic load is limited by the height of the structure and other factors. But the maximum hydrostatic load may be from water levels that exceed the top of a structure.

3. In some cases, hydrostatic loading may be experienced from differential head across a structure in either direction.

4. When uplift is the part of the principal load, it is determined from the condition that creates the maximum loading on the structure.

5. Determination of the maximum differential head condition should be made in consultation with the project's hydraulic engineers.

(d) Companion hydrostatic loads. When Hs is a companion load (Hs_c), values to be used for design are normal operating conditions as defined in paragraph 3–2c(2).

(6) *IX, Ice, thermal expansion*. Forces from thermal expansion of ice. Design values for principal loads should be based on upper bound values and designed as extreme loads. IX is a temporary load. See EM 1110-2-1612 for more information.

(7) *ES, Earth surcharge*. Temporary loads due to stockpiled material, machinery, roadways, and other influences resting on the soil surface near the structure that increase the lateral or vertical pressures on the structure. Surcharge loads should be considered temporary, principal loads in load combinations unless the surcharge is permanent. Typical surcharge forces used for design accounting for nominal vehicle or fill loading should be considered unusual loads. Surcharge loads for the extreme case are considered the upper limits of possible loads.

(8) *Q, Operational loads*. Reactions from operating equipment and hydraulic gates. See USACE guidance for hydraulic steel structures.

(9) *L, Vertical live load from personnel, equipment, vehicles, or temporary storage on operating surfaces*. Live load is determined and factored according to ASCE 7-22.

(10) *T, Self-straining*. Self-straining forces from constrained structures that experience dimensional changes. Dimensional changes primarily come from settlement and temperature changes. Temperature differentials are created by environmental temperature changes or by hydration of concrete during the curing process. Additional guidance for determining T is provided in ASCE 7-22. For self-straining forces from concrete curing processes, see paragraph 2–9h.

(11) *V, Vehicle loads*. When vehicle loads are applied to RCHS as principal loads, see paragraph 3–1g. When vehicle loads for RCHS are companion loads, RCHS are designed with or without the vehicle loads present, whichever has the greatest effect. The vehicle loads are selected from service (unfactored) loads according to industry standards. For instance, companion vehicle loads are derived from the service load combinations in the AASHTO LRFD Bridge Design Specifications.

(12) *Hd, Dynamic*. Hydrodynamic loads from thrust from vessels (propwash), downdrag, temporal head, inertial resistance, overtopping impingement, hydraulic jumps, etc. Generally, these loads are estimated with much uncertainty in expected values. Extreme case design values are based on maximum expected loading. Hydrodynamic forces from earthquakes are covered under paragraph 3-2f(17), EQ, earthquake.

(13) *Hw, Wave*. Wave loads are computed as described in EM 1110-2-1100. Wind events used to generate wave loads must account for the location of the structure and characteristics of the hydraulic loading.

(a) For RCHS on reservoirs and other locations with permanent water loading, the design should consider a case with wave loads as principal loads (Hw_{pr}). These waves are generated by extreme wind events with minimum return periods as defined in paragraph 3–3e. When the wind and water elevation are independent, the wave loads are combined with a companion hydrostatic load, Hs_c. For other load cases with independent pool elevation and wind/wave events where wave loads are companion loads, design wave loads are determined as described in paragraph 3–2c(2) (return period of 10 years).

(b) For coastal situations with correlation between surge and wave, annual exceedance of combined loads must be computed by a coastal hydraulic engineer using a coupled analysis of the water elevation and wave heights. The surge level and wave force computed as a function of annual probability of exceedance will be provided by the hydraulic engineer.

(14) *IM, Impact from debris or floating ice*.

(a) Impact from debris and floating ice may be perpendicular to the structure for piers, weirs, and other RCHS in flow. Impact may be from glancing blows from debris in flow parallel to a structure, or it may be driven onto the wall by wind.

(b) Debris loads may be correlated with flood loads (Hs) and are combined with flood loads as companion loads when debris may be experienced at a site during flooding. Impact that is applied as a principal load should be based on upper-bound, extreme loading values.

(c) Impacts should be determined from an assessment of probable debris and from past experience. Impact loads should be considered to act at or below the water surface level for the hydrostatic loading being considered.

(15) *BI, Barge (and vessel) impact*.

(a) *General*. This load is caused by impact from aberrant vessels or barges moved by wind or current, or by impact from powered vessels entering or exiting locks, in channels, at wharfs and piers, etc. The type and size of vessels, barges, and barge tows that may impact a structure is site-dependent and should be determined by the engineer after careful research of local conditions. For coastal locations, barges may be moved many miles during high surge and wind events.

(b) *Load combinations*. Impact loads applied as principal loads should be based on guidance in EM 1110-2-3402. Barge impact loads may also be applied as companion loads and should be based on the normal impact loads expected for the site during the conditions that create the principal load.

(c) *Design loads*. Impact loads from vessels and barges are a function of the kinetic energy of the vessel, the deformation of the vessel or barge during impact, and the response of the wall. Barge and vessel impact loads are computed according to EM 1110-2-3402 and other applicable engineer manuals.

(16) *W, Wind*.

(a) Principal and companion wind loads are computed according to ASCE 7-22. Wind loads for critical structures are calculated using criteria for Risk Category IV structures if a failure from the wind load would result in consequences that meet the definition of a critical structure in paragraph 3–1d. Wind loads for all other structures are calculated using criteria for Risk Category II structures.

(b) Wind loads included in serviceability cases are computed using serviceability wind loads from ASCE 7-22 that meet limits of return period for usual loads described in paragraph 3–2d(1). Design should generally be performed using a wind velocity with a 10-year return period.

(17) *EQ, Earthquake*. See paragraph 3–3h.

(18) *HA, Hawser*. See EM 1110-2-2602 for a description of the hawser loads associated with locks. The design hawser force in EM 1110-2-2602 is based on the nominal breaking strength of a single line. Hawser loads for piers, wharfs, and other structures may be more or less than the load provided in EM 1110-2-2602. Hawser loads for these sites should be determined by the characteristics of the vessels and hawser lines at the particular site. Hawser loads based on the strength of a broken line should generally be considered unusual loads. Hawser loads that are based on loads less than the breaking strength of the line should be considered usual loads.

3–3. Strength design

a. *Required strength*. The required strength computed from the effects of factored loads must be less than the design strength, calculated as:

$$\sum \gamma_i L_{ni} \leq \varphi R_n \tag{3–1}$$

where:

$\sum \gamma_i L_{ni}$ = U = required strength, the effect of factored loads
γ_i = load factors that account for bias and variability in loads to which they are assigned
L_{ni} = nominal (code-specified) load effects
φ = resistance factor from ACI 318-19
R_n = nominal resistance from ACI 318-19 and Chapter 4

b. *Load factors and load combinations*. Load factors and general load combination equations for LRFD are provided. Specific load combinations must be determined by the design engineer for each structure type. Examples of load combinations for typical structures are shown in Appendix E. Consultation with and approval by CECW-EC is required for loads not covered in this manual.

c. Load combinations. The general equation for combining loads for LRFD is as follows:

$$U = \Sigma\gamma_p\, Lp + \gamma_{pr} L_{pr} + \Sigma\, \gamma_c Lt_c + \gamma_c Ld_c \qquad (3\text{–}2)$$

where:

U = factored applied load
γ_p = load factor applied to permanent loads
Lp = permanent loads
γ_{pr} = load factor applied to principal loads
L_{pr} = principal load
Lt = temporary loads
Ld = dynamic loads
c = designates companion loads; see paragraph 3–2c(2)
γ_c = load factor applied to companion loads

(1) Lp, Lt, and Ld are defined in paragraph 3–2e. Ld is not included as a companion load (Ld_c) when the principal load is a dynamic load (Ld_{pr}).

(2) For principal loads that are correlated, such as hydrostatic and wave forces from storm-created surge and wave, the applied principal load must be determined from the combined, correlated loading. The combined load will not be combined with other temporary or dynamic loads.

d. Permanent load factors (γ_p). Permanent loads on RCHS consist of dead loads, vertical and lateral earth loads, and gravity loads. Maximum and minimum load factors must be applied to provide the greatest effect.

D, when combined with other loads, γ_p = 1.2 or 0.9

D, when applied alone, γ_p = 1.4

EV, γ_p = 1.35 or 1.0

EH, at-rest pressure, driving γ_p = 1.35, resisting γ_p = 0.9

EH, active pressure, γ_p = 1.5, passive pressure: γ_p = 0.5

G, γ_p = 1.6 or 0

e. Principal load factors (γ_{pr}). Principal load factors are used to provide low probability of failure. Normally, extreme loads are used for design loads, but for some sites the maximum principal load on a structure may occur with a return period in the unusual or usual load category. Except as where defined for loads that use other industry standards as specified in paragraph 3–3e(4), or for earthquake as described in paragraph 3–3h, load factors, γ_{pr}, applied to principal loads are as follows:

(1) *Principal Load Condition 1*. Maximum loading is not limited by the geometry of the structure or other physical factors. The return period of the load can be estimated. Examples include most wave loads. Nominal loads for design are based on return periods that provide very low probability of exceedance. Minimum design return periods are greater than or equal to 3,000 years for normal structures and to 10,000 years for critical structures.

γ_{pr} = 1.2

(2) *Principal Load Condition 2*. The maximum loading that can be applied is limited by the geometry of the structure or other physical factors. The return period of the load can be calculated or estimated. An example is differential hydrostatic loading limited by the height of a floodwall or dam.

(a) Principal Load Condition 2 (extreme). The load factor below is applied unless the principal load has a return period greater than those in Principal Load Condition 1, or unless otherwise stated in paragraph 3–2f. Return periods in the extreme range are less than 3,000 years but greater than 300 years for normal structures and less than 10,000 years but greater than 750 years for critical structures.

γ_{pr} = 1.3

(b) Principal Load Condition 2 (unusual). The principal load is limited at a maximum value with a return period in the unusual range of less than 300 years but greater than 10 years for normal structures and less than 750 years but greater than 10 years for critical structures.

γ_{pr} = 1.4

(c) Principal Load Condition 2 (usual). The load is limited at a maximum value with a return period in the usual range of less than 10 years.

γ_{pr} = 1.5

(3) *Principal Load Condition 3*. The return period of the principal load is unknown. Design loads are considered to be upper bound loads. Usually, these loads are of very low expected probability of exceedance (extreme). Examples are typically impact loads, thermal expansion of ice, barge impact, and many hydrodynamic loads. The load factor below accounts for the uncertainty in the knowledge of these loads. If the expected return period of the upper bound loads is more frequent than extreme, the appropriate load factors from paragraph 3–3e(2) should be used.

γ_{pr} = 1.3

(4) Loads derived from ASCE 7-22 (live load, self-straining, wind).

L, γ_{pr} = 1.6

T, γ_{pr} = 1.0

W, γ_{pr} = 1.0

f. Companion load factors (γ_c). Companion load factors depend on the source of the load. For live load, self-straining, and wind companion, load factors are taken from load combinations in ASCE 7-22. For all other temporary and dynamic companion loads, the companion load factor is 1.0. Summarizing γ_c:

L, γ_c = 1.0 (ASCE 7-22)

T, γ_c = 0.75 (ASCE 7-22)

W, γ_c = 0.5 (ASCE 7-22)

All other Lt and Ld, γ_c = 1.0

g. Load factors summary. Load factors used for design of RCHS are summarized in Table 3–2.

Table 3–2
Minimum load factors for strength design

Load Category		**Permanent and Companion**	**Principal Load Factors, γ_{pr}**		
			Usual	**Unusual**	**Extreme**
Return Period – Critical		≤ 10	≤ 10	> 10, ≤ 750	> 750
Return Period – Normal		≤ 10	≤ 10	> 10, ≤ 300	> 300
Permanent Loads, Lp		γ_p			
Dead	D	1.2[1], 0.9[2], 1.4[1]	–	–	–
Vertical Earth	EV	1.35[1], 1.0[2]	–	–	–
Lateral Earth	EH	See Note 7	–	–	–
Gravity (Mud/Ice)	G	1.6[1], 0[2]	–	–	–
Temporary Loads, Lt		γ_c	–	–	–
Hydrostatic	Hs	1.0	1.5[3]	1.4[3]	1.3[6]
Ice, Thermal Expansion	IX	1.0	–	N/A	1.3[6]
Soil Surcharge	ES	1.0	–	1.6	1.3[6]
Operating Equipment	Q	1.0	1.5[3]	1.4[3]	1.3[6]
Live Load	L	1.0[4]	–	1.6[4]	N/A
Self-Straining	T	0.75[4]	–	1.0[4]	1.0[4]
Vehicle Live Loads	V	1.0	–	AASHTO[8]	AASHTO[8]
Dynamic Loads, Ld		γ_c	–	–	–
Hydrodynamic	Hd	1.0	–	–	1.3[6]

Load Category		Permanent and Companion	Principal Load Factors, γ_{pr}		
			Usual	Unusual	Extreme
Wave	Hw	1.0	–	–	1.2[6]
Debris/Floating Ice	IM	1.0	–	–	1.3[6]
Barge/Vessel Impact	BI	1.0	2.2[9]	1.6[9]	1.3[9]
Wind	W	0.5[4]	–	–	1.0[4]
Earthquake	EQ	N/A	–	1.5	1.0 or 1.25[5]
Hawser	HA	1.0	–	1.6	–

Notes:
1. Applied when loads add to the predominant load effect. See paragraph 3–3d.
2. Applied when loads subtract from the predominant load effect.
3. Unusual loads used as principal loads for strength design when they are the maximum possible loads.
4. ASCE 7-22. See paragraph 3–3f.
5. For site-specific earthquake load factor are 1.0. Otherwise, the higher load factor is used. See paragraph 3–3h(3).
6. Typical load factors for design are shown. See paragraph 3–3e for selection of load factors.
7. Load Factors for Lateral Earth Pressure: See paragraph 3–3d.
8. For cases with vehicle lives loads as principal loads, see paragraph 3–1g.
9. Load factors from EM 1110-2-3402.

h. Earthquake load (effects).

(1) *General*. In developing earthquake loads, two levels of design earthquakes are considered for serviceability and strength, as defined in ER 1110-2-1806. The Operating Basis Earthquake (OBE) is an unusual load, and the Maximum Design Earthquake (MDE) is an extreme load. For critical features, the Maximum Credible Earthquake (MCE) is used for the MDE. The design earthquakes, ground motions, and performance requirements for the OBE and MDE are determined according to ER 1110-2-1806. Guidance for analysis and design of concrete hydraulic structures for earthquake is provided in EM 1110-2-6050, EM 1110-2-6051, and EM 1110-2-6053.

(2) *Combination with other loads*. Earthquake loads are of low probability of occurrence and short duration. Since they are principal loads, they are combined with normal operating loads when developing load combinations according to paragraph 3–2c. The other static loads typically consist of self-weight, uplift, internal and external water pressure, and lateral soil pressures. Earthquake-induced lateral earth pressures are defined in EM 1110-2-2100. Dynamic analysis (response spectrum or time history analysis) of earthquake should be performed using at-rest lateral earth pressures as the structure and soil can move in either direction.

(3) *Load combination for earthquake loads.* Only one temporary load need be included at a time (if applicable) with an earthquake load. For dynamic analysis, the load factors are applied to the load effects (computed member forces) instead of to the applied loads.

(a) For standard and site-specific OBE ground motion analysis:

$$U = \Sigma\, \gamma_p\, Lp + 1.5\, EQ + \gamma_c\, Lt_c \quad (3\text{–}3)$$

(b) For standard MDE ground motion analysis:

$$U = \Sigma\, \gamma_p\, Lp + 1.25\, EQ + \gamma_c\, Lt_c \quad (3\text{–}4)$$

(c) For site-specific MDE and MCE ground motion analysis:

$$U = 1.0\, \Sigma\, Lp + 1.0\, EQ + 1.0\, Lt_c \quad (3\text{–}5)$$

3–4. Serviceability design

a. Service stresses.

(1) *Maximum service stresses.* To meet the serviceability requirement in paragraph 3–1e, RCHS are designed by limiting stresses for flexure, shear reinforcement, and direct tension. Maximum service (unfactored) stresses in the reinforcing steel (fs) are provided in Table 3–3. These allowable stress requirements apply to all steel grades. In addition to the maximum stresses in Table 3–3, the reinforcing spacing requirements of paragraph 2–6b and the reinforcement limits of paragraph 3–6 provide additional requirements for serviceability. There are no serviceability stress requirements for extreme load combinations.

Table 3–3
Maximum service stresses

Load Category	Reinforcement Flexure and Shear Stress, fs, ksi (MPa)	Reinforcement Direct Tension Stress, fs, ksi (MPa)
Usual	25 (170)	20 (140)
Unusual	35 (240)	27.5 (190)

(2) *Alternate serviceability stress design*. Design sections of beams and one-way slabs with service stresses meeting Table 3–3 can be determined using the strength design method. Service loads are multiplied by a single load factor provided in Table 3–4 to compute a factored design load. These load factors apply only to design using reinforcing steel with yield strength of 60,000 psi (414 MPa).

Table 3–4
Single-load factors for approximating serviceability design with f_y = 60,000 psi (414 MPa)

Load Category	Flexure and Shear (Reinforcement)	Direct Tension
Usual	2.2	2.8
Unusual	1.6	2.0

b. *Deflection*.

(1) Deflections are computed using service (unfactored) loads. Acceptable deflection limits are determined by the engineer from the operational requirements of the structure. Some structures can tolerate much more deflection than others depending on the situation. Where waterstops are present, differential deflections across joints should be limited to prevent damage to them. This will be based on the ability of the selected waterstop to move without damage. Where an RCHS connects to or supports other structures or elements that may be sensitive to or damaged by movement, deflections must be controlled to ensure satisfactory performance of the system.

(2) Limiting the tension reinforcement ratio in flexural members to a maximum of 25 percent of the balanced reinforcement ratio (0.25 ρ_b) was required in past versions of this manual to help limit deflection. It is recommended to generally design within this limit. Additional guidance for deflection may be provided in other applicable engineer manuals.

3–5. Design strength of reinforcement

Design should normally be based on 60,000 psi (414 MPa), the yield strength of ASTM Grade 60 reinforcement. Other grades, up to a maximum of 80,000 psi (552 MPa), may be used subject to the provisions of ACI 318-19 and this manual. The yield strength used in the design should be indicated on the drawings. Reinforcement with yield strength in excess of 80,000 psi (552 MPa) must not be used unless a detailed investigation of ductility and serviceability requirements is conducted in consultation with and approved by CECW-EC.

3–6. Reinforcement limits

For all load cases, the tension reinforcement ratio must not exceed 50 percent of the balanced reinforcement ratio (0.50 ρ_b) to ensure that the strength limit state is a ductile failure mode.

3–7. Minimum thickness of walls

Walls with height greater than 10 ft (3 m) must be a minimum of 12 in. (30.5 cm) thick. Walls 10 in. (25 cm) or greater in thickness must have reinforcement in both faces. Walls must not be less than 8 in. (20 cm) thick.

3–8. Mandatory requirements

a. RCHS must be designed to satisfy all serviceability, strength, and stability requirements in accordance ACI 318-19 with the exceptions to the provisions of ACI 318-19 for load factors, load combinations, and reinforcement limits contained herein.

b. According to paragraph 3–1d, structures at high hazard potential projects must be considered critical where failure will result in loss of life; all other structures will be classified as normal.

c. Stability analyses of RCHS must be performed according to the requirements of paragraph 3–1f.

d. Hydraulic structures supporting vehicles must be designed according to paragraph 3–1g.

e. Loads used for design must conform to the definitions in paragraph 3–2.

f. The required strength computed from the effects of factored loads must be less than the capacity as defined by equation 3–1.

g. Load factors must be used according to paragraph 3–3.

h. Regarding loading conditions, as a minimum, the loading cases provided in Appendix E must be satisfied.

i. If resistance to earthquake loads, EQ, is required, the requirements of paragraph 3–3h must be met.

j. Design service stresses must meet the requirements of paragraph 3–4a.

k. Tension reinforcement ratios must meet the requirements of paragraph 3–6.

l. Walls must meet the thickness and reinforcement requirements of paragraph 3–7.

Chapter 4
Flexure and Axial Loads

4–1. Design assumptions and general requirements

a. This chapter covers general design requirements for the strength design of RCHS subject to combined loadings. The general design procedure for these members is:

(1) Determine the member's loadings and reinforcement configuration.

(2) Determine the eccentricity ratio of the loading.

(3) Design the member using the specified ϕP_n and ϕM_n equations for the given member and eccentricity ratio.

b. Members subject to flexural and axial loads are considered in six categories:

(1) Members that contain only tension reinforcement and are loaded in flexure and compression. Design of these members is covered in paragraph B–2.

(2) Members with both tension and compression reinforcement and are loaded in flexure and compression. Design of these members is covered in paragraph B–3.

(3) Members loaded in tension and flexure. Design of these members is covered in paragraph B–4.

(4) Members that support axial loadings and are subject to biaxial bending. Design of these members is covered in paragraph 4–3.

(5) Members that contain only tension reinforcement and are loaded in flexure only. Design of these members is covered in paragraph B–5.

(6) Members that contain tension and compression reinforcement and are loaded in flexure only. Design of these members is covered in paragraph B–6.

c. The eccentricity of axial load (e') is a critical component in determining the effect that a given loading has on a member.

(1) The eccentricity of axial load is a distance measured from the centroid of the tensile reinforcement, and is taken as:

$$e' = \frac{M_u}{P_u} + d - \frac{h}{2} \qquad (4\text{–}1)$$

where P_u is considered positive for compression and negative for tension.

(2) For equation 4–1, the applied moment and axial loadings are the resultants of all applied loadings. Tension loadings and moments causing the bottom of the member to act in compression are taken as negative by convention.

(3) The eccentricity ratio normalizes eccentricities such that a ratio of one represents an axial loading acting at the member's extreme compression fiber, and a ratio of zero represents a loading acting directly at the centroid of tensile reinforcement. The eccentricity ratio for all members is defined as:

$$\frac{e'}{d} \tag{4–2}$$

d. Additional general requirements that apply to all members covered by this chapter are:

(1) The assumed maximum usable strain ε_c at the extreme concrete compression fiber must be equal to 0.003, according to ACI 318-19.

(2) Balanced conditions for hydraulic structures exist at a cross section when the tension reinforcement ρ_b reaches the strain corresponding to its specified yield strength f_y as the concrete in compression reaches its design strain ε_c. Tensile reinforcement must be provided such that ρ complies with paragraph 3–6.

(3) Concrete stress of $0.85f_c'$ must be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = \beta_1 c$ from the fiber of maximum compressive strain, where c is the distance from the extreme compression fiber to the neutral axis. The free body diagrams shown in Figure B–1 through Figure B–3 illustrates these conditions.

(4) Factor β_1 will be taken as specified in ACI 318-19.

(5) Factor k_b represents the ratio of stress block depth (a) to the effective depth (d) at balanced strain conditions. Its value can be determined for all members using:

$$k_b = \frac{\beta_1 E_s \varepsilon_c}{E_s \varepsilon_c + f_y} \tag{4–3}$$

e. Appendix B contains the applicable design equations for each member type described in paragraphs 4–1a through 4–1d, along with the derivations of those equations.

4–2. Interaction diagrams

a. An interaction diagram is a plot of the axial loads and bending moments that cause a concrete member of specified size and reinforcement to fail. Figure 4–1 diagrammatically shows the compression failure, tension failure, and balance point.

Figure 4–2 shows strain condition at the compression failure, tension failure, and balance point.

b. Interaction diagrams can be developed using the computer-aided software engineering program Concrete General Strength Investigation (CGSI) or commercial software. Appendix C includes an example using computer program CGSI.

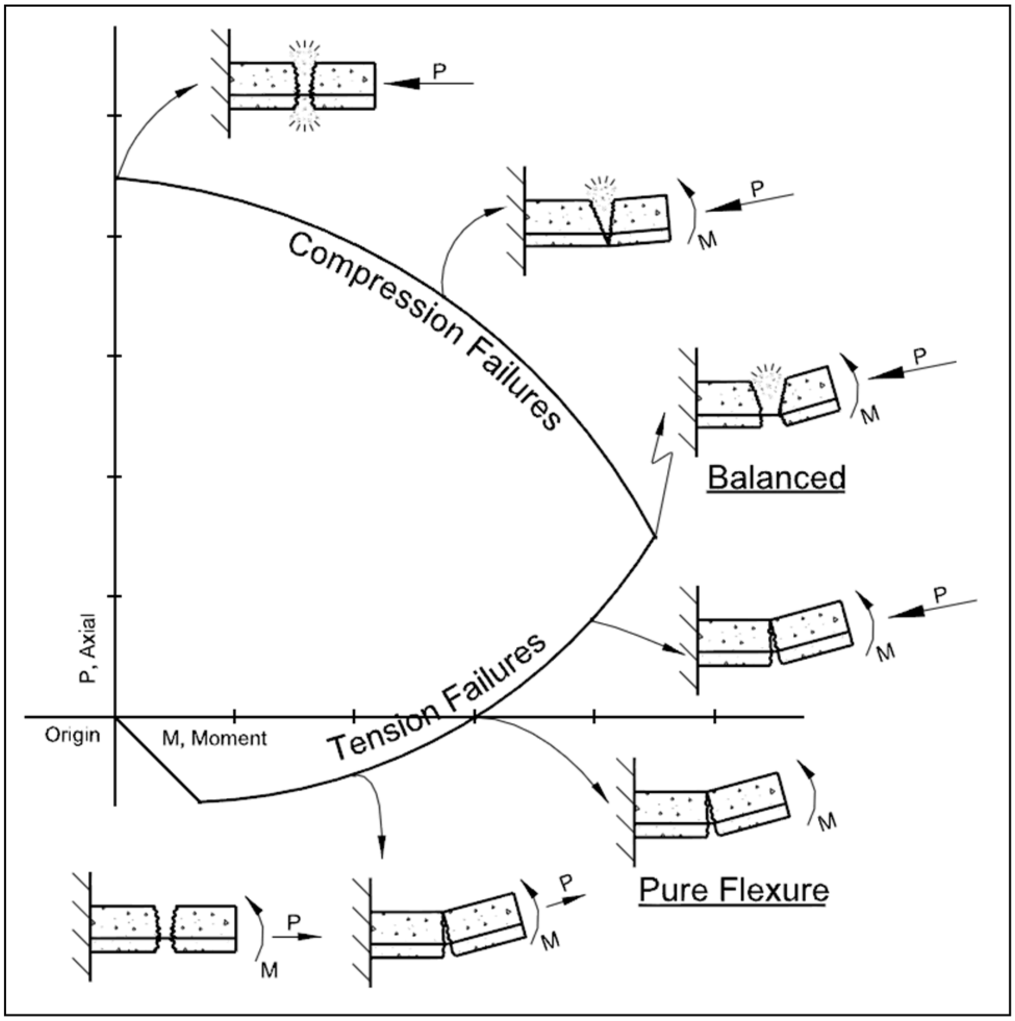


Figure 4–1. Interaction diagram with illustrated failure modes

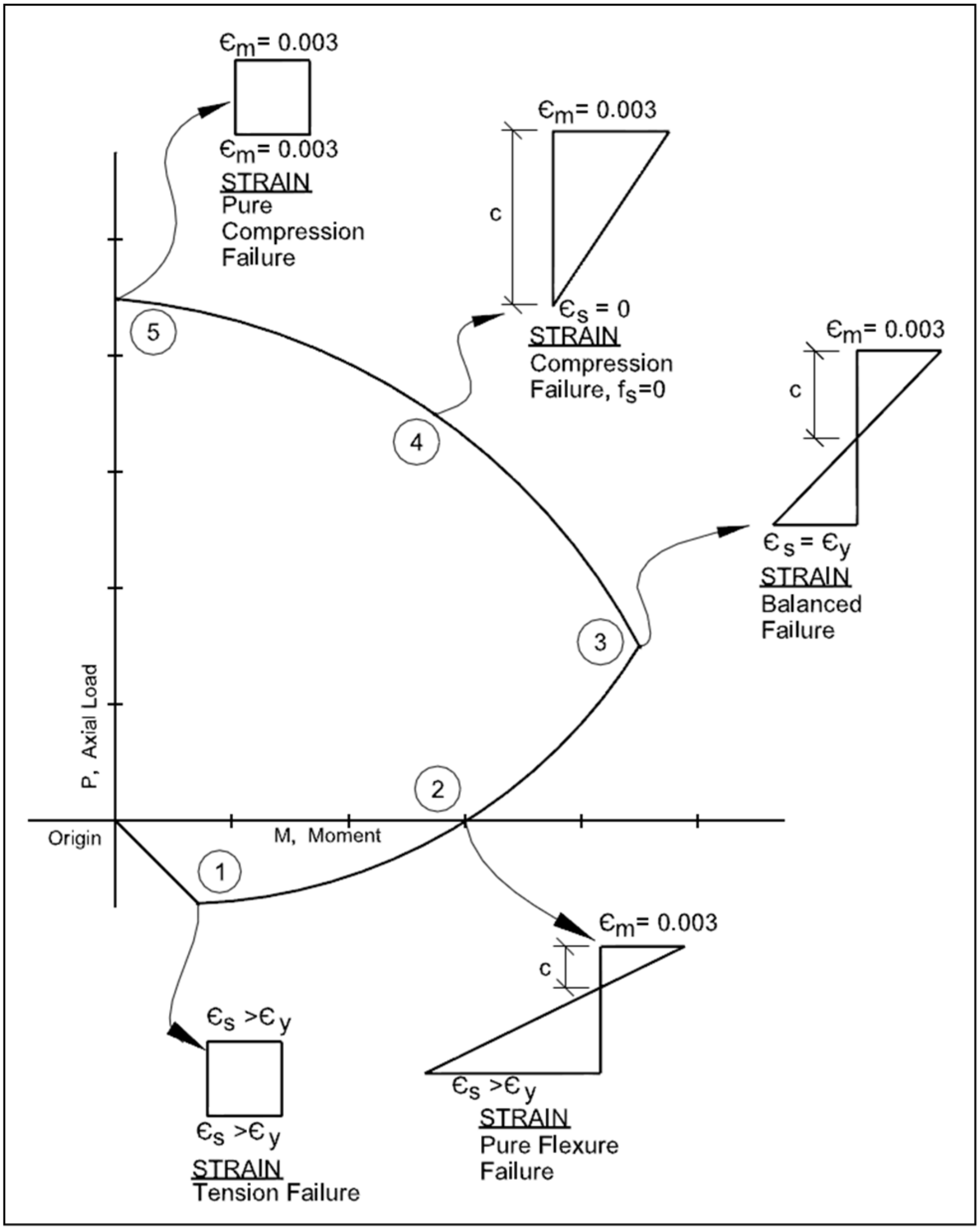


Figure 4–2. Interaction diagram illustrating strain conditions

4–3. Biaxial bending and axial load for all members

a. Paragraph 4–3 applies to all reinforced concrete members subjected to biaxial bending.

b. The load contour method and equation, known as the Bresler Approach, is used for investigation or design of a square or rectangular section that is subjected to an axial compression in combination with bending moments about both the x and y axes. The method is described in Bresler 1960. The Bresler load contour equation describing the capacity of a section with axial load and biaxial bending is:

$$\left[\frac{M_{nx}}{M_{0x}}\right]^K + \left[\frac{M_{ny}}{M_{0y}}\right]^K = 1.0 \tag{4–4}$$

where:

M_{nx}, M_{ny} = nominal biaxial moment strengths with respect to the x and y axes, respectively

M_{0x}, M_{0y} = uniaxial nominal bending strength at P_n about the x and y axes, respectively

For use in design, the equation is modified as shown in equation 4–5. For a given nominal axial load $P_n = \frac{P_u}{\phi}$, the following nondimensional equation must be satisfied:

$$\left[\frac{M_{ux}}{\varphi M_{0x}}\right]^K + \left[\frac{M_{uy}}{\varphi M_{0y}}\right]^K \leq 1.0 \tag{4–5}$$

where:

M_{ux}, M_{uy} = factored bending moments with respect to the x and y axes, respectively

M_{0x}, M_{0y} = uniaxial nominal bending strength at P_n about the x and y axes, respectively

M_{0x} = capacity at P_n when M_{uy} is zero

M_{0y} = capacity at P_n when M_{ux} is zero

K = 1.5 for rectangular members
= 1.75 for square or circular members

c. M_{0x} and M_{0y} are determined according to paragraphs B–2 through B–4 as applicable.

d. Whenever possible, columns subjected to biaxial bending should be circular in cross section. If rectangular or square columns are necessary, the reinforcement should be uniformly spread around the perimeter.

e. Appendix C includes an example of a rectangular column of axial load with biaxial bending using the computer program CGSI.

