Final Draft Development of

Chapter 30 - Concrete

of the

National Engineering Handbook, Part 636

Prepared for:

NRCS
Natural Resources Conservation Service

of the

USDA
United States Department of Agriculture

Prepared by:

Gannett Fleming
# Chapter 30 – Concrete
National Engineering Handbook, Part 636

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A. Purpose of Documents

This handbook contains minimum criteria and procedures for use in reinforced concrete design practice in the Natural Resources Conservation Service. The American Concrete Institute Building Code Requirements for Structural Concrete (ACI 318) and the American Concrete Institute Code Requirements for Environmental Concrete Structures (ACI 350) serve as the reference for this handbook and NRCS concrete design criteria in general. Their provisions apply as the general design codes for NRCS except as modified or otherwise stated herein, either directly or indirectly. This handbook supplements, expands on, and/or emphasizes subject matter particularly pertinent to NRCS requirements.

B. History of SCS/NRCS Design Methodology

In the Beginning

The Soil Conservation Service (SCS) published a series of handbooks in the early 1950s to guide engineering work in the agency. The series included National Engineering Handbook 6 (NEH-6) “Structural Engineering” including Chapter 4 “Concrete” which covered concrete design criteria as well as design aids, steel detailing guidance, and structural drawing layout. Specific dates are lacking from copies of the original handbook, but references are made to the new ASTM 305-49 “Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement” and that Committee 318 of the American Concrete Institute (ACI) has proposed revising the ACI Building Code to permit the use of these higher bond stresses. The basic design code reference is Chapter VIII of the Joint Committee Report entitled “Recommended Practice
and Standard Specifications for Concrete and Reinforced Concrete”. For reference here, the original NEH-6 outlined the following concrete design criteria for Class A Concrete:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'_c$</td>
<td>3750 psi</td>
<td>Class A, maximum considered</td>
</tr>
<tr>
<td>$f_c$</td>
<td>1500 psi</td>
<td>i.e. 0.40 $f'_c$</td>
</tr>
<tr>
<td>$v_c$</td>
<td>113 psi</td>
<td>beams with no web reinforcing</td>
</tr>
<tr>
<td>$f_s$</td>
<td>20000 psi</td>
<td>intermediate grade steel bars</td>
</tr>
<tr>
<td>$\rho_{ts}$</td>
<td>0.20%</td>
<td>exposed face</td>
</tr>
<tr>
<td>$\rho_{ts}$</td>
<td>0.10%</td>
<td>unexposed face</td>
</tr>
</tbody>
</table>

These criteria were used to design a series of “Standard Structural Detail Drawings” which covered a range of sizes of standardized Type B Drop Spillways titled ES2xxx where xxx is a size designation. The drawings were self-contained sets of structural and reinforcement details that could be readily adapted to a particular site in the field. The drawings were prepared in the early 1950’s and widely used across the country. Some installations suffered from cracking in the sidewalls of the structure. The problem was attributed to low design loading and poor structural analysis assumptions, not the concrete design.

The 1960’s

ACI Committee 318 did follow through with their proposed revision to the ACI Building Code and published ACI 318-63. It presents equally both Working Stress Design (WSD) based on allowable stresses, service loads, and accepted straight line theory of stresses and strains in flexure and Ultimate Strength Design (USD) based on proportioning reinforced concrete members based on calculation of their ultimate strength and controlling deflections and cracking under service loads to assure serviceability. This control was principally accomplished by
limiting the amount of reinforcement to 0.75 of the ratio ($\rho_b$) that would produce balanced strain conditions at ultimate strength. NEH-6 was updated in 1964 to incorporate most of the new criteria from ACI 318-63, however, NEH-6 was still limited to WSD. For reference here, this revised NEH-6 outlined the following concrete design criteria for all SCS work:
Table 2. Criteria in 1964 revision to NEH-6 Chapter 4

<table>
<thead>
<tr>
<th>$f'_c$</th>
<th>4000 psi</th>
<th>Class 4000 for moderate exposure conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_c$</td>
<td>1600 psi</td>
<td>still 0.40 $f'_c$ while ACI318-63 used 0.45 $f'_c$</td>
</tr>
<tr>
<td>$v_c$</td>
<td>70 psi</td>
<td>beams with no web reinforcing, same as ACI 318-63</td>
</tr>
<tr>
<td>$f_s$</td>
<td>20000 psi</td>
<td>Grade 40 steel bars</td>
</tr>
<tr>
<td>$\rho_{ts}$</td>
<td>0.30%</td>
<td>exposed face w/ 30ft+ between joints</td>
</tr>
<tr>
<td>$\rho_{ts}$</td>
<td>0.20%</td>
<td>exposed face, unexposed face w/ 30ft+ between joints</td>
</tr>
<tr>
<td>$\rho_{ts}$</td>
<td>0.10%</td>
<td>unexposed face w/ less than 30ft between joints</td>
</tr>
</tbody>
</table>

These criteria were used to design the two series of “Standard Detail Drawings” covering a range of sizes of standardized Covered Top Risers titled ES30xx and Open Top Risers titled ES31xx where xx is a size designation. The drawings were prepared in the mid 1960’s and have been widely used for thousands of dams since then. These criteria were also used for hundreds of special designs, typically larger than covered by the standard sizes. All the structures have performed very well and virtually all are still in service. The concrete design criteria utilized should be considered very adequate.

The 1970’s

ACI Committee 318 updated the ACI Building Code and published ACI 318-71 and ACI 318-77. The 1971 Code primarily adopts the strength design method, but accepts the partial use of the working stress design method within the body of the Code, terming it the Alternate Design Method (ADM). The 1977 Code moves the ADM to an Appendix. The plan and practice of the ACI Committee 318 at the time was to publish a new Code every six years and introduce extensive new material into an Appendix for one Code cycle before bringing it into the main body of the Code. Likewise old material would be carried in an Appendix for one Code cycle before completely deleting it.
ACI 318-71 introduced additional requirements to control cracking under service loads and assure serviceability by adding requirements to detail and distribute reinforcing steel in beams and one way slabs to limit z-values to 175 and 145 kips per inch for interior and exterior exposure respectively. The Commentary on ACI 318-71 explains that the 175 and 145 values correspond to theoretical crack widths to 0.016 and 0.013 inches.

ACI Committee 350 published a Report in 1977 on “Concrete Sanitary Engineering Structures” which cited special serviceability requirements needed for their structures including limited deflections and cracking, good durability in contact with water and wastewater, and low permeability. The Report recommends that structural design generally conform with ACI 318-63, but with lower allowable stresses for $f_c$, $v_c$, $f_s$ and lower allowable z-values of 115 and 95 kips per inch for normal sanitary and very severe exposures respectively.

The 1980’s

As ACI 318 continued to evolve, it became clear that the working stress design method that had been used to successfully design virtually all existing and successfully functioning dam infrastructure would become passé, be lost from the primary concrete design code and be replaced by strength design. New engineers would only learn strength design in college and new design aids would only be developed for strength design. The SCS, the Corps of Engineers, and Sanitary Engineering designers started to examine how they could determine alternate criteria parameters for the strength design method to produce the same concrete and reinforcing proportions and stress levels as the old WSD that had performed successfully on so many dam and sanitary projects.
The SCS published Technical Release 67 (TR-67) “Reinforced Concrete Strength Design” in 1980 broadly modeled on the ACI 318-77 Code and refined to duplicate results from old WSD. The SCS approach was to increase load factors for both dead and live loads in most cases to 1.8, limit the design yield strength of Grade 60 reinforcing steel to 40 ksi for Service hydraulic structures (SHY), limit z-values, and limit the maximum reinforcing steel ratios. The z-values are limited to 130 kips per inch for Service hydraulic structures and 145 for other structures. A Service hydraulic structure is defined as a Soil Conservation Service structure subject to hydrostatic or hydrodynamic pressures. For a SHY, the maximum permitted steel ratio cannot exceed that allowed by SCS WSD criteria in NEH-6 for the same material combination. Hence, the ratio of the maximum steel ratios permitted for SHY to the steel ratio producing balanced strain conditions in SD accordingly varies from approximately 0.24 to 0.40 for various materials combinations. For the de facto standard, the ratio of the SHY ratio to the balanced strength steel ratio is 0.31. The maximum steel ratio permitted for SHY is:

$$\rho_{shy} = 0.40 \left( \frac{f'_c}{f_y} \right) \frac{1.0}{\left[ 1.0 + \left( 1.25 \frac{f_y}{nf'_c} \right) \right]} \quad \text{where} \quad n = \frac{503.3}{(f'_c)^{0.5}}$$

For comparison here, the maximum allowable reinforcement ratio, $\rho_{SHY}$, with 4000 psi concrete and 40 ksi steel, would be 0.01556.

The SCS also revised NEH-6 Chapter 4 in 1980 so that concrete design for Service hydraulic structures by either SD in the new TR-67 or WSD in the revised NEH-6 would yield very similar designs. Basic WSD parameter of $f'_c$, $v_c$, $f_y$ were unchanged from the 1964 revision and remain consistent with the concrete design criteria for all standard detailed drawings developed since the mid 1960’s. Development and splicing of reinforcing was updated to agree with major changes.
in ACI 318-77. A z-value limit of 130 kips per inch was added to match TR-67. Example design problems and design monographs remained the same as the 1964 revision. Some steel detailing guidance stayed the same as the original 1950 publication. This 1980 revision was the last update to NEH-6.

The Corps published ETL 1110-2-265 “Strength Design Criteria for Reinforced Concrete Hydraulic Structures” in 1981. Their approach was to increase load factors for dead and live loads to 1.5 and 1.9 respectively, limit the design yield strength of Grade 60 reinforcement to 48 ksi, and limit maximum reinforcement ratios for various situations. For comparisons here, the maximum allowable reinforcement ratio for a simple rectangular section with tension reinforcement only, 4000 psi concrete, and Grade 60 reinforcing, would be 0.0097 for a hydraulic structure. This is about 2/3 of the maximum allowable SCS TR-67 limit.

The ACI 350 Committee published ACI 350R-83 “Concrete Sanitary Engineering Structures” in 1983. They introduced the concept of sanitary durability coefficients to provide conservative service load stresses for strength design with Grade 60 steel. They also state in the Report that the coefficients were selected to provide crack control equivalent to that obtained with working stress design. For basic comparisons here, the sanitary coefficient is 1.3. For calculations for reinforcement in flexure, the required strength should be 1.3 U where U is the total factored load according to ACI 318 except factors of 1.7 are used for lateral earth pressure, H, and lateral liquid pressure, F, respectively. Maximum allowable steel ratios are not explicitly limited, but z-values of 115 and 95 kips per inch are carried from the previous report.
The ACI 350 Committee was reorganized and renamed “Environmental Engineering Concrete Structures” in 1984. The new mission of the Committee became developing an ACI 350 Code to cover Environmental Engineering Concrete Structures. Only two other ACI Committees produced Codes at the time, ACI 318 of course and ACI 349 “Concrete Nuclear Structures”. ACI leadership directed the ACI 350 Committee to develop an ACI 350 Code as a dependent code on the ACI 318 Code, i.e. the ACI 350 Code would be a complete stand alone document including everything in ACI 318 Code and any changes, deletions, or additions, the ACI 350 Committee determined were appropriate for Environmental Engineering Concrete Structures. Both SCS and the Corps were members of the reorganized ACI 350 Committee with the similar vision of having a consensus standard Code that covered reinforced concrete design by both SD and WSD methods and produced designs that were similar to those on thousands of dams that have been performing successfully. The ACI 350 Committee drafted and balloted several versions of the ACI 350 Code during the decade, but none came close to being published as the ACI 318 Code kept moving forward and the ACI 350 work was always a Code cycle behind. The ACI 350 Committee did republish the ACI 350R-89 to maintain a current document in the ACI library.

*The 1990’s*

The Corps published ETL 1110-2-2104 “Strength Design of Reinforced-Concrete Hydraulic Structures” in 1992. They cite adapting an approach similar to ACI 350R-89. The concrete stress-strain relationship and the yield strength of Grade 60 reinforcement given in ACI 318 were adapted. The specified load factors are closer to those in ACI 318 and are modified by a hydraulics factor, \( H_f \), to account for the serviceability needs of hydraulic structures.
The ACI 350 Committee continued to redraft and re-ballot individual chapters of a proposed ACI 350 Code. One of the items balloted and established was to define Environmental Engineering Concrete Structures as including “…ancillary structures for dams, spillways, and channels.” The first ACI 350 Code was finally published in 2001.

The Millennium

The ACI 318 Committee deleted the Alternate Design Method altogether, even from the Appendix, with publication of the ACI 318-02 Code. However, language was added in the Commentary of Chapter 1, General Requirements, of ACI 318-05 that allowed designers to use the Alternate Design Method of the ACI 318-99 and earlier Codes.

Serviceability and durability are both major design considerations for the structural design of hydraulic structures and are just as important as the strength requirements. ACI 318, which is the basis of the strength design procedure of ACI 350 and NRCS TR-67, notes that its serviceability requirements are not adequate for liquid retention structures. ACI 350-01 used the same methods as ACI 318 to address serviceability and durability and simply placed more restrictions on the magnitudes of the z-values to meet the more demanding hydraulic structure environment. ACI 350-06 introduced service durability factors for flexure, shear, and axial tension design in addition to the modified ACI 318 z-values.

Since ACI 350 follows the requirements of ACI 318 by one or two ACI 318 code cycles, the 1999 and 2002 edition of the ACI 318 code caused the ACI 350 committee to look at its durability and strength design calculation procedures for the update to ACI 350-01. Brief
descriptions of the changes in flexural design procedures to address strength and serviceability are described for each code change in the following sections.

ACI 318-99 eliminated the z-value calculation and replaced it with a direct solution for the spacing of reinforcing. This decision was based on extensive laboratory work involving deformed bars that confirmed that crack width at service loads is proportional to steel stress. The significant variables were found to be thickness of concrete cover and the area of concrete in the zone of maximum tension surrounding each individual reinforcing bar. These variables pertain to the detailing of the concrete members. A review of Table 3 below gives a bird’s eye view of how ACI 318-99, ACI 350-01, and NRCS TR-67 compare to each other. As can be seen, durability and serviceability are dealt with in terms of bar spacing and clear cover with ACI 318-99 and in terms of bar spacing and steel stress with ACI 350-01 and TR-67. See Table 3 below:
Table 3. Comparison of ACI 318-99, ACI 350-01, and NRCS TR 67

<table>
<thead>
<tr>
<th></th>
<th>ACI 318-99</th>
<th>ACI 350-01</th>
<th>NRCS TR-67 1980</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Factor</td>
<td>1.4(D+F) + 1.6(L+H)</td>
<td>1.4(D) + 1.7(F+L+H)</td>
<td>1.8(D) + 1.8(L+H)</td>
</tr>
<tr>
<td>Durability Factor</td>
<td>N/A</td>
<td>1.3 – flexure &amp; shear</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.65 – axial tension</td>
<td></td>
</tr>
<tr>
<td>Crack control,</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>normal exposure</td>
<td>$s = \left( \frac{540}{f'_s} \right) - 2.5c_c$</td>
<td>$Z = 115$ kips/in</td>
<td>$Z = 130$ kips/in</td>
</tr>
<tr>
<td>Crack control,</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>severe exposure</td>
<td>N/A</td>
<td>$Z = 95$ kips/in</td>
<td>N/A</td>
</tr>
<tr>
<td>Max spacing, s</td>
<td>18 inches</td>
<td>12 inches</td>
<td>18 inches</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>4,000 psi</td>
<td>4,000 psi</td>
<td>4,000 psi</td>
</tr>
<tr>
<td>$f_y$</td>
<td>60,000 psi</td>
<td>60,000 psi</td>
<td>40,000 psi</td>
</tr>
<tr>
<td>$f_s$</td>
<td>as high as 36,000 psi</td>
<td>27,000 psi</td>
<td>20,000 psi</td>
</tr>
<tr>
<td>$\rho_{\text{design}}$</td>
<td>only upper limit of $0.75\rho_b$</td>
<td>only upper limit of $0.75\rho_b$</td>
<td>$\rho_{shy} = 0.31\rho_b$</td>
</tr>
<tr>
<td>$\rho_{\text{min}}$</td>
<td>0.00333</td>
<td>0.00333</td>
<td>0.00500</td>
</tr>
<tr>
<td>Min T &amp; S ratio</td>
<td>0.0018</td>
<td>0.0030</td>
<td>0.0030</td>
</tr>
</tbody>
</table>
ACI 318-02 changes from the 1999 code were even more dramatic. The concept of Unified Design and net tensile strain were introduced. Actually, both appeared in Appendix B of the 1995 and 1999 ACI 318 Code. The steel ratio based approach that was the standard method of flexural design since the inception of the strength design method, was shifted to Appendix B of the ACI 318-02 Code. The Unified Design method is similar to the Strength Design Method. It uses factored loads and strength reduction factors to proportion the members. The main difference is that in the Unified Design Provisions, a concrete section is defined as either compression-controlled or tension-controlled, depending on the magnitude of the net tensile strain in the layer of reinforcement closest to the tension face of a member. The \( \phi \)-factor is then determined by the strain conditions at a section at nominal strength. Another change for ACI 318-02 was the adoption of ASCE 7 load factors and combinations. In effect, the load factors were reduced slightly. The corresponding \( \phi \)-factors were reduced accordingly, except for the \( \phi \)-factor for tension control design, which is the category for flexural members. The \( \phi \)-factor for tension control design is 0.9 as for the old flexural member designation. The net effect of this revision is an increase in flexural steel stress at service loads. A comparison of ACI 318-02, ACI 350-01, and NRCS TR 67 is shown in Table 4 below.
<table>
<thead>
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<th>ACI 318-02</th>
<th>ACI 350-01</th>
<th>NRCS TR-67 1980</th>
</tr>
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<tbody>
<tr>
<td><strong>Load Factor</strong></td>
<td>1.4(D+F)</td>
<td>1.2(D+F) + 1.6(L+H)</td>
<td>1.4(D) + 1.7(F+ L+H)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.8(D) + 1.8(L+H)</td>
</tr>
<tr>
<td><strong>Durability Factor</strong></td>
<td>N/A</td>
<td>1.3 – flexure and shear</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.65 – axial tension</td>
<td></td>
</tr>
<tr>
<td><strong>Crack control, normal exposure</strong></td>
<td>$s = \left( \frac{540}{f_s} \right) - 2.5c_c$</td>
<td>$Z = 115$ kips/in</td>
<td>$Z = 130$ kips/in</td>
</tr>
<tr>
<td><strong>Crack control, severe exposure</strong></td>
<td>N/A</td>
<td>$Z = 95$ kips/in</td>
<td>N/A</td>
</tr>
<tr>
<td><strong>Max spacing, s</strong></td>
<td>18 inches</td>
<td>12 inches</td>
<td>18 inches</td>
</tr>
<tr>
<td><strong>$f'_c$</strong></td>
<td>4,000 psi</td>
<td>4,000 psi</td>
<td>4,000 psi</td>
</tr>
<tr>
<td><strong>$f_y$</strong></td>
<td>60,000 psi</td>
<td>60,000 psi</td>
<td>40,000 psi</td>
</tr>
<tr>
<td><strong>$f_s$</strong></td>
<td>as high as 36,000 psi</td>
<td>27,000 psi</td>
<td>20,000 psi</td>
</tr>
<tr>
<td><strong>$\rho_{design}$</strong></td>
<td>only upper limit of $\rho_t$</td>
<td>only upper limit of 0.75$\rho_b$</td>
<td>$\rho_{asy} = 0.31\rho_b$</td>
</tr>
<tr>
<td><strong>$\rho_{min}$</strong></td>
<td>0.00333</td>
<td>0.00333</td>
<td>0.00500</td>
</tr>
<tr>
<td><strong>Min T &amp; S ratio</strong></td>
<td>0.0018</td>
<td>0.0030</td>
<td>0.0030</td>
</tr>
</tbody>
</table>
ACI 318-05 changes from the 2002 Code were not substantive. All notations and definitions were moved to Chapter 2. This was an improvement that was needed for uniformity and consistency. The equation for maximum spacing to address crack control was adjusted due to the higher flexural stresses that accompany the reduction in load factors that were introduced in the 2002 Code. Service load flexural stresses could be as high as 40,000 psi. ACI 350 was in the process of finalizing revisions for the next update. Likewise, NRCS was incorporating the proposed ACI 350 revisions into the draft of its new concrete design handbook. ACI 318-05, ACI 350-01, and TR 67 are compared in Table 5 below.
Table 5. Comparison of ACI 318-05, ACI 350-01, and NRCS TR-67

<table>
<thead>
<tr>
<th></th>
<th>ACI 318-05</th>
<th>ACI 350-01</th>
<th>NRCS TR-67 1980</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Factor</td>
<td>1.4(D+F)</td>
<td>1.2(D+F) + 1.6(L+H)</td>
<td>1.4(D) + 1.7(F+L+H)</td>
</tr>
<tr>
<td>Durability Factor</td>
<td>N/A</td>
<td>1.3 – flexure and shear</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.65 – axial tension</td>
<td></td>
</tr>
<tr>
<td>Crack control, normal exposure</td>
<td>$s = 15 \left( \frac{40,000}{f_s} \right) - 2.5c_c$</td>
<td>Z = 115 kips/in</td>
<td>Z = 130 kips/in</td>
</tr>
<tr>
<td>Crack control, severe exposure</td>
<td>N/A</td>
<td>Z = 95 kips/in</td>
<td>N/A</td>
</tr>
<tr>
<td>Max spacing, s</td>
<td>18 inches</td>
<td>12 inches</td>
<td>18 inches</td>
</tr>
<tr>
<td>$f_c$</td>
<td>4,000 psi</td>
<td>4,000 psi</td>
<td>4,000 psi</td>
</tr>
<tr>
<td>$f_y$</td>
<td>60,000 psi</td>
<td>60,000 psi</td>
<td>40,000 psi</td>
</tr>
<tr>
<td>$f_s$</td>
<td>as high as 36,000 psi</td>
<td>27,000 psi</td>
<td>20,000 psi</td>
</tr>
<tr>
<td>$\rho_{design}$</td>
<td>only upper limit of $\rho_t$</td>
<td>only upper limit of 0.75$\rho_b$</td>
<td>$\rho_{shy} = 0.31\rho_b$</td>
</tr>
<tr>
<td>$\rho_{min}$</td>
<td>0.00333</td>
<td>0.00333</td>
<td>0.00500</td>
</tr>
<tr>
<td>Min T &amp; S ratio</td>
<td>0.0018</td>
<td>0.0030</td>
<td>0.0030</td>
</tr>
</tbody>
</table>
ACI 350-06 was introduced with a different look and feel driven by the previous revisions leading up to ACI 318-02 and new research on flexural cracking analysis. The new equation for addressing flexural cracking is based on correlations between clear cover and crack spacing. The crack width model clearly illustrates that the crack spacing and width are functions of the distance between the reinforcing steel. Therefore, crack control is achieved by limiting the spacing of the reinforcing steel. Maximum bar spacing is determined by limiting the crack widths to acceptable limits. The maximum allowable stresses are now specified directly as a function of bar spacing, allowable crack width, and a clear cover of 2 inches. The design crack width depends on the level of environmental exposure and is approximately 0.01 inch for normal exposure and 0.009 inch for severe exposure. The durability factor is now dependent on the yield stress and service level stress of the reinforcement instead of a constant value. This was required to address the use of new load factors and combinations from ASCE 7 that resulted in higher service load stresses. The unified design provisions of ACI 318-02 have been adopted for strength design. ACI 318-05, ACI 350-06, and TR 67 are compared in Table 6 below.
Table 6. Comparison of ACI 318-05, ACI 350-06, and NRCS TR 67