4–4. Mandatory requirements

a. Design of members for flexure and axial load must meet the requirements of paragraph 4–1.

b. Design of members for biaxial bending and axial load must meet the requirements of paragraph 4–3.

Chapter 5
Shear

5–1. Shear strength

The shear strength (V_n) provided by concrete (V_c) and reinforcement (V_s) must be computed according to ACI 318-19, except in the cases described in this chapter. Guidance is provided in EM 1110-2-2400 (for outlet works) and EM 1110-2-6053 (for performance-based design) to calculate shear capacity for seismic loads, when applicable. Some RCHS members may meet the definition of a deep beam. These members must be designed according to the deep beam provisions of ACI 318-19.

5–2. Shear strength for one-way slabs in reinforced concrete hydraulic structures

a. A substantial amount of RCHS is comprised of large, flat elements such as walls and foundation slabs. These elements are found in I-walls, T-walls, and L-walls, U-framed structures, pumping station wells, gate wells, intake towers, hydraulic control structures, and powerhouse substructures. Shear reinforcement is not normally used for these structures. For these elements, the nominal shear strength of the unreinforced concrete sections is determined from equation 5–1. The requirements of ACI 318-19 for shear capacity in members without shear reinforcement are waived for these member types.

$$V_c = \left[2\sqrt{f_c'} + \frac{N_u}{6\,A_g}\right] bd \text{ lbs } \left(\left[0.17\sqrt{f_c'} + \frac{N_u}{6\,A_g}\right] bd \text{ N}\right) \tag{5–1}$$

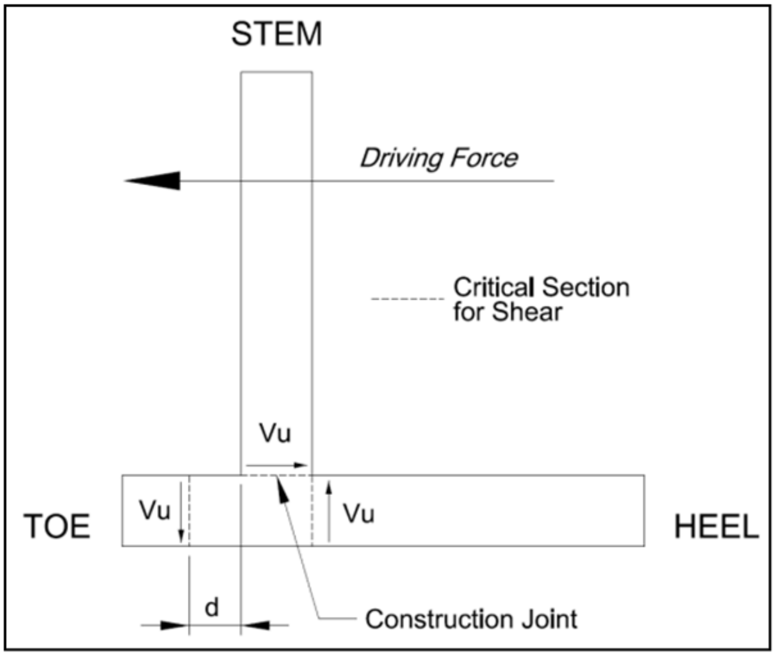
where:

V_c = nominal shear capacity, lbs
f_c' = concrete compressive strength, psi (MPa) f_c'
N_u = factored axial load, lbs (N)
A_g = gross area of design section, in^2 (mm^2)
b = section width, in. (mm)
d = distance from extreme compression fiber of driving-side leg to primary wall reinforcement, in. (mm)

b. The factored shear in one-way slabs must be less than ϕV_c unless shear reinforcement is provided. There is no requirement to limit the shear capacity to $V_c / 2$ when shear reinforcement is not provided, as is required for beams. Shear reinforcement is not commonly used in walls and slabs. They should normally be designed to eliminate the need for shear reinforcement for constructability and economy.

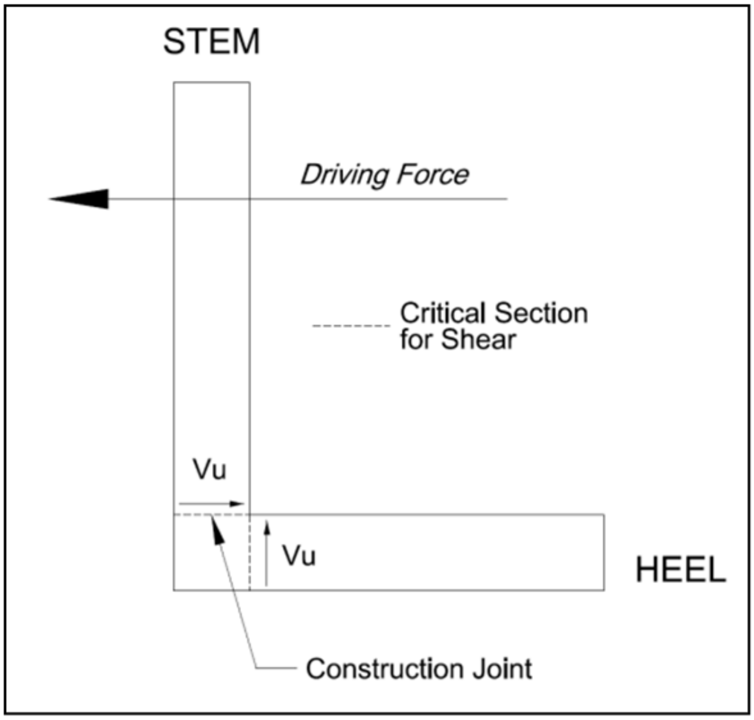
5–3. Design sections for cantilever walls

The critical section for shear depends on the support conditions. When a construction joint is used at the base of a cantilevered stem, the critical section for shear must be taken at the base of the stem. Additionally, vertical reinforcement in both faces of the stem should be developed into the base to ensure shear friction across the joint. The critical section for shear in a base is taken at a distance, d, from the front face of the wall stem for the toe section and at the back face of the wall stem for the heel section. Figure 5–1 and Figure 5–2 shows the critical sections for shear at T-type and L-type walls. U-frame structures are similar.



(a)

Figure 5–1. Critical sections for shear in cantilever T-type walls



(b)

Figure 5–2. Critical sections for shear in cantilever L-type walls

5–4. Shear strength for special straight members

a. The provisions of this paragraph apply only to straight members of box culvert sections, gate wells, or similar structures that satisfy the requirements of paragraphs 5–4b and 5–4c. The stiffening effects of wide supports and haunches are included in determining moments, shears, and member properties. The ultimate shear strength of the member is considered to be the load capacity that causes formation of the first inclined crack. The equations in this section are available only in U.S. (standard) units.

b. Members that are subjected to uniformly (or approximately uniformly) distributed loads that result in internal shear, flexure, and axial compression (but not axial tension).

c. Members having all of the following properties and construction details:

(1) Rectangular cross-sectional shapes.

(2) ℓ_n / d between 1.25 and 9, where ℓ_n is the clear span in inches.

(3) f_c' not more than 6,000 psi.

(4) Rigid, continuous joints or corner connections.

(5) Straight, full-length reinforcement. Flexural reinforcement must not be terminated even though it is no longer a theoretical requirement.

(6) Extension of the exterior face reinforcement around corners such that a vertical lap splice occurs in a region of compression stress.

(7) Extension of the interior face reinforcement into and through the supports.

d. The shear strength provided by the concrete is computed as:

$$V_c = \left[\left(11.5 - \frac{\ell_n}{d}\right) \sqrt{f_c'} \sqrt{1 + \frac{\frac{N_u}{A_g}}{5\sqrt{f_c'}}} \right] bd \text{ lbs} \qquad (5\text{–}2)$$

at a distance of 0.15 ℓ_n from the face of the support.

e. The shear strength provided by the concrete is not to be taken greater than

$$V_c = 2\left[12 - \left(\frac{\ell_n}{d}\right)\right] \sqrt{f_c}' \, bd \text{ lbs} \qquad (5\text{–}3)$$

and not exceed $10\sqrt{f_c'}bd$ lbs.

5–5. Shear strength for curved members

At points of maximum shear, for uniformly loaded, curved, cast-in-place members with $R\ /\ d > 2.25$, where R is the radius curvature to the centerline of the member, the shear strength provided by the concrete is computed as shown below. This equation is available only in U.S. (standard) English units.

$$V_c = \left[4\sqrt{f_c'} \sqrt{1 + \frac{\frac{N_u}{A_g}}{4\sqrt{f_c'}}} \right] bd \text{ lbs} \qquad (5\text{–}4)$$

and not exceed: $10\sqrt{f_c'}bd$ lbs.

5–6. Mandatory requirements

a. The shear strength V_c provided by concrete must be computed according to ACI 318-19 except in the cases described in this chapter.

b. Shear strength of cantilever walls must meet the requirements of paragraph 5–2.

c. Shear strength of special straight members must meet the requirements of paragraph 5–4.

d. Shear strength of curved members must meet the requirements of paragraph 5–5.

Appendix A
References

Unless otherwise indicated, USACE publications are available at https://www.publications.usace.army.mil. Army publications are at https://armypubs.army.mil/.

Section I

Required Publications

AASHTO LRFD Bridge Design Specifications
9th ed. (Available at https://store.transportation.org/item/collectiondetail/202?AspxAutoDetectCookieSupport=1)

ACI 318-19
Building Code Requirements for Structural Concrete and Commentary. (Available at https://www.concrete.org/tools/318buildingcodeportal.aspx.aspx)

AREMA Manual for Railway Engineering
(Available at https://www.arema.org/AREMA_MBRR/AREMA_MBRR/AREMAStore/MRE.aspx)

ASTM A615-20
Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement. (Available at https://www.astm.org/a0615_a0615m-04.html)

ASCE 7-22
Minimum Design Loads and Associated Criteria for Buildings and Other Structures. (Available at https://www.asce.org/publications-and-news/civil-engineering-source/article/2021/12/02/updated-asce-7-22-standard-now-available)

Bresler 1960
Bresler, B. 1960. “Design Criteria for Reinforced Concrete Columns under Axial Load and Biaxial Loadings.” *ACI Journal*. 57(5). (Available at https://www.concrete.org/publications/internationalconcreteabstractsportal/m/details/id/8031)

EM 1110-2-1100
Coastal Engineering Manual. Part VI

EM 1110-2-1612
Ice Engineering

EM 1110-2-2000
Standard Practice for Concrete for Civil Works Structures

EM 1110-2-2100
Stability Analysis of Concrete Structures

EM 1110-2-2400
Structural Design and Evaluation of Outlet Works

EM 1110-2-2502
Floodwalls and Other Hydraulic Retaining Walls

EM 1110-2-2602
Planning and Design of Navigation Locks

EM 1110-2-3402
Barge Impact Forces for Hydraulic Structures

EM 1110-2-6050
Response Spectra and Seismic Analysis for Concrete Hydraulic Structures

EM 1110-2-6051
Time-History Dynamic Analysis of Concrete Hydraulic Structures

EM 1110-2-6053
Earthquake Design and Evaluation of Concrete Hydraulic Structures

ER 1110-2-1806
Earthquake Design and Evaluation of Civil Works Projects

Environmental Operating Principles
http://www.usace.army.mil/Missions/Environmental/EnvironmentalOperatingPrinciples.aspx

Section II

Prescribed Forms

This section contains no entries.

Appendix B
Design Equations for Flexural and Axial Loads

B–1. General

Design equations for each of the member types described in paragraph 4–1b are presented below, along with their derivations. The design equations provide a general procedure that may be used to design members for combined flexural and axial load.

B–2. Flexural and compressive capacity for members with tension reinforcement only (refer to Figure B–1)

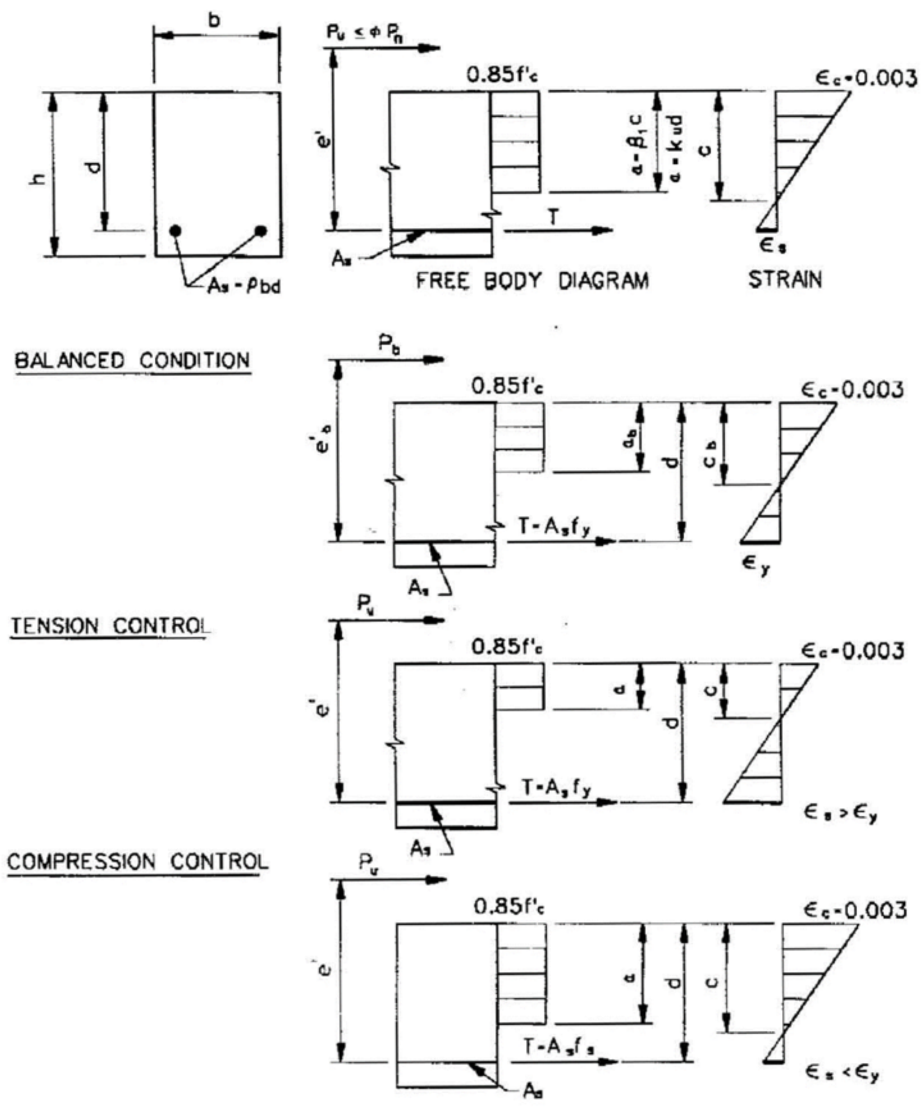


Figure B–1. Axial compression and flexure, single reinforcement

a. Introduction. For members governed by this section, the relationship between the actual eccentricity ratio and the balanced eccentricity ratio will determine whether the strength of a given section is controlled by its strength in tension or its strength in compression. The designer should determine the member's eccentricity using equation 4–1 and compare it to the balanced eccentricity found using equation B–9. If the member's eccentricity ratio is greater than the balanced eccentricity ratio, the strength of the section will be controlled by its strength in tension and the section should be designed using paragraph B–2d. Otherwise, the strength of the section will be controlled by its strength in compression and should be designed using paragraph B–2e.

b. Design axial load. Regardless of whether tension or compression controls, the design axial load strength ϕP_n for members with tie reinforcement is limited by ACI 318-19 and should not be greater than:

$$\phi P_{n(max)} = 0.8\phi\left[.85{f'}_c\left(A_g - \rho bd\right) + f_y \rho bd\right] \quad \text{(B–1)}$$

c. Balanced eccentricity ratio. Equation B–10, which calculates the balanced eccentricity ratio of a member with flexural and compressive loadings and tension reinforcement, is derived as follows:

From equilibrium,

$$\frac{P_u}{\phi} = 0.85 f_c' \ b \ k_u \ d - A_s f_s \quad \text{(B–2)}$$

let

$$j_u = d - \frac{a}{2} = d - \frac{k_u d}{2} \quad \text{(B–3)}$$

from moment equilibrium.

$$\frac{P_u e'}{\phi} = (0.85 f_c' \ b \ k_u \ d)(j_u d) \quad \text{(B–4)}$$

Rewrite equation B–4 as:

$$\frac{P_u e'}{\phi} = (0.85 f_c' \ b \ k_u \ d)\left(d - \frac{k_u d}{2}\right)$$

$$= (0.85 f_c' \ bd^2)\left(k_u - \frac{{k_u}^2}{2}\right)$$

$$= 0.425 f_c'\left(2k_u - {k_u}^2\right)bd^2 \quad \text{(B–5)}$$

From the strain diagram at balanced condition (Figure B–1):

$$\frac{c_b}{d} = \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_y}$$

$$\frac{\left(\frac{k_b d}{\beta_1}\right)}{d} = \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_y} \quad \text{(B–6)}$$

since:

$\varepsilon_y = \frac{f_y}{E_s}$:

$$k_b = \frac{\beta_1 E_s \varepsilon_c}{E_s \varepsilon_c + f_y} \quad \text{(B–7)}$$

since:

$$e_b' = \frac{P_b e'}{P_b} \quad \text{(B–8)}$$

e_b' is obtained by substituting equations B–5 and B–2 into equation B–8 with: $k_u = k_b, f_s = f_y,$ and $P_u = P_b$.

$$e_b' = \frac{0.425 f_c' (2k_b - k_b^2) bd^2}{0.85 f_c' \, k_b bd - f_y \rho bd} \quad \text{(B–9)}$$

Therefore:

$$\frac{e_b'}{d} = \frac{2k_b - k_b^2}{2k_b - \frac{f_y \rho}{0.425 f_c'}} \quad \text{(B–10)}$$

d. Design of sections controlled by tension. Members that have an eccentricity ratio higher than the balanced eccentricity ratio are controlled by their strength in tension and should be designed according to equations B–11 and B–13. The derivations of the design equations and their relevant terms are shown below. ϕP_n is obtained from equation B–2 with $f_s = f_y$ as:

$$\phi P_n = \phi\left(0.85 f_c' b k_u d - A_s f_y\right)$$

$$\phi P_n = \phi\left(0.85 f_c' k_u - \rho f_y\right) bd \quad \text{(B–11)}$$

The design moment ϕM_n is expressed as:

$$\phi M_n = \phi P_n e$$

$$\phi M_n = \phi P_n \left[\frac{e'}{d} - \left(1 - \frac{h}{2d}\right)\right] d \quad \text{(B–12)}$$

Therefore,

$$\phi M_n = \phi\left(0.85 f_c' k_u - f_y \rho\right) \left[\frac{e'}{d} - \left(1 - \frac{h}{2d}\right)\right] bd^2 \quad \text{(B–13)}$$

Substituting equation B–2 with fs = f_y into equation B–5 gives:

$$\left(0.85 f_c' k_u bd - f_y \rho bd\right) e' = 0.425 f_c' (2k_u - k_u^2) bd^2 \quad \text{(B–14)}$$

which reduces to:

$$k_u^2 + 2\left(\frac{e'}{d} - 1\right) k_u - \frac{f_y \rho e'}{0.425 f_c' d} = 0 \quad \text{(B–15)}$$

Solving by the quadratic equation:

$$k_u = \sqrt{\left(\frac{e'}{d} - 1\right)^2 + \left(\frac{\rho f_y}{0.425 f_c'}\right)\frac{e'}{d}} - \left(\frac{e'}{d} - 1\right) \qquad \text{(B–16)}$$

e. Sections controlled by compression. Members that have an eccentricity ratio less than or equal to the balanced eccentricity ratio are controlled by their strength in compression and should be designed according to equations B–17 and B–18 below. The derivation of the design equations and their relevant terms is shown below. ϕP_n is obtained from equation B–2.

$$\phi P_n = \phi(0.85 f_c' k_u - \rho f_s)\, bd \qquad \text{(B–17)}$$

and ϕM_n is obtained by multiplying equation B–17 by e.

$$\phi M_n = \phi\left(0.85 f_c' k_u - \rho f_s\right)\left[\frac{e'}{d} - \left(1 - \frac{h}{2d}\right)\right] bd^2 \qquad \text{(B–18)}$$

The steel stress, f_s, is expressed as $f_s = E_s \varepsilon_c$.
From Figure B–1:

$$\frac{c}{d} = \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_s}$$

or

$$\frac{\left(\frac{k_u d}{\beta_1}\right)}{d} = \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_s}$$

therefore,

$$f_s = \frac{E_s \varepsilon_c (\beta_1 - k_u)}{k_u} \left(\geq -f_y\right) \text{ equation B–19}$$

Substituting equations B–2 and B–19 into B–5 gives:

$$0.85 f_c' k_u b d e' - \left[\frac{E_s \varepsilon_c (\beta_1 - k_u)}{k_u}\right] \rho b d e' = 0.425 f_c' (2k_u - k_u^2) bd^2 \qquad \text{(B–20)}$$

which can be arranged as:

$$k_u^3 + 2\left(\frac{e'}{d} - 1\right) k_u^2 + \left(\frac{E_s \varepsilon_c \rho e'}{0.425 f_c' d}\right) k_u - \frac{\beta_1 E_s \varepsilon_c \rho e'}{0.425 f_c' d} \qquad \text{(B–21)}$$

B–3. Flexural and compressive capacity for members with both tension and compression reinforcement (refer to Figure B–2)

a. Introduction. For structures governed by this subchapter, the relationship of the actual eccentricity ratio to the balanced eccentricity ratio will determine whether the strength of a given section is controlled by its strength in tension or its strength in compression. The designer should determine the member’s eccentricity using equation 4–1 and compare it to the balanced eccentricity found using equation B–27 below. If the member’s eccentricity ratio is greater than the balanced eccentricity ratio, the strength of the section will be controlled by its strength in tension and the section should be designed using paragraph B–3e. Else, the strength of the section will be controlled by its strength in compression and should be designed using paragraph B–3f.

b. Detailing. Design for flexure using compression reinforcement is discouraged. However, if compression reinforcement is used in members controlled by compression, lateral reinforcement must be provided according to the ACI 318-19.

c. Design Axial Strength. Regardless of whether tension or compression controls, the design axial load strength ϕP_n for sections governed by this subchapter is limited by ACI 318-19 and should not be taken as greater than:

$$\phi P_{n(max)} = 0.8\phi\left[.85f'_c\left(A_g - (\rho + \rho')\,bd\right) + f_y(\rho + \rho')\,bd\right] \qquad \text{(B–22)}$$

d. Balanced condition. Equation B–28, which calculates the balanced eccentricity ratio of a member with flexural and compressive loadings and both tension and compression reinforcement, is derived as follows.

From equilibrium shown in Figure B–2:

$$\frac{P_u}{\phi} = 0.85f_c^{'}k_u bd + f_s^{'}\rho^{'}bd - f_s\rho bd \qquad \text{(B–23)}$$

In a manner similar to the derivation of equation B–6, moment equilibrium results in:

$$\frac{P_u e^{'}}{\phi} = 0.425f_c^{'}(2k_u - k_u^2)bd^2 + f_s^{'}\rho^{'}bd\left(d - d^{'}\right) \qquad \text{(B–24)}$$

As in equation B–7:

$$k_b = \frac{\beta_1 E_s \varepsilon_c}{E_s\varepsilon_c + f_y} \qquad \text{(B–25)}$$

since

$$e_b^{'} = \frac{P_b e^{'}}{P_s} \qquad \text{(B–26)}$$

and using equations B–23 and B–24:

$$e_b^{'} = \frac{0.425f_c^{'}\left(2k_b - k_b^2\right)bd^2 + f_s^{'}\rho^{'}bd\left(d - d^{'}\right)}{0.85f_c^{'}k_b bd + f_s^{'}\rho^{'}bd - f_s\rho bd} \qquad \text{(B–27)}$$

which can be rewritten as:

$$e_b^{'} = \frac{\left(2k_b - k_b^2\right)d + \frac{f_s^{'}\rho^{'}}{0.425f_c^{'}}\left(d - d^{'}\right)}{2k_b + \frac{f_s\rho^{'}}{0.425f_c^{'}} - \frac{f_y\rho}{0.425f_c^{'}}}$$

or:

$$\frac{e_b^{'}}{d} = \frac{2k_b - k_b^2 + \frac{f_s^{'}\rho^{'}\left(1 - \frac{d^{'}}{d}\right)}{0.425f_c^{'}}}{2k_b - \frac{f_y\rho}{0.425f_c^{'}} + \frac{f_s^{'}\rho^{'}}{0.425f_c^{'}}} \qquad \text{(B–28)}$$

The value of k_b is given in equation B–25 and $f_s^{'}$ is given in equation B–31 with $k_u = k_b$.

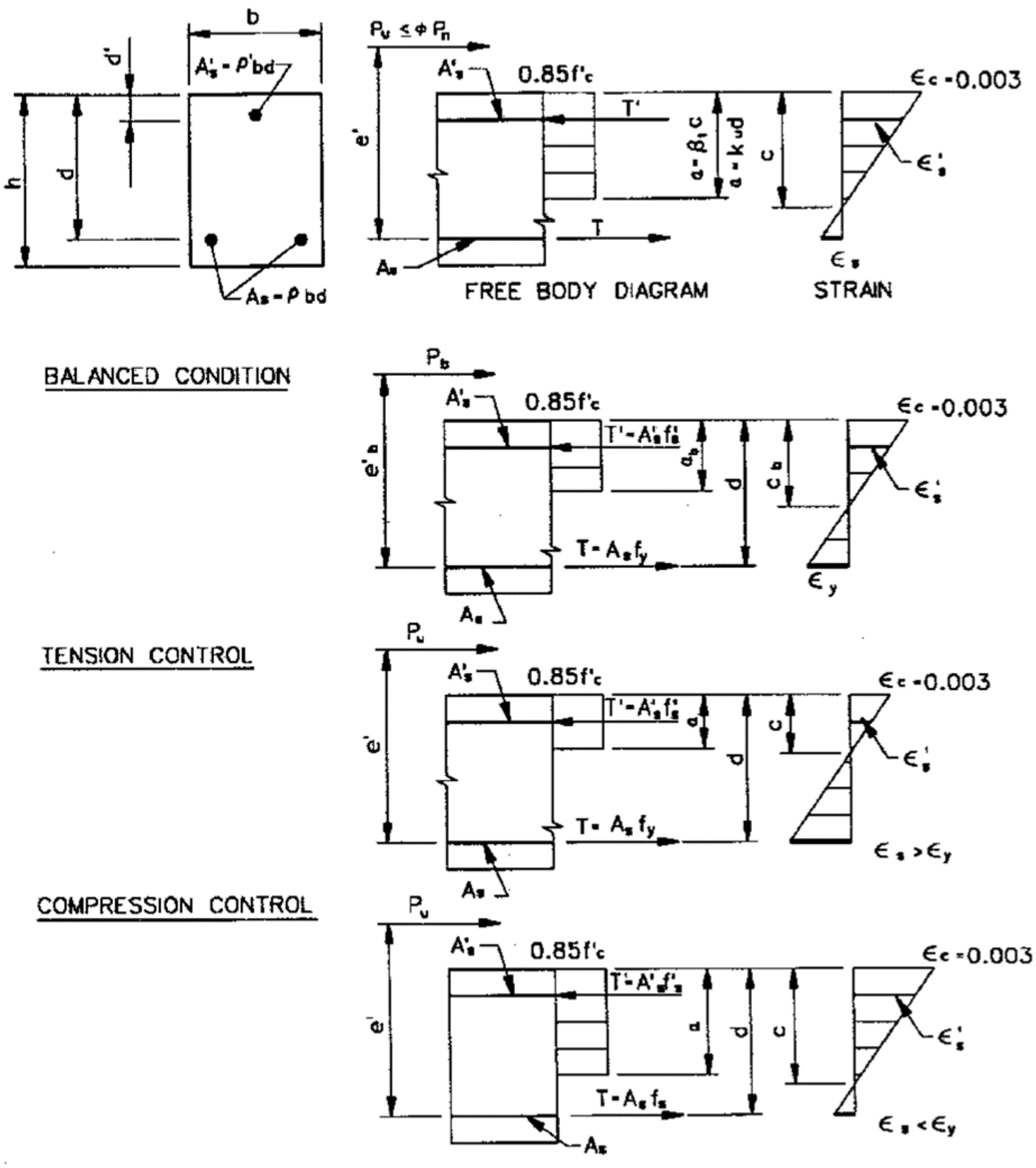


Figure B–2. Axial compression and flexure, double reinforcement

e. *Sections controlled by tension*. Members that have an eccentricity ratio higher than the balanced eccentricity ratio are controlled by their strength in tension and should be designed according to equations B–29 and B–30. The derivation of the design equations and their relevant terms is shown below; ϕP_n is obtained as equation B–23 with $f_s = f_y$:

$$\phi P_n = \phi\left(0.85 f_c^{'} k_u + \rho^{'} f_s^{'} - \rho f_y\right) bd \tag{B–29}$$

Using equations B–12 and B–29:

$$\phi M_n = \phi\left(0.85 f_c^{'} k_u + \rho^{'} f_s^{'} - \rho f_y\right)\left[\frac{e^{'}}{d} - \left(1 - \frac{h}{2d}\right)\right] bd^2 \qquad \text{(B–30)}$$

From Figure B–2:

$$\frac{\varepsilon_s^{'}}{c-d^{'}} = \frac{\varepsilon_y}{d-c}\ ; \quad f_s^{'} = E_s \varepsilon_s^{'}\ ; \quad c = \frac{k_u d}{\beta_1}$$

Therefore:

$$\frac{f_s'}{E_s} = \left(\frac{k_u d}{\beta_1} - d'\right)\left(\frac{\varepsilon_y}{d - \dfrac{k_u d}{\beta_1}}\right)$$

or:

$$f_s^{'} = \frac{\left(k_u - \beta_1 \dfrac{d^{'}}{d}\right)}{(\beta_1 - k_u)} E_s \varepsilon_y\ (\leq\ f_y) \text{ equation B– 31}$$

Substituting equation B–23 with $f_s = f_y$ into equation B–24 gives:

$$\left(0.85 f_c^{'} k_u bd + f_s^{'} \rho^{'} bd - f_y \rho bd\right) e^{'} = 0.425 f_c^{'} (2k_u - k_u^2) bd^2 + f_s^{'} \rho^{'} bd\left(d - d^{'}\right) \qquad \text{(B–32)}$$

Using equation B–31, equation B–32 can be written as:

$$\begin{aligned} & k_u^3 + \left[2\left(\frac{e'}{d} - 1\right) - \beta_1\right] k_u^2 \\ & - \left\{\frac{f_y}{0.425 f_c'}\left[\rho'\left(\frac{e'}{d} + \frac{d'}{d} - 1\right) + \frac{\rho e'}{d}\right] + 2\beta_1\left(\frac{e'}{d} - 1\right)\right\} k_u \\ & + \frac{f_y \beta_1}{0.425 f_c'}\left[\rho' \frac{d'}{d}\left(\frac{e'}{d} + \frac{d'}{d} - 1\right) + \frac{\rho e'}{d}\right] = 0 \end{aligned} \qquad \text{(B–33)}$$

f. *Sections controlled by compression*. Members that have an eccentricity ratio less than or equal to the balanced eccentricity ratio are controlled by their strength in compression and should be designed according to equations B–34 and B–35 below. The derivation of the design equations and their relevant terms is shown below.

ϕP_n is obtained from equilibrium:

$$\phi P_n = \phi\left(0.85 f_c^{'} k_u + \rho^{'} f_s^{'} - \rho f_s\right) bd \qquad \text{(B–34)}$$

Using equations B–12 and B–34,

$$\phi M_n = \phi\left(0.85 f_c^{'} k_u + \rho^{'} f_s^{'} - \rho f_s\right)\left[\frac{e^{'}}{d} - \left(1 - \frac{h}{2d}\right)\right] bd^2 \qquad \text{(B–35)}$$

From Figure B–2:

$$\frac{\varepsilon_s}{d-c} = \frac{\varepsilon_c}{c}; \quad f_s = E_s \varepsilon_s; \quad c = \frac{k_u d}{\beta_1}$$

which can be written as:

$$f_s = \frac{E_s \varepsilon_c (\beta_1 - k_u)}{k_u} \tag{B–36}$$

Also,

$$\frac{\varepsilon_s^{'}}{c-d^{'}} = \frac{\varepsilon_c}{c}$$

which can be rewritten as:

$$f_s^{'} = \frac{E_s \varepsilon_c \left[k_u - \beta_1 \left(\frac{d^{'}}{d}\right)\right]}{k_u} \left(\geq -f_y\right) \text{ equation B– 37}$$

From equations B–23 and B–24,

$$(0.85 f_c' k_u bd = f_s' \rho' bd - f_s \rho bd)e' \quad = 0.425 f_c' (2k_u - k_u^2) bd^2 + f_s' \rho' bd(d - d') \tag{B–38}$$

Substituting equations B–36 and B–37 with $k_b = k_u$ into equation B–38 gives:

$$k_u^3 + 2\left(\frac{e'}{d} - 1\right) k_u^2 + \frac{E_s \varepsilon_c}{0.425 f_c'} \left[(\rho + \rho')\left(\frac{e'}{d}\right) - \rho'\left(1 - \frac{d'}{d}\right)\right] k_u$$
$$- \frac{\beta_1 E_s \varepsilon_c}{0.425 f_c'} \left[\rho'\left(\frac{d'}{d}\right)\left(\frac{e'}{d} + \frac{d'}{d} - 1\right) + \rho\left(\frac{e'}{d}\right)\right] = 0 \tag{B–39}$$

B–4. Flexural and tensile capacity (refer to Figure B–3)

a. This section should be used to design sections subject to tension and uniaxial flexure, regardless of reinforcement pattern.

b. The design axial strength ϕP_n of members loaded in tension is limited by ACI 318-19 and should not be taken greater than allowed by:

$$\phi P_{n(max)} = 0.8\phi\left(\rho + \rho^{'}\right) f_y bd \tag{B–40}$$

c. Tensile reinforcement should be provided in both faces of the member if the load has an eccentricity ratio e'/d in the range shown in equation B–41. Sections with eccentricity ratios in this range will generally be loaded axially between the two layers of reinforcement and both layers of reinforcement will be in tension. A section under a tensile load with an eccentricity ratio higher than ($1 - \left(\frac{h}{2d}\right)$) will place the assumed tensile reinforcement into compression. Therefore, the extreme compressive fiber should be assumed to be at the opposite face of the section and the eccentricity ratio should be recalculated.

$$\left(1 - \frac{h}{2d}\right) \geq \frac{e^{'}}{d} \geq 0 \tag{B–41}$$

d. Members governed by this chapter should be designed according to the criteria listed below.

e. Sections that are purely in tension and subject to no flexural load should be designed according to paragraph B–4i.

f. Sections under tension and uniaxial flexure that have $e'/d > 0$ should be designed according to paragraph B–4j.

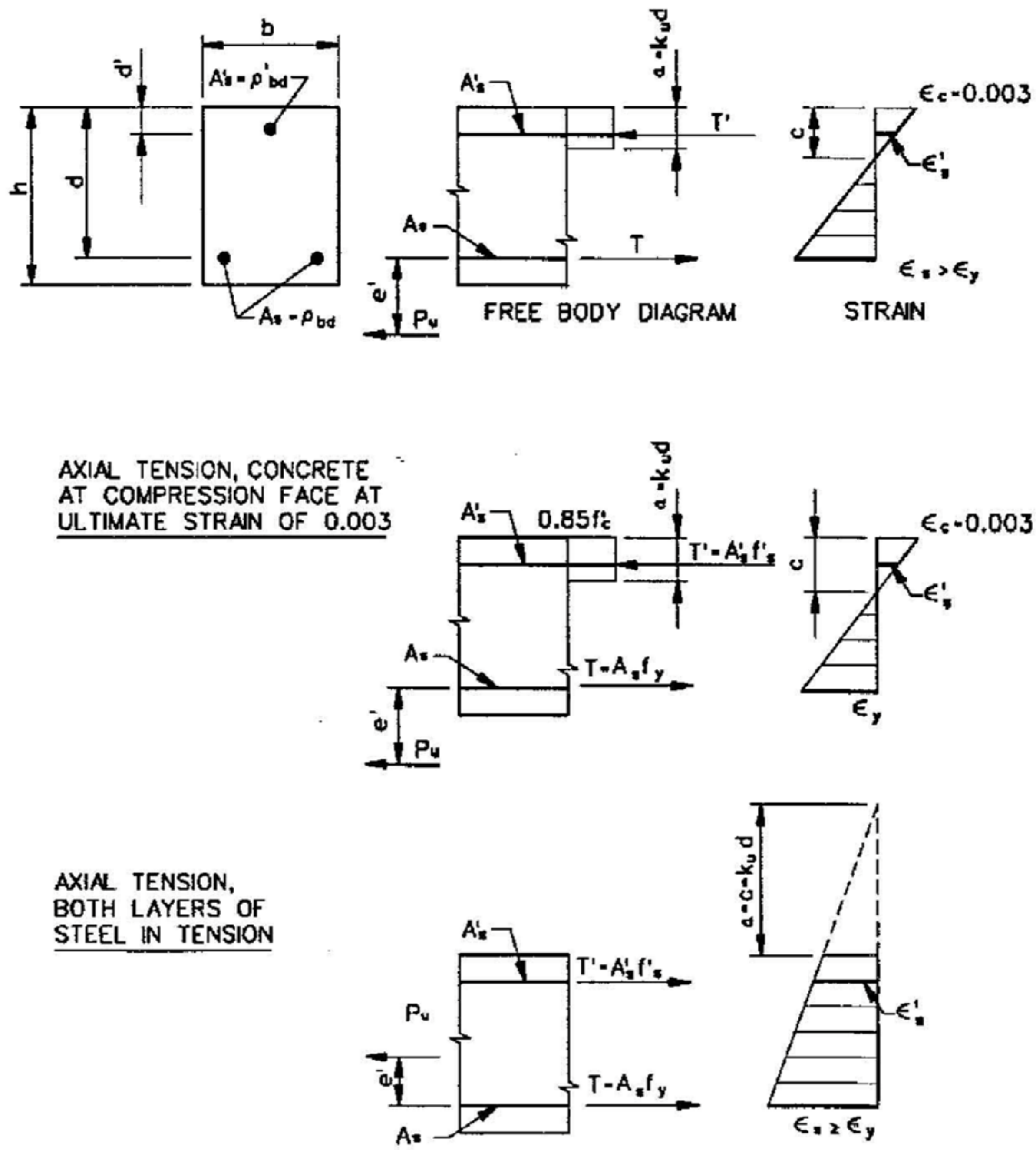


Figure B–3. Axial tension and flexure, double reinforcement

g. Sections subjected to a tensile load with an eccentricity ratio $\frac{e'}{d} < 0$ should be designed using paragraph B–2e, when compressive reinforcement is not present ($A_s' = 0$) or is not subject to a compressive load ($c \le d'$). This case is similar to the case where the member is subject to compression and uniaxial flexure, where the member's strength is controlled by its tensile strength. However, the k_u is slightly different. The derivation of k_u for this case follows the derivation presented in equation B–15, with the exception that the tensile load results in k_u having the value presented below in equation B–42:

$$k_u = -\left(\frac{e'}{d} - 1\right) - \sqrt{\left(\frac{e'}{d} - 1\right)^2 + \left(\frac{\rho f_y}{0.425 f_c'}\right)\frac{e'}{d}} \quad \text{(B–42)}$$

h. Sections subject to a tensile load with an eccentricity ratio $\frac{e'}{d} < 0$ should be designed using paragraph B–3e when compressive reinforcement is present ($A_s' > 0$) and subject to a compressive load ($c > d'$). This case is similar to the case where the member is subject to compression and uniaxial flexure, and the member's strength is controlled by its tensile strength.

i. Members that are purely in tension should be designed according to equation B–40. The derivation of the design equation is shown below.

From equilibrium (double reinforcement):

$$\phi P_n = \phi\left(A_s + A_s'\right) f_y \quad \text{(B–43)}$$

For design, the axial load strength of tension members is limited to 80 percent of the design axial load strength at zero eccentricity.
Therefore, equation B–43 can simply be converted to equal equation B–40, restated as:

$$\phi P_{n(max)} = 0.8\phi(\rho + \rho') f_y bd$$

j. For the case where $1 - \frac{h}{2d} \ge \frac{e'}{d} \ge 0$, the applied tensile resultant P_u/ϕ lies between the two layers of steel. Members loaded in this manner should be designed according to equations B–44 and B–45 below. The derivation of the design equations and their relevant terms is shown below:

from equilibrium:

$$\phi P_n = \phi\left(A_s f_y + A_s' f_s'\right)$$

or:

$$\phi P_n = \phi\left(\rho f_y + \rho' f_s'\right) bd \quad \text{(B–44)}$$

and:

$$\phi M_n = \phi P_n \left[\left(1 - \frac{h}{2d}\right) - \frac{e'}{d}\right] d$$

or:

$$\phi M_n = \phi\left(\rho f_y + \rho' f_s'\right) \left[\left(1 - \frac{h}{2d}\right) - \frac{e'}{d}\right] bd^2 \qquad \text{(B–45)}$$

From Figure B–3:

$$\frac{\varepsilon_s'}{a+d'} = \frac{\varepsilon_y}{a+d}$$

which can be rewritten as:

$$f_s' = f_y \frac{\left(k_u + \frac{d'}{d}\right)}{(k_u + 1)} \qquad \text{(B–46)}$$

From Figure B–3, equilibrium requires:

$$A_s f_s e' = A_s' f_s' \left(d - d' - e'\right) \qquad \text{(B–47)}$$

Substituting equation B–46 and $f_s = f_y$ into equation B–47 results in:

$$k_u = \frac{\rho'\left(\frac{d'}{d}\right)\left(1 - \frac{d'}{d} - \frac{e'}{d}\right) - \rho\left(\frac{e'}{d}\right)}{\rho\left(\frac{e'}{d}\right) - \rho'\left(1 - \frac{d'}{d} - \frac{e'}{d}\right)} \qquad \text{(B–48)}$$

B–5. Flexural capacity for members with tension reinforcement only (refer to Figure B–1)

The design moment ϕM_n is expressed as:

$$\phi M_n = T\left(d - \frac{a}{2}\right) \qquad \text{(B–49)}$$

where:

$$a = \frac{A_s F_y}{0.85 f_c' b} \qquad \text{(B–50)}$$

B–6. Flexural capacity for members with tension and compression reinforcement (refer to Figure B–2)

The design moment ϕM_n is expressed as:

$$\phi M_n = C_c\left(d - \frac{a}{2}\right) + C_s(d - d') \qquad \text{(B–51)}$$

where:

$$C_c = 0.85 f_c' ba \qquad \text{(B–52)}$$

$$C_s = 0.85\, f_s' - 0.85\, f_c'\, A_s' \qquad \text{(B–53)}$$

Appendix C
Investigation Examples

C–1. General

This appendix provides four investigative examples. The purpose of these examples is to illustrate the application of this engineer manual and ACI 318-19, to determine the flexural capacity of existing concrete sections of a single and reinforced beam (paragraph C–2) and of a beam with reinforcement on both faces (paragraph C–3); to create an interaction diagram using computer program CGSI (paragraph C–4); and to calculate the capacity of an existing concrete section with axial load and biaxial bending (paragraph C–5).

C–2. Example – Analysis of a singly reinforced beam (Figure C–1)

a. Given:

$\beta_1 = 0.85$ $\quad f_c^{'} = 4\ ksi$

$E_s = 29{,}000\ ksi$ $\quad f_y = 60\ ksi$

$A_s = 1.58\ in.^2$

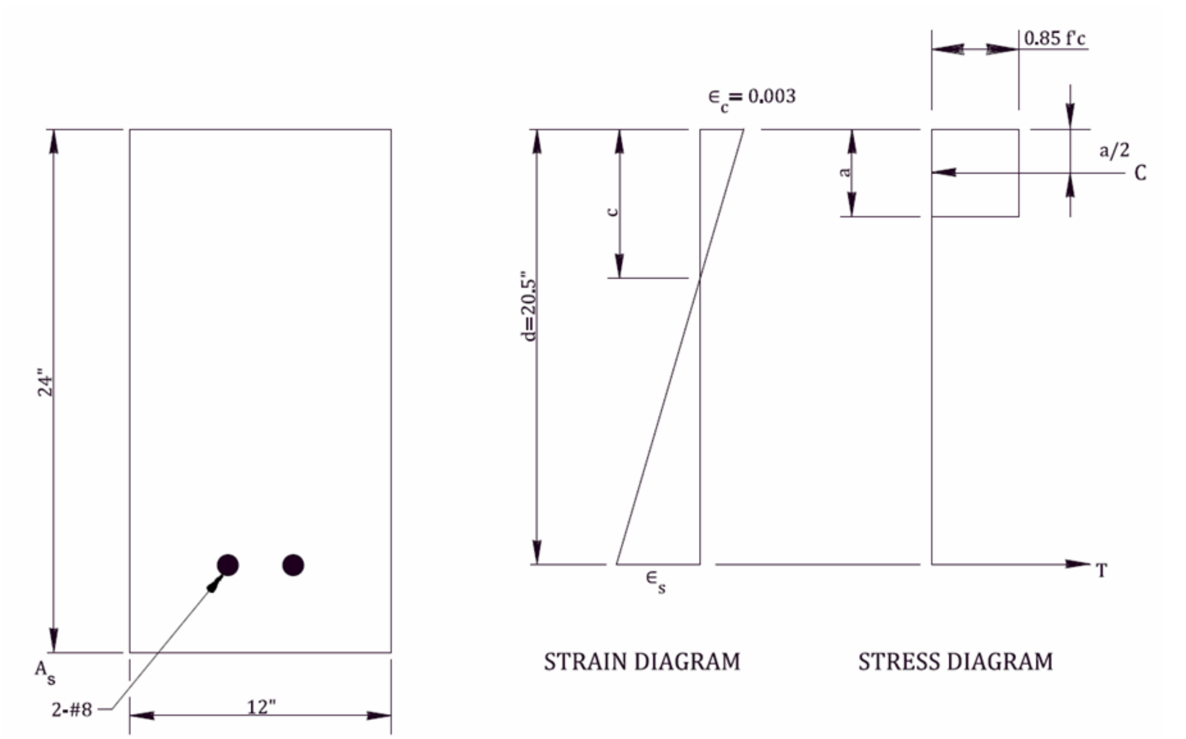


Figure C–1. Diagram of singly reinforced beam cross section, strain, and stress

b. Solution:

(1) Check steel ratio.

$$\rho_{act} = \frac{A_s}{bd}$$
$$= \frac{1.58}{12(20.5)}$$
$$= 0.006423$$

$$\rho_b = 0.85\beta_1 \frac{f_c^{'}}{f_y}\left(\frac{87,000}{87,000+f_y}\right)$$
$$= 0.85(0.85)\left(\frac{4}{60}\right)\left(\frac{87,000}{87,000+60,000}\right)$$
$$= 0.02851$$

According to paragraph 3–5:

$0.25\rho_b = 0.00713$

$\rho_{act} = 0.00642$

$\rho_{act} < 0.25\rho_b$

ρ_{act} is less than the limit not requiring special study or investigation. Therefore, no special consideration for serviceability, constructability, and economy is required. This reinforced section is satisfactory.

(2) Assume the steel yields and compute the internal forces.

$T = A_s f_y = 1.58(60) = 94.8\ kips$

$C = 0.85 f_c' ba$

$C = 0.85(4)(12)a = 40.8a$

(3) From equilibrium set T = C and solve for a:

$94.8 = 40.8a \rightarrow a = 2.324\ in.$

Then, $a = \beta_1 c \rightarrow c = \frac{2.324}{0.85} = 2.734\ in.$

(4) Check ε_s to demonstrate steel yields prior to crushing of the concrete:

$$\frac{\varepsilon_s}{20.5 - c} = \frac{0.003}{c}$$

$$\varepsilon_s = (20.5 - 2.734)\left(\frac{0.003}{2.734}\right) = 0.01949$$

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29{,}000} = 0.00207$$

Given the results of this calculation, steel yields:

$$\varepsilon_s > \varepsilon_y$$

(5) Compute the flexural capacity:

$$\phi M_n = \phi(A_s f_y)\left(d - \frac{a}{2}\right)$$

$$= 0.90(1.58)(60)\left(20.5 - \frac{2.324}{2}\right)$$

$$= 1649.9\ in. - k$$

$$= 137.5\ ft - k$$

C–3. Example – Analysis of a slab with reinforcement in both faces (see Figure C–2)

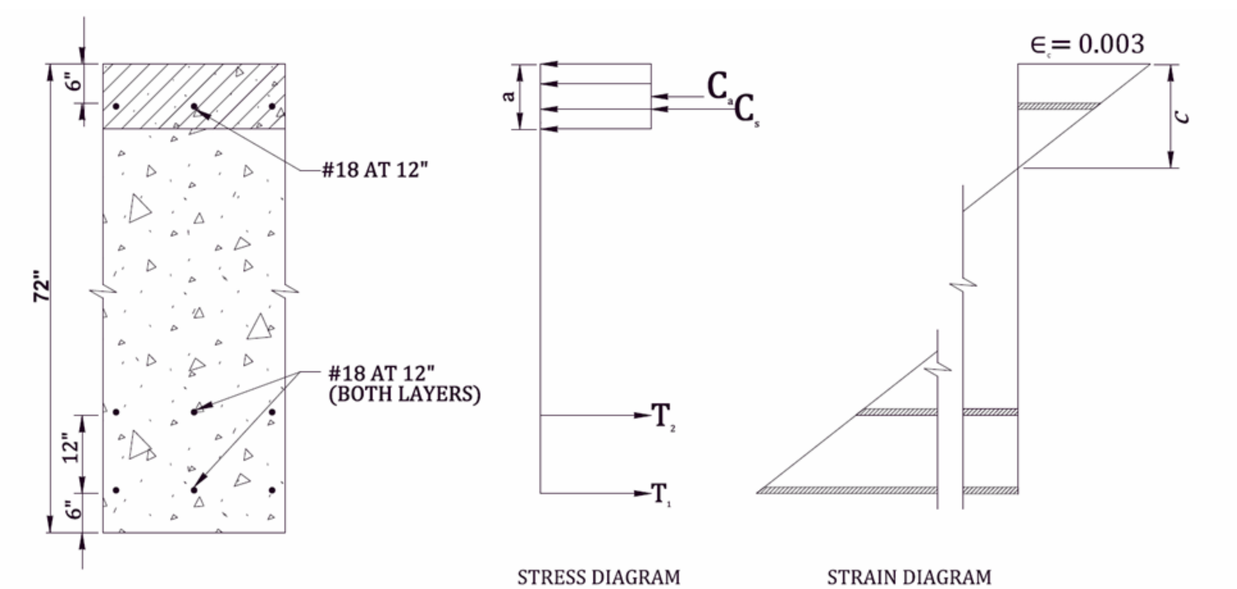


Figure C–2. Slab with reinforcement on both faces, with diagram of stress and strain

a. Given:

$$f_c' = 4{,}000\ psi \qquad \varepsilon_c = 0.003$$
$$f_y = 60{,}000\ psi \qquad \beta_1 = 0.85$$
$$A_s = 8.00\ in.^2 \qquad E_s = 29{,}000{,}000\ psi$$
$$A_s' = 4.00\ in.^2 \qquad b = 12\ in.$$

b. Solution:

(1) First analyze considering steel in tension face only.

$$\rho = \frac{A_s}{bd} = \frac{8}{(12)(60)} = 0.011$$

$$\rho_{ba1} = 0.85\frac{\beta_1 f_c'}{f_y}\left(\frac{87{,}000}{87{,}000 + f_y}\right) = 0.0285$$

$$\rho = \frac{0.011}{0.0285}\rho_{ba1} = 0.39\rho_b$$

$$T = A_s f_y$$

$$T = 8(60) = 480\ kips$$

$$C_c = 0.85 f_c^{'} ba = 40.8a$$

Then:

$$T = C_c$$

$$\therefore a = 11.76\ in. \text{ and } c = 13.84\ in.$$

By similar triangles, demonstrate that steel yields.

$$\frac{\varepsilon_c}{13.84} = \frac{\varepsilon_{s(2)}}{54 - 13.84} \Rightarrow \varepsilon_{s(2)} = \ 0.00871 > \varepsilon_y = 0.0021$$

Given the results of this calculation, both layers of steel yield.

Moment capacity = 480 kips (d – a / 2)

$$= 480\ kips\ (60\ - 5.88)$$

$$M = 25{,}976.5\ in. - k$$

(2) Next, analyze the situation considering steel in compression face.

$$\rho^{'} = \frac{A_s^{'}}{bd} = \frac{4}{12(60)} = 0.0056$$

$$\rho - \rho' = 0.0054$$

$$= 0.85\frac{\beta_1 f_c'}{f_y} \bullet \frac{d'}{d}\left(\frac{87{,}000}{87{,}000 - f_y}\right) = 0.01552$$

$$\rho - \rho' \leq 0.01552$$

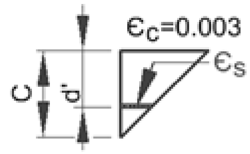
∴ Compression steel does not yield, must do general analysis using $\sigma : \varepsilon$ compatibility.

Locate neutral axis:

$T = 480\ kips$

$C_c = 0.85 f_c b a = 40.8a$

$C_s = A_s'(f_s' - 0.85 f_c') = 4(f_s' - 3.4)$



By similar triangles:

$$\frac{\varepsilon_s'}{c-6} = \frac{0.003}{c}$$

Substitute:

$$c = \frac{a}{0.85} = 1.176a$$

Then:

$$\varepsilon_s' = 0.003 - \frac{0.0153}{a}$$

Since:

$$f_s' = E\varepsilon_s' \Rightarrow f_s' = \left(87 - \frac{443.7}{a}\right) ksi$$

Then:

$$C_s = 4\left(87 - 3.4 - \frac{443.7}{a}\right) kips$$

$T = C_c + C_s = 480\ kips$

Substitute for C_c and C_s and solve for *a*:

$$40.8a + 334.4 - \frac{1774.8}{a} = 480$$

$$a^2 - 3.57a - 43.5 = 0$$

Then:

$a = 8.62\ in.$

And:

$c = 10.14\ in.$

Check:

$\varepsilon_s' > \varepsilon_y$

By similar triangles:

$$\frac{0.003}{10.14} = \frac{\varepsilon_s'}{d - 10.14}$$

$\varepsilon_s' = 0.0148 > 0.0021$

$C_c = 40.8a = 351.6\ kips$

$C_s = 4(32.11) = 128.4\ kips$

$C_c + C_s = 480kips = T$

Resultant of C_c and $C_s = \frac{351.6\left(\frac{8.62}{2}\right)+(128.4)(6)}{480} = 4.76\ in.$

Internal Moment Arm = $60 - 4.76 = 55.24\ in.$

$M = 480\ (55.24) = 26{,}515.2\ in.-k$

c. Table C–1 lists the a, c, Arm, and M (moment capacity) for a beam with tension steel only and a beam with reinforcement on both faces.

Table C–1
Moment capacity of a beam with tension steel only and of a beam with the addition of compression steel

	Tension Steel Only	Compression Steel
a	11.76 in.	8.62 in.
c	13.84 in.	10.14 in.
Arm	54.11 in.	55.24 in.
M	25,976.5 in. – k	26,515.2 in. – k

Note: A 2.1 percent increase in moment capacity is observed with steel reinforcement in the compression zone.