<table>
<thead>
<tr>
<th></th>
<th>ACI 318-05</th>
<th>ACI 350-06</th>
<th>TR-67 1980</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Factor</td>
<td>1.4(D+F)</td>
<td>1.4(D+F)</td>
<td>1.8D+1.8(L+H)</td>
</tr>
<tr>
<td></td>
<td>1.2(D+F)+1.6(L+H)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Durability Factor</td>
<td>N/A</td>
<td>$S_d = \frac{f_{zy}}{f_{ys}} \geq 1.0$</td>
<td>N/A</td>
</tr>
<tr>
<td>Crack control,</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>normal exposure</td>
<td>$s = 15 \left( \frac{40,000}{f_s} \right) - 2.5c_c$</td>
<td>$f_{s,max} = 20 ksi \leq \frac{320}{\beta \sqrt{s^2 + 4(2 + d_s / 2)^2}} \leq 36 ksi$</td>
<td>$Z = 130$ kips/in</td>
</tr>
<tr>
<td>Crack control,</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>severe exposure</td>
<td>N/A</td>
<td>$f_{s,max} = 17 ksi \leq \frac{260}{\sqrt{s^2 + 4(2 + d_s / 2)^2}} \leq 36 ksi$</td>
<td>N/A</td>
</tr>
<tr>
<td>Max spacing, s</td>
<td>18 inches</td>
<td>12 inches</td>
<td>18 inches</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>4,000 psi</td>
<td>4,000 psi</td>
<td>4,000 psi</td>
</tr>
<tr>
<td>$f_y$</td>
<td>60,000 psi</td>
<td>60,000 psi</td>
<td>40,000 psi</td>
</tr>
<tr>
<td>$f_s$</td>
<td>as high as 40,000 psi</td>
<td>based on bar spacing as above</td>
<td>20,000 psi</td>
</tr>
<tr>
<td>$\rho_{design}$</td>
<td>only upper limit of $\rho_t$</td>
<td>only the upper limit of $\rho_t$</td>
<td>$\rho_{shy} = 0.31 \rho_b$</td>
</tr>
<tr>
<td>$\rho_{min}$</td>
<td>0.00333</td>
<td>0.00333</td>
<td>0.0050</td>
</tr>
<tr>
<td>Min T &amp; S ratio</td>
<td>0.00180</td>
<td>0.0030</td>
<td>0.0030</td>
</tr>
</tbody>
</table>
NEH 636 Chapter 30, Concrete Design is drafted and will contain the new design criteria for NRCS concrete hydraulic structures. The criteria will be in accordance with ACI 350 provisions; however, NRCS will place an upper limit of 20,000 psi on flexural steel service level stresses. There are also two other slight departures from ACI 350-06. NRCS criteria will require a maximum steel ratio that is based on balanced design requirements of the old working stress design criteria. This was formerly designated as $\rho_{shy}$ in TR 67. Maximum bar spacing also will be calculated for this allowed steel tensile stress. This is similar to the methodology used in ACI 318. The equation is the same as the ACI 350 equation for $f_{\text{sm}}$, except the maximum flexural stress is known, and the maximum bar spacing is calculated. By requiring a maximum steel ratio based on the working stress balanced design method and using the new hydraulic durability factor, working stress design and strength design are drawn together. The durability factor, which is now based on the yield stress and service level stress, plays an important role in melding the two design concepts together.
Table 7. Comparison of ACI 318-08, ACI 350-06, and Draft NEH 636.30

<table>
<thead>
<tr>
<th></th>
<th>ACI 318-08</th>
<th>ACI 350-06</th>
<th>NEH 636.30</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Load Factor</strong></td>
<td>1.4(D+F)</td>
<td>1.4(D+F)</td>
<td>1.4(D+F)</td>
</tr>
<tr>
<td></td>
<td>1.2(D+F)+1.6(L+H)</td>
<td>1.2 (D+F) + 1.6(L+H)</td>
<td></td>
</tr>
<tr>
<td><strong>Durability Factor</strong></td>
<td>N/A</td>
<td>( S_d = \frac{f_{y}}{f_s} \geq 1.0 )</td>
<td>( S_d = \frac{f_{y}}{f_s} \geq 1.0 )</td>
</tr>
<tr>
<td><strong>Crack control, normal exposure</strong></td>
<td>( s = 15 \left( \frac{40,000}{f_s} \right) - 2.5c_c )</td>
<td>( f_{s,max} = \frac{320}{2s^2 + 4(2 + d_s/2)^2} \leq 36ksi )</td>
<td>( s = \left( \frac{320}{f_s \beta} \right) - 4 \left( 2 + \frac{d_s}{2} \right)^2 )</td>
</tr>
<tr>
<td><strong>Crack control, severe exposure</strong></td>
<td>N/A</td>
<td>( f_{s,min} = \frac{260}{2s^2 + 4(2 + d_s/2)^2} \leq 36ksi )</td>
<td>Special Design required</td>
</tr>
<tr>
<td><strong>Max spacing, s</strong></td>
<td>18 inches</td>
<td>12 inches</td>
<td>12 inches</td>
</tr>
<tr>
<td><strong>f_c</strong></td>
<td>4,000 psi</td>
<td>4,000 psi</td>
<td>4,000 psi</td>
</tr>
<tr>
<td><strong>f_y</strong></td>
<td>60,000 psi</td>
<td>60,000 psi</td>
<td>60,000 psi</td>
</tr>
<tr>
<td><strong>f_s</strong></td>
<td>high as 40,000 psi</td>
<td>based on bar spacing as above</td>
<td>20,000 psi</td>
</tr>
<tr>
<td><strong>( \rho_{design} )</strong></td>
<td>only upper limit of ( \rho_t )</td>
<td>only the upper limit of ( \rho_t )</td>
<td>( \rho_{shy} = 0.546 \rho_b )</td>
</tr>
<tr>
<td><strong>( \rho_{min} )</strong></td>
<td>0.00333</td>
<td>0.00333</td>
<td>0.00333</td>
</tr>
<tr>
<td><strong>Min T&amp;S ratio</strong></td>
<td>0.00180</td>
<td>0.0030</td>
<td>0.0030</td>
</tr>
</tbody>
</table>
Future revisions to ACI 350 and NRCS criteria will focus on the need to address serviceability and durability requirements. Undoubtedly, new crack prediction models will be developed based on new research. Discussions of the return to working stress design have and will continue to occur. However, the concept of cracking being directly proportional to flexural stress will remain. NRCS TR 67 criteria is nearly 30 years old, but is still appropriate because of the limits that were placed on service level stresses and requiring specific load factors and steel ratios. NRCS will follow the guidelines of ACI 350 for the design of NRCS hydraulic structures. Although the 20,000 psi service level stress requirement will continue to be an NRCS staple, future developments in crack control spacing could change as well as other design requirements/methodology being introduced. The focus of this paper has been on serviceability and durability requirement changes in the form of reinforcement distribution. The more things change, the more they stay the same. Service level steel stress is the most important design consideration for crack control in hydraulic/environmental structures. The next revision for ACI 350 is scheduled for 2011 with no planned changes in the serviceability requirements.

And of course “what goes around comes around”, the NRCS in NEH 636 Chapter 30 will continue to accept the well-proven working stress design method as presented in earlier 1964 and 1980 versions of NEH-6.
Abbreviations and Common Terminology
The following are terms defined for use in this document.

\[ a = \text{Depth of equivalent stress block} \]

\[ A = \text{Effective tension area of concrete per bar, determined as the tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as the reinforcement, divided by the number of bars, inches}^2 \]

\[ A_c = \text{Area of the core measured to the outside diameter of the spiral, in}^2 \]

\[ A_g = \text{Gross area of section} \]

\[ A_s = \text{Area of Reinforcement} \]

\[ A_{st} = \text{Total area of longitudinal reinforcement, in}^2 \]

\[ A_{tr} = \text{Total cross-sectional area of all transverse reinforcement that is within the spacing} s \text{ and that crosses the potential plane of splitting through the reinforcement being developed, in}^2 \]

\[ \beta = \text{Ratio of clear spans in long and short directions of two way slabs} \]

\[ b = \text{Width of member} \]

\[ C = \text{Compression Force} \]

\[ c_c = \text{Clear cover from the nearest surface in tension to the surface of the flexural tension reinforcement, in.} \]
\( c_b \) = Smallest side cover measured from the edge of concrete to center of bar or the center-to-center spacing of bars.

\( \mathbf{D} \) = Dead Load

\( d \) = Distance from extreme compression fiber to centroid of tension reinforcement, in

\( d_c \) = Thickness of concrete cover measured from the extreme tension fiber to the center of the longitudinal bar located closest to the extreme fiber, inches

\( \mathbf{E} \) = Earthquake (seismic) Load

\( E_c \) = Modulus of Elasticity of Concrete, psi

\( E_{cb} \) = Modulus of elasticity of beam concrete, psi

\( E_{cs} \) = Modulus of elasticity of slab concrete, psi

\( E_s \) = Modulus of Elasticity of Steel, psi

\( \varepsilon_t \) = Net tensile strain in extreme tension steel at nominal strength

\( f'_c \) = Specified compressive strength of concrete

\( f_r \) = Modulus of rupture of concrete, psi.

\( f_s \) = Calculated stress in reinforcement at service loads calculated by the unfactored moment divided by the product of steel area and the internal moment arm, psi.

\( f_y \) = Specified yield strength of the reinforcement, psi.

\( f_{yt} \) = Specified yield strength of the transverse reinforcement, psi.

\( \mathbf{H} \) = Earth Pressure Load or water pressure in backfill

\( h \) = Overall thickness of member

\( I_b \) = Moment of inertia about centroidal axis of gross section of beam, in\(^4\)

\( I_{cr} \) = Moment of inertia of cracked section, transformed to concrete, in\(^4\).

\( I_g \) = Moment of inertia of gross concrete section about centroid axis and neglecting reinforcement, inches\(^4\).
\( I_s \) = Moment of inertia about centroidal axis of gross section of slab, in\(^4\).
\( k \) = Effective length factor.
\( K tratt \) = Transverse reinforcement factor.
\( L \) = Live Load
\( \ell_d \) = development length for deformed straight bars in tension, in.
\( \ell_n \) = clear span for positive moment or shear and average of adjacent clear spans for negative moment.
\( \lambda \) = Multiplier for additional long term deflection
\( M_a \) = Maximum moment in member, or portion of member, for loading for which deflection is computed, in-lbs.
\( M_{cr} \) = Cracking moment, in-lbs.
\( M_u \) = Required moment strength
\( n \) = number of bars being spliced or developed along the plane of splitting.
\( P_{u} \) = Required axial load strength
\( r \) = Radius of gyration
\( \rho \) = Ratio of tension reinforcement
\( \rho' \) = Ratio of compression reinforcement
\( \rho_b \) = Reinforcement ratio producing balanced strain
\( R_n \) = Nominal strength
\( s \) = maximum center-to-center spacing of transverse reinforcement within \( \ell_d \), in.
\( S \) = Environmental Durability Factors
\( S_2 \) = Service loads
$T = \text{Cumulative effect of temperature, creep, shrinkage, differential settlement and shrinkage compensating concrete}$

$T = \text{Tensile Force}$

$U = \text{Required strength to resist factored loads or related to internal moments or forces}$

$w = \text{Unit weight of concrete, pcf}$

$W = \text{Wind Load}$

$y_t = \text{Distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension, inches.}$

$A = \text{Ratio of flexural stiffness to section flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panels of each side of the beam}$

$\alpha_m = \text{Average value of } \alpha \text{ for all beams along edges of a panel}$

$\alpha_n = \text{Load factors}$

$\Phi = \text{Strength reduction factor}$

$\phi M_n = \text{Design moment strength}$

$\phi P_n = \text{Design axial load strength}$
A. Classes of Concrete

Concrete by definition is a mixture of Portland cement or any other hydraulic cement, fine aggregate, coarse aggregate, water and admixtures.

For each project, careful consideration should be given to requirements for durability, strength and service, costs and availability of materials, work force and equipment required for the concrete that is needed. Additionally, workability and consistency to permit concrete to be worked into forms and around reinforcement during placement without excessive segregation or bleeding should be considered. Finally, the necessity for the concrete to resist ‘special’ exposures should be investigated.

There are ten classes of concrete that have been established for use. They cover the various conditions of design and construction encountered on NRCS projects. The number associated with the class is the specified compressive strength (f’c) of the concrete in pounds per square inch (psi).

Class 5000 or 5000X – Concrete used in special structures; precast or prestressed construction and extreme exposure conditions. (i.e. prestressed piles)

Class 4500 or 4500X – Concrete used in standard types of structures, with severe exposure conditions. (i.e. risers, box conduits, impact basins, waste storage structures)
**Class 4000 or 4000X** – Concrete used in standard types and sizes of structures, with moderate exposure conditions. (i.e. risers, box conduits, impact basins, waste storage structures)

**Class 3000 or 3000X** – Concrete used in simple structures and mass foundations. (i.e. slabs-on-grade, retaining wall footings, concrete in general applications)

**Class 4000M or 3000M** – Concrete used in minor structures (where the concrete quantity is less than 5 cubic yards) and where the location of the concrete will permit easy maintenance and replacement.

**Class 2500 or 2500X** – Concrete used in small structures (built by unskilled labor) and plain concrete construction.

For Class 5000, Class 4500, Class 4000, Class 3000 and Class 2500 concrete the Contractor is responsible for the design of the concrete mix based on the performance criteria specified by the engineer. This is referred to as ‘Method 1’ in NEH Part 642. For Class 5000X, Class 4500X, Class 4000X, Class 4000M, Class 3000X, Class 3000M and Class 2500X concrete the Engineer is responsible for the design and proportioning of the concrete mix which is referred to as ‘Method 2’ in NEH Part 642.

Different cements from the same manufacturer or the same cement from different manufacturers should not be used interchangeably in the same structural element or portion of the work. Additional guidance may be found in ACI 225R.

**B. Acceptable Reinforcing Steel Grades**

Steel reinforcement bars are manufactured from billet steel, rail steel and axle steel.
Deformed reinforcing bars are divided into three grades: Grade 40, 50 and 60. The number associated with the grade is the specified yield strength of the reinforcement (fy) in kips per square inch (ksi). Grade 60 reinforcement is typically used in most structures today as Grade 40 is scarce to not available. However, higher grades of reinforcing steel are available such as Grade 75.

Welded plain wire fabric for concrete reinforcement and welded deformed wire fabric for reinforcement shall have a minimum yield strength of 60,000 psi.

C. Water to Cementitious Material Ratios

The water to cement ratio gives a general indication of the permeability of the concrete. For higher durability, a lower water cement ratio and higher strength concrete should be used. The water to cementitious materials ratio shall be calculated using the weight of cement plus fly ash and other pozzolans, slag, and silica fume. If a special exposure condition is present during the final use of the structure, the following table provides requirements.

<table>
<thead>
<tr>
<th>Exposure Condition</th>
<th>Maximum water-cementitious material ratio by weight</th>
<th>Minimum $f'_{c}$, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete intended to have low permeability when exposed to water, wastewater and</td>
<td>0.45</td>
<td>4000</td>
</tr>
<tr>
<td>corrosive gasses</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete exposed to freezing and thawing in a saturated condition or to deicing</td>
<td>0.45</td>
<td>4500</td>
</tr>
<tr>
<td>chemical</td>
<td></td>
<td></td>
</tr>
<tr>
<td>For corrosion protection of reinforcement in concrete exposed to chlorides in tanks</td>
<td>0.40</td>
<td>5000</td>
</tr>
<tr>
<td>containing brackish water and concrete exposed to deicing chemicals, seawater or</td>
<td></td>
<td></td>
</tr>
<tr>
<td>spray from seawater</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Concrete exposed to sulfate containing solutions or soils that contain significant amounts of sulfate shall conform to Table 3002-2 or shall be mixed using sulfate resistant cement. The concrete shall have a maximum water-cementitious material ratio and minimum compressive strength as listed below.

<table>
<thead>
<tr>
<th>Sulfate Exposure</th>
<th>Water soluble sulfate (SO₄) in soil, percent by weight</th>
<th>Sulfate (SO₄) in water, ppm</th>
<th>Cement Type¹</th>
<th>Maximum water-cementitious ratio, by weight²</th>
<th>Minimum specified compressive strength f’ₜ, psi.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negligible</td>
<td>0.00-0.10</td>
<td>0-150</td>
<td>I, III, IV</td>
<td>0.45</td>
<td>4000</td>
</tr>
<tr>
<td>Moderate³</td>
<td>0.10-0.20</td>
<td>150-1500</td>
<td>II, IP(MS), IS (MS), I (PM)(MS), I (SM)(MS)</td>
<td>0.45</td>
<td>4500</td>
</tr>
<tr>
<td>Severe</td>
<td>0.20-2.00</td>
<td>1500-10,000</td>
<td>V</td>
<td>0.40</td>
<td>5000</td>
</tr>
<tr>
<td>Very Severe⁴</td>
<td>Over 2.00</td>
<td>Over 10,000</td>
<td>V plus pozzolan</td>
<td>0.40</td>
<td>5000</td>
</tr>
</tbody>
</table>

1) Cement Types
   Type I - Ordinary Portland Cement
   Type IP (MS) - Portland Cement (blended cement) with pozzolan, moderate sulfate resistance (equivalent to Type II cement)
   Type IS (MS) - Portland Cement (blended cement) with ground granulated blast slag, moderate sulfate resistance (equivalent to Type II cement)
   Type I (PM) (MS) - Portland Cement (blended cement) with pozzolan, modified, moderate sulfate resistance (equivalent to Type II cement)
   Type I (SM) (MS) - Portland Cement (blended cement) with ground granulated blast slag, modified, moderate sulfate resistance (equivalent to Type II cement)
   Type II – Portland Cement with moderate sulfate resistance
   Type III – High Early strength Portland Cement
   Type IV – Low Heat of Hydration (mass concrete)
   Type V – Sulfate resistant used only in concrete exposed to severe sulfate action.

2) A lower water-cementitious material ratio or higher strength may be required for corrosion protection for concrete exposed to chlorides.

3) Seawater exhibits moderate sulfate exposure characteristics.
4) Additional corrosion barriers such as coatings or liners shall be required for very severe exposures.

Concrete exposed to freezing and thawing or deicing chemicals shall be air-entrained with the air content indicated in Table 3002-3. Tolerance on air content shall be not greater that ± 1.5%.

**Table 3002-3  Total air content for frost-resistant concrete**

<table>
<thead>
<tr>
<th>Nominal maximum aggregate size, in</th>
<th>Air Content, percent</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Severe Exposure²</td>
</tr>
<tr>
<td>3/8</td>
<td>7.5</td>
</tr>
<tr>
<td>½</td>
<td>7</td>
</tr>
<tr>
<td>¾</td>
<td>6</td>
</tr>
<tr>
<td>1</td>
<td>6</td>
</tr>
<tr>
<td>1 ½</td>
<td>5.5</td>
</tr>
</tbody>
</table>

1 See ASTM C33 for tolerance on oversize and various nominal size designations
2 Severe Exposure is where concrete in cold climates will be in continuous contact with moisture prior to freezing, or where deicing salts are used. (e.g. pavements, sidewalks, tanks, etc.)
3 Moderate Exposure is where concrete in cold climates will only occasionally be exposed to moisture prior to freezing, and where no deicing salts are used. (e.g. certain walls, beams, girders or slabs not in contact with soil)

**D. Concrete Testing**

The acceptance criteria for concrete strength are outlined in NEH Part 642, 31 – Concrete for Major Structures, section 23, Acceptance of Concrete Work. ACI 301 and ACI 318 can also be used as references for information regarding standard deviation analysis for concrete compressive strength break tests.

The use of laboratory trial batches or field experience for establishing concrete mix proportions is the primary basis for selecting the required water-cementitious material ratios. ACI 318 utilizes a statistical approach for establishing the target strength of the concrete. The target strength is required to ensure attainment of the specified compressive strength. If an applicable standard deviation from strength tests of the concrete has been developed, the target
strength level for which the concrete must be proportioned is established. Otherwise, concrete material proportioning must be determined to produce sufficiently conservative target strengths to allow for a high degree of variability in strength test results.

Concrete used in tests to determine standard deviation is similar to the specified proposed work if the concrete is made with the same general types of ingredients and reasonable control over material quality and production methods as the specified proposed work. A change in type of concrete, aggregate, or an increase in strength may increase the standard deviation. When there is doubt as to the reliability of the standard deviation, the estimated standard deviation should be conservative (high).

Standard deviations are generally established by at least 30 consecutive tests. Statistical methods provide the tools for assessing the strength test results.

When the concrete production facility has strength tests based on 30 consecutive strength tests that are similar to those expected, the strength used as the basis for selecting concrete proportions for specified concrete compressive strengths less than or equal to 5,000 psi must be the larger of:

\[ f'_{cr} = f'_c + 1.34s_s \]  (3002-1)

\[ f'_{cr} = f'_c + 2.33s_s - 500 \]  (3002-2)

in which:

\[ s_s = \left[ \frac{\sum(x_i - \bar{x})^2}{(n-1)} \right]^{1/2} \]  (3002-3)
where:

\[
\begin{align*}
ss & = \text{sample standard deviation, psi} \\
xi & = \text{individual strength tests} \\
\bar{x} & = \text{average of n strength test results} \\
n & = \text{number of consecutive strength tests}
\end{align*}
\]

If the standard deviation is not known, the required average strength used as the basis for determining concrete proportions must be selected from Table 3002-4.

<table>
<thead>
<tr>
<th>If ( f_c' )</th>
<th>Then ( f_{cr} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_c' &lt; 3,000 )</td>
<td>( f_{cr} = f_c' + 1000 ) (3002-4)</td>
</tr>
<tr>
<td>( 3,000 \leq f_c' \leq 5,000 )</td>
<td>( f_{cr} = f_c' + 1200 ) (3002-5)</td>
</tr>
<tr>
<td>( f_c' &gt; 5,000 )</td>
<td>( f_{cr} = 1.1f_c' + 700 ) (3002-6)</td>
</tr>
</tbody>
</table>

The average levels provided above (and in ACI 318) are intended to provide a level of assurance that the concrete strengths are acceptable under the following circumstances (for concrete 5,000 psi and below): 1) the average of strength tests (3 consecutive tests) is equal to or greater than the specified compressive strength 2) an individual strength is not more than 500 psi below the specified compressive strength.
Mix approval procedures ensure that the furnished concrete meets or exceeds the specified strength requirements (compressive strength). The following steps could be used to review a mix design:

1) Determine the expected standard deviation from past experience. This can be done by examining a record of 30 consecutive tests made from a similar mix. If a similar mix is difficult to find, two similar mixes can be combined to establish a statistical average value of standard deviation (as long as the total of the tests made from these mixes equals or exceeds 30 tests).

2) Use the standard deviation to select the appropriate target strength as described previously.
   a. As an example, if the specified compressive strength, $f'_c$, is 4,000 psi, and the standard deviation is 500 psi. The required average compressive strength is the larger of:
      i. $f'_{cr} = 4,000 \text{ psi} + 1.34 (500) = 4,670 \text{ psi}$
      ii. $f'_{cr} = 4,000 \text{ psi} + 2.33 (500) - 500 = 4,665 \text{ psi}$
         (The required average strength to be used as the basis for selecting concrete mixture proportions must be 4,670 psi.)
   b. If there is no acceptable test record available, the required average compressive strength will have to be at least $f'_{cr} = 4,000 \text{ psi} + 1,200 \text{ psi} = 5,200 \text{ psi}$.

3) Data furnished that documents the mix proposed will give the average strength needed.
   This could consist of
   a. A set of 30 concrete tests (from field concrete) or
   b. Laboratory strength data obtained from a series of trial batches.
If any strength test of field cured cylinders fall below the specified value by more than the value given in the NEH Part 642, Construction Specification 31, or if tests of field cured cylinders indicate deficiencies, steps shall be taken to assure the load carrying capacity of the structure is not jeopardized. The engineer will determine the effects on the load carrying capacity of the structure based on the actual compressive strength. If it is determined that the load carrying capacity of the structure is compromised, tests of cores drilled from the area in question in accordance with Construction Specification 31 and ASTM C42, Method of Obtaining and Testing Drilled Cores and Sawed Beams in Concrete shall be permitted. The cores will be studied to determine their actual compressive strength.

Evaluation and Acceptance of Concrete

Concrete testing is to be performed by qualified personnel with certified knowledge and skills. This includes both the field and lab personnel. The testing frequency is a basic requirement for proportioning concrete on the probabilistic basis. ACI requires that strength tests be performed according to a prescribed minimum frequency as a statistical basis for acceptance. The frequency of tests required:

Minimum Number of Strength Tests Per Day (number of tests must be not less than)

- One test per day
- One test for each 150 cubic yards of concrete placed
- One test for each 5,000 square feet of surface area of slabs or walls placed.

Minimum number of Strength Tests Per Project (number of tests must be not less than)
• Five strength tests from five randomly selected batches or from each batch if fewer than five batches.

• According to ACI, if the total quantity of concrete placed on a project is less than 50 cubic yards, the concrete testing may be waived.

Testing cylinders should be 6 inches in diameter by 12 inches long or 4 inches in diameter by 8 inches long according to ASTM C31, which is the guideline for making concrete test specimens in the field.

The number of cylinders cast for each required test will normally exceed the minimum number required. Per ACI 5.6.2.4, a strength test is to be the average of the strengths of two cylinders made from that same sample of concrete and tested 28 days or at the age designated for the determination of the concrete strength. The additional number of cylinders typically required includes cylinders for information (7 day tests) or field cured cylinders (to check early strength to facilitate form stripping) plus one or two extra, for verification if a cylinder break occurring at the 28 day age test would be below acceptable levels.

The concrete is considered acceptable when the strength level of a class of concrete has met both of the following requirements:

• The average of any three consecutive strength tests equals or exceeds the specified compressive strength.
• No individual strength test (the average of at least two cylinders) falls below the specified compressive strength by more than 500 psi when the compressive strength is 5000 psi or less.
The overall performance of a concrete structure is not only dependent on the ability of the structure to resist the imposed loads through proper sizing of the members and adequate reinforcement for strength, but it is also heavily dependent on the proper detailing of the reinforcement. This section will summarize specific guidelines and code requirements for various aspects of concrete reinforcement detailing. Additionally, the engineer may refer to the ACI Detailing Manual, reported by ACI Committee 315, and the CRSI Manual of Standard Practice for situations not covered in this section.

Table 3003-1 provides the standard geometry and designations for all of the common reinforcing steel sizes.