C–4. Example – Construction of interaction diagram (see Figure C–3)

A complete discussion on the construction of interaction diagrams is beyond the scope of this manual. However, to demonstrate how the equations presented in Chapter 4 are used to construct a diagram, a few basic points will be computed. Note that the effects of φ, the strength reduction factor, have not been considered. Using the example cross section shown below, compute the points defined by 1, 2, 3 notations shown in Figure C–3.

a. Given:

$f_c' = 4.0\ ksi$

$f_y = 60\ ksi$

$A_s = \rho bd = 2.0\ sq.in.$

$d = 22\ in.$

$h = 24\ in.$

$b = 12\ in.$

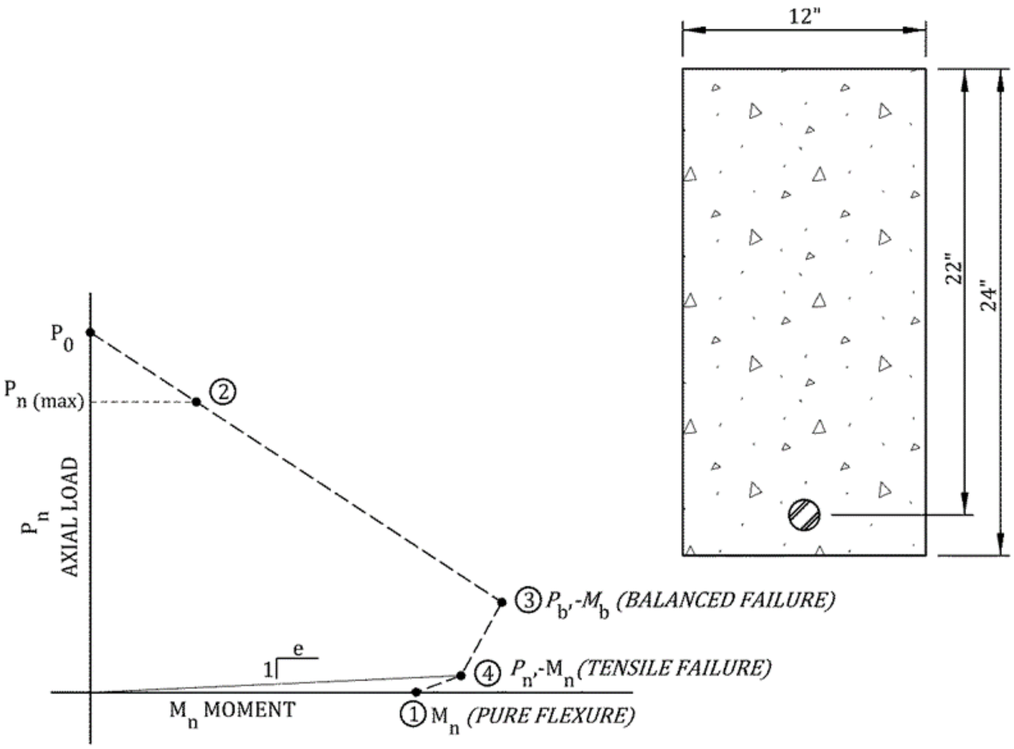


Figure C–3. General interaction diagram points and given cross section

b. Determination of Point 1, Pure Flexure (presented in Figure C–4).

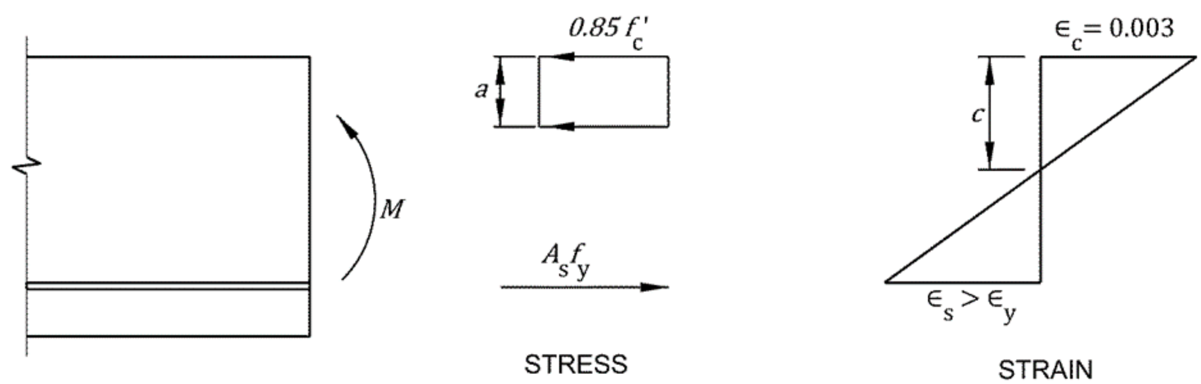


Figure C–4. Stress and strain under pure flexure

$$\phi M_n = \phi 0.85\, f_c' ab\left(d - \frac{a}{2}\right)$$

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{(2.0)(60.0)}{(0.85)(4.0)(12)} = 2.941\ in.$$

$$M_n = (0.85)(4.0)(2.941)(12)\left(22 - \frac{2.941}{2}\right)$$

$$M_n = 2{,}463.5\ k - in.$$

$$M_n = 205.3\ k - ft$$

c. Determination of Point 2, Maximum Axial Capacity (presented in Figure C–5).

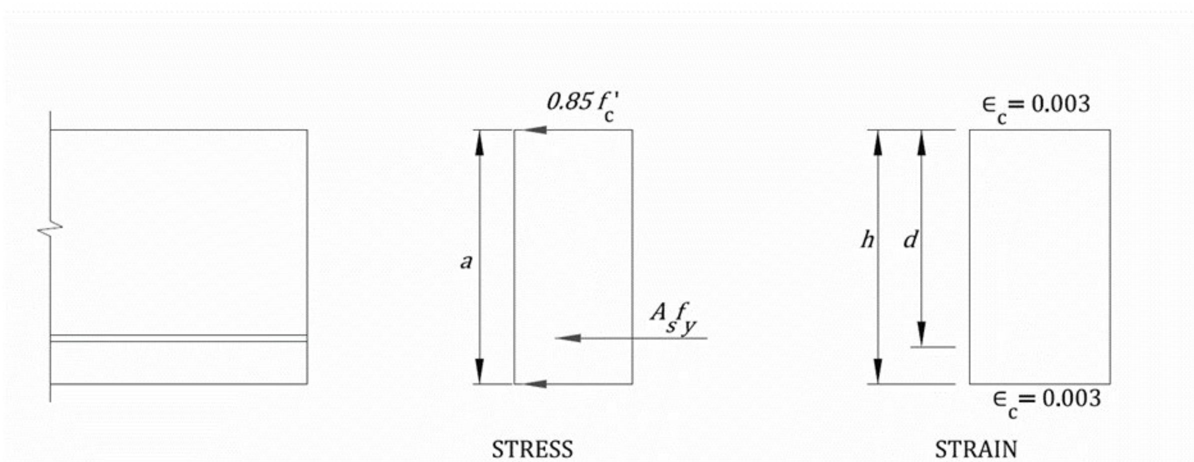


Figure C–5. Stress and strain under maximum axial load

$$\phi P_{n(max)} = \phi 0.80 P_o$$

$$\phi P_{n(max)} = \phi 0.80\left[0.85 f_c'\left(A_g - \rho bd\right) + f_y \rho bd\right]$$

$$P_{n(max)} = 0.80[(0.85)(4.0)(288 - 2.0) + (60.0)(2.0)]$$

$$P_{n(max)} = 0.80(1092.4) = 873.9\ kips$$

d. Determination of Point 3, Balanced Point (presented in Figure C–6).

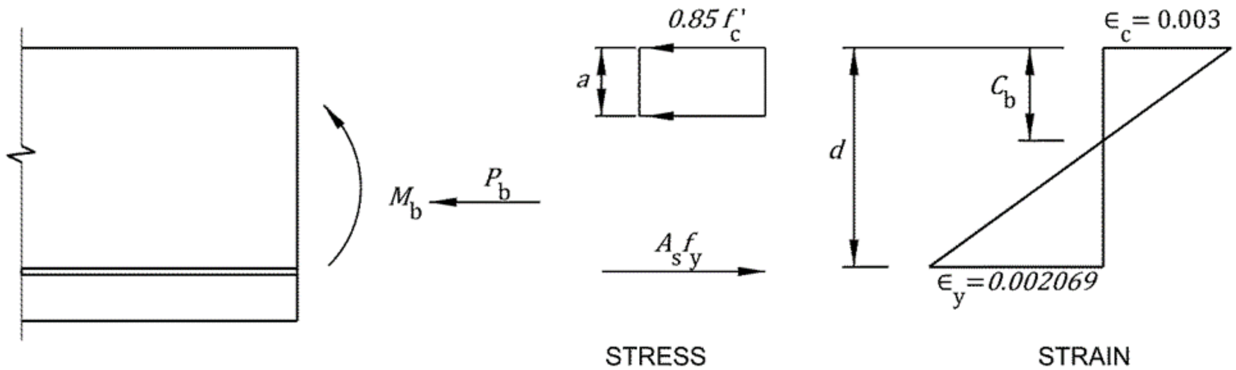


Figure C–6. Stress and strain at balanced point

(1) Find k_b:

$$k_b = \frac{\beta_1 E_s \epsilon_c}{E_s \epsilon_c + f_y}$$

$$k_b = \frac{(0.85)(29{,}000)(0.003)}{(29{,}000)(0.003) + 60} = 0.5031$$

Then k_b is equal to k_u when balanced condition.

(2) Find $\frac{e_b'}{d}$:

$$\frac{e_b'}{d} = \frac{2k_u - k_u^2}{2k_u - \frac{\rho f_y}{0.425 f_c'}}$$

$$\frac{e_b'}{d} = \frac{2(0.5031) - (0.5031)^2}{2(0.5031) - \frac{(0.00758)(60)}{(0.425)(4.0)}} = 1.01952$$

(3) Find P_b:

$$\phi P_b = \phi\left[0.85 f_c' k_b - \rho f_y\right] bd$$

$$P_b = [(0.85)(4.0)(0.5031) - (0.00758)(60)](12)(22)$$

$$P_b = 331.52\ kips$$

(4) Find M_b:

$$\phi M_b = \phi\left[0.85 f_c' k_b - \rho f_y\right]\left[\frac{e_b{}'}{d} - \left(1 - \frac{h}{2d}\right)\right] b d^2$$

$$M_b = [(0.85)(4.0)(0.5031) - (0.00758)(60)] \cdot \left[1.01952 - \left(1 - \frac{24.0}{2(22.0)}\right)\right](12)(22.0)^2$$

$$M_b = 4120.55\ k - in.$$

$$M_b = 343.38\ k - ft$$

e. The interaction diagram shown in Figure C–7 illustrates the results of the calculation.

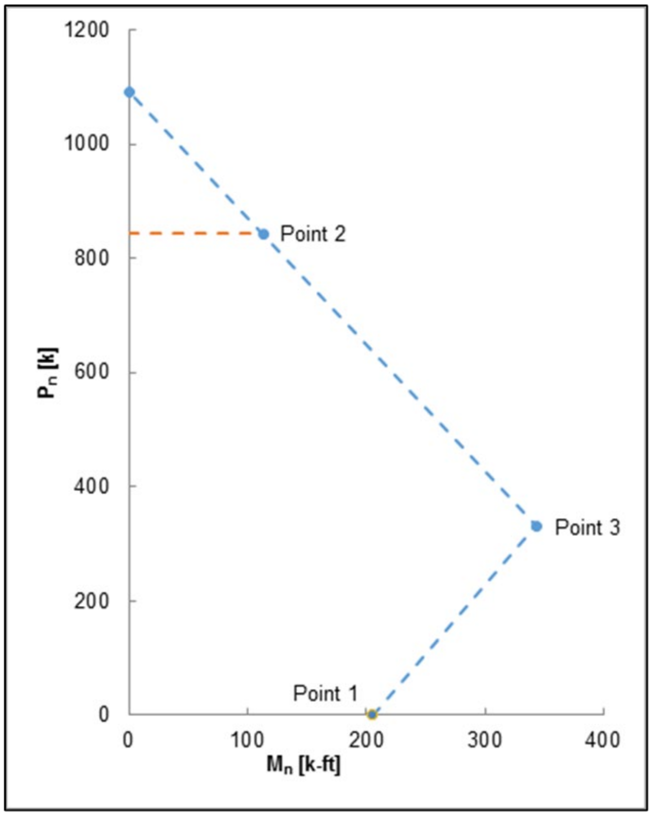


Figure C–7. Interaction diagram for combined bending and axial forces

f. The interaction diagram can also be formed using the program CGSI or with commercial software. An arbitrary load case of a moment of 100 k-ft was applied to generate the curve, but any load case can be used; the interaction diagram will remain the same. The nominal strength, M_n and P_n, interaction curve is the dashed line produced in CGSI. The interaction diagram developed using CGSI is presented in Figure C–8.

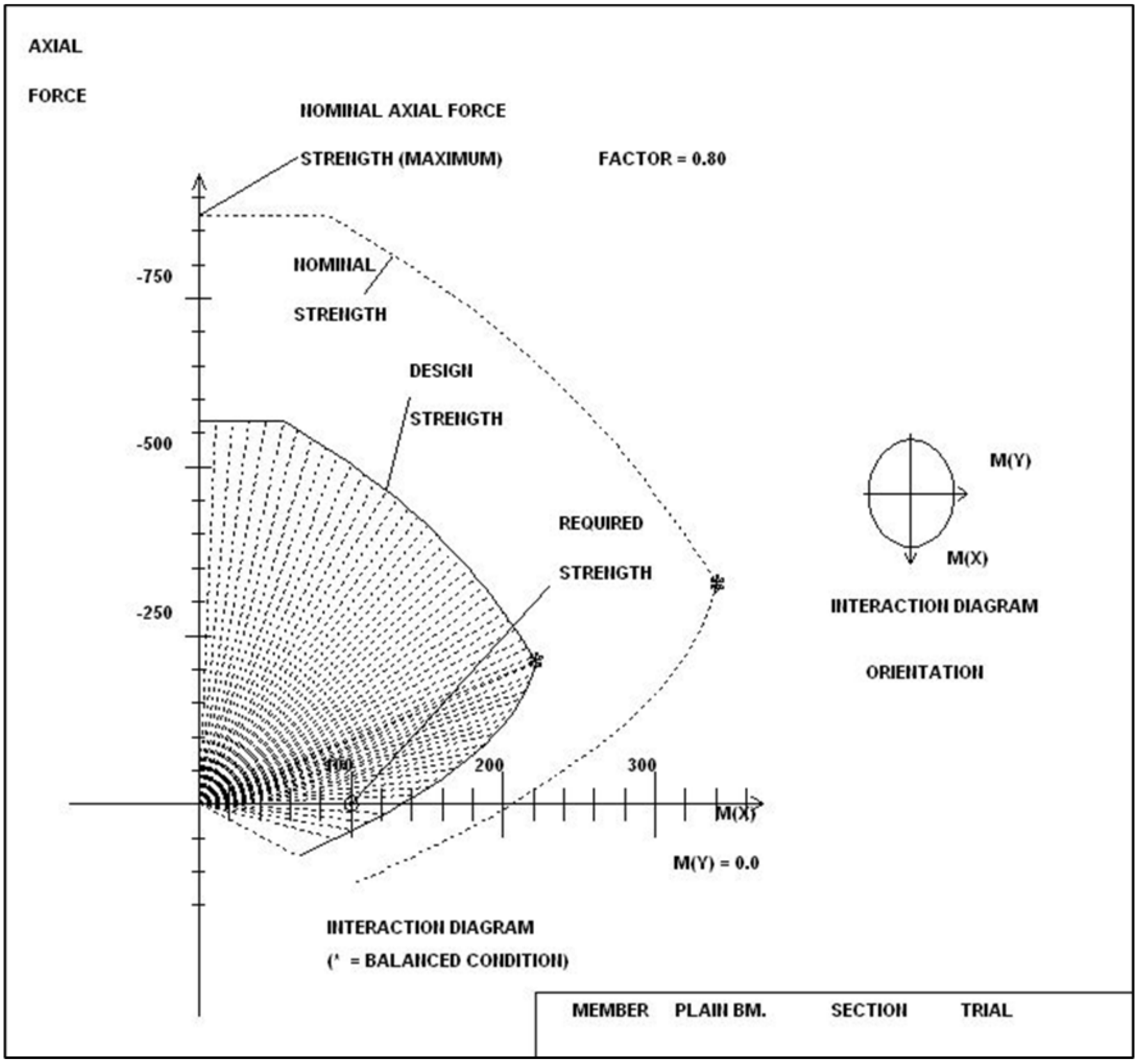


Figure C–8. Interaction diagram produced in the computer program CGSI

C–5. Example – Axial load with biaxial bending (see Figure C–9)

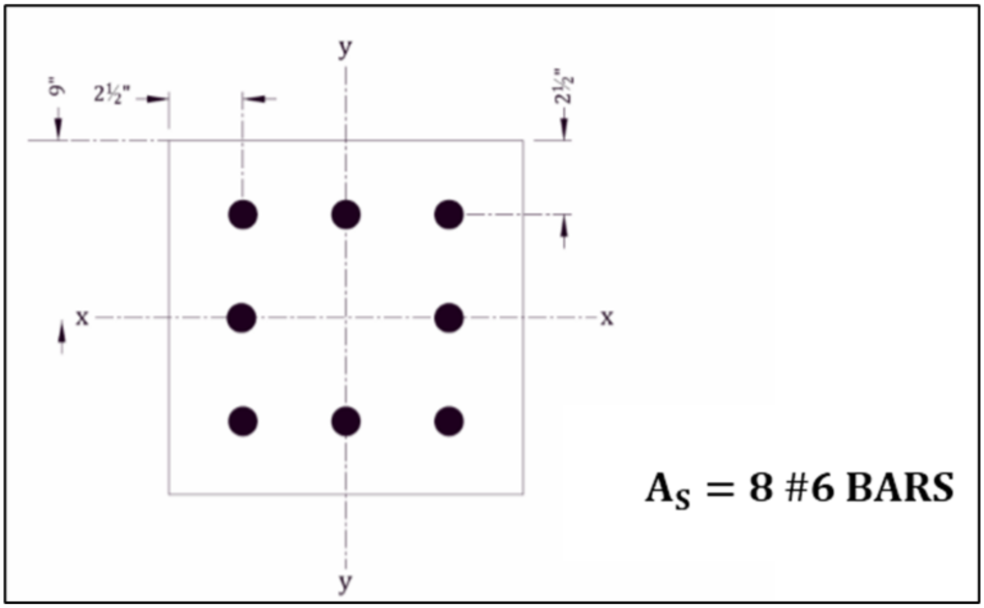


Figure C–9. Cross section of column with 8 #6 bars

a. According to paragraph 4–3, design an 18 x 18-in. reinforced concrete column for the following conditions:

$$f_c' = 4{,}000\ psi$$

$$f_y = 60{,}000\ psi$$

$\varphi = 0.65$ – for compression controlled regions

$$P_u = 300\ kips,\ Required\ P_n = \frac{P_u}{0.65} = 462\ kips$$

$$M_{ux} = 94\ ft - k, Required\ M_{rx} = \frac{M_{ux}}{0.65} = 145\ ft - k$$

$$M_{uy} = 30\ ft - k,\ Required\ M_{ry} = \frac{M_{uy}}{0.65} = 46\ ft - k$$

Let concrete cover plus one-half a bar diameter equal 2.5 in.

b. Using uniaxial design procedures (Appendix B), select reinforcement for P_n and bending about the x-axis, since M_{nx} > M_{ny}. The resulting cross section is given below.

c. Then use CGSI to develop the interaction diagram. In Figure C–10 and Figure C–11 show screenshots of the user inputs into the CGSI interface. The key changes to make are to select User Input under Solution Type, and then, in the bottom right corner, input the current ACI code factors. The modulus of elasticity of the concrete was found according to ACI 318-19. Material strength can also be adjusted. Also, define the reinforcement locations and the load case data.

d. In this example, the solution for both axes are found with a single load case, so both moments and axial load are input into one load case. With user input, a single strength reduction factor (PHI) is entered, so check that the value of PHI matches the required valued from ACI 318-19. The value of PHI depends on whether the member is loaded in the tension control region, compression control region, or in between (see Figure 4–1 for an illustration.

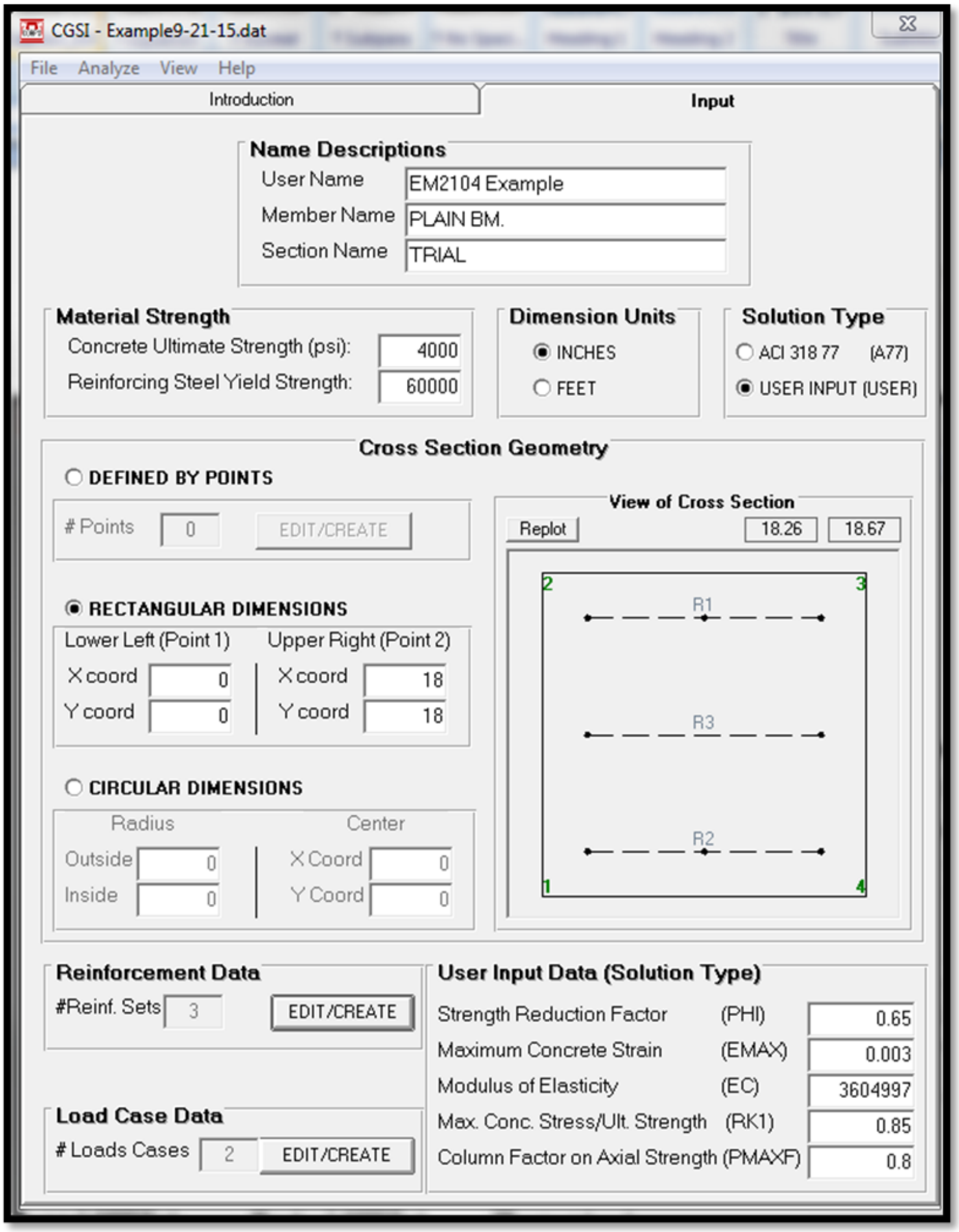


Figure C–10. Inputs for the computer program CGSI

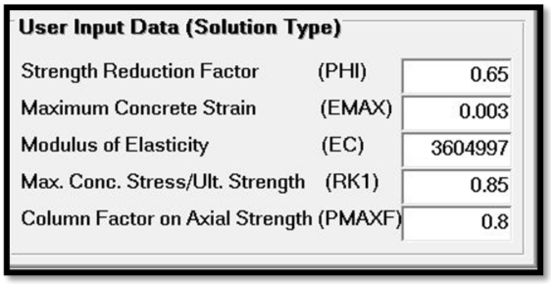


Figure C–11. User inputs for the computer program CGSI

e. Once the user has input all of the data and load cases, the analysis can be performed by selecting Analyze, then Go. The interaction diagram is created as shown in Figure C–12. The point lies in the shaded region, so the capacity does exceed the demand. The value is in the compression controlled region above the balance point, so PHI = 0.65.

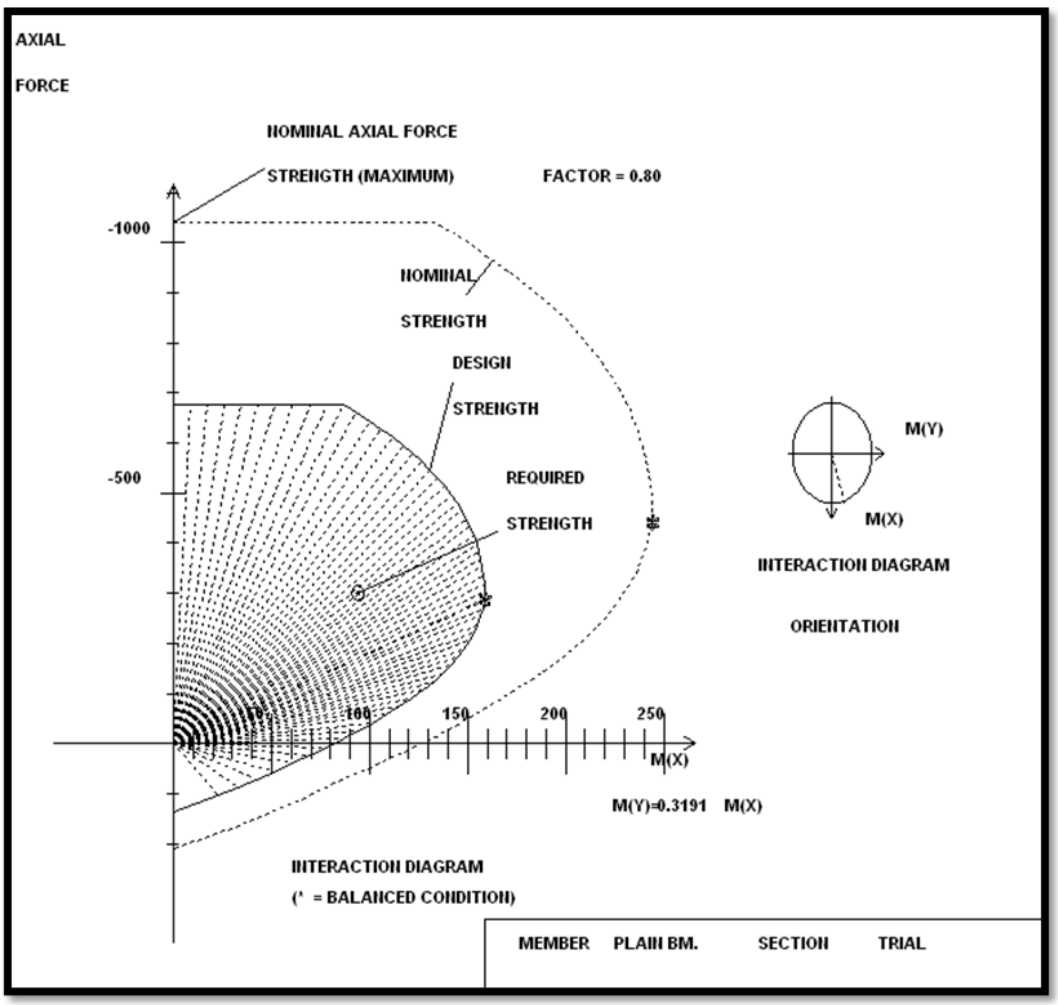


Figure C–12. Flexural strength when both bending moments are acting simultaneously

f. The other method includes using the interaction equation, which is outlined below in combination with CGSI by applying M_{ny} and M_{nx} separately and generating two interaction diagrams, then by combining the results in the equation.

g. Figure C–13 and Figure C–14, created in CGSI, present the nominal strength interaction diagrams about x and y axes. Figure C–14 shows that the member is adequate for uniaxial bending about the y-axis with P_n = 462 kips and M_{ny} = 46 ft-kips. From Figure C–13 and Figure C–14 at P_n = 462 kips:

$$M_{nx} = 280\, ft - k$$

$$M_{ny} = 280\, ft - k$$

Which, for a square column, must satisfy:

$$\left(\frac{M_{rx}}{M_{nx}}\right)^{1.75} + \left(\frac{M_{ry}}{M_{ny}}\right)^{1.75} \leq 1.0$$

$$\left(\frac{145}{280}\right)^{1.75} + \left(\frac{46}{280}\right)^{1.75} = 0.36$$

h. If a value greater than 1.0 is obtained, increase reinforcement and/or increase member dimensions. This confirms the initial results from CGSI (Figure C–12).