<table>
<thead>
<tr>
<th>Bar Designation</th>
<th>Cross Sectional Area (in²)</th>
<th>Diameter, ( d_b ) (in)</th>
<th>Unit Weight per Foot (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3 #10</td>
<td>0.11</td>
<td>0.375</td>
<td>0.376</td>
</tr>
<tr>
<td>#4 #13</td>
<td>0.20</td>
<td>0.500</td>
<td>0.668</td>
</tr>
<tr>
<td>#5 #16</td>
<td>0.31</td>
<td>0.625</td>
<td>1.043</td>
</tr>
<tr>
<td>#6 #19</td>
<td>0.44</td>
<td>0.750</td>
<td>1.502</td>
</tr>
<tr>
<td>#7 #22</td>
<td>0.60</td>
<td>0.875</td>
<td>2.044</td>
</tr>
<tr>
<td>#8 #25</td>
<td>0.79</td>
<td>1.000</td>
<td>2.670</td>
</tr>
<tr>
<td>#9 #29</td>
<td>1.00</td>
<td>1.128</td>
<td>3.400</td>
</tr>
<tr>
<td>#10 #32</td>
<td>1.27</td>
<td>1.270</td>
<td>4.303</td>
</tr>
<tr>
<td>#11 #36</td>
<td>1.56</td>
<td>1.410</td>
<td>5.313</td>
</tr>
<tr>
<td>#14 #43</td>
<td>2.25</td>
<td>1.693</td>
<td>7.650</td>
</tr>
<tr>
<td>#18 #57</td>
<td>4.00</td>
<td>2.257</td>
<td>13.600</td>
</tr>
</tbody>
</table>

\(^1\)The nominal dimensions of a deformed bar (diameter and area) are equivalent to those of a plain round bar having the same weight per foot as the deformed bar.
A. Standard Hooks

In certain detailing situations such as for stirrups and ties or where there is not sufficient length to develop a given bar it is necessary to provide standard hooks at the ends of the reinforcement. The ACI Code limitations for standard hooks are presented in Figure 3003-1.

**Figure 3003-1  Standard hook details**

* Bend diameters for primary reinforcement are as follows:
  - #3 thru #8 = 6\(d_b\)
  - #9 thru #11 = 8\(d_b\)
  - #14 and #18 = 10\(d_b\)

** Bend diameters for stirrups and ties are as follows:
  - #3 thru #5 = 4\(d_b\)
  - #6 thru #8 = 6\(d_b\)

B. General Spacing Limits for Reinforcement

There are multiple general guidelines for the proper spacing of reinforcement within a member. These limits were developed to allow concrete to adequately flow between and around
individual bars as long as the limitations on maximum aggregate size given in the ACI Code are adhered to. Development and splice lengths are a function of the clear spacing between bars and spliced bars. Therefore the minimum clear spacing between bars must be used to determine development and splice lengths. The following criteria are illustrated in Figure 3003-2.

- The clear spacing between parallel bars in the same layer of beams or slabs may be as small as the bar diameter, $d_b$, but not less than 1 in or 4/3 the maximum aggregate size.
- For cases with multiple layers of tension reinforcement, each bar must be placed directly above the lower bar with the clear distance between layers not less than 1 in.
- The vertical reinforcement in columns and walls may not have a clear spacing between bars of less than 1.5 $d_b$, 1½ in, or 4/3 the maximum aggregate size.
- The maximum spacing of primary flexural reinforcement in walls and slabs has limitations based on whether the member is part of an environmental or other structure as given below:
  
  - Other: three times the wall or slab thickness but not more than 18 inches (14 inches maximum recommended for ag waste structures).
  - Environmental: two times the wall or slab thickness but not more than 12 in.
C. Concrete Protection for Reinforcement

In order to protect the reinforcement from corrosion due to liquids or gases, minimum cover dimensions have been established in the ACI Code. The classifications of the various cover scenarios differ for non-environmental and environmental structures, and thus, are presented separately in Tables 3003-1 and 3003-2 respectively. It should be noted that according to the ACI Code, if the general building code requires a concrete cover for fire protection that is greater than the values given in the tables, the general building code would govern. Also, the values given in the tables do not apply to bundled bars. Refer to the ACI Code for specific concrete cover criteria for bundled bars.
### Table 3003-2  Minimum cover conditions for other structures

<table>
<thead>
<tr>
<th>Cover Condition</th>
<th>Minimum Cover, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Concrete cast against and permanently exposed to earth</td>
<td>3</td>
</tr>
<tr>
<td>(b) Concrete exposed to earth or weather:</td>
<td></td>
</tr>
<tr>
<td>#6 through #18 bars</td>
<td>2</td>
</tr>
<tr>
<td>#5 bar, W31 or D31 wire and smaller</td>
<td>1-½ ¹</td>
</tr>
<tr>
<td>(c) Concrete not exposed to weather or in contact with ground:</td>
<td></td>
</tr>
<tr>
<td><strong>Slabs, walls, and joists:</strong></td>
<td></td>
</tr>
<tr>
<td>#14 and #18 bars</td>
<td>1-½</td>
</tr>
<tr>
<td>#11 bar and smaller</td>
<td>¾</td>
</tr>
<tr>
<td><strong>Beams and columns:</strong></td>
<td></td>
</tr>
<tr>
<td>Primary reinforcement, ties, stirrups, and spirals</td>
<td>1-½</td>
</tr>
</tbody>
</table>

¹ 2 in. recommended for ag waste structures.

### Table 3003-3  Minimum cover conditions for Environmental Structures and Ag Waste Structures

<table>
<thead>
<tr>
<th>Cover Condition</th>
<th>Minimum Cover, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Concrete cast against and permanently exposed to earth</td>
<td>3</td>
</tr>
<tr>
<td>(b) Concrete exposed to earth, liquid, weather, or cast against a concrete work mat:</td>
<td></td>
</tr>
<tr>
<td><strong>Slabs and joist</strong></td>
<td>2</td>
</tr>
<tr>
<td><strong>Beams and columns:</strong></td>
<td></td>
</tr>
<tr>
<td>Stirrups, spirals, and ties</td>
<td>2</td>
</tr>
<tr>
<td>Primary reinforcement</td>
<td>2</td>
</tr>
<tr>
<td><strong>Walls</strong></td>
<td>2</td>
</tr>
<tr>
<td><strong>Footings and base slabs:</strong></td>
<td></td>
</tr>
<tr>
<td>Formed surfaces</td>
<td>2</td>
</tr>
<tr>
<td>Top of footings and base slabs</td>
<td>2</td>
</tr>
</tbody>
</table>

Note: For environmental structures, cover should be increased in areas of high velocity or abrasive flow.
D. Specific Reinforcement Criteria for Compression Members

NRCS structures generally do not include typical column elements as seen in building construction; therefore, the primary compression members for NRCS structures would be walls designed as short compression members, pedestals, or strut type members between two walls. The ACI Code allows walls to be designed as compression members using the criteria given in Section 3007; however, the requirement for lateral reinforcement as given in this section is removed as long as less than 1% longitudinal steel is used in the wall. If a compression member is not a wall (i.e. an isolated column) then it must meet the lateral reinforcement requirements of this section. Axial compression forces acting on compression members of typical NRCS structures are, in general, less than a force equal to 10% of the 28-day compression strength times the gross cross-sectional area of the member \( (P_u \leq 0.1 f'_c A_g) \). These members should be designed as tension-controlled members taking into account the combined flexure and axial forces acting on the section.

Isolated compression members (columns) are unique in that they require confinement reinforcement, as given in the “Flexure and Axial Loads” chapter of the ACI Code. Confinement reinforcement can also serve as shear reinforcement to resist shear forces acting on the element. Along with these requirements and other more situation specific stipulations given in the “Details of Reinforcement” chapter of the ACI Code, the following general criteria must be met for ties when used.
1. **Ties for use in isolated compression members (i.e. columns)**

   a. All longitudinal bars must be enclosed within lateral ties with the following minimum size restrictions:
      - #3 bar for longitudinal bars #10 and smaller.
      - #4 bar for longitudinal bars #11, #14, #18 or bundled bars.

   b. The vertical spacing of ties must meet the smaller of the following maximum limits:
      - 16 longitudinal bar diameters.
      - 48 tie bar diameters.
      - Least dimension of the compression member.
      - 12 inches (for environmental structures only)

E. **Temperature and Shrinkage Reinforcement**

   The function of temperature and shrinkage reinforcement is not to eliminate cracks but minimize the widths of cracks and ensure the cracks are evenly distributed. Properly laid out temperature and shrinkage steel also serves the important auxiliary function of tying the structure together. Where principal steel is required in only one direction, it shall ordinarily be placed nearer the concrete surface than the temperature steel.

   Due to the differing use classifications between environmental and other structures, the minimum amount of steel required for temperature and shrinkage forces is not the same.
Therefore, the criterion is presented independently for each type of structure. Also, information is given for determining the proper amount of reinforcement to control cracking in slabs on grade.

1. Other Structures

The minimum temperature and shrinkage reinforcement ratios given in this section pertain to the reinforcement that is perpendicular to the main one-way flexural reinforcement in structural slabs and walls. If the reinforcement required by design strength is less than that given in this section, then these criteria would apply in any direction and on any face.

When designing cantilever retaining walls, the engineer must provide the minimum horizontal temperature reinforcement in the wall stem as a ratio of the reinforcement area to the gross concrete area for the wall as given in Table 3003-4.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Minimum shrinkage and temperature reinforcement ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing between movement joints less than or equal to 30 feet</td>
<td>0.0025</td>
</tr>
<tr>
<td>Spacing between movement joints greater than 30 feet</td>
<td>See Table 3003-5</td>
</tr>
</tbody>
</table>

Two-thirds of the steel ratios in Table 3003-4 for retaining walls should be placed in the exposed face with the remaining reinforcement in the back face of the wall. If both faces of the
wall will be exposed for an extended period of time during construction, half of the required temperature steel should be placed at each face of the wall. In addition, the bars must not be spaced more than the lesser of three times the wall thickness or 18 inches (or 14 inches for ag waste storage structures.)

The specific ratios of temperature reinforcement that must be provided based on the gross concrete area for structural slabs, mats, isolated and combined footings, and footings of retaining walls are given in Table 3003-5. The required steel can be divided into two layers with a layer at each face. At least one-third of the steel must be at one face with the remainder at the other face of the member. Should a situation exist where the member is significantly restrained against movements caused by extreme temperature changes or loads due to creep and settlement, then the values in the table may need to be increased to properly control cracking. Typical examples of movement and non-movement joints are shown in Section 3004.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Minimum shrinkage and temperature reinforcement ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slabs where Grade 40 or 50 deformed bars are used</td>
<td>0.0020</td>
</tr>
<tr>
<td>Slabs where Grade 60 deformed bars or welded wire fabric (plain or deformed) are used</td>
<td>0.0018</td>
</tr>
<tr>
<td>Slabs where reinforcement with yield stress exceeding 60,000 psi measured at a yield strain of 0.35 percent is used</td>
<td>$\frac{0.0018 \times 60,000}{f_y}$</td>
</tr>
</tbody>
</table>
Unless specifically dictated elsewhere, all temperature and shrinkage reinforcement must meet the following stipulations:

a. The bars must not be spaced more than the lesser of five times the slab thickness or 18 in (14 inches for ag waste storage structures).

b. The reinforcement must be able to develop the yield strength, \( f_y \), in tension as defined in Section 3009.

2. Environmental Structures and Ag Waste Storage Structures

The minimum temperature and shrinkage reinforcement ratios given in this section pertain to the reinforcement that is perpendicular to the main one-way flexural reinforcement in structural slabs, mat foundations, and walls to include retaining walls. Also, if the reinforcement required by design strength is less than that given in this section, then these criteria would apply in any direction and on any face.

When designing cantilever retaining walls, the engineer must provide minimum temperature and shrinkage reinforcement in the horizontal direction as given in this section. The minimum reinforcement for cantilevered retaining walls in the vertical direction should be provided in accordance with Section 3007.

The specific ratios of reinforcement that must be provided based on the gross concrete area are given in Table 3003-6. Should a situation exist where the member is significantly
restrained against movements caused by extreme temperature changes or loads due to creep and settlement, then the values in the table may need to be increased to properly control cracking. Typical examples of movement and non-movement joints are shown in Section 3004.

<table>
<thead>
<tr>
<th>Length between movement joints, ft</th>
<th>Minimum shrinkage and temperature reinforcement ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Grade 40</td>
</tr>
<tr>
<td>Less than 20</td>
<td>0.0030</td>
</tr>
<tr>
<td>20 to less than 30</td>
<td>0.0040</td>
</tr>
<tr>
<td>30 to less than 40</td>
<td>0.0050</td>
</tr>
<tr>
<td>40 and greater</td>
<td>0.0060*</td>
</tr>
</tbody>
</table>

* To be used when no movement joints are provided unless analysis indicates a greater amount is required.