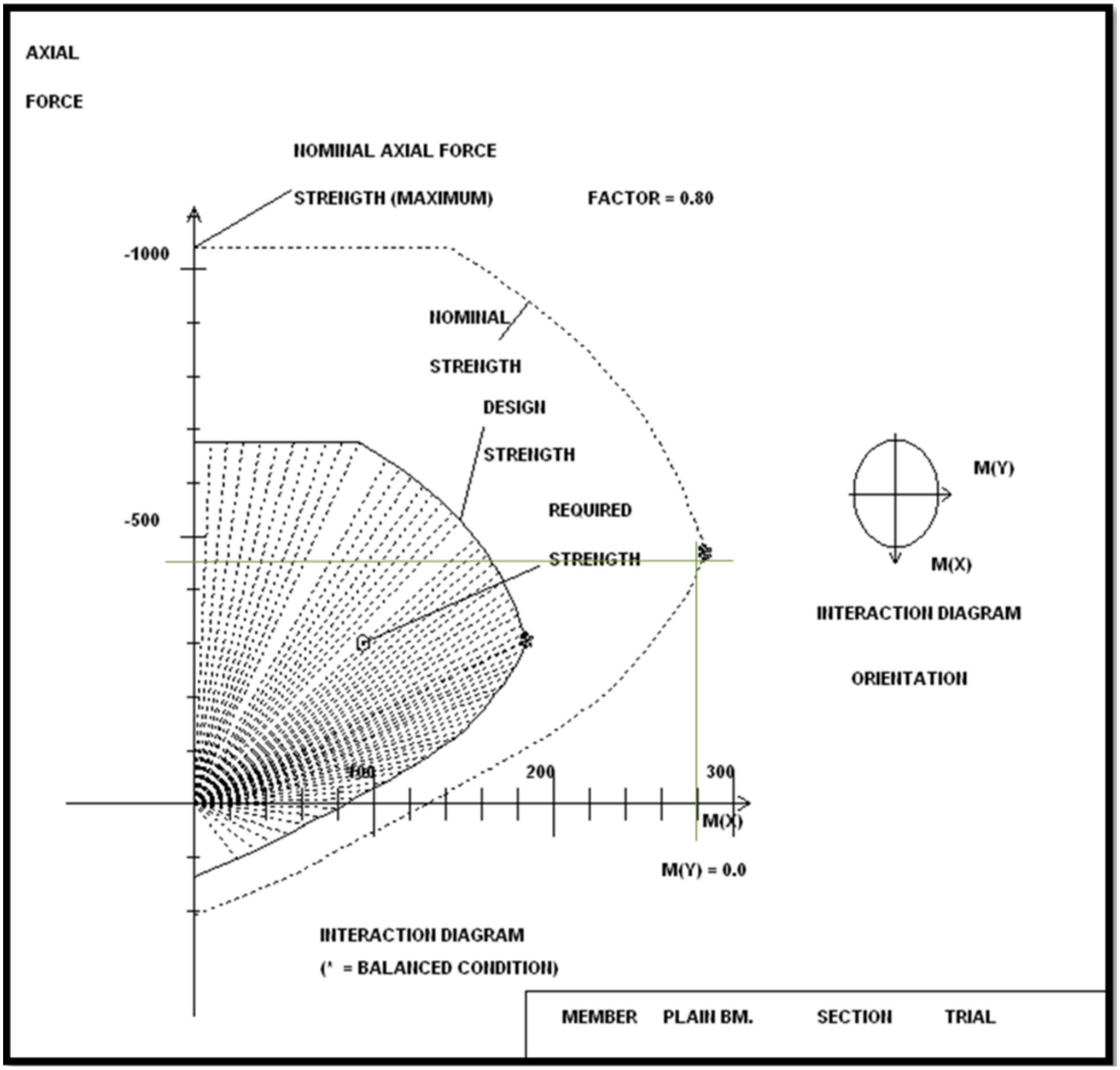


Figure C–13. Nominal flexural strength about the x-axis

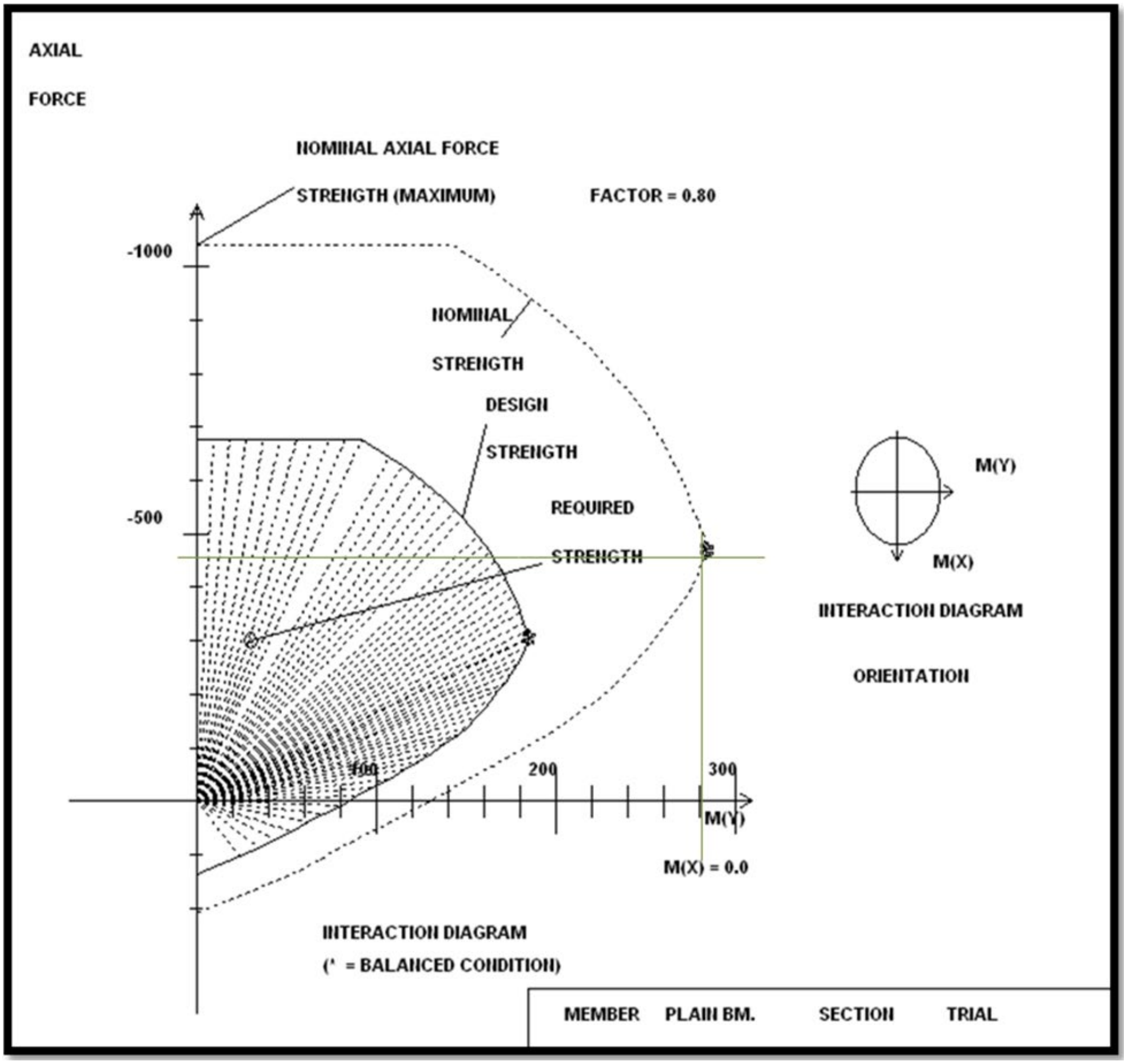


Figure C–14. Nominal flexural strength about the y-axis

Appendix D
Design Examples

D–1. General

This appendix provides derivation of design equations for singly and doubly reinforced members and design procedures (paragraph D–2); design examples of a singly reinforced retaining wall (paragraph D–3), a doubly reinforced retaining wall (paragraph D–4), and a combined flexure plus axially loaded retaining wall (paragraph D–5); a design example of a coastal floodwall (paragraph D–6); and a shear strength example for a special straight member (paragraph D–7) and for a curved member (paragraph D–8).

D–2. Design equations and procedures

The following paragraphs provide derivation of design equations for the singly reinforced and doubly reinforced members and design procedures.

a. Derivation of design equations for singly reinforced members. Figure D–1 shows the conditions of stress on a singly reinforced member subjected to a moment M_n and load P_n. Equations for design may be developed by satisfying conditions of equilibrium on the section.

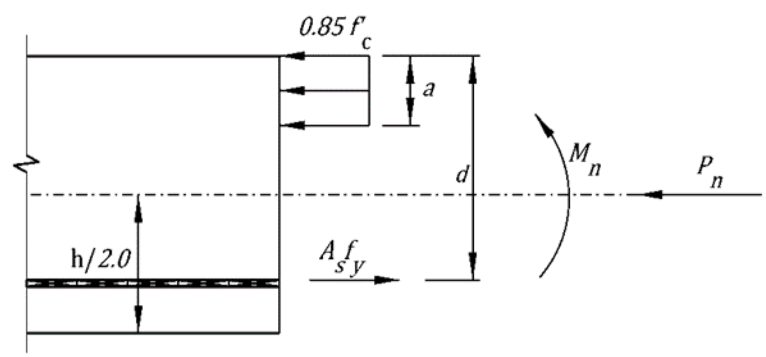


Figure D–1. Section of stress for singly reinforced member

By requiring the $\sum M$ about the tensile steel to equal zero:

$$M_n = 0.85 f_c' ab\left(d - \frac{a}{2}\right) - P_n\left(d - \frac{h}{2}\right) \quad \text{(D–1)}$$

By requiring the $\sum H$ to equal zero:

$$A_s f_y = 0.85 f_c' ab - P_n \quad \text{(D–2)}$$

Expanding equation D–1 yields:

$$M_n = 0.85f_c'abd - 0.425f_c'a^2b - P_n\left(d - \frac{h}{2}\right)$$

Let $a = K_u d$, then:

$$M_n = 0.85f_c'K_u bd^2 - 0.425f_c'K_u^2 d^2 b - P_n\left(d - \frac{h}{2}\right)$$

The above equation may be solved for K_u using the solution for a quadratic equation:

$$K_u = 1 - \sqrt{1 - \frac{M_n + P_n\left(d - \frac{h}{2}\right)}{0.425f_c'bd^2}}$$

Substituting K_{ud} for "a" in equation D–2 then yields:

$$A_s = \frac{0.85f_c'K_u bd - P_n}{f_y}$$

b. Derivation of design equations for doubly reinforced members. Figure D–2 shows the conditions of stress and strain on a doubly reinforced member subjected to a moment M_n and load P_n. Equations for design are developed in a manner identical to that shown previously for singly reinforced beams.

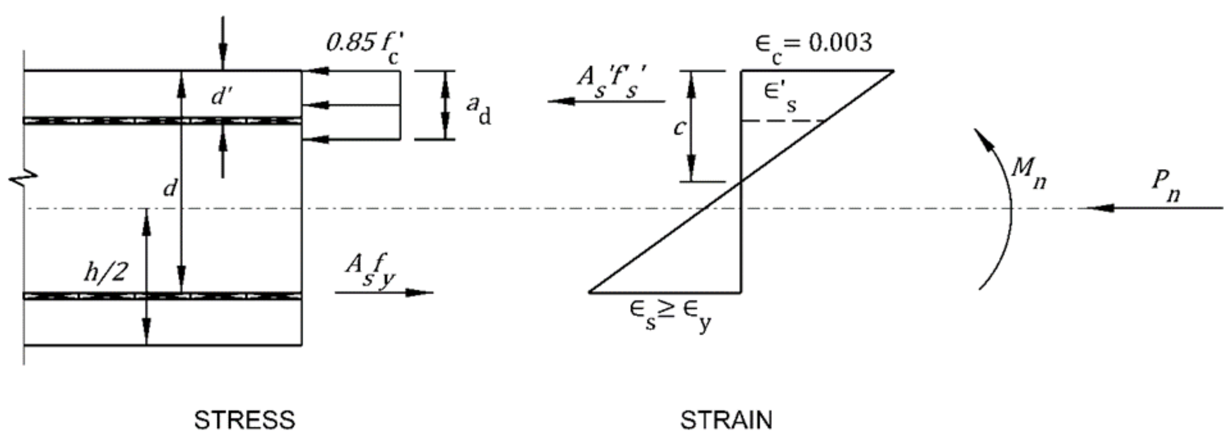


Figure D–2. Section of stress and strain for doubly reinforced member

By requiring the $\sum H$ to equal zero yields:

$$A_s = \frac{0.85f_c'K_d bd - P_n + A_s'f_s'}{f_y} \tag{D–3}$$

By setting $a_d = \beta_1 c$ and using the similar triangles from the strain diagram above, ϵ_s' and f_s' may be found:

$$\epsilon_s' = \frac{\epsilon_y(c-d')}{d-c} = \frac{\epsilon_y\left(\frac{a_d}{\beta_1} - d'\right)}{d - \frac{a_d}{\beta_1}}$$

$$f_s' = \frac{(a_d - \beta_1 d')\epsilon_c E_s}{a_d}$$

An expression for the moment carried by the concrete (M_{DS}) may be found by summing moments about the tensile steel of the concrete contribution.

$$M_{DS} = 0.85 f_c' a_d b\left(d - \frac{a_d}{2}\right) - \left(d - \frac{H}{2}\right)P_n$$

Finally, an expression for $A_s^{'}$ may be found by requiring the compression steel to carry any moment above that which the concrete can carry ($M_n - M_{DS}$).

$$A_s' = \frac{M_n - M_{DS}}{f_s'(d-d')} \quad \text{(D–4)}$$

c. Derivation of the expression, d_d. The term d_d is the minimum effective depth a member may have and meet the limiting requirements on steel ratio. The expression for d_d is found by substituting $a_d = k_d d_d$ in the equation shown above for M_{DS} and solving the resulting quadratic expression for d_d.

$$d_d = \sqrt{\frac{M_n}{0.85 f_c' k_d b\left(1 - \frac{k_d}{2}\right)}} \quad \text{(D–5)}$$

d. Design procedure. For convenience, a summary of the steps used in the design examples in this appendix is provided below. This procedure may be used to design flexural members subjected to pure flexure or flexure combined with axial load. The axial load may be tension or compression.

(1) *Step 1*. Compute the required nominal strength, M_n and P_n, where M_u and P_u are determined according to paragraph 4–1:

$$M_n = \frac{M_u}{\phi}$$

$$P_n = \frac{P_u}{\phi}$$

Note. Step 2 below provides a convenient and quick check to ensure that members are sized properly to meet steel ratio limits. The expressions in Step 2a are adequate for flexure and small axial load. For members with significant axial loads, the somewhat lengthier procedures of Step 2b should be used.

(2) *Step 2*.

(a) (Step 2a). Compute d_d from Table D–1. If $d \geq d_d$, the member is of adequate depth to meet steel ratio requirements and A_s is determined using Step 3.

(b) (Step 2b). When significant axial load is present, the expressions for d_d become cumbersome and it is easier to check the member size by determining M_{DS}. M_{DS} is the maximum bending moment a member may carry and remain within the specified steel ratio limits.

$$M_{DS} = 0.85 f_c' a_d b \left(d - \frac{a_d}{2}\right) - \left(d - \frac{h}{2}\right) P_n \qquad \text{(D–6)}$$

where:

$$a_d = k_d d \qquad \text{(D–7)}$$

and k_d is found from Table D–1.

(3) *Step 3*. Singly reinforced; when $d \geq d_d$ (or $M_n \leq M_{DS}$), the following equations are used to compute A_s:

$$K_u = 1 - \sqrt{1 - \frac{M_n + P_n\left(d - \frac{h}{2}\right)}{0.425 f_c' b d^2}} \qquad \text{(D–8)}$$

$$A_s = \frac{0.85 f_c' K_u b d - P_n}{f_y} \qquad \text{(D–9)}$$

Table D–1
Minimum effective depth

f'_c (psi)	f_y (psi)	ρ^*/ρ_b	$K_d = \frac{\left(\frac{\rho}{\rho_b}\right)\beta_1\epsilon_c}{\epsilon_c + \frac{f_y}{E_s}}$¶	d_d (in.)
3,000	60,000	0.25	0.125765	$\sqrt{\frac{3.3274M_n^{**}}{b}}$
3,000	60,000	0.5	0.251531	$\sqrt{\frac{1.7834M_n^{**}}{b}}$
4,000	60,000	0.25	0.125765	$\sqrt{\frac{2.4956M_n^{**}}{b}}$
4,000	60,000	0.5	0.251531	$\sqrt{\frac{1.3375M_n^{**}}{b}}$
5,000	60,000	0.25	0.118367	$\sqrt{\frac{2.1129M_n^{**}}{b}}$
5,000	60,000	0.5	0.236735	$\sqrt{\frac{1.1274M_n^{**}}{b}}$

Notes:
* See paragraph 3–6, “Reinforcement limits.” The traditional limit of 0.25 and upper limit of 0.5 are shown.
** M_n units are inch-kips.

where:

$$K_d = \frac{\left(\frac{\rho}{\rho_b}\right)\beta_1\epsilon_c}{\epsilon_c + \frac{f_y}{E_s}}$$

$$d_d = \sqrt{\frac{M_n}{0.85 f_c' k_d b\left(1 - \frac{k_d}{2}\right)}}$$

D–3. Singly reinforced example

Figure D–3 shows a singly reinforced retaining wall demonstrating the design procedure outlined in paragraph D–2 for a singly reinforced beam with a maximum design steel ratio of 0.25 ρ_b. This limit is used to provide adequate section thickness to control deflection. The required area of steel is computed to carry the moment at the base of a retaining wall stem.

a. Given: $M = 5\ k - ft$ (unfactored moment, usual loads)

$f_c' = 4.0\ ksi$

$f_y = 60\ ksi$

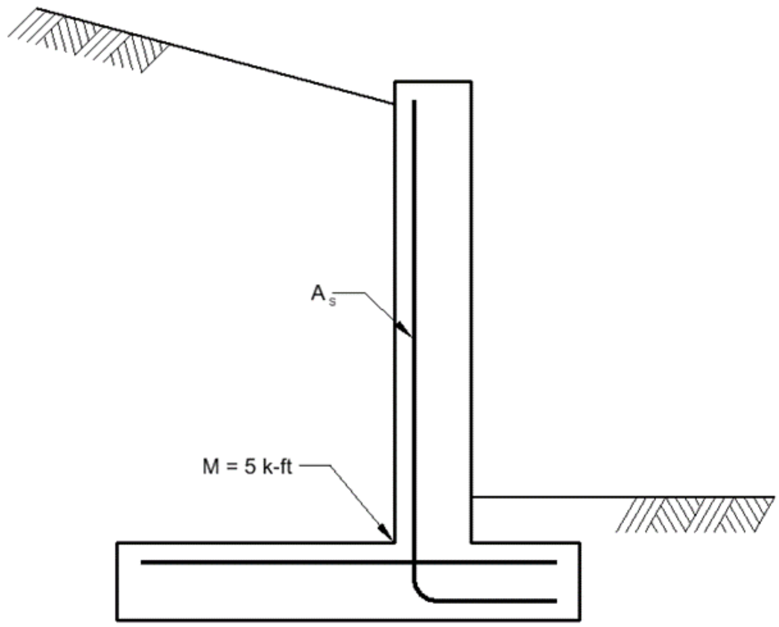


Figure D–3. Retaining wall with moment at the base of stem

(1) *Step 1*. Compute nominal moments using alternate serviceability design (single load factor).

$$M_u = 2.2(Lp + Lt + Ld) = 2.2M$$

$$M_u = (2.2)(5) = 11\ k\text{–}ft$$

$$M_n = \frac{M_u}{\phi} = \frac{11(12)}{0.9} = 147\ k\text{–}in.$$

(2) *Step 2*. Determine minimum depth to reinforcement, *d*.

From Table D–1:

$$d_d = \sqrt{\frac{2.4956M_n}{b}} = \sqrt{\frac{2.4956(147)}{12}} = 5.53\ in.$$

Use d = 6 in. so that $d > d_d$.

(3) *Step 3*. Calculate K_u and A_s from equations D–8 and D–9:

$$K_u = 1 - \sqrt{1 - \frac{M_n + P_n\left(d - \frac{h}{2}\right)}{0.425 f_c' b d^2}} = 1 - \sqrt{1 - \frac{(147)}{(0.425)(4.0)(12)(6)^2}} = 0.105$$

$$A_s = \frac{0.85 f_c' \; K_u b d}{f_y} = \frac{(0.85)(4.0)(0.105)(12)(6)}{60} = 0.43\ sq.in.$$

Use No. 6 bars at 12-in. on center (o.c.) spacing, $A_s = 0.44\ sq.in.$

(4) *Step 4*. Check reinforcement ratio:

$$0.25\rho_b = 0.25\left(0.85\beta_1 \frac{f_c'}{f_y}\left(\frac{87000}{87000 + f_y}\right)\right) = 0.25(0.85)(0.85)\frac{4}{60}(0.5918) = 0.0071$$

$$\rho = \frac{A_s}{bd} = \frac{0.44}{12(6)} = 0.0061$$

$\rho < 0.25\rho_b, therefore\ OK$

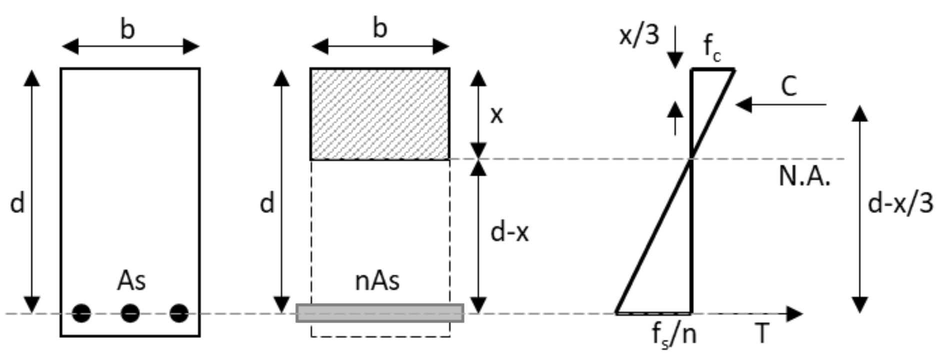
Determine wall thickness, h:

$$h = d + \frac{d_b}{2} + cover = 6 + \frac{0.75}{2} + 2 = 8.375\ in.$$

Use *h* = 9 in.

(5) *Step 5*. Check reinforcement stress and reinforcement spacing.

(a) Calculate f_s for *M* = 5 *kip-ft*



$$n = \frac{E_s}{E_c} = \frac{29000}{3605} = 8.04$$

$$bx\left(\frac{x}{2}\right) = nA_s(d - x)$$

$$x = \frac{-nA_s \pm \sqrt{(nA_s)^2 - 4(\frac{b}{2})(-nA_s d)}}{2(\frac{b}{2})} = \frac{-3.54 \pm \sqrt{(3.54)^2 - 4(6)(-21.24)}}{2(6)} = 1.61\ in.$$

$$I_{NA} = \frac{bx^3}{3} + nA_s(d-x)^2 = \frac{(12)(1.61)^3}{3} + (8.04)(0.44)(6-1.61)^2 = 84.9 in.^4$$

$$\frac{f_s}{n} = \frac{M(d-x)}{I_{NA}} = \frac{5(12)(6-1.61)}{84.9} = 3.10\ ksi,\ f_s = (8.04)(3.10) = 24.96\ ksi$$

$$f_s < 25 \text{ ksi (Table 3–3)}, therefore, OK$$

(b) Calculate the maximum reinforcement spacing, s_{max}, according to ACI 318-19, section 24.3.2. Use c_c = 2″ since actual cover is less, according to paragraph 2–6b(1) of this manual.

$$\mathrm{S}_{max} = 15\left(\frac{40{,}000}{f_s}\right) - 2.5c_c = 15\left(\frac{40{,}000}{24{,}959}\right) - 2.5(2) = 19\ in.$$

$$s < \mathrm{s}_{max}, therefore\ OK$$

b. At this point, go through and re-compute the design capacities based on these final determinations, then proceed with the remaining checks (shear, etc.). Since a maximum reinforcement ratio of 0.25 ρ_b was used, assumed that the section thickness will be sufficient to control deflection and a detailed deflection check is not required for this retaining wall.

D–4. Combined flexure plus axial load example

The wall shown in Figure D–4 is an existing wall designed for flexure only. Due to changing site conditions, it now needs to support an axial load of 25 kips/ft length, which includes self-weight. Determine if the wall section, as is, can handle the additional axial load.

a. Given:

M = 95 k-ft (unfactored moment)
P = 25 kips (unfactored stem weight plus axial load)
f_c' = 4.0 ksi
f_y = 60 ksi
cover = 3 in.
d = 16.295 in.
d′ = 3.705 in.
A_s = 3.40 sq. in. ($\rho = A_s/bd = 0.0174$), No. 11 bars at 5.5″ o.c.
A_s' = 1.87 sq. in. ($\rho' = A'_s/bd = 0.0096$), No. 11 bars at 10″ o.c.
e = 3.5 in. (center of wall section to axial load)

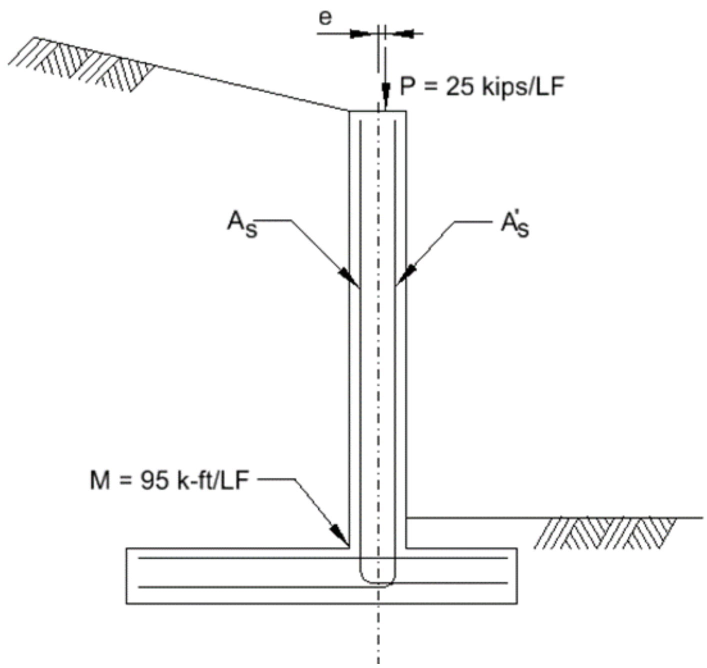


Figure D–4. Retaining wall with moment at the base of stem plus axial load

(1) *Step 1*. Compute nominal moments using Alternate Serviceability Design (single load factor), M_u, P_u:

$$M_u = 2.2(Lp + Lt + Ld) = 2.2M$$

$$M_u = (2.2)(95) = 209\ k - ft$$

$$P_u = 2.2(Lp + Lt + Ld) = 2.2\ P$$

$$P_u = 2.2(25) = 55\ kips$$

$$M_{u1} = M_u + P_u e(2.2)(95) = 209\ k - ft$$

$$M_{u1} = (209) + (55)(3.5) = 225\ k - ft$$

(2) *Step 2*. Determine balanced eccentricity ratio to determine if the section is controlled by strength in tension or strength in compression. Considering the reinforcement in both faces, the eccentricity can be calculated using equation 4–1 and the balanced eccentricity can be calculated based on equation B–27. The balanced condition exists when the tension reinforcement reaches the strain, ε_y, corresponding to its specified yield strength as the concrete in compression reaches its design strain, ε_c.

$$\epsilon_c = 0.003$$

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29{,}000} = 0.0021$$

$$K_b = \frac{\beta_1 E_s \epsilon_c}{E_s \epsilon_c + f_y} = \frac{0.85(29000)(0.003)}{29000(0.003) + 60} = 0.503$$

$$f_s' = \frac{(k_b - \beta_1 \frac{d'}{d})}{\beta_1 - k_b} E_s \varepsilon_y = \frac{\left(0.503 - 0.85\frac{3.705}{16.295}\right)}{0.85 - 0.503}(29000)(0.0021) = 53.6ksi$$

$$e' = \frac{M_u}{P_u} + d - \frac{h}{2} = 51.9in$$

$$e_b' = \frac{0.425f_c'(2K_b - K_b^2)bd^2 + f_s'\rho'bd(d - d')}{0.85f_c'K_bbd + f_s'\rho'bd - f_s\rho bd}$$
$$= \frac{0.425(4)[2(0.503) - 0.503^2](12)(16.295)^2 + 53.6(0.0096)(12)(16.295)(16.295 - 3.705)}{0.85(4)(0.503)(12)(16.295) + (53.6)(0.0096)(12)(16.295) - 0.0174(60)(12)(16.295)}$$
$$= \frac{4079 + 1261}{334.5 + 100.2 - 204} = 23.2\ in$$

$Since\ e' > e_b', section\ is\ controlled\ by\ strength\ in\ tension$

(3) *Step 3*. Calculate axial and moment capacity of tension controlled section using equations in paragraph B–3e. Use ϕ = 0.9.