When required, temperature and shrinkage reinforcement must meet the following stipulations:

a. The joint spacing needs to be multiplied by 1.5 if no more than 50% of the reinforcement passes through the joint.

b. If the section is at least 24 in. thick, the minimum amount of reinforcement may be based on a 12 in thick layer on each face.

c. The bars must not be spaced more than 12 in. apart.

d. The minimum bar size is a #4 bar.
e. At least 1/3 of the required area of steel needs to be distributed along one of the faces.

f. The reinforcement must be able to develop the yield strength, \( f_y \), in tension as defined in Section 3009.

### 3. Non-structural Slabs On Grade

For situations where liquid retention is not necessary, or when the imposed loads do not include heavy storage or racking, the minimum temperature and shrinkage reinforcement for a slab on grade may be determined per the criteria that is presented in ACI 360R. The key for controlling cracks in slabs on grade is striking the right balance between joint spacing and the amount of reinforcement. ACI 360R-06 states that as long as a steel ratio of 0.5% is achieved for the slab cross-section area, that sawcut contraction joints may be eliminated. This option may be useful for slabs that are not protected from the elements and thus could develop problems at the joint locations if they are not properly filled and sealed. As a point of reference, for a 6” thick slab, this would be equivalent to providing #5 bars at 10” in both directions or 6x6-W18xW18 welded wire fabric.

The alternative to providing a 0.5% steel ratio would be to provide 0.1% steel and utilize sawcut contraction joints at the maximum spacing given in Table 3003-7. The reinforcement should be discontinued at expansion joints with a spacing of 120 feet. This will further ensure crack development at the sawcut locations. To eliminate the concern for differential settlement across the expansion joints, sleeved dowel bars could be added at each joint.
in which:

\[ A_s = 0.001bh \]  (3003-1)

\[ A_s = \text{cross-sectional area in square inches of steel per lineal feet.} \]

\[ b = 12 \text{ in. (based on a one foot strip of slab)} \]

\[ h = \text{slab thickness, in.} \]

**Table 3003-7** Maximum saw-cut contraction joint spacing for slabs on grade

<table>
<thead>
<tr>
<th>Slab Thickness, in.</th>
<th>Joint Distance, ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>8</td>
</tr>
<tr>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>7</td>
<td>14</td>
</tr>
<tr>
<td>8</td>
<td>15</td>
</tr>
<tr>
<td>9</td>
<td>15</td>
</tr>
<tr>
<td>10</td>
<td>15</td>
</tr>
<tr>
<td>11</td>
<td>15</td>
</tr>
<tr>
<td>12</td>
<td>15</td>
</tr>
</tbody>
</table>
A. Lap Splices

Lap splice classification, and hence, required lap length is dependent upon, among other things, whether or not the splice is staggered. In a staggered splice, the offset distance of splices shall be the larger of a lap length or 3 feet. In non-prestressed circular tanks designed for hoop tension, splices in hoop reinforcement shall be Class B and the location of splices shall be staggered. Adjacent hoop reinforcement splices shall be staggered horizontally (center of lap below to center of lap above) by not less than one lap length nor 3 feet, and shall not coincide in vertical arrays more frequently than every third bar. Splices that do not meet these requirements shall be considered non-staggered.

Figure 3004-1 Staggered and non-staggered lap splices for horizontal and vertical members

![Diagram showing staggered and non-staggered lap splices](image-url)
B. Joints

1. Movement

Figure 3004-2 Typical slab-on-grade dowelled construction joint detail

**Dowel Bar Table**

<table>
<thead>
<tr>
<th>Slab Thickness</th>
<th>Bar Dia &amp; Length</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>5&quot; - 6&quot;</td>
<td>3/4&quot; x 18&quot;</td>
<td>12&quot;</td>
</tr>
<tr>
<td>7&quot; - 8&quot;</td>
<td>1&quot; x 18&quot;</td>
<td>12&quot;</td>
</tr>
<tr>
<td>9&quot; - 11&quot;</td>
<td>1 1/4&quot; x 18&quot;</td>
<td>12&quot;</td>
</tr>
</tbody>
</table>

*Note: Taken from ACI 308 LR.*
Figure 3004-3  Typical slab-on-grade doweled expansion joint detail

**NOTE:**
1. Thickened slab area required for slabs less than 7 inches thick.

<table>
<thead>
<tr>
<th>DOWEL BAR TABLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>SLAB THICKNESS</td>
</tr>
<tr>
<td>5&quot; - 6&quot;</td>
</tr>
<tr>
<td>7&quot; - 8&quot;</td>
</tr>
<tr>
<td>9&quot; - 11&quot;</td>
</tr>
</tbody>
</table>

*Taken from ACI 302.1R.*
Figure 3004-4  Expansion joint details for structural slabs and walls

2. Non-Movement

FLUSH  DOWELED FLUSH
Figure 3004-5  Saw cut slab-on-grade contraction joint detail
Figure 3004-6  Slab-on-grade keyed construction joint detail

1/2" X 1/2" JOINT FILLER
IN JOINTS EXPOSED TO VIEW

SECTION
Figure 3004-7  Construction joint details for structural slabs and walls

**Notes:**
- Provide waterproof sealant in all required surfaces of construction joints.
- Provide reinforcement for removal of formwork.

**Schedule of Dimensions:**

<table>
<thead>
<tr>
<th>D</th>
<th>C</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>6&quot;</td>
<td>6&quot;</td>
<td>6&quot;</td>
</tr>
<tr>
<td>8&quot;</td>
<td>6&quot;</td>
<td>6&quot;</td>
</tr>
<tr>
<td>10&quot;</td>
<td>6&quot;</td>
<td>6&quot;</td>
</tr>
<tr>
<td>More than 10&quot;</td>
<td>6&quot;</td>
<td>6&quot;</td>
</tr>
</tbody>
</table>
C. Miscellaneous Details

Figure 3004-8  Reinforcing at wall and slab openings

NOTES:
1. Circular opening shown, rectangular opening similar.
2. Extra reinforcing shall be provided in both faces unless typical reinforcing is placed in single layer only.
3. When dowel bars are cut provide dowels each side of opening in accordance with this detail.
Figure 3004-9  Typical wall corner reinforcement

NOTE:
WHERE 3 DIFFERENT BAR SIZES OCCUR AT CORNER, THE CORNER BAR SHALL BE THE LARGER BAR SIZE.

Figure 3004-10  Typical wall dowels
Figure 3004-11  Typical retaining wall section

NOTE:
REINFORCEMENT SIZE AND SPACING TO BE DETERMINED BY THE DESIGNER
Figure 3004-12  Typical box culvert section

NOTE:
REINFORCEMENT SIZE AND SPACING TO BE DETERMINED BY THE DESIGNER
Reinforced concrete design was the first to include the strength design methodology in American structural engineering practice. In fact, strength design concepts have been included in the ACI 318 building code since the 1950’s under the title of “Ultimate Strength Design”, and in 1977 the method was recognized as the preferred approach to concrete design by placing the “Working Stress” (Alternate Design) method in the appendix where it stayed until the 2002 edition of the code. This fact shows the gradual change of the industry to a full implementation of the strength design methodology. However, that does not mean that the “Working Stress” method is not still utilized for concrete design. It is still the predominant method used for environmental structures designed by ACI 350. This is due to the proven serviceability results achieved by the “Working Stress” method. The stresses are limited based on working loads of the structure and not percentages of the ultimate strength; therefore, thicker sections are a result which contributes to less cracking and fewer deflection problems. Another reason why “Working Stress” is still widely used is the fact that many legacy software applications and standard drawings were developed based on this method. For many organizations, it would require a significant expenditure of time for training and financial resources to convert entirely to the strength design methodology. Given the fact that the “Working Stress” method is still in use today, it will be included in Appendix 30A.

A. **Basic Methodology of Strength Design**

As stated above, starting with the 2002 edition of the ACI 318 Building Code, the traditional “Working Stress” design methodology is no longer included within the code except
by reference in the commentary to the 1999 edition of the code and in the appendix of the 2001 edition of the ACI 350 Building Code for environmental concrete structures. Therefore, except for Appendix 30A and relevant design examples in Appendix 30B, this handbook is based entirely on the “Strength Design” methodology. For this reason this section is dedicated to describing the fundamental components of strength design and their methods of application.

For those users of this handbook who come from a working stress design background, strength design concepts may seem complex and laborious to implement into design. However, the basic premise is that strength design focuses on the strength of materials rather than the stress that is produced by imposed loads. The engineer checks for failure that would be caused by specific load results such as bending, axial compression, and shear. This allows for greater precision in determining the factor of safety, by simultaneously factoring the anticipated loads and the ultimate strength of the concrete member under that load.

B. Overview of the Minimum Criteria for Strength Design

As defined in the ACI Code, “The strength design method requires service loads or related internal moments and forces to be increased by specified load factors (required strength) and computed nominal strengths to be reduced by specified strength reduction factors, $\phi$ (design strength).” The basic requirement for strength design may thus be expressed as:
Design strength ≥ Required strength.  \hspace{1cm} (3005-1)

\[ \varphi R_n \geq U \]

1. **Required Strength vs. Design Strength**

Required Strength – Required strength, delineated by the letter \( U \) is obtained by multiplying the service loads by load factors. Load factors provide for excess load effects from sources such as overloads and simplified structural analysis assumptions. The service loads are those that are specified in the general building code for basic loads such as dead, live, and wind. Load factors are given in Section 3005 of this handbook for both environmental and other structures. The basic expression for required strength can be given as follows:

\[ U = \alpha_1 S_1 + \alpha_2 S_2 + ... \] \hspace{1cm} (3005-2)

in which:

\[ \alpha_n \quad = \quad \text{load factors} \]

\[ S_n \quad = \quad \text{service loads or load effects} \]

As a new modification in the 2006 edition of the ACI 350 code, the environmental durability factor, \( (S_d) \), is no longer directly stipulated for design parameters such as flexural, axial or shear strength, but rather it is now a function
of the steel stress and the controlling load combination. The full discussion of how to use this factor is contained in Section 3006.

Design Strength is computed by multiplying the “nominal strength” by a strength reduction factor, \( \phi \) (always less than one). The strength reduction factor accounts for the possibility that small adverse variations in material strengths, workmanship, and dimensions may combine to result in a strength that is less than that specified. The “nominal strength” is calculated in accordance with the ACI Standard in terms of flexure, axial load, shear, and torsion.

In light of these definitions, another way to write Equation 3005-1 is as follows:

\[
\phi R_n \geq U \quad \text{other structures} \tag{3005-3}
\]

\[
\phi R_n \geq S_d U \quad \text{environmental structures}
\]

in which:

\[
\phi = \text{strength reduction factor}
\]

\[
R_n = \text{nominal strength}
\]

\[
U = \text{factored load or load effect}
\]

\[
S_d = \text{environmental durability factor}
\]

C. Strength Design Equations
1. Flexure

The general principles of flexural design are illustrated in Figure 3005-1, which can be used to generate the standard equilibrium equations that are given below. These equations are developed further in Section 3007 for specific reinforcement configurations.

Figure 3005-1 Strain and equivalent stress distribution in rectangular section with tension reinforcement only

The above relationships lead to the development of the following equations:

a. Force Equilibrium

\[ C = T \]
b. Moment Equilibrium

\[ M_n = (C \text{ or } T) \left( d - \frac{a}{2} \right) \]  

(3005-5)

Thus for analyzing a given section the following two equations may be used, assuming the tension steel has yielded:

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

(3005-6)

or

\[ \phi M_n = \phi \left[ 0.85 f_c' ba \left( d - \frac{a}{2} \right) \right] \]  

(3005-7)

2. Combined Flexure and Axial Load

In order to develop the procedures for designing a short compression member it is necessary to recognize that a compression member with moment and axial load applied is...
equal to a compression member with the same axial load applied at a given eccentricity as shown in Figure 3005-2.

Based on this relationship it is now possible to develop the design equations for determining the ultimate strength of a compression member. There are an infinite number of strength combinations of moment and axial load that when plotted lie on a curve known as the column interaction diagram. Refer to Figure 3005-3 for an example.
of a column interaction diagram, which includes demarcations for the areas that define the tension controlled and compression controlled limits and the transition zone.

Figure 3005-3  Strength interaction equation design categories

Design of members subject to flexure and axial load is based on the equations of equilibrium and strain compatibility with the equivalent rectangular concrete compression stress block being assumed as shown in Figure 3005-4. Note that since NRCS structures do not typically utilize compression members with the longitudinal reinforcement enclosed in ties, the effect of the compression reinforcement is ignored in the equilibrium equations.
Figure 3005-4  Strain and stresses of a compression member with an eccentric load

\[ P_n = C - T \]  \hspace{1cm} (3005-8)  
\[ P_n = 0.85 f'_c ba - A_s f_y \]  \hspace{1cm} (3005-9) 

b. Moment Equilibrium about the plastic centroidal axis

\[ M_n = P_n e = C \left( \frac{h}{2} - \frac{a}{2} \right) + T \left( d - \frac{h}{2} \right) \]  \hspace{1cm} (3005-10)  
\[ M_n = P_n e = 0.85 f'_c ba \left( \frac{h}{2} - \frac{a}{2} \right) + A_s f_y \left( d - \frac{h}{2} \right) \]  \hspace{1cm} (3005-11)
3. **Serviceability**

The strength design method not only provides for adequate strength to support the anticipated factored loads; it also includes provisions to assure adequate performance at service load levels. The serviceability provisions can be found within the following sections.

- Deflections ..................................................Section 3006
- Crack Control..............................................Section 3007
- Distribution of reinforcement .....................Section 3007

4. **Special Requirements for Environmental Structures**

Environmental Structures should be designed and constructed to be essentially liquid-tight with minimal leakage under normal service conditions. The liquid-tightness of a structure is reasonably assured when:

- concrete mixture is well proportioned
- concrete is well consolidated and without segregation
- concrete is properly cured
- adequate reinforcing steel is provided
- reinforcing is well distributed to minimize crack widths and depth
- joints are properly spaced, sized, water-stopped, and constructed
- reinforcing steel is properly detailed, fabricated, and placed
- impervious protective coatings are used when required
The specific criteria provided in this document for Environmental Structures represent minimum standards of design. Occasionally, however, factors warrant that the engineer should follow more conservative approaches. An example would be a structure designed for the primary containment of a hazardous material such as agri-chemical facilities or other hazardous materials as defined by the Environmental Protection Agency (EPA). This handbook should not be used for the design of any structure intended for those purposes.
A. Required Strength – Non-Environmental and Environmental Structures

Per ACI 318 the required strength, \( U \), expressed in terms of factored loads or related internal moments and forces is as follows. Due regard must be given to sign convention in determining \( U \) for combinations of loadings, as one type of loading may produce effects of opposite sense to that produced by another type. Consideration must be given to various combinations of loadings to determine the most critical design condition. Required strength may be expressed in several different ways. The engineer has the choice of multiplying the service loads by the load factors before computing the load effects, or computing the individual load effects of the unfactored loads and multiplying individual load effects by their respective load factors.

\[
U = 1.4(D + F)
\]  

\[
U = 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)
\]

\[
U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W)
\]

\[
U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R)
\]

\[
U = 1.2D + 1.0E + 1.0L + 0.2S
\]
\[ U = 0.9D + 1.6W + 1.6H \] (3006-6)

\[ U = 0.9D + 1.0E + 1.6H \] (3006-7)

Note that there are a few exceptions to the above factors and combinations per the “Required Strength” section in the ACI Code. The ones most likely to effect NRCS structures are listed below, but the ACI Code should be referred to for more information.

- It is permissible to use 0.5L for L in Equations 3006-3 through 3006-5 except for garages, places of public assembly and all areas where the live load is greater than 100 lb/ft².
- It is required to set H equal to zero in Equations 3006-6 and 3006-7 if H counteracts W or E. Also, H shall not include lateral earth pressure if that pressure negates other forces.

1. **Environmental Structures – Additional Requirements**

As defined in Section 3000, many of the structures designed by NRCS are considered environmental structures due to their conveyance or storage of water. When designing environmental structures per the “Strength Design” method, the load factors and strength reduction factors as presented for other structures should be used for environmental structures as well. However, an environmental durability factor, \( S_d \), shall be applied to the factored loads or load effects of environmental structures unless they are...
considered compression-controlled as defined in the “Design Strength” section. The environmental durability factor is used to address the various serviceability requirements of environmental structures. Note, that the durability factor shall not be used for designs using working stress where unfactored loads are used to determine stresses. The environmental durability factor is developed below.

\[ S_d = \frac{\phi \gamma_f}{f_s} \geq 1.0 \]  

(3006-8)

in which:

\[ \gamma = \frac{\text{Factored Load}}{\text{Unfactored Load}} \]  

(3006-9)

and \( f_s \) is the allowable tensile stress in the reinforcement due to unfactored loads as given below:

a. Flexural tensile stress (maximum for NRCS environmental structures)

\[ f_s = 20\text{ksi} \]

- Higher values for \( f_s \) are allowed based on Equation 3007-14 presented in Section 3007, Part 5b. However, previous designs of NRCS environmental structures have been based on a steel stress at service loads of 20,000 psi.

b. Direct tensile stress due to direct axial tension
\[ f_s = 20\text{ksi} \]

c. Shear reinforcement tensile stress

\[ f_s = 24\text{ksi} \]

Table 3006-1 shows the calculated environmental durability factor for standard steel stresses and common NRCS loading conditions. Note, that \( S_d \) should be set equal to 1.0 for compression controlled sections.

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Environmental Durability Factor, ( S_d )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flexure or Tension ( f_s = 20\text{ksi} )</td>
</tr>
<tr>
<td>Dead or Fluid ( (\gamma = 1.4) )</td>
<td>1.93</td>
</tr>
<tr>
<td>Dead or Fluid ( (\gamma = 1.2) )**</td>
<td>2.25</td>
</tr>
<tr>
<td>Live or Soil ( (\gamma = 1.6) )</td>
<td>1.69</td>
</tr>
</tbody>
</table>

* Acting alone
** Acting in combination with other loads

2. **Ag Waste Storage Structures – Additional Requirements**

As defined in Section 3000, many of the ag waste storage structures designed by NRCS are considered lower level environmental structures due to their requirement to convey or store liquid and solid wastes and be relatively liquid tight. When designing ag waste storage structures in this category per the “Strength Design” method, the load...
factors and strength reduction factors as presented for other structures should be used. However, an environmental durability factor, $S_d$, shall be applied to the factored loads or load effects, theoretically unless they are considered compression-controlled as defined in the “Design Strength” section. The environmental durability factor is used to address the various serviceability requirements of environmental structures. Note, that the durability factor shall not be used for designs using working stress where unfactored loads are used to determine stresses. The environmental durability factor is developed below.

$$S_d = \frac{\phi_{\gamma}}{f_s} \geq 1.0$$ \hfill (3006-8)

in which:

$$\gamma = \frac{\text{Factored Load}}{\text{Unfactored Load}}$$ \hfill (3006-9)

and $f_s$ is the allowable tensile stress in the reinforcement due to unfactored loads as given below:

d. Flexural tensile stress (maximum for NRCS ag waste storage structures)

$$f_s = 36ksi$$

- Higher values for $f_s$ are not allowed in order to meet serviceability requirements.

e. Direct tensile stress due to un-factored direct axial tension
\[ f_s = 30ksi \]

f. Shear reinforcement tensile stress due to un-factored loads

\[ f_s = 36ksi \]

Table 3006-1 shows the calculated environmental durability factor for standard steel stresses and common NRCS loading conditions. Note, that \( S_d \) should be set equal to 1.0 for compression controlled sections.

**Table 3006-1 Standard environmental durability factors for flexure and shear**

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Environmental Durability Factor, ( S_d )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flexure ( f_s = 36ksi ) ( \phi = 0.90 )</td>
</tr>
<tr>
<td>Dead or Fluid (( \gamma = 1.4 ))</td>
<td>1.07</td>
</tr>
<tr>
<td>Dead or Fluid (( \gamma = 1.2 ))**</td>
<td>1.25</td>
</tr>
<tr>
<td>Live or Soil (( \gamma = 1.6 ))</td>
<td>1.00</td>
</tr>
</tbody>
</table>

* Acting alone
** Acting in combination with other loads

**B. Design Strength – Non-Environmental/Other and Environmental Structures**
Per the ACI Code the design strength is obtained by multiplying the nominal strength by the following strength reduction factors. The strength reduction factor, $\phi$, represents an attempt to take into account the following issues.

- To allow for under-strength due to variations in material strengths and dimensions
- To allow for inaccuracies in design equations
- To reflect the ductility and failure mode of the member under the type of loads under consideration
- To reflect the importance of the member in the structure.

a. Tension-controlled sections $\phi = 0.90$

b. Compression-controlled sections
   - Other than spiral reinforcement $\phi = 0.65$

(Note: refer to the “Design strength” section in the ACI Code and Figure 3006-1, with accompanying discussion below, for members in which the net tensile strain is between the limits for compression-controlled and tension-controlled sections, which allows for a linear increase in $\phi$.)

c. Shear and torsion $\phi = 0.75$

d. Bearing on concrete (except for post-tensioned anchorage)
zones and strut-and-tie models) \( \phi = 0.65 \)
e. Post-tensioned anchorage zones \( \phi = 0.85 \)
f. Strut-and-tie models or deep beams \( \phi = 0.75 \)

The exact code definitions from the “Flexure and Axial Loads” chapter of ACI 318 are given below for Tension-controlled, Compression-controlled, and the Transition region along the steel strain equivalents.

The exact code definitions from the “Flexure and Axial Loads” chapter of ACI 318 are given below for Tension-controlled, Compression-controlled, and the Transition region along the steel strain equivalents.
with a graphical explanation in Figure 3006-2. A further discussion on this topic is given in the “Notes on ACI 318-08 Building Code Requirements for Structural Concrete,” which is produced by the Portland Cement Association (PCA) hereinafter referred to as the PCA Notes.

**Compression-controlled** – “Sections are compression-controlled when the net tensile strain in the extreme tension steel is equal to or less than 0.002 at the time the concrete in compression reaches its strain limit of 0.003.

**Tension-controlled** – “Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 just as the concrete in compression reaches a strain of 0.003.

**Transition region** – Sections with net tensile strain in the extreme layer of tension steel between 0.002 and 0.005 constitute a transition region between compression-controlled and tension-controlled sections. The $\phi$ -factor varies from 0.65 to 0.90 in this region as shown in Figure 3006-2.

**Figure 3006-2** Tension-controlled and compression-controlled strain conditions
C. Deflection Limits

To serve its intended function, a structure must be safe and serviceable throughout its design life. Serviceability and durability require that deflections and attendant cracking be kept within allowable limits. With strength design and the use of high strength steels, there is a tendency toward shallow sections. Shallow sections produce larger deflections. Hence, deflections assume an increased importance in strength design procedures.

The deflections of interest are those that occur at service load levels. They may be deflections that occur immediately on application of load or they may be long-term deflections caused by shrinkage and by creep under sustained load. Under service loads the steel and concrete stresses are essentially within their elastic ranges so that deflections that occur immediately can be calculated by methods based on elastic behavior.
Two approaches are provided for controlling deflections. Deflection requirements are considered satisfied if at least a specified minimum thickness for the type of member is used. For members that do not meet the minimum thickness criteria, deflections must be calculated to verify they are within acceptable limits. However, the limits on member thicknesses presented in the ACI Code may not be applicable to many NRCS structures due to the types and magnitudes of typical NRCS structure loadings.

Most NRCS structures, when designed using strength design criteria of this document, are not generally controlled by deflection. This is due to the careful specification of strength and serviceability factors that minimize the negative effects of deflections as well as cracking. If excessive deflection would be detrimental to the structure, methods for calculating the deflection are provided in the ACI Code.

One of the strength and serviceability requirements enforced for NRCS structures is the maximum steel ratios determined as a percentage of the balance steel ratio, $\rho_b$. Table 3006-2 provides the maximum steel ratios to be used for typical NRCS structures. The use of steel ratio limits greater than those presented will require the investigation of deflection and distribution reinforcement in accordance with the requirements of this document. For additional information showing how the steel ratios in Table 3006-2 relate to the net tensile strain requirements of strength design see Section 3007 of this handbook.
Table 3006-2  Maximum steel ratios with Grade 60 steel

<table>
<thead>
<tr>
<th>$f'_c$ (psi)</th>
<th>Other Structures</th>
<th>Environmental Structures</th>
</tr>
</thead>
<tbody>
<tr>
<td>3000</td>
<td>*0.5 $\rho_b$</td>
<td>-----</td>
</tr>
<tr>
<td>4000</td>
<td>*0.5 $\rho_b$</td>
<td>0.546 $\rho_b$</td>
</tr>
</tbody>
</table>

*Use 0.3 $\rho_b$ for ag waste structures.
For the design or investigation of beams, walls, and slabs that are subjected to flexure and/or axial loads, two conditions must be satisfied at nominal strength.

- Static equilibrium of compressive and tensile forces must be achieved.
- Stress and strain must be compatible.