(a) Using equation B–33:

$$k_u^3 + \left[2\left(\frac{e'}{d} - 1\right) - \beta_1\right]k_u^2 - \left\{\frac{f_y}{0.425f_c'}\left[\rho'\left(\frac{e'}{d} + \frac{d'}{d} - 1\right) + \frac{\rho e'}{d}\right] + 2\beta_1\left(\frac{e'}{d} - 1\right)\right\}k_u + \frac{f_y\beta_1}{0.425f_c'}\left[\rho'\frac{d'}{d}\left(\frac{e'}{d} + \frac{d'}{d} - 1\right) + \frac{\rho e'}{d}\right] = 0$$

$$k_u^3 + \left[2\left(\frac{51.9}{16.295} - 1\right) - 0.85\right]k_u^2 - \left\{\frac{60}{0.425(4)}\left[(0.0096)\left(\frac{51.9}{16.295} + \frac{3.705}{16.295} - 1\right) + \frac{(0.0174)(51.9)}{16.295}\right] + 2(0.85)\left(\frac{51.9}{16.295} - 1\right)\right\}k_u + \frac{(60)(0.85)}{0.425(4)}\left[(0.0096)\frac{3.705}{16.295}\left(\frac{51.9}{16.295} + \frac{3.705}{16.259} - 1\right) + \frac{(0.0174)(51.9)}{16.295}\right] = 0$$

$$K_u^3 + 3.519K_u^2 - 6.483K_u + 1.819 = 0$$

(b) Solving the cubic equation, $K_u = 0.357$.

$$f_s' = \frac{(k_u - \beta_1 \frac{d'}{d})}{\beta_1 - k_u} E_s \varepsilon_y = \frac{\left(0.357 - 0.85\frac{3.705}{16.295}\right)}{0.85 - 0.357}(29000)(0.0021) = 19.85\ ksi$$

$$\phi P_n = \phi\left(0.85f_c'K_u + \rho'f_s' - \rho f_y\right)bd$$
$$= 0.9[(0.85(4)(0.503) + 0.0096(19.85) - 0.0174(60)](12)(16.295)$$
$$= (1.21 + 0.191 - 1.044)195.5 = 63\ kips$$

b. Please note that regardless of whether tension or compression controls, the design axial load strength ϕP_n for members with tie reinforcement is limited by ACI 318-19 and should not be greater than:

$$\phi P_{n(max)} = 0.8\phi\left[.85f'_c\left(A_g - (\rho + \rho')\,bd\right) + f_y(\rho + \rho')\,bd\right]$$
$$= 0.8\phi[.85(4)(240 - (0.0174 + 0.0096)\,195.5)$$
$$+ 60(0.0174 + 0.0096)\,195.5] = 802kips < 63kips$$

$$\phi P_n > P_u, therefore\ OK$$

$$\phi M_n = \phi\left(0.85 f_c' K_u + \rho' f_s' - \rho f_y\right)\left[\frac{e'}{d} - \left(1 - \frac{h}{2d}\right)\right] bd^2$$
$$= [(0.85(4)(0.503) + 0.0096(19.85) - 0.0174(60)]\left[\frac{51.9}{16.295}\right.$$
$$\left. - \left(1 - \frac{20}{2(16.295)}\right)\right](12)16.295^2 = (1.21 + 0.191 - 1.044)8917$$
$$= 2880\ k - in.$$

$$\phi M_n > M_{u1}, therefore\ OK$$

D–5. Design example of coastal floodwall

The following design example (see Table D–2 and Figure D–5) shows the expanded use of the general guidance found in this manual.

a. Task: Design authorization has been approved for the design of a coastal floodwall near a navigational canal for a 100-year storm event (90 percent confidence with sea level change). This floodwall is considered a critical structure because failure would result in probable loss of life. Landside flooding will occur before water reaches the top of the floodwall due to overtopping at other locations within the polder. It was determined that this would occur for the event with a return period of 500 years (50 percent confidence with sea level change).

b. Determination: Based on geotechnical investigations, it was determined that the coastal floodwall is pile-founded. The coastal floodwall will be reinforced concrete-founded on steel piles with a seepage cut-off wall. The floodwall will be protected by dolphin structures, so only a debris loading is considered. The pile foundation has been designed and the global pile reactions provided. Determine the wall stem and base slab thicknesses and reinforcement details.

Table D–2
Design example of coastal floodwall

Given Wall Geometrics			
Top of Wall:	16.0 ft	Top of Base:	-12.33 ft
Base Width*:			29 ft
Distance to C/L of Stem from F.S. Toe*:			17 ft
Distance to Sheet Pile Cut-off from F.S. Toe*:			11.5 ft
Given Hydraulic Data			
100-year	11.0 ft	500-year:	14.8 ft
100-year Wave Load:	5.32 k/ft	500-year Wave Load:	6.16 k/ft
100-year Wave Resultant Elevation (EL):	-0.32 ft	500-year. Wave Resultant EL:	4.63 ft
Normal Water EL:	1.0 ft	Landside Low Water EL:	-1.9 ft
Normal Wave Load:	1.5 k/ft	γ_w:	64.2 pcf
Normal Wave Resultant EL:	-6 ft	–	–
Seismic Hazard:	Low	–	–
Given Geotechnical Data			
F.S. Soil EL:	-10.0 ft	K_o:	0.524
P.S. Soil EL:	-10.0 ft	γ_s:	0.115 kcf
Other Given Design Data			
Debris Impact:	0.5 k/ft	Construction Surcharge:	0.20 ksf
Aberrant Barge Impact:	N/A**	f'_c:	5 ksi

Notes:
[*] Base slab pile cap widths and sheet pile cut-off location are based on pile foundation design not shown.
[**] Floodwall is protected from barge impact by dolphins.

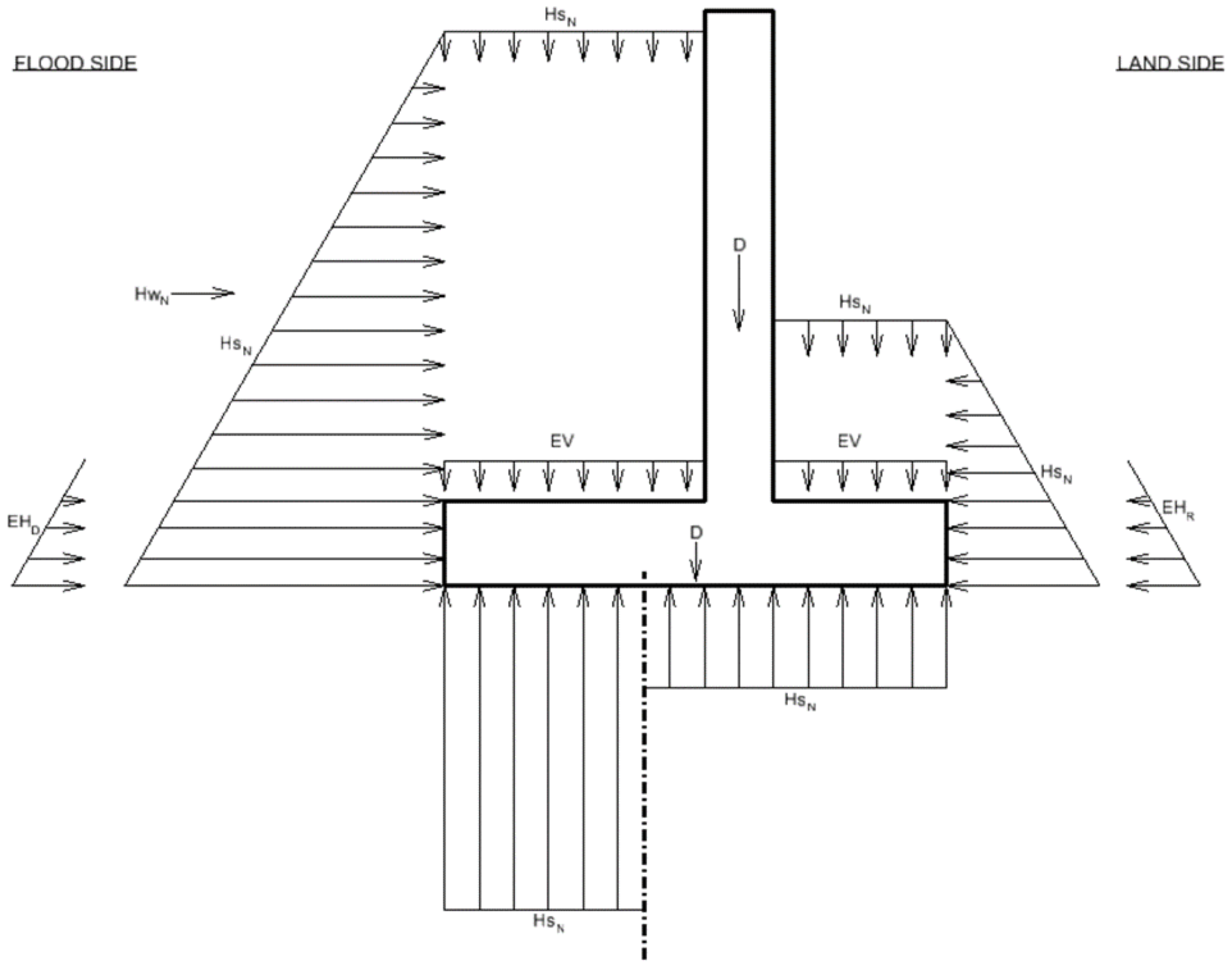


Figure D–5. Coastal floodwall with Load Case 1.1 loads (Table D–3a)

(1) *Step 1*. Determine loads and load combinations according to paragraph E–5. Table D–3a shows the load combinations determined to be applicable for this example. Some of the load combinations were designed for the maximum possible loading before the landside is flooded (paragraph 3–3e). Since this would occur for an event with a return period of 500 years, it falls under the unusual load category for a critical structure (paragraph 3–2d(2)). For earthquake load cases, the wall is in a low seismic hazard region and water is on the wall for a short period of time on a very infrequent basis. Therefore, earthquake loads will not control design and are not considered. Table D–3b shows the design water levels for the load cases.

Table D–3a
Loads and load combinations according to paragraph E–5

Load Case	Load Description	Category	Principal Load	Load Combination	Load Combination	Load Combination
				Serviceability (Service Stress)	Serviceability (Alternate Design)	Strength
1.1	Maximum Surge + Wave + Impervious Uplift	Maximum (Unusual)	Hs + Hw Hs + Hw	1.0 ($D + EH_D – EH_R + EV + Hs_N + Hw_N$)	1.6 ($D + EH_D – EH_R + EV + Hs_N + Hw_N$)	$1.2\ D + 1.35\ EH_D – 0.9\ EH_R + 1.35\ EV + 1.4\ (Hs_N + Hw_N)$ $0.9\ D + 1.35\ EH_D – 0.9\ EH_R + 1.00\ EV + 1.4\ (Hs_N + Hw_N)$
1.2	Maximum Surge + Wave + Pervious Uplift	Maximum (Unusual)	Hs + Hw Hs + Hw	1.0 ($D + EH_D – EH_R + EV + Hs_N + Hw_N$)	1.6 ($D + EH_D – EH_R + EV + Hs_N + Hw_N$)	$1.2\ D + 1.35\ EH_D – 0.9\ EH_R + 1.35\ EV + 1.4\ (Hs_N + Hw_N)$ $0.9\ D + 1.35\ EH_D – 0.9\ EH_R + 1.00\ EV + 1.4\ (Hs_N + Hw_N)$
2.1	Normal Operating + Wave + Impervious Uplift	Usual	Hs + Hw	1.0 ($D + EH_D – EH_R + EV + Hs_N + Hw_N$)	2.2 ($D + EH_D – EH_R + EV + Hs_N + Hw_N$)	N/A
2.2	Normal Operating + Wave + Pervious Uplift	Usual	Hs + Hw	1.0 ($D + EH_D – EH_R + EV + Hs_N + Hw_N$)	2.2 ($D + EH_D – EH_R + EV + Hs_N + Hw_N$)	N/A
3.1	Maximum surge + Debris Impact + Impervious Uplift	Maximum (Unusual)	Hs Hs	1.0 ($D + EH_D – EH_R + EV + Hs_N + IM$)	1.6 ($D + EH_D – EH_R + EV + Hs_N + IM$)	$1.2\ D + 1.35\ EH_D – 0.9\ EH_R + 1.35\ EV + 1.4\ Hs_N + 1.0\ IM$ $0.9\ D + 1.35\ EH_D – 0.9\ EH_R + 1.00\ EV + 1.4\ Hs_N + 1.0\ IM$

Load Case	Load Description	Category	Principal Load	Load Combination	Load Combination	Load Combination
3.2	Maximum surge + Debris Impact + Pervious Uplift	Maximum (Unusual)	Hs Hs	1.0 (D + EH_D – EH_R + EV + Hs_N + IM)	1.6 (D + EH_D – EH_R + EV + Hs_N + IM)	1.2 D + 1.35 EH_D – 0.9 EH_R + 1.35 EV + 1.4 Hs_N + 1.0 IM 0.9 D + 1.35 EH_D – 0.9 EH_R + 1.00 EV + 1.4 Hs_N + 1.0 IM
4A	OBE	Unusual	–	(not considered)	(not considered)	(not considered)
4B	MCE	Extreme	–	(not considered)	(not considered)	(not considered)
5	Construction	Unusual	Es Es	1.0 (D + EH_D – EH_R + EV + Hs_N + ES_N + W)	1.6 (D + EH_D – EH_R + EV + Hs_N + ES_N + W)	1.2 D + 1.35 EH_D – 0.9 EH_R + 1.35 EV + 1.6 ES_N + 0.5 W 0.9 D + 1.35 EH_D – 0.9 EH_R + 1.0 EV + 1.6 ES_N + 0.5W

Table D–3b
Design water levels

	Floodside	Landside	Return Period
Normal Operating	EL. 1	EL. -1.9	10-year. (Usual)
Maximum Surge	El. 14.8	EL. -1.9	Between 10-year. and 750-year. (Unusual)

(2) *Step 2*. Determine M_u and V_u for wall stem. Weight of stem is neglected. The governing load case was found to be Load Case 1. The resulting factored loads, per foot length of wall, are shown in Table D–4.

Table D–4
Factored loads for the predetermined governing Load Case 1

Load Type	Load Factor	Horiz. Load	Moment Arm	Moment
F.S. Hs_N:	1.4	33.08 kips/ft	9.04 ft	299 k-ft/ft
L.S. Hs_N:	1.4	-4.89 kips/ft	3.48 ft	-17 k-ft/ft
F.S. Lateral Earth, EH_A:	1.35	0.10 kips/ft	0.78 ft	0.08 k-ft/ft
L.S. Lateral Earth, EH_P:	0.9	-0.07 kips/ft	0.78 ft	-0.05 k-ft/ft
Wave Load, Hw_N:	1.4	8.62 kips/ft	16.96 ft	146 k-ft/ft
Resulting Loads at Base of Stem:	–	36.85 kips/ft	–	428 k-ft/ft

V_u = 36.9 kips/ft, M_u = 428 k-ft/ft

(3) *Step 3*. Determine wall stem thickness and reinforcement.

(a) Assume a trial thickness, *h*. Try $\ell\ /\ 8$, which is minimum for non-prestressed cantilever beams and one-way slabs according to ACI-318.

$$h = \frac{\left((28.33ft)(12in/ft)\right)}{8} = 42\ in.$$

(b) Design data:

f_c' = 5,000 psi
f_y = 60,000 psi
b_w = 12 in. (design width)
For shear; ϕ = 0.75 (ACI 318-19)
For bending; ϕ = 0.90 (ACI 318-19)
λ = 1.0 (concrete weight factor for normal weight concrete)

(c) Validate if the wall thickness is adequate for shear at base of stem. It must satisfy $\phi V_c \geq V_u$.

(d) Compute *d*: *d* = *h* – cover – 0.5 (bar diameter)

Assume maximum bar diameter #18 bars, d_b = 2.257 in.

d = 42 in. – 4 in. – 0.5(2.257 in.) = 36.9 in.

(e) Use: *d* = 36.5 in.

(f) Compute d_d:

$$M_n = \frac{M_u}{\phi} = \frac{428}{0.9} = 476\ k\text{–}ft/ft$$

(g) From Table D–1 with target reinforcing ratio of 0.25:

$$d_d = \sqrt{\frac{2.4956M_n}{b_w}} = \sqrt{\frac{2.4956(476)(\ 12)}{12}} = 34.47\ in.$$

$D > d_d$, therefore, stem width is adequate.

(h) Determine if $\phi V_c \geq V_u$.

$V_c = 2\sqrt{f_c'}b_w d$ (equation 5–1 without axial terms)

$$V_c = \frac{2\sqrt{5000}(12)(36.5)}{1000} = 61.9\ kips/ft$$

$$\phi V_c = 0.75(61.9) = 46.5\ kips/ft$$

46.5 kips ≥ 36.9 kips, thus $\Phi V_c \geq V_u$

(i) Compute required A_s:

$$K_u = 1 - \sqrt{1 - \frac{M_n + P_n\left(d - \frac{h}{2}\right)}{0.425f_c'b_w d^2}}$$

$$K_u = 1 - \sqrt{1 - \frac{476(12)}{0.425(5)(12)(36.5^2)}} = 0.088$$

$$A_s = \frac{0.85f_c'K_u b_w d}{f_y} = \frac{0.85(5)(0.111)(12)(36.5)}{60} = 2.73\ sq.in./ft$$

(j) Try No. 11 bars, A_{11} = 1.56 sq. in.

$$s = \frac{A_{11}}{A_s}b_w = \frac{1.56}{2.73}(12) = 6.86\ in.$$

(k) Use No. 11 bars at 6 in. center to center, tension face. (Compression face reinforcement computations not shown.)

(4) *Step 4*. Check reinforcement ratio with target ratio = 0.25 ρ_b:

$$0.25\rho_b = 0.25\left(0.85\beta_1 \frac{f_c'}{f_y}\left(\frac{87000}{87000 + 60000}\right)\right)$$

$$= 0.25(0.85)(0.80)\frac{5}{60}(0.5918) = 0.0083$$

$$\rho = \frac{A_s}{bd} = \frac{3.12}{12(36.5)} = 0.0071$$

$$\rho < 0.25\rho_b, therefore\ meets\ target$$

Since the reinforcement ratio is less than 0.25 ρ_b, deflection is not checked for this floodwall.

(5) *Step 5*. Check reinforcement stress and reinforcement spacing.

(a) Calculate service shear stress, f_s, for governing load case, M_n = 306 kip-ft (linear feet (LF) = 1.0)

$$n = \frac{E_s}{E_c} = \frac{29000}{4031} = 7.2$$

$$x = \frac{-nA_s \pm \sqrt{(nA_s)^2 - 4(\frac{b_w}{2})(-nA_s d)}}{2(\frac{b_w}{2})} = \frac{-22.45 \pm \sqrt{(22.45)^2 - 4(6)(-837.2)}}{2(6)}$$

$$= 9.96\ in.$$

$$f_s = \frac{M}{A_s(d - \frac{x}{3})} = \frac{306(12)}{3.12(36.5 - \frac{10.09}{3})} = 34.7\ ks$$

$$f_s < 35 \text{ ksi (Table 3– 3)}, therefore\ OK$$

(b) Check reinforcement spacing according to paragraph 2–6b. Use c_c = 2.5 in. since actual cover is greater than 2.5 in. From ACI 318-19:

$$s_{max} = 15\left(\frac{40{,}000}{f_s}\right) - 2.5c_c = 15\left(\frac{40{,}000}{34{,}700}\right) - 2.5(2.5) = 11 \text{ in.}$$

$$6\ in. < 11\ in., therefore\ OK$$

(6) *Step 6*. Determine embedment depth into base slab.

(a) The tension reinforcement in the wall must be fully developed in the base slab. The base slab should be sized so that this development length can be achieved.

(b) Per ACI 318-19: It may not be economically feasible to develop the vertical tension steel (no bends) within the required base slab depth. Thus, it is recommended to hook the bars, 90 or 180 degrees. The correct direction of the hook is shown in

Figure 2–1. From ACI 318-19, the development length (l_{dh}) of a standard hook is calculated by:

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r \psi_o}{55\lambda\sqrt{f_c'}}\right) d_b^{1.5}$$

ψ_e = epoxy coating factor = 1.0 for uncoated bars
ψ_c = concrete strength = f_c'/15,000 + 0.6 for f_c' < 6,000 psi
ψ_r = confining reinforcement factor = 1.0 when no confining reinforcement is provided.
ψ_o = location factor = 1.0 for no. 11 and smaller bars with a side cover ≥ 6db.

(c) Other terms were previously provided, so:

$$l_{dh} = \frac{(60{,}000)(1.0)(0.93)(1.0)(1.0)}{55(1.0)\sqrt{5000}}(1.41)^{1.5} = 24.1\ in.$$

(d) Use a minimum embedment depth from the base of the stem to a distance 24.5 in. below. Typically, the hooks will rest on the bottom mat and be tied to the top mat of reinforcement in the base slab. Thus, when designing the base slab, it is important to match the reinforcement spacing of the vertical bars.

(7) *Step 7*. Determine temperature and shrinkage reinforcement for stem.

(a) Per paragraph 2–9 for monolith greater than 40 feet in length:

$$A_{T\&S} = \frac{0.005A_g}{2} \leq\ 1.00\ sq.in./ft,.ea.face$$

$$A_{T\&S} = \frac{0.005(42)(12)}{2} =\ 1.26\ sq.in./ft$$

(b) Since this is greater than #9 bars at 12 in. (1.0 sq. in./ft), by paragraph 2–9b use #9 at 12 in. spacing each face and the concrete mix design must account for possible thermal effects. Per paragraph 2–9f, a contraction joint can be provided in the center of the monolith, thereby reducing the reinforcement ration to 0.003. This will result in #8 bars for temperature and shrinkage. The cost difference of the bar size reduction and contraction joint construction versus larger bar size with no contraction joint will need to be evaluated.

(8) *Step 8*. Determine base slab thickness and reinforcement. The base slab reinforcement can be designed using the same procedures as the stem reinforcement. For guidance on the application of pile reactions to the base slab and the calculation of shear and moment in the base slab, refer to EM 1110-2-2502.

D–6. Shear strength example for special straight members

Paragraph 5–2 describes the conditions for which a special shear strength criterion applies for straight members. The following example demonstrates the application of equation 5–1. Figure D–6 shows a rectangular conduit with factored loads. The following parameters are given or computed for the roof slab of the conduit.

$f_c' = 4{,}000\ psi$

$l_n = 10.0\ ft = 120\ in.$

$d = 2.0\ ft = 24.0\ in.$

$b = 1.0\ ft\ (unit\ width) = 12\ in.$

$N_u = 6.33\ (5) = 31.7\ kips$

$A_g = 2.33\ sq.ft = 336\ sq.in.$

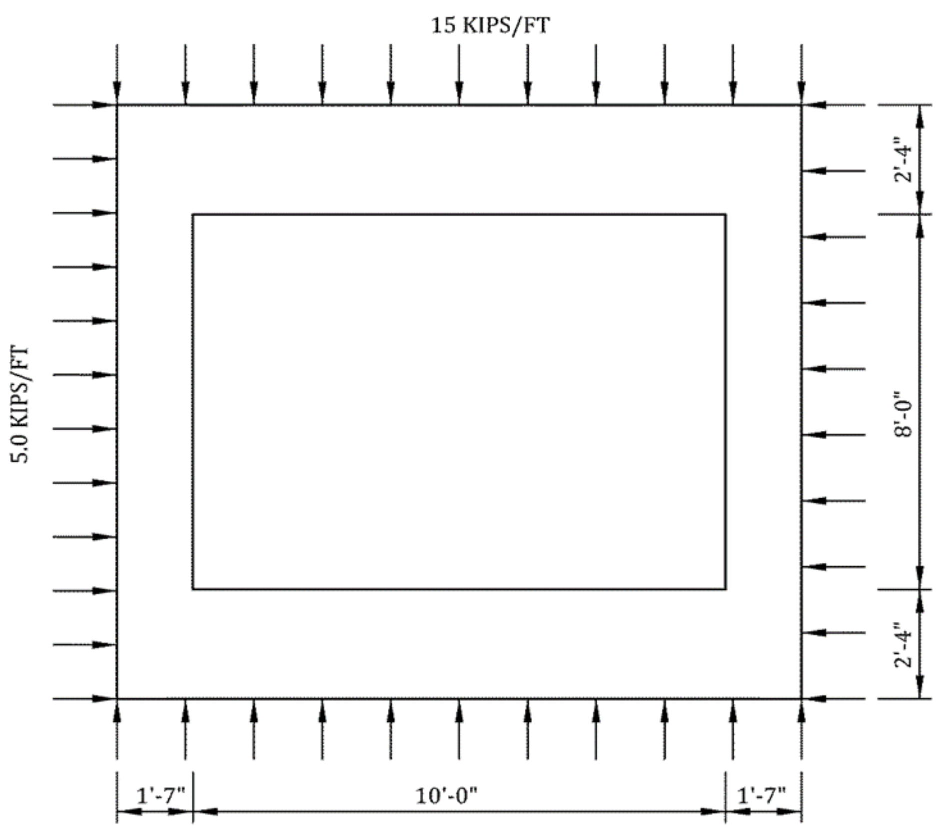


Figure D–6. Rectangular conduit

Calculate V_c using equation 5–2 from Chapter 5:

$$V_c = \left[\left(11.5 - \frac{120\ in.}{24\ in.}\right)\sqrt{4{,}000}\sqrt{1 + \left(\frac{\frac{31{,}700 lb}{336\ in^2}}{5\sqrt{4{,}000}}\right)}\right](12\ in.)(24\ in.)$$

$$V_c = 134{,}906\ lb = 134.9\ kips$$

a. Check limit: $V_c = 10\sqrt{f_c'}bd = 10\sqrt{4{,}000}(12\ in.)(24\ in.) = 182{,}147\ lb$

b. Compare shear strength with applied shear:

$$\phi V_c = 0.75(134.9\ kips) = 101.2\ kips$$

c. V_u at 0.15(l_n) from face of the support is:

$$V_u = w\left(\frac{l_n}{2} - 0.15 l_n\right)$$

$V_u = 15\frac{kips}{ft}\left[\left(\frac{10ft}{2}\right) - 0.15(10ft)\right] = 52.5\ kips \leq \phi V_c$; shear strength is adequate

D–7. Shear strength example for curved members

Paragraph 5–5 described the conditions for which equation 5–4 applies. The following example applies equation 5–4 to the circular conduit presented in Figure D–7. Factored loads are shown, and the following values are given or computed:

$f_c' = 4{,}000\ psi$

$b = 12\ in.$

$d = 43.5\ in.$

$A_g = 576\ sq.in$

$N_u = 162.5\ kips$

a. V_u = 81.3 *kips* at a section 45 degrees from crown.

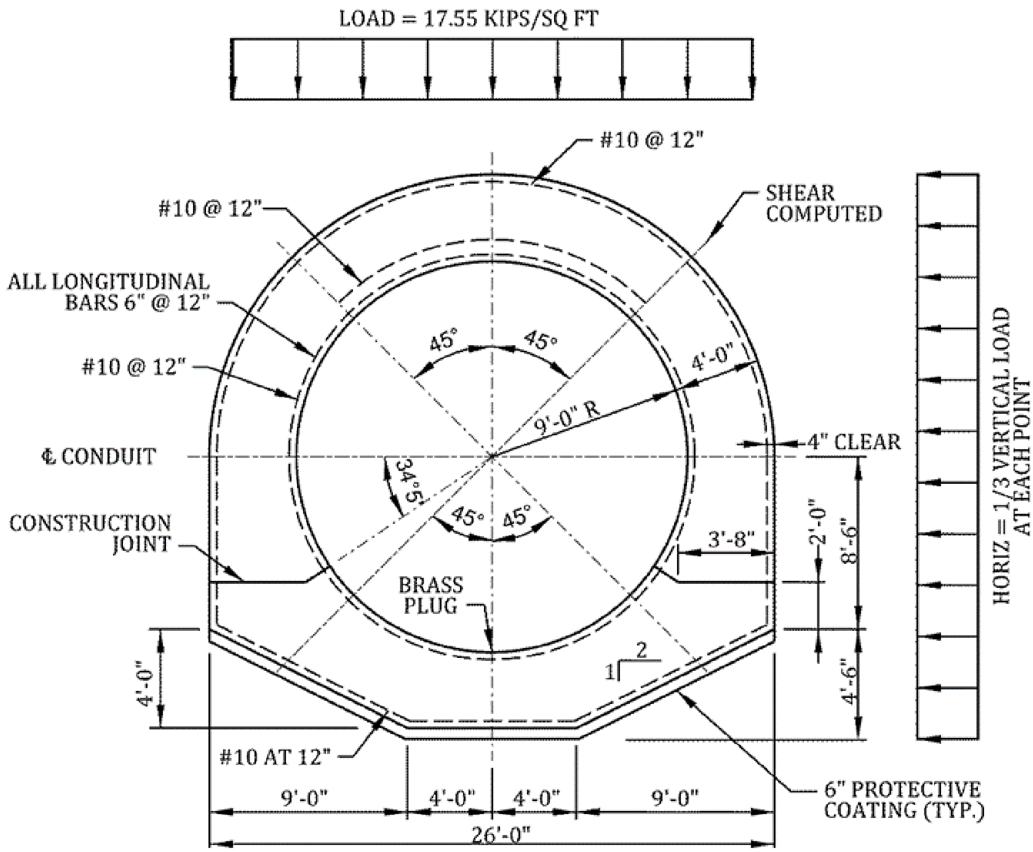


Figure D–7. Circular conduit

b. Calculate V_c using equation 5–4 from Chapter 5:

$$V_c = 4\sqrt{4{,}000}\left[\sqrt{1+\left(\frac{\frac{162{,}500\ lb}{576\ in.^2}}{4\sqrt{4{,}000}}\right)}\right](12\ in.)(43.5\ in.)$$

$$V_c = 192{,}058\ lb = 192.1\ kips$$

c. Check limit: $V_c = 10\sqrt{f_c'}bd = 10\sqrt{4{,}000}(12\ in.)(43.5\ in.) = 330{,}142\ lb$

d. Compare the strength with the applied shear:

$\phi V_c = 0.75(192.1\ kips) = 144.1\ kips$

$V_u \le \phi V_c$; shear strength is adequate

Appendix E
Load Combinations for Design of Typical Reinforced Concrete Hydraulic Structures

E–1. Purpose

a. This appendix provides load combinations for typical concrete hydraulic structures. See applicable engineer manuals for structures not included here. This includes with load descriptions, load categories, principal loads, and load factors, based on the criteria in Chapter 3. The load case combination described should be used as a guide. Other combinations may be required based on site-specific conditions.

b. Typical load categories are provided. For some load types (such as hydrostatic loading), the load category is determined by the return period of the maximum possible loading. These load combinations are noted by using the word “Maximum” for the load category.

c. Serviceability and strength load combinations are provided. Serviceability combinations may be multiplied by a load factor of 1.0 for calculations of deflection or stress. Serviceability combinations may be multiplied by a single load factor for usual and unusual cases as described in paragraph 3–4a(2). The single load factor depends on the load category.

d. For earthquake load combinations, the load factors are applied to the load effects.

E–2. Variable names

a. Table E–1 shows load types applied to walls covered in this manual and their variable names.

Table E–1
Load names

Permanent Loads, Lp	**Variable**
Dead	D
Vertical Earth	EV
Lateral Earth	EH
Gravity	G
Temporary Loads, Lt	**Variable**
Hydrostatic	Hs
Thermal Expansion of Ice	IX
Soil Surcharge	ES
Operating Equipment	Q
Live Load	L
Self-Straining	T
Vehicle Live Loads	V
Dynamic Loads, Ld	**Variable**
Hydrodynamic (except earthquake)	Hd
Wave	Hw
Debris/Floating Ice Impact	IM
Barge/Boat Impact	BI
Wind	W
Earthquake	EQ
Hawser	HA

b. Subscripts $_U$, $_N$, and $_X$ designate usual, unusual, and extreme load categories, respectively. The subscript $_{pr}$ designates a principal load, and the subscript $_c$ designates a companion load consistent with paragraph 3–3c.

E–3. Earth retaining wall (normal structure)

Load combinations for an earth retaining wall is shown in Table E–2.

Table E–2
Load combinations for an earth retaining wall

Load Case	Load Description	Load Category	Principal Load	Serviceability	Strength
1	Normal Operating	Usual	EH	(1.0 or 2.2)(D + EH + EV + Hs_U)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.0 Hs_c
2	Normal Operating + Surcharge	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N + Hs_c)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.6ES_N + 1.0 Hs_c
3A	Earthquake – OBE	Unusual	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.5EQ + 1.0 Hs_c
3B	Earthquake – MDE	Extreme	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.25EQ + 1.0 Hs_c
4	Maximum Hydrostatic Loading	Maximum	Hs	(1.0 or 2.2 or 1.6)(D + EH + EV + Hs_{pr})	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + γ_{pr} Hs_{pr}
5	Construction	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N)	1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.6ES_N

a. Load Case 1, Normal Operating.

(1) Dead load, lateral soil pressure, and weight of soil that may be present.

(2) Water levels on each side of the wall that create a differential hydrostatic loading with approximately 10-year return.

b. Load Case 2, Normal Operating + Surcharge Loads. This case is the same as Case 1 except a temporary surcharge is applied. The load factors shown for EH assume that sufficient movement occurs for a design condition using active and passive earth pressures. This depends on the expected movement of the structure.

c. Load Case 3A, Earthquake – OBE. This is the same as Case 1 except with the addition of OBE-induced lateral and vertical loads.

d. Load Case 3B, Earthquake – MDE. This is the same as Case 3A except with MDE instead of OBE. The equation in the table assumes that standard ground motions are used.

e. Load Case 4, Maximum Hydrostatic Load. This case is the same as Case 1 except the water table level in the backfill and the water on the resisting side are at levels creating the maximum possible differential loading. The load category and load factor γ_{pr} depends on the return period of the maximum water loading, Hs_{pr}.

f. *Load Case 5, Construction*. Wall is in place with the loads that are possible during the construction period.

(1) Dead load, lateral soil pressure, and weight of soil that may be present. The load factors shown for EH assume that sufficient movement occurs for a design condition using active and passive earth pressures. This depends on the expected movement of the structure.

(2) Soil surcharge.

E–4. Inland floodwall (critical structure)

Load combinations for an inland floodwall are shown in Table E–3.

Table E–3
Load combinations for an inland floodwall

Load Case	Load Description	Load Category	Principal Load	Serviceability	Strength
1	Maximum Hydrostatic	Maximum	Hs	(1.0 or 2.2 or 1.6)(D + (EH + EV + Hs_{pr} + (Hw_c or IM_c))	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + γ_{pr} Hs_{pr} + 1.0(Hw_c or IM_c)
2A	Earthquake – OBE	Unusual	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.5EQ + 1.0 Hs_c
2B	Earthquake – MDE	Extreme	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	1.0D + 1.0EH + 1.0EV + 1.0EQ + 1.0Hs_c
3	Construction	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.6ES_N

a. *Load Case 1. Maximum Hydrostatic*.

(1) Combination of water on the flood side and landside that produces the maximum structural loading. The load category depends on the return period of the maximum hydrostatic force.

(2) Dead load, lateral soil pressure, and weight of soil that may be present. The load factors shown for EH assume that sufficient movement occurs for a design condition using active and passive earth pressures. This depends on the expected movement of the structure.

(3) Wave force from a wind event with 10-year return period or debris or ice impact loads that may be present as companion loads.

(4) Typically, the principal load is unusual or extreme.

b. Load Case 2A, Earthquake – OBE. (*Note*: This load case needs to be considered only if the wall has a significant loading during the nonflood stage.)

(1) Water levels on each side of the wall that create a differential hydrostatic loading with approximately 10-year return.

(2) Dead load, lateral soil pressure, and weight of soil that may be present.

(3) OBE-induced lateral and vertical loads.

c. Load Case 2B, Earthquake – MDE.

(1) Please note that this load case needs to be considered only if the wall has a significant loading during the nonflood stage.

(2) Loads are the same as Case 2A except with MDE (MDE equal to MCE for a critical structure).

d. Load Case 3, Construction.

(1) Dead load, lateral soil pressure, and weight of soil that may be present. The load factors shown for EH assume that sufficient movement occurs for a design condition using active and passive earth pressures. This depends on the expected movement of the structure.

(2) Soil surcharge.

E–5. Coastal floodwall (critical structure)

Load combinations for a coastal floodwall are shown in Table E–4.

Table E–4
Load combinations for a coastal floodwall

Load Case	Load Description	Load Category	Principal Load	Serviceability	Strength
1	Maximum Surge + Wave	Maximum	Hs + Hw	(1.0 or 2.2 or 1.6)(D + EH + EV + Hs_U + Hw_U)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + γ_{pr} $(Hs+ Hw)_{pr}$
2	Normal Operating	Usual	Hs + Hw	(1.0 or 2.2)(D + EH + EV + Hs_U + Hw_U)	N/A
3	Maximum Surge + Impact	Maximum	Hs + BI	(1.0 or 2.2 or 1.6)(D + EH + EV + Hs_{pr} + BI_{pr})	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + γ_{pr} Hs_{pr} + γ_{pr} BI_{pr}
4A	Earthquake – OBE	Unusual	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.5EQ + $1.0Hs_c$
4B	Earthquake – MDE	Extreme	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	1.0D + 1.0EH + 1.0EV + 1.0EQ + $1.0Hs_c$
5	Construction	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + $1.6ES_N$

a. *Load Case 1, Maximum Surge + Correlated Wave Force*.

(1) Maximum considered possible force or moment from surge still water plus the nonbreaking, breaking, or broken wave coincident with that condition. The load category of this principal load depends on the return period of the maximum loading.

(2) Dead load, lateral soil pressure, and weight of soil that may be present. The load factors shown for EH assume that sufficient movement occurs for a design condition using active and passive earth pressures. This depends on the expected movement of the structure.

(3) Typically, the principal loads are unusual or extreme.

b. *Load Case 2, Normal Operating*.

(1) Water is at the highest level with up to 10-year return period on the flood side.

(2) Wave force from a wind event with up to 10-year return period.

(3) Dead load, lateral soil pressure, and weight of soil that may be present.

c. *Load Case 3, Maximum Surge + Aberrant Barge, Boat, or Debris Impact*.

(1) Water levels create a maximum possible hydrostatic loading from differential head.

(2) Companion vessel impact load during conditions that create the maximum hydrostatic loading.

(3) Dead load, lateral soil pressure, and weight of soil that may be present. The load factors shown for EH assume that sufficient movement occurs for a design condition using active and passive earth pressures. This depends on the expected movement of the structure.

(4) Typically, this load is unusual or extreme.

d. Load Case 4A, Earthquake – OBE. (*Note*: This load case needs to be considered only if the wall has a significant loading during the nonflood stage.)

(1) Water levels on each side of the wall that create a differential hydrostatic loading with approximately 10-year return.

(2) OBE-induced lateral and vertical loads, if applicable.

(3) Dead load, lateral soil pressure, and weight of soil that may be present.

e. Load Case 4B, Earthquake – MDE. (*Note*: This load case needs to be considered only if the wall has a significant loading during the nonflood stage.)

(1) Same as 4A, except with MDE (Equal to MCE for a critical structure).

(2) Critical structure with site-specific ground motion determination assumed in the table.

f. Load Case 5, Construction. Floodwall is in place with the loads that are possible during the construction period.

(1) Dead load, lateral soil pressure, and weight of soil that may be present. The load factors shown for EH assume that sufficient movement occurs for a design condition using active and passive earth pressures. This depends on the expected movement of the structure.

(2) Soil surcharge.

E–6. Intake tower (normal structure)

Load combinations for an intake tower are shown in Table E–5.

Table E–5
Load combinations for an intake tower

Load Case	Load Description	Load Category	Principal Load	Serviceability	Strength
1A	Normal Pool, All Gates Open	Usual	Hs	(1.0 or 2.2)(D + EH + EV + Hs_U + Hw_U)	N/A
1B	Normal Pool, One or More Gates Closed	Usual	Hs	(1.0 or 2.2)(D + EH + EV + Hs_U + Hw_U)	N/A
1C	Normal Pool, All Gates Closed	Usual	Hs	(1.0 or 2.2)(D + EH + EV + Hs_U + Hw_U)	N/A
1D	Normal Pool with Silt	Usual	Hs	(1.0 or 2.2)(D + EH + EV + Hs_U + Hw_U)	N/A
2	Minimum Pool	Usual	Hs	(1.0 or 2.2)(D + EH + EV + Hs_U + Hw_U)	N/A
3	Diversion	Unusual	Hs	(1.0 or 1.6)(D + EH + EV + Hs_N)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + 1.4Hs_N
4	Maintenance Bulkheads in Place	Unusual	Hs	(1.6 or 1.6)(D + EH + EV + Hs_N + (Hw_c or IM_c))	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + 1.4Hs_N + 1.0(Hw_c or IM_c))
5	Maximum Pool	Extreme	Hs	(1.0)(D + EH + EV + Hs_X + (Hw_c or IM_c))	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0) EV + 1.2Hs + 1.0(Hw_c or IM_c)
6A	Maximum Wave	Extreme	Hw	(1.0)(D + EH + EV + Hw_X + Hs_c)	(1.2D or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + 1.2Hw_X + 1.0Hs_c
6B	Maximum Ice	Extreme	IX or IM	(1.0)(D + EH + EV + (IX_X or IM_X) + Hs_c)	(1.2D or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + 1.3(IX_X or IM_X) + 1.0Hs_c
7A	Earthquake – OBE	Unusual	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.5EQ + 1.0Hs_c
7B	Earthquake – MDE	Extreme	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	1.0D + 1.0EH + 1.0EV + 1.0EQ + 1.0Hs_c
8	Construction	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.6ES_N

a. Load Case 1A, Normal Pool, All Gates Open.

(1) Dead load of structure.

(2) Reservoir at normal high pool.

(3) Lateral and vertical earth loads (if any).

(4) Wave force from a wind event with 10-year return or impact load with similar expected return period (if any).

(5) Water surface inside structure drawn down to hydraulic gradient with all gates fully opened.

b. Load Case 1B, Normal Pool, One or More Gates Closed.

(1) Dead load of structure.

(2) Reservoir at normal high pool.

(3) One or more gates closed with other gates fully opened and water surface drawn down to hydraulic gradient in remainder of structure in combinations that produce the most unstable conditions.

(4) Lateral and vertical earth loads (if any).

(5) Wet well full of water upstream from closed gate.

(6) Wave force from a wind event with 10-year return or impact load with similar expected return period (if any).

c. Load Case 1C, Normal Pool, All Gates Closed.

(1) Dead load of structure.

(2) Reservoir at normal pool.

(3) Earth load (if any).

(4) Wave force from a wind event with 10-year return or impact load with similar expected return period (if any).

d. Load Case 1D, Normal Pool with Silt. Reservoir with silt for the most critical of Load Cases 1A through 1C.

e. Load Case 2, Minimum Pool.

(1) Reservoir empty or at minimum pool.

(2) Dead load of structure.

(3) Earth load (if any). The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(4) Wind load in the direction that would produce the most severe foundation pressures.

(5) Wave force from a wind event with 10-year return or impact load with similar expected return period (if any).

f. Load Case 3, Diversion.

(1) Reservoir at maximum elevation expected during diversion.

(2) Dead load of structure at diversion level completion.

(3) Earth load (if any). The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

g. Load Case 4, Maintenance Bulkheads in Place.

(1) Bulkheads in place, no water in structure downstream of bulkheads.

(2) Dead load of structure.

(3) Reservoir at maximum pool level at which bulkheads are used.

(4) Earth loads (if any). The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(5) Wave force from a wind event with 10-year return or impact load with similar expected return period (if any).

h. Load Case 5, Maximum Pool.

(1) Pool at probable maximum flood (PMF) elevation. Load factor in table assumes PMF has a return period of greater than 10,000 years.

(2) All gates opened or closed, depending on project operating criteria.

(3) Dead load of structure.

(4) Earth loads (if any). The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(5) Wave force from a wind event with 10-year return or impact load correlated to the height of the pool (if any).

i. Load Case 6A, Maximum Wave.

(1) Pool at elevation with 10-year return period.

(2) Gates at normal settings (multiple setting if needed).

(3) Dead load of structure.

(4) Earth loads (if any). The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(5) Wave load from wind event of 10,000-year return period (assuming the structure is critical).

j. Load Case 6B, Maximum Ice.

(1) Pool at elevation with 10-year return period.

(2) Gates at normal settings during ice season.

(3) Dead load of structure.

(4) Earth loads (if any). The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(5) Upper bound ice load (return period unknown).

k. Load Case 7A, Earthquake – OBE. OBE during the most critical of the Load Cases 1A through 1D.

l. Load Case 7B, Earthquake – MDE. MDE during the most critical of the Load Cases 1A through 1D. The equation in the table assumes site-specific ground motion is used.

m. Load Case 8, Construction.

(1) Reservoir empty.

(2) Dead load of structure (partially or fully completed).

(3) Earth load (if any). The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(4) Surcharge from construction operations.

E–7. Navigation lock wall (normal structure)

Load combinations for a navigation lock wall are shown in Table E–6.

Table E–6
Load combinations for a navigation lock wall

Load Case	Load Description	Load Category	Principal Load	Serviceability	Strength
1A	Normal Operating, Empty	Usual	Hs	(1.0 or 2.2)(D + EH + EV + Hs_U + BI_U)	N/A
1B	Normal Operating, Filled	Usual	Hs	(1.0 or 2.2)(D + EH + EV + Hs_U + BI_U)	N/A
1C1, 1C2	Normal Operating, Hawser	Unusual	HA	(1.0 or 1.6)(D + EH + EV + HA + Hs_c)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + 1.6HA + 1.0Hs_c
2	Maximum Barge Impact	Maximum	BI	(1.0)(D + EH + EV + BI_X)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0) EV + 1.2BI_X
3	Maintenance	Unusual	Hs	(1.0 or 1.6)(D + EH + EV + Hs_N)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + 1.4Hs_N
4	Maximum Hydrostatic	Maximum	Hs	(1.0 or 1.6)(D + EH + EV + Hs_{pr} + BI_U)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + γ_{pr} Hs_{pr} + 1.0BI_U
5A	Earthquake – OBE	Unusual	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.5EQ + 1.0 Hs_c
5B	Earthquake – MDE	Extreme	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	1.0D + 1.0 EH + 1.0EV + 1.0EQ + 1.0 Hs_c
6	Construction	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.6ES_N

a. Load Case 1A, Normal Operating.

(1) Dead load with lateral and vertical earth loads.

(2) Lower pool in landward lock chamber, upper pool in riverward lock chamber at differential levels with an average annual return period of 10 years.

(3) Vessel impact from normal operation if additive to forces.

b. Load Case 1B, Normal Operating.

(1) Dead load with lateral and vertical earth loads.

(2) Upper pool in landward lock chamber, lower pool in riverward lock chamber at differential levels with an average annual return period of 10 years.

(3) Vessel impact from normal operation if additive to forces.

c. Load Case 1C1, Normal Operating with Hawser Load.

(1) Dead load with lateral and vertical earth loads. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(2) Hawser load.

(3) Lower pool in landward lock chamber, upper pool in riverward lock chamber with 10-year hydrostatic differential force.

d. Load Case 1C2, Normal Operating with Hawser Load.

(1) Dead load with lateral and vertical earth loads. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(2) Hawser load.

(3) Upper pool in landward lock chamber, lower pool in riverward lock chamber at differential levels with an average annual return period of 10 years.

e. Load Case 2, Maximum Barge Impact.

(1) Dead load with lateral and vertical earth loads. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(2) Head differential across lock with 10-year return period and chambers filled with maximum effect with barge impact.

(3) Maximum barge impact.

(4) Soil pressures in the equation assume a very stiff structure with little movement and at-rest lateral earth pressures.

f. Load Case 3, Maintenance.

(1) Dead load with lateral and vertical earth loads. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(2) Lock chamber unwatered to a predetermined level. Water in backfill at maximum expected level during dewatering.

(3) This case normally provides the maximum hydrostatic loading from head differential across the monolith. If it does not, the case with maximum hydrostatic loading must be analyzed with load factors appropriate for the expected return period of the loading.

g. Load Case 4, Maximum Hydrostatic (may coincide with Load Case 3, Maintenance).

(1) Dead load with lateral and vertical earth loads. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(2) Pool level in lock chamber and pool outside of lock at levels that create the maximum hydrostatic loading from differential head.

(3) Vessel impact from normal operation, if additive to forces.

h. Load Case 5A, Earthquake – OBE. The same loads as Load Cases 1A and 1B except for OBE earthquake load added in the most critical direction.

i. Load Case 5B, Earthquake – MDE. The same loads as Load Cases 1A and 1B except for MDE earthquake load added in the most critical direction. The equation shown assumes a site-specific ground motion is used.

j. Load Case 6, Construction.

(1) Dead load, lateral soil pressure, and weight of soil that may be present. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(2) Soil surcharge.

E–8. Navigation lock gate monolith (normal structure)

Load combinations for a navigation lock wall monolith are shown in Table E–7.

Table E–7
Load combinations for a navigation lock gate monolith

Load Case	Load Description	Load Category	Principal Load	Serviceability	Strength
1A	Normal Operating, Gates Loaded	Usual	Hs	(1.0 or 2.2)(D + EH + EV + Hs_U)	N/A
1B	Normal Operating, Gates Unloaded	Usual	Hs	(1.0 or 2.2)(D + EH + EV + Hs_U)	N/A
2	Maximum Operating Forces	Maximum	Q	(1.0)(D + EH + EV + Hs_U + Q_{pr})	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0) EV + 1.3Q_{pr}
3	Maintenance	Unusual	Hs	(1.0 or 1.6)(D + EH + EV + Hs_N)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + 1.4Hs_N
4	Maximum Hydrostatic	Maximum	Hs	(1.0 or 1.6)(D + EH + EV + Hs_{pr})	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + γ_{pr} Hs_{pr}
5A & 5B	Earthquake – OBE	Unusual	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.5EQ + 1.0 Hs_c
5C & 5D	Earthquake – MDE	Extreme	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	1.0D + 1.0EH + 1.0EV + 1.0EQ + 1.0 Hs_c
6	Construction	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N)	(1.2 or 0.9)D + (1.3 or 0.9)EH + (1.35 or 1.0)EV + 1.6ES_N

a. *Load Case 1A, Normal Operating*.

(1) Dead load with lateral and vertical earth loads.

(2) Upper pool upstream of gates.

(3) Lower pool downstream of gates.

(4) Hydrostatic force from head differential with an average annual return period of 10 years.

b. *Load Case 1B, Normal Operating*.

(1) Dead load with lateral and vertical earth loads.

(2) Gates closed.

(3) For upper gate bay, upper pool in gate bay.

(4) For lower gate bay, lower pool in lock chamber.

(5) Hydrostatic force from head differential with an average annual return period of 10 years.

c. Load Case 2, Normal Operating, Gates Operating.

(1) Dead load with lateral and vertical earth loads.

(2) Reactions from gate operation with gate stuck and operating equipment at maximum output.

(3) For upper gate bay, upper pool in gate bay.

(4) For lower gate bay, lower pool in lock chamber.

d. Load Case 3, Maintenance.

(1) Dead load with lateral and vertical earth loads. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(2) Lock chamber unwatered to a predetermined level. Water in backfill at maximum expected level during dewatering.

(3) This case normally provides the maximum hydrostatic loading from head differential across the monolith. If it does not, the case with maximum hydrostatic loading must be analyzed with load factors appropriate for the expected return period of the loading.

e. Load Case 4, Maximum Hydrostatic (may coincide with Load Case 3, Maintenance).

(1) Dead load with lateral and vertical earth loads. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(2) Pool level in lock chamber and pool outside of lock at levels that create the maximum hydrostatic loading from differential head.

f. Load Cases 5A and 5B, Earthquake – OBE. The same loads as Load Cases 1A and 1B except for OBE loads added in the most critical direction.

g. Load Cases 5C and 5D, Earthquake – MDE. The same loads as Load Cases 1A and 1B except for MDE loads added in the most critical direction.

h. Load Case 6, Construction.

(1) Moist backfill to a predetermined level. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(2) Permanent or construction surcharge.

(3) Gates swinging freely in appropriate mitered position.

(4) Hydrostatic forces are active according to construction or cofferdam plans.

E–9. Navigation lock approach wall (normal structure)

Load combinations for a navigation approach wall are shown in Table E–8.

Table E–8
Load combinations for a navigation lock approach wall (for approach walls that also retain fill, see paragraph E–2 for additional load cases)

Load Case	Load Description	Load Category	Principal Load	Serviceability	Strength
1A	Normal Operating + Vessel Impact	Usual	BI	(1.0 or 2.2)(D + EH + EV + BI + Hs_U)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + $2.2BI_U$ + $1.0Hs_c$
1B	Normal Operating + Vessel Impact	Unusual	BI	(1.0 or 1.6)(D + EH + EV + BI_N + Hs_c)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + $1.6BI_X$ + $1.0Hs_c$
1C	Normal Operating + Vessel Impact	Extreme	BI	(1.0)(D + EH + EV + BI_X + Hs_c)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + $1.2BI_X$ + $1.0Hs_c$
2	Normal Operating,+ Vessel Thrust	Extreme	Hd	(1.0)(D + EH + EV + Hs_U +Hd_X)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV+ $1.3Hd_X$ + $1.0Hs_c$
3	Normal Operating+ Hawser	Unusual	HA	(1.0 or 1.6)(D + EH + EV + HA + Hs_c)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + 1.6HA + $1.0Hs_c$
4A	Earthquake – OBE	Unusual	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.5EQ + 1.0 Hs_c
4B	Earthquake – MDE	Extreme	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.25EQ + 1.0 Hs_c
5	Construction	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + $1.6ES_N$

a. Load Case 1A, Usual Vessel Impact Loading.

(1) Dead load with lateral and vertical earth loads.

(2) The pool is at a level of interest for performance under routine vessel impacts. A range between low and high operational pools may be used.

(3) Normal operating vessel impact force on face of wall at most critical angle of incidence.

b. Load Case 1B, Unusual Vessel Impact Loading.

(1) Dead load with lateral and vertical earth loads.

(2) Unusual impact force on face of wall at most critical angle of incidence.

(3) Water at high operational pool with 10-year return period.

c. Load Case 1C, Extreme Vessel Impact Loading.

(1) Dead load with lateral and vertical earth loads. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(2) The same requirements as Load Case 1A except that the vessel impact is at maximum design force level.

(3) Water at high operational pool with 10-year return period.

d. Load Case 2, Vessel Thrust Loading. The same loads as Load Case 1A except the maximum vessel thrust load is applied. This could be considered an unusual service load if the probability of the maximum thrust is not extremely low and/or it is desired to perform under this load with highly elastic response and little cracking.

e. Load Case 3, Hawser Loading. The same loads as Load Case 1A except that the hawser load is applied instead of impact. Water is at level most critical for hawser load with maximum of 10-year return period.

f. Load Case 4A, Normal Operating + OBE.

(1) Most critical normal operating pool condition.

(2) OBE loads in the most critical direction.

g. Load Case 4B, Normal Operating + MDE.

(1) Most critical normal operating pool condition.

(2) MDE loads in the most critical direction.

(3) Equation in table assumes standard ground motions are used.

h. Load Case 5, Construction. Temporary construction conditions exist. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

E–10. Spillway approach channel walls (critical structure)

Load combinations for a spillway approach channel wall are shown in Table E–9.

Table E–9
Load combinations for spillway approach channel walls

Load Case	Load Description	Load Category	Principal Load	Serviceability	Strength
1A	Usual Low Pool	Usual	EH	(1.0 or 2.2)(D + EH + EV + Hs_U)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.0 Hs_U
1B	Low Pool + Surcharge	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N + Hs_c)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.6ES_N + 1.0 Hs_c
1C	Maximum Hydrostatic	Maximum	Hs	(1.0 or 2.2 or 1.6)(D + (EH + EV + Hs_{pr})	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + γ_{pr} Hs_{pr}
2	Partial Sudden Drawdown, PMF	Extreme	Hs	(1.0)(D + (EH + EV + Hs_X)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.2Hs_X
3	Sudden Pool Rise, PMF	Extreme	Hs	(1.0)(D + (EH + EV + Hs_X)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.2Hs_X
4:00 AM	Earthquake – OBE	Unusual	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.5EQ + 1.0 Hs_c
4 B	Earthquake – MDE	Extreme	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	1.0D + 1.0EH + 1.0EV + 1.0 Hs_c + 1.0EQ
5	Construction	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.6ES_N

a. Load Case 1A, Channel Empty.

(1) Pool at low usual level with a 10-year average annual return period.

(2) Backfill submerged to elevation of line of drains, and naturally drained above this elevation. If no drains exist, the engineers must determine the reasonable level of usual head differential across the wall.

(3) Dead load, lateral soil pressure, and weight of soil that may be present. Drained backfill conditions.

b. Load Case 1B, Low Pool + Surcharge. This Load Case is the same as 1A + lateral soil pressure from temporary surcharge. The load factors shown for EH assume that sufficient movement occurs for a design condition using active and passive earth pressures. This depends on the expected movement of the structure.

c. Load Case 1C, Maximum Hydrostatic. This Load Case is the same as 1A except the water table level in the backfill and the water on the resisting side are at levels creating the maximum possible differential loading. The load category and load factor, γ_{pr}, depends on the return period of the maximum water loading, Hs_{pr}. The load factors shown for EH assume that sufficient movement occurs for a design condition using active and passive earth pressures. This depends on the expected movement of the structure.

d. Load Case 2, Partial Sudden Drawdown.

(1) Partial rapid drawdown of reservoir from PMF elevation (assumed return period for load factor greater than 10,000 years).

(2) Water in channel to drawdown elevation, which may occur suddenly.

(3) Fill submerged to profile reached during PMF, drained above.

(4) Dead load, lateral soil pressure, and weight of soil that may be present. Undrained backfill conditions. The load factors shown for EH assume that sufficient movement occurs for a design condition using active and passive earth pressures. This depends on the expected movement of the structure.

e. Load Case 3, Sudden Rise of Reservoir.

(1) Rapid rise of reservoir to PMF elevation (assumed return period for load factor > 10,000 years).

(2) Water in channel to PMF conditions.

(3) Fill submerged to concurrent water surface in fill.

(4) Water above fill to PMF elevation.

(5) Dead load, lateral soil pressure, and weight of soil that may be present. Drained backfill conditions. The load factors shown for EH assume that sufficient movement occurs for a design condition using active and passive earth pressures. This depends on the expected movement of the structure.

f. Load Case 4A, Earthquake – OBE.

(1) Water levels on each side of the wall that create a differential hydrostatic loading with approximately 10-year return.

(2) OBE loads in most critical direction.

(3) Dead load, lateral soil pressure, and weight of soil that may be present.

g. Load Case 4B, Earthquake – MDE. The same requirements as for Load Case 4A except the MDE is used instead of the OBE. The equation assumes site-specific ground motions are used.

h. Load Case 5, Construction. Wall is in place with the loads possible during the construction period.

(1) Dead load, lateral soil pressure, and weight of soil that may be present. The load factors shown for EH assume that sufficient movement occurs for a design condition using active and passive earth pressures. This depends on the expected movement of the structure.

(2) Soil surcharge.

E–11. Spillway chute slab walls (normal structure)

Load combinations for spillway chute slab walls are shown in Table E–10.

Table E–10
Load combinations for spillway chute slab walls

Load Case	Load Description	Load Category	Principal Load	Serviceability	Strength
1A	Channel Empty	Usual	EH	(1.0 or 2.2)(D + EH + EV + Hs_U)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.0 Hs_U
1B	Channel Empty + Surcharge	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N + Hs_c)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + 1.6ES_N + 1.0Hs_c
1C	Maximum Hydrostatic	Maximum	Hs	(1.0 or 2.2. or 1.6)(D + (EH + EV + Hs_{pr})	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.25 or 0.9)EV + γ_{pr} Hs_{pr}
2	Water in Channel, PMF	Extreme	Hs	(1.0)(D + (EH + EV + Hs_X)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 0.9)EV + 1.3Hs_X
3A	Earthquake – OBE	Unusual	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	(1.2 or 0.9)D + (1.5 or 0.5)EH + (1.35 or 1.0)EV + 1.5EQ + 1.0 Hs_c
3B	Earthquake – MDE	Extreme	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	1.0D + 1.0EH + 1.0EV + 1.0EQ + 1.0 Hs_c
4	Construction	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + 1.6ES_N

a. Load Case 1A, Channel Empty.

(1) Channel empty. (This assumes that the channel frequently has little or no water in it. If it normally has water, the level is the low usual level with a 10-year average annual return period. In other words, the level with an annual exceedance probability of 0.90)

(2) Backfill submerged to elevation of drains. Backfill naturally drained above elevation of drains. If no drains exist, the engineers must determine a reasonable level of water in the backfill.

(3) Dead load, lateral soil pressure, and weight of soil that may be present. Drained backfill conditions. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

b. Load Case 1B. Channel Empty + Surcharge. Same as Load Case 1A plus load from temporary surcharge. Assumes a maintenance condition with channel unwatered.

c. Load Case 1C. Maximum Hydrostatic. Same as Load Case 1A except the water table level in the backfill and the water on the resisting side are at levels creating the maximum possible differential loading. The load category and load factor, γ_{pr}, depends on the return period of the maximum water loading, Hs_{pr}.

d. Load Case 2, Water in Channel to PMF Elevation.

(1) Water in channel to PMF conditions (assumed return period for load factor greater than 750 years but less than 10,000 years).

(2) Backfill submerged to elevation of drains. If no drains, water in backfill at normal level is determined by the engineer.

(3) Dead load, lateral soil pressure, and weight of soil that may be present. Undrained backfill conditions. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

e. Load Case 3A, Earthquake – OBE.

(1) Water levels on each side of the wall that create a differential hydrostatic loading with approximately 10-year return.

(2) OBE loads in most critical direction.

(3) Dead load, lateral soil pressure, and weight of soil that may be present.

f. Load Case 3B, Earthquake – MDE. The same requirements as for Load Case 3A, except the MCE is used instead of the OBE. The equation assumes site-specific ground motions are used.

g. Load Case 4, Construction. Wall is in place with the loads possible during the construction period.

(1) Dead load, lateral soil pressure, and weight of soil that may be present. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

(2) Soil surcharge.

E–12. Spillway stilling basin walls (normal structure)

Load combinations for spillway stilling basin walls are shown in Table E–11.

Table E–11
Load combinations for spillway stilling basin walls

Load Case	Load Description	Load Category	Principal Load	Serviceability	Strength
1	Maintenance	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N + Hs_c)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + 1.6ES_{prN} + 1.0 Hs_c
2	Rapid Closure of Gates	Extreme	Hs	(1.0)(D + EH + EV + Hs_{pr})	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.25 or 0.9)EV + γ_{pr} Hs_{pr}
3A	Frequent Flood Discharge	Usual	Hs	(1.0 or 2.2(D + EH + EV + Hs_U)	N/A
3B	Maximum Hydrostatic – In	Maximum	Hs	(1.0 or 1.6 or 2.2)(D + EH + EV + Hs + Hd)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.25 or 0.9)EV + γ_{pr} $(Hs + Hd)_{pr}$
3C	Maximum Hydrostatic – Out	Maximum	Hs	(1.0 or 1.6 or 2.2)(D + EH + EV + Hs + Hd)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.25 or 0.9)EV + γ_{pr} $(Hs + Hd)_{pr}$
4A	Earthquake – OBE	Unusual	EQ	(1.0)(D + EH + EV + EQ + Hsc)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + 1.5EQ + 1.0 Hsc
4B	Earthquake – MDE	Extreme	EQ	(1.0)(D + EH + EV + EQ + Hs_c)	1.0D + 1.0EH + 1.0EV + 1.0EQ + 1.0 Hs_c
5	Construction	Unusual	ES	(1.0 or 1.6)(D + EH + EV + ES_N)	(1.2 or 0.9)D + (1.35 or 0.9)EH + (1.35 or 1.0)EV + 1.6ES_N

a. Load Case 1, Maintenance.

(1) Stilling basin empty.

(2) Backfill submerged to drain or higher if, during construction or maintenance, higher elevation is anticipated with stilling basin unwatered.

(3) Surcharge, if applicable.

(4) Dead load, lateral soil pressure, and weight of soil that may be present. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

b. Load Case 2, Rapid Closure of Gates or Reduction of Discharge of Ungated Spillway.

(1) Maximum reduction of discharge and tailwater, which is expected to occur rapidly. The load category of this principal hydrostatic load depends on the return period of the rapid closure condition.

(2) Water surface inside stilling basin at tailwater corresponding to reduced flow conditions.

(3) Backfill submerged to elevation midway between tailwater before and after reduction (corresponding to 50 percent reduction by drainage).

(4) Dead load, lateral soil pressure, and weight of soil that may be present. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

c. Load Case 3A, Frequent Operating.

(1) Water surface inside at hydraulic jump profile frequent (usual) discharge condition. This condition creates the greatest differential head between outside and inside faces of the wall.

(2) Backfill submerged to the corresponding tailwater conditions.

(3) Backfill above tailwater is naturally drained.

(4) Uplift across base varying uniformly from tailwater at heel to a value midway between tailwater and jump profile at toe (the latter corresponds to 50 percent relief of unbalanced pressure by floor drainage).

(5) Dead load, lateral soil pressure, and weight of soil that may be present.

d. Load Case 3B, Maximum Hydrostatic Loading – In. Maximum water loading from differential head across the wall from the outside of the stilling basin.

(1) Water surface combination of hydraulic jump profile and tailwater outside of the wall to create the highest water loading (combination of hydrostatic and hydrodynamic loads from the jump). The load category of this principal load and load factor depends on the return period of the conditions creating the maximum hydrostatic load.

(2) Backfill submerged to the corresponding tailwater condition or drains, whichever are higher.

(3) Uplift across base varying uniformly from tailwater at heel to a value midway between tailwater and jump profile at the toe (the latter corresponds to 50 percent relief of unbalanced pressure by floor drainage).

(4) Dead load, lateral soil pressure, and weight of soil that may be present. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

e. Load Case 3C, Maximum Hydrostatic Loading – Out. Maximum head across the wall from inside of the stilling basin.

(1) At the downstream of the expected hydraulic jump location, cross waves create a differential from inside the stilling basin to the outside. The combination of cross-wave size and water depth that create the highest water loading (combination of hydrostatic and hydrodynamic loads from the jump) is used. The load category of this principal load and load factor depends on the return period of the conditions creating the maximum hydrostatic load.

(2) Backfill submerged to the corresponding tailwater conditions.

(3) Uplift equal to tailwater.

(4) Dead load, lateral soil pressure, and weight of soil that may be present. The load factors shown for EH assume that at-rest earth pressures are used for design. This depends on the expected movement of the structure.

f. Load Case 4A, Earthquake – OBE.

(1) Water levels on each side of the wall that create a differential hydrostatic loading with approximately 10-year return.

(2) OBE loads in most critical direction.

(3) Dead load, lateral soil pressure, and weight of soil that may be present.

g. Load Case 4 B, Earthquake – MDE. The requirements are the same as for Load Case 4A except the MDE is used instead of the OBE. The equation assumes site-specific ground motions are used.

h. Load Case 5, Construction. Wall is in place with the loads possible during the construction period.

(1) Dead load, lateral soil pressure, and weight of soil that may be present.

(2) Soil surcharge.

Appendix F
Commentary on Chapter 3

F–1. Introduction

Table F–1 lists brief commentary on the provisions in Chapter 3.

F–2. Discussion of changes

Table F–1
Commentary on Chapter 3

Paragraph	Title	Comment(s)
Chapter 3		
3–1a	ACI 318-19	ACI 318-19 is appropriate for computing member capacity; resistance factors are provided in ACI 318-19.
3–1b	Serviceability Limit States	For hydraulic structures, cracking, deflection, durability, vibrations, and stability are major serviceability concerns. Some general observations regarding the cracking phenomenon in RCHS (Liu and Gleason 1981, pp. A1–A2) are that: • Crack widths due to applied loads are essentially proportional to the stress in the reinforcement. • Crack widths at any load level are minimized if the tension reinforcement is well distributed across the width of the concentric, concrete cross section. • Only deformed bars should be used for tension reinforcement. • The average crack width for an RCHS should not exceed 0.01 in. Practical methods for analyzing or predicting cracking are either very complex and costly or unreliable, so accounting for crack widths and spacing is based on design experience and observed performance of RCHS. Therefore, cracking is controlled by limiting the service stresses in the reinforcing steel and also by limiting the spacing of flexural reinforcement.
3–1c	Strength Limit States	The intention is for designs under the strength limit state to have uniform minimum design reliability. As the range and uncertainty in loads on hydraulic structures can vary greatly across load types and project locations, uniform reliability can be achieved only by using loads with very low likelihood of exceedance.
3–1f	Stability Analysis	EM 1110-2-2100 provides criteria for stability of RCHS. *At the publishing time of this engineer manual, stability analyses of concrete structures is performed using limit equilibrium methods to compute a factor of safety. Therefore, stability analyses for all load cases must be performed both according to EM 1110-2-2100 to check factors of safety for stability limit states, and according to (using) the load factor methods herein to compute moments and shears for structural design of members*. For load cases that include earthquake effects, the seismic coefficient used for stability may be different from what is used for strength analysis.

Paragraph	Title	Comment(s)
3–2d	Load Probability	Usual, unusual, and extreme load categories and the limits of AEP were selected to be consistent with other USACE design guidance. See paragraph 3–2e for discussion of loads and paragraph 3–3 for load factors.
3–2f	Load Definitions	Loads and load combinations applicable to hydraulic structures are generally different from those provided for buildings in ACI-318 and ASCE-7. Therefore, loads typically applied to RCHS were defined and developed for this manual. ASCE, AASHTO, and others have done much detailed work to define load types, load probability, and design loads for buildings and bridges. Comparatively little has been done for hydraulic structures. For most projects, hydraulic engineers have developed stage frequency relationships for hydrostatic loads. Wind and seismic information is available from ASCE, U.S. Geological Survey, and other sources. For other loads, there is generally less information. Chapter 3 provides guidance based on information available at the time this manual was developed. General guidance is provided for loads for which little information is available. In many cases, these loads will be site-dependent: • *Load combinations with hydrostatic forces*. For many hydraulic structures, hydrostatic forces from a pool are combined with other independent loads. Examples are wind driven waves on dam, thermal expansion ice loads on a dam, debris loads on a dam, wave loads on a floodwall, vessel impact forces, gate operation loads, and others. In some cases, the hydrostatic loads may not be important, but the height of applied loads, determined by pool levels, may be. Studies for the engineer manual showed that pool levels to be combined with independent loads for the strength case must have low probability of exceedance. Reasons are similar to that presented in the comments on paragraph 3–5 (p 27). • *Design water levels for extreme load cases*. Load factors are intended to account for uncertainty in the nominal load and to provide adequate reliability. For lower nominal loads relative to possible loads, higher load factors are required. The predominant load on most RCHS is hydrostatic loading. The load depends on the differential head across the structure. The uncertainty of hydrostatic loading is highly site-dependent. For this reason, nominal loads must be selected from the highest possible hydrostatic head. If the RCHS provides a higher level of protection than its authorized level of protection, the RCHS are designed to withstand the forces produced by water at the higher level of protection (for example, at the top of a wall) if life or significant economic losses initiate when the higher hydraulic events are experienced.

<table>
<tr>
<td>3–3</td>
<td>Minimum load factors for design of RCHS</td>
<td>
Compared to the previous version of this manual, the LRFD load combination equation has been simplified. Serviceability requirements were embedded in the previous LRFD equations but have now been broken out into separate criteria in paragraph 3–4.
<u>Strength Limit States</u>
Reliability-based load factors were developed for the strength limit states used with extreme loads. General steps in developing load factors are:

Review existing structures and identify loads.
Compute reliability of the existing structures that were designed with previous engineer manuals.
Establish reliability targets for new structures.
Perform trial designs with trial load factors and load combinations.
Calculate reliability of the trial designs.
Adjust load factors and nominal loads and repeat steps 4 and 5 to achieve targets for reliability.

In addition, load factors and target reliability in ASCE 7 were used as a guide.
<u>Reliability Analyses</u>
Before developing new sets of load factors for design of RCHS, it was important to understand the reliability of structures designed under previous guidance. Existing RCHS have performed very well from the standpoint of structural strength. Analyses were performed on existing inland and coastal floodwalls in three project sites designed from the 1960s to the 2000s. The existing structures had very low probabilities of load effects exceeding ACI 318-19 limit states for bending moment and shear (less than 10^{-6} over a service life of 100 years). This showed that designs performed with previous guidance resulted in structures with very high reliability for concrete strength.
<u>Target Reliability</u>
Using reliability of existing RCHS and reliability targets provided in ASCE 7 as a guide, reliability targets for design expressed as β were determined for 100-year project service lives as shown in
Table F–2. The definition of Critical and Normal structures is provided in Chapter 3.
where:
$\beta = E[SM]/\sigma[SM]$
E[SM] is the mean of the safety margin and σ[SM] is standard deviation of the safety margin. This can be rewritten as:
$B = E[R - Q]/(\sigma_C^2 + \sigma_D^2)^{½}$
where R is the resistance and Q is the demand or load effect.
Table F–2
Target reliability for 100-year service life, β
<table>
<tr><th></th><th>Normal</th><th>Critical</th></tr>
<tr><td>Redundant load path</td><td>3.0</td><td>3.5</td></tr>
<tr><td>Single load path</td><td>3.5</td><td>4.0</td></tr>
</table>
</td>
</tr>
</table>

Note that these targets are for the probability of failure over the service life of the structure. They include the probability loading. They are not conditional on a load occurring.

Structures with single load paths have higher probability of collapse after reaching the limit state and, therefore, have higher targets for Beta and lower target probabilities of failure. Structures with redundant load paths can sustain more load and/or damage before collapse. Most RCHS are cantilever structures with single load paths. Therefore, load and load factor developed for RCHS are for structures with single load path.

From β, the probability of failure (exceeding a specific limit state) P_f can be determined by:

$P_f \approx \phi(-\beta)$

where Φ is the standard normal deviate.

For the load factor calibration that was performed, the P_f was computed using Monte Carlo simulation, and β then computed from P_f. See Figure F–1 for an illustration of these concepts.

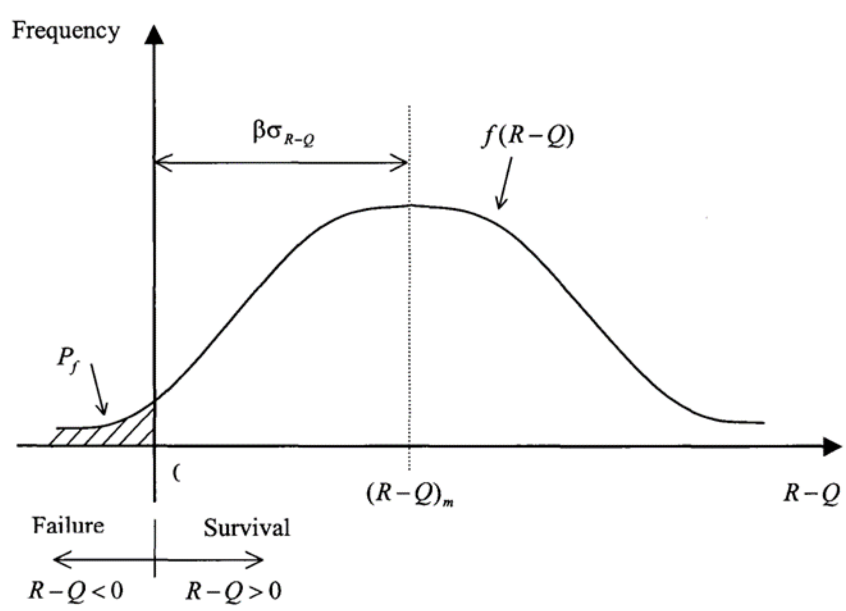


Figure F–1. Reliability concepts

<u>Calibration</u>. Calibration was performed for loads designed at the minimum for the following structures and load types:

1. *Inland floodwall*. Trial designs for soil-founded floodwalls on the Red River of the North at Grand Forks, North Dakota, and at Pembina, North Dakota. The design water surface was at the top of the wall, which was considered the maximum possible hydrostatic loading condition before inundation of the Landside.
2. *Coastal floodwall*. Cantilever floodwall stem subject to surge and wave with no tailwater. The example section was in Freeport, Texas, for which distributions of annual frequency vs. wave and surge loads had been developed by the U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory.

3. Earth retaining wall with no water present.
4. Vessel impact on a navigation approach wall with no other lateral loads (hydrostatic or earth pressures) present.
5. Wave plus independent hydrostatic loads on cantilever wall stems on Big Bend and Fort Randall dams, South Dakota.

<u>Nominal Loads and Load Factors.</u>

ASCE 7 has been moving to the principle of Uniform Risk (uniform reliability) for establishing nominal loads and load factors, rather than Uniform Hazard, as had been used previously. Under the Uniform Risk principle in ASCE 7, loads and load factors are chosen to provide uniform reliability for structures with a given risk category. To obtain this, nominal loads are selected with low probability of exceedance (large return period).

Nominal loads for design of strength limit states for RCHS are intended to be determined similarly. Both the load factor and the resistance factor determine the reliability of the designed section. For RCHS, the decision was to use the resistance factors in ACI 318-19 without modification. The total reliability targets for RCHS are higher than used for the development of the resistance factors in ACI 318-19. Therefore, the load factor must be made sufficient to provide the intended total reliability. For RCHS, the minimum load factor is 1.2 rather than 1.0 used in ASCE 7-16.

Minimum return periods for design of RCHS structures were determined by calculation and comparison with ASCE 7-16 design for wind forces. Design for wind load in ASCE 7 is based on an extreme value distribution of maximum annual wind velocities. Other temporary and dynamic loads on RCHS may have a similar distribution of annual maximums. The return period of 3,000 years for design wind velocities in ASCE 7 provides a structure with reliability, β, of 3.5 in 50 years. This was considered adequate for normal RCHS structures with a single load path. A return period of 10,000 years for design of critical structures will provide a corresponding β of approximately 4.0, which sufficiently meets reliability targets. A load factor of 1.2 is used when nominal loads are selected at these return periods to meet reliability targets.

For hydrostatic loading on many RCHS, geometry, ground levels, height of the structure, etc., will limit the maximum differential head that a structure will see. The maximum loading is typically less than the return periods stated in the previous paragraph. For these structures, the maximum possible load must be used for design. With design load return periods potentially at the lower limits shown in Figure 3–1, higher load factors are required to provide adequate reliability.

A load factor of 1.3 was found to meet reliability targets for floodwalls with maximum differential head at a 300-year return period for normal structure and at a 750-year return period for critical structures. Based on this analysis, the load factor of 1.3 was adopted for all loads potentially limited to return periods at the lower end of the limits for extreme loads. A load factor of 1.4 was found to meet reliability targets for loads with a maximum value at a return period of 10 years. This is the lower end of the unusual load category.

For many loads on hydraulic structures, such as debris impact, thermal expansion ice force, equipment loads, and hydrodynamic forces, insufficient data exists from actual events to determine return periods of loading. Design is performed with loads considered upper bounds of

Paragraph	Title	Comment(s)
		possible loading; because of the uncertainty in the actual loading, a load factor of 1.3 is applied. This load factor was chosen based on past experience, and to maintain consistency with the load factor developed in the previous paragraph. Load Factors for Soil Pressures Strength limit state load factors for soil vertical and lateral loads were derived from AASHTO LRFD Bridge Design Specifications 2014, 7th ed. Work has been performed by and in support of AASHTO to determine appropriate load factors for soil. No new work was performed for this engineer manual. The load factors for lateral earth loads meet target reliability using soil pressures computed using soil parameters with an inherent conservative bias. This is described in EM 1110-2-2502. Critical and Normal Structures For simplicity and consistency with EM 1110-2-2100, required reliability for critical and normal structures is accounted for by the minimum annual probability of the exceedance used to define the strength limit state/extreme load case. The load factor applied to both critical and normal structures is the same. Serviceability requirements for critical and normal structures are also the same.
3–3h	Earthquake Load (effects)	This section has been simplified from previous versions to a reference to ER 1110-2-1806 and three engineer manuals to be used for earthquake design of reinforced concrete. Equations for calculating factored earthquake loads are provided.
3–4	Serviceability Design	In this version of the manual, serviceability design requirements have been separated from strength design. This is intended to make the design requirements more transparent and allow the LRFD design to be performed consistent with other components of hydraulic structures, such as structural steel components and deep foundations. The revised provisions also clarify the design limit states to which the serviceability requirements are applied. Service Stresses. Maximum service stresses were determined from requirements in past versions of the manual. The stresses are intended to meet the performance requirements in paragraph 3–1e. They will provide designs consistent with past versions of this manual. Since the allowable stresses control deformation and cracking in the concrete, they are independent of the reinforcing steel strength. Alternate Serviceability Design. Single load factors are provided to design by using the strength method. The resulting design sections should provide service stress that meet the previous paragraph. These single load factors are consistent with the previous version of the manual.

Paragraph	Title	Comment(s)
3–6	Reinforcement Limits	The minimum tension reinforcement ensures that the strength limit state is a ductile failure mode that exceeds the cracking moment as estimated from its modulus of rupture. The minimum requirements have been changed from past versions of the manual. The previous limit was derived from the initial change from working stress to strength design in the 1980s. The intent was to provide thick sections equivalent to those designed under working stress. Crack control has become a prime performance requirement and the limits to reinforcing ratio do not have a significant effect on this performance aspect. Guidance for design for deflection is covered in paragraph 3–4.

Appendix G
Commentary on Chapter 5

G–1. Introduction

RCHS are primarily made of one-way slabs in bending and shear. These slabs are frequently cantilevers and are typically thick, of low reinforcement ratio, and with no shear reinforcement. ACI 318-19 greatly revised the computation of shear capacity of concrete beams without shear reinforcement. Chapter 5 of this manual provides shear capacity equations from ACI guidance prior to ACI 318-19 for use in design of RCHS. This appendix describes why the newer shear capacity equations were not adopted in this version of EM 1110-2-2104.

G–2. Shear capacity computation

a. Prior to ACI 318-19. The general equation for shear capacity of members without reinforcement before ACI 318-19 was:

$$V_c = \left[2\sqrt{f'_c} + \frac{N_u}{6\,A_g}\right] b\,d$$

where:

V_c = nominal shear capacity

f'_c = concrete compressive strength

N_u = factored axial load

A_g = gross area of design section

b = section width

d = distance from extreme compression fiber of driving side leg to primary wall reinforcement, inches

b. ACI 318-19. In ACI 318-19, a new shear equation was provided for members without shear reinforcement:

$$V_c = \left[8\,\lambda_s\,\rho_w^{1/3}\sqrt{f'_c} + \frac{N_u}{6\,A_g}\right] b\,d$$

where terms are defined above except:

λ_s = size effect modification factor

ρ_w = reinforcement ratio

c. Comparing the shear equations. In the ACI 318-19 shear capacity equation, rather than a fixed coefficient of 2.0 for all members, the capacity decreases as the member thickness increases and the reinforcement ratio decreases. RCHS members tend to be thick and reinforcement ratios small. The shear capacity of RCHS may be computed to be much less using ACI 318-19 compared to the prior version. Table G–1 shows the computation of the shear coefficient for different thickness of sections assuming that the longitudinal reinforcement is 0.25 of the balanced reinforcing ratios. The coefficient is less than 2 in all cases and becomes much lower at large thickness.

Table G–1
Shear coefficient for ρ_w = 0.25 ρ_b

Member Thickness, in.	$8\ \lambda_s\ \rho_w^{1/3}$
12	1.54
24	1.25
36	1.06
48	0.93
60	0.84
72	0.78
84	0.72
96	0.68
108	0.64
120	0.61

Notes:
1. Assumptions are f'_c = 4,000 psi,
2. F_y = 60,000 psi, depth of cover = 3 in.

d. *Meeting shear capacity requirements of ACI 318-19.* To provide sufficient shear capacity according to ACI 318-19, thick members may be needed for much more longitudinal reinforcement (to increase ρ_w) or else shear reinforcement may need to be used. Many existing RCHS would not meet the shear capacity requirements of ACI 318-19.

G–3. Considerations for EM 1110-2-2104

a. USACE contacted ACI after learning of the revised shear requirements in ACI 318-19. The U.S. Bureau of Reclamation also contacted ACI with similar concerns about the new shear capacity requirements. ACI reported some performance concerns with shear in beams in buildings. Most research for the revised capacity equation was based on testing of simple span members with concentrated loads in the middle of the beam. RCHS tend to be cantilever structures with maximum shear where the base of the cantilever intersects another member.

b. Reinforced concrete members in hydraulic structures can be very thick and have low reinforcement ratios. USACE has not experienced shear failures of concrete members using the previous shear capacity equations. There are many very large earths retaining wall and abutment structures in the USACE inventory that are loaded near to the design loading without incident. The applicability of the revised shear capacity equations to RCHS is therefore questionable. Based on this experience, EM 1110-2-2104 uses shear capacity equations from standards prior to ACI 318-19 until the need for changes to the shear capacity computation can be verified.

Glossary of Terms

Section I

List of Acronyms

Term	Definition
AASHTO	Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AEP	Annual Exceedance Probability
AREMA	American Railway Engineering and Maintenance-of-Way Association
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing And Materials
CGSI	Concrete General Strength Investigation
DA	No Definition
EL	Elevation
EM	Engineer Manual
ER	Engineer Regulation
GFRP	Glass Fiber-Reinforced Polymer
HSS	Hydraulic Steel Structures
L&D	Locks and Dam
LRFD	Load and Resistance Factor Design
MCE	Maximum Credible Earthquake
MDE	Maximum Design Earthquake
N/A	Not Applicable
OBE	Operating Basis Earthquake
PHI	Strength Reduction Factor
PMF	Probable Maximum Flood
RCHS	Reinforced Concrete Hydraulic Structures
USACE	U.S. Army Corps of Engineers

Section II

Notation

A_d = Depth of stress block at limiting value of balanced condition
b = Design width of member, in
d = Distance from extreme compression flange to centroid of longitudinal tension reinforcement, in
d_d = Minimum effective depth that a singly reinforced member may have and maintain steel ratio requirements
e' = Eccentricity of axial load measured from the centroid of the tension reinforcement
e'_b = Eccentricity of nominal axial load strength, at balanced strain conditions, measured from the centroid of the tension reinforcement
f'_c = Specified compressive strength of concrete, psi
f_y = specified yield strength of reinforcement, psi
k_b = Ratio of stress block depth (a) to the effective depth (d) at balanced strain conditions
k_u = Ratio of stress block depth (a) to the effective depth (d)
K = Exponent, equal 1.5 for rectangular members and 1.75 for square or circular members, used in nondimensional biaxial bending expression
ℓ_n = Clear span between supports
M_{DS} = Bending moment capacity at limiting value of balanced condition
M_{nx}, M_{ny} = Nominal biaxial bending strengths with respect to the x and y axes, respectively
M_{0x}, M_{0y} = Uniaxial nominal bending strength at P_n about the x and y axes, respectively
M_{ux}, M_{uy} = Factored biaxial bending moments with respect to the x and y axes, respectively
R = Radius of curvature to centerline of curved member

Section III

Unit conversion factors

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic inches	1.63871E-05	cubic meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
inches	0.0254	meters
pounds (force)	4.448222	newtons
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
square feet	0.09290304	square meters
square inches	6.4516E-04	square meters
square miles	2.59E+06	square meters
cubic meters	35.31466	cubic feet
cubic meters	61023.7	cubic inches
radians	57.295788	degrees (angle)
meters	3.2808	feet
meters	39.37	inches
newtons	0.224809	pounds (force)
pascals	0.020885	pounds (force) per square foot
kilopascals	0.145038	pounds (force) per square inch
square meters	10.763910	square feet
square meters	1550.0	square inches
square meters	3.86E-07	square miles