A. Flexure

For rectangular members subjected to flexure, the following condition must be satisfied, along with the deflection limitations discussed in Section 3005 and the distribution of reinforcement for crack control that is discussed later in this section.

\[ \phi M_n \geq M_u \]  

(3007-1)

in which:

\[ \phi M_n = \text{design strength} \]

\[ M_u = \text{required strength} \]
1. **Design Assumptions**

When designing for flexure it is important to note the following assumptions that are relied upon for the development of the standard design equations that are discussed later in this section.

a. Sections perpendicular to the axis of bending which are plane before bending remain plane after bending.

   • This is another way of stating that the strain distribution is linear.

b. The strain in the reinforcement is equal to the strain in the concrete at the same level.

   • This is necessary because the steel and the concrete must act compositely to carry the applied load.

c. The stresses in the concrete and reinforcement can be computed from the strains using stress-strain curves for concrete and steel.

d. The tensile strength of concrete is neglected in flexural strength calculations.
e. The compressive strain limit for concrete is assumed to be 0.003.

f. The compressive stress distribution for concrete may be assumed to be rectangular, trapezoidal, parabolic, or any other shape that results in prediction of strength in substantial agreement with the results of comprehensive tests.

- For simplification of the design equations, a rectangular compressive stress block will be assumed for the analysis and design of NRCS structures. The average stress of the compressive block will be taken as $0.85f_c$.

g. Modulus of elasticity in psi, shall be as follows:

\[ E_s = 29,000,000 \quad (3007-2) \]

\[ E_c = w_c^{1.5} 33 \sqrt{f_c} \quad (3007-3) \]

in which:
\[ w_c = \text{unit weight of concrete, pcf} \]
\[ f'_c = \text{compressive strength of concrete, psi} \]

for normal weigh concrete, the following relationship may be used:

\[ E_c = 57,000 \sqrt{f'_c} \]  \hspace{1cm} (3007-4)

h. Stress block factor shall be as follows:

\[ \beta_i = 0.85 - 0.05\left(\frac{f'_c - 4000}{1000}\right) \geq 0.65 \]  \hspace{1cm} (3007-5)

for \( 2500 \leq f'_c \leq 4000 \) the stress block factor, \( \beta_i \) is equal to 0.85

2. General Principles and Requirements

The general principles of flexural beam design are based on the assumptions mentioned above and can be illustrated as shown in Figure 3007-1.
a. Design Equations

There are various methods for determining the most efficient size and amount of steel required for concrete beams. Design aids are published and may be purchased for use as well as design software. Another option, aside from hand calculations, is to develop a spreadsheet program using software such as Microsoft Excel. This allows for the iterative processes to be performed by the computer and can be time saving and thus cost reducing. However, as with any other computer design program the assumptions made by the program must be fully understood and sound engineering judgment must be employed by the design engineer when interpreting the results.
Rectangular Section with Tension Reinforcement Only

Assuming the tension steel has yielded, the basic method for rectangular beams with tension reinforcement only is to solve directly for the required area of steel for a given cross section and loading. This is accomplished by setting the design strength equal to the required strength and solving the following quadratic equation.

\[
\phi M_u = M_u = \phi \left( A_s f_y \left( d - \frac{A_s f_y}{0.85 f_c b} \right) \right)
\]

Which can be reduced to:

\[
(A_s)^2 \left[ \frac{\phi f_y^2}{1.7 f_c b} \right] - (A_s) \left[ \phi f_y d \right] + [M_u] = 0
\]

\[
(x^2) \left[ a \right] - (x) \left[ b \right] + \left[ c \right] = 0 \quad \text{(Quadratic form)}
\]
With the standard solution from algebra, the solution for area of steel becomes:

\[
A_s = \frac{bd \cdot 0.85 \cdot f_c' \left(1 - \sqrt{1 - 2 \cdot \frac{M_u}{\phi \cdot 0.85 \cdot f_c' \cdot bd^2}}\right)}{f_y}
\]  

(3007-8)

Once the required area of steel is calculated it is necessary to determine whether the section is tension controlled, compression controlled, or in the transition zone, so that the proper strength reduction factor, \(\phi\), from Section 3006 Part B, can be applied and the final moment capacity checked. The steel ration must also be checked to ensure it is less than or equal to the maximum value presented in Table 3006-2.

There are multiple methods for determining whether a section is tension controlled, compression controlled, or in the transition zone. One such method is given below. Note: refer to Figure 3007-1 for graphical depiction of “\(a\)” and “\(d_t\)”. 

\[
\frac{a_{\text{ctl}}}{d_t} = 0.600 \beta_1
\]

(3007-9)

\[
\frac{a_{\text{ctl}}}{d_t} = 0.375 \beta_1
\]

(3007-10)

If \(\frac{a}{d_t} \leq \frac{a_{\text{ctl}}}{d_t}\) then the section is tension controlled and \(\phi = 0.90\).
If \( \frac{a_{det}}{d_t} < \frac{a}{d_t} < \frac{a_{det}}{d_t} \), then the section is in the transition zone and \( \phi \) is determined by Section 3006, Part B.

**Determine if the tension steel yields as assumed:**

\[
\frac{a_b}{d} = \beta_i \frac{87000}{87000 + f_y}
\]

(3007-11)

If \( \frac{a}{d} \leq \frac{a_b}{d} \), then the tension steel yields and the assumption was correct.

After determining the proper strength reduction factor, it is necessary to check the final moment capacity, \( \phi M_n \) as given in Equation 3007-6.

**Rectangular Section with Compression Reinforcement**

In design, the addition of compression steel is often ignored since it does not significantly increase the moment capacity of the member; however, there are some benefits for using compression reinforcement such as:

- smaller long term deflections,
• increased ductility, and
• the ability to change a section from compression controlled to tension controlled.

If flexural members are designed utilizing compression reinforcement, they are required to have all longitudinal reinforcement enclosed within closed ties in accordance with ACI requirements. In general, the design of wall and slab type elements of NRCS structures should not include compression reinforcement to develop the required strength of a member. However, if a situation arises where the use of compression reinforcement is required, the engineer should reference the ACI Code and other design handbooks for guidance on how to incorporate compression reinforcement into the design.

3. Maximum Reinforcement

The maximum amount of reinforcement for flexural members in all structures is limited to that which will have a net tensile strain, \( \varepsilon_t \), equal to 0.005. A lower net tensile strain limit of 0.004 is allowed for flexural members or flexural members with a factored axial load less than \( 0.10f'cA_g \).

It is recommended that the lower strain limit for flexural members of NRCS structures be based on a net tensile strain of 0.005 instead of
0.004. The latter is in the transition zone; therefore, the strength reduction factor, $\phi$, would be less than 0.90, which subsequently reduces the strength gained by having a higher reinforcement ratio. NRCS flexural members are to be designed with a net tensile strain no lower than 0.005.

Table 3007-1 provides relationships between the traditional steel ratios and the current net tensile strain, $\varepsilon_t$, for sections with one layer of tensile reinforcement only. The table values were developed based on 3,000 and 4,000 psi concrete and 60,000 psi steel. Note, that the tables contain recommended minimum net tensile strain values and upper limit steel ratios for both other and environmental structures. These values, in conjunction with load factors and environmental durability factors, have been provided to ensure that the required serviceability limit states will not be exceeded. Even though the steel ratios and the corresponding net tensile strains presented in Table 3007-1 are based on a single layer of tension steel, they can be used for members containing multiple layers of tension steel. However; it should be noted that in members containing multiple layers of tension steel the net tensile strain will be higher than the values shown in Table 3007-1, which is conservative.
### Table 3007-1  Relationship between steel ratio and net tensile strain (one layer of tension steel only)

<table>
<thead>
<tr>
<th>Steel Ratio</th>
<th>Net Tensile Strain, $\varepsilon_t$</th>
<th>Steel Ratio</th>
<th>Net Tensile Strain, $\varepsilon_t$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho_b = 0.0214$</td>
<td>0.00207</td>
<td>$\rho_b = 0.0285$</td>
<td>0.00207</td>
</tr>
<tr>
<td>$0.6\rho_b = 0.0128$</td>
<td>0.00545</td>
<td>$0.6\rho_b = 0.0171$</td>
<td>0.00545</td>
</tr>
<tr>
<td>*$0.5\rho_b = 0.0107$</td>
<td>0.00714</td>
<td>*$0.546\rho_b = 0.01556$</td>
<td>0.00629</td>
</tr>
<tr>
<td>**$0.5\rho_b = 0.0107$</td>
<td>0.00714</td>
<td>**$0.5\rho_b = 0.0143$</td>
<td>0.00714</td>
</tr>
<tr>
<td>$0.375\rho_b = 0.0080$</td>
<td>0.01052</td>
<td>$0.375\rho_b = 0.0107$</td>
<td>0.01052</td>
</tr>
<tr>
<td>***$0.30\rho_b = 0.00642$</td>
<td>0.01388</td>
<td>***$0.30\rho_b = 0.00855$</td>
<td>0.01390</td>
</tr>
</tbody>
</table>

* Maximum for “environmental” Structures  
** Recommended maximum for “Other” Structures  
*** Maximum for ag waste structures

#### 4. **Minimum Reinforcement** (applies to both other and environmental structures except where noted otherwise)

The minimum amount of tensile reinforcement for flexural members shall be that given in Equation 3007-12 except as modified by Parts 1, 2, 3, and 4.

\[
A_{s,\text{min}} = \frac{3\sqrt{f'_c}}{f_y} b_w d \geq \frac{200 b_w d}{f_y} \quad (3007-12)
\]
1. For statically determinate members with a flange in tension, the minimum area of steel shall be greater than or equal to that given above with \( b_w \) being replaced with the smaller of \( 2b_w \) or the flange width.

2. If the amount of reinforcement provided at every section is one-third greater than the amount required by analysis, then the minimum steel requirements given in Equation 3007-12 need not be satisfied.

3. For structural slabs, mats, walls, and footings of other structures with a constant thickness, the minimum amount of reinforcement in the direction of span must be according to the limits given in Table 3003-4.

4. For structural slabs, mats, walls, and footings of environmental structures with a constant thickness, the minimum amount of reinforcement in the direction of span must be according to the limits given in Table 3003-5

5. Distribution of Flexural Reinforcement

   a. Other Structures
Large crack widths are unsightly and allow the entrance of water or other corrosive liquids which can result in corrosion of the reinforcement. To assure protection of the reinforcement against corrosion, well distributed fine cracks are preferable to a few wide cracks. When reinforcing is used at high service load stresses, excessive flexural crack widths may be expected unless adequate precautions are taken in detailing the reinforcement.

Size and spacing of flexural cracks are functions of steel stress at service load levels, amount of concrete cover, and the area of concrete surrounding and tributary to each individual reinforcing bar. The importance of controlling cracking increases with the exposure or environment class of the structural component.

Flexural tension reinforcement at both maximum positive and maximum negative moment locations, in beams and one-way slabs, shall be distributed so that the maximum spacing, $s$, shall be that given below.

$$s = 15 \left( \frac{40,000}{f_s} \right) - 2.5c_c \leq 12 \left( \frac{40,000}{f_s} \right)$$  \hspace{1cm} \text{(3007-13)}$$

in which:
\( f_s = \) calculated stress in reinforcement at service loads calculated by the unfactored moment divided by the product of steel area and the internal moment arm, or in lieu of such calculations may be taken as \( 2/3f_y \), psi, when the recommended maximum steel ratio is used for the design of the member.

\( c_c = \) the least distance from the surface of the reinforcement steel to the tension face, in.

Flexural cracking behavior of two-way slabs is significantly different from that in one-way members. For the same total load, the crack widths in two-way action will usually be less than those experienced in one-way action. The maximum spacing of reinforcement in two-way slabs is limited to two times the slab thickness.

Refer to the ACI Code for other more specific flexural reinforcement distribution criteria, such as providing skin reinforcement in the webs of flexural members that are greater than 36 inches deep, but do not meet the requirements of a deep beam as given in Section 3011.

b. Environmental Structures

Crack widths in environmental structures are of a greater concern due to the presence of liquids in contact with environmental structures. To
assure protection of the reinforcement against corrosion, crack widths should be minimized and should be evenly distributed across the span of the flexural member. Many fine cracks are preferable to a few wide cracks. When reinforcing is used at high service load stresses, excessive flexural crack widths should be expected unless adequate precautions are taken in detailing the reinforcement.

Width and spacing of flexural cracks are functions of steel stress at service load levels, amount of concrete cover, and the area of concrete surrounding and tributary to each individual reinforcing bar. The importance of controlling cracking increases with the exposure or environment class of the structural component.

Cross sections at both maximum positive and maximum negative moment locations of flexural members shall be proportioned so that in normal environmental exposure areas:

\[
f_s = \frac{320}{\beta_g \sqrt{s^2 + 4 \left(2 + \frac{d_d}{2}\right)^2}}
\]

but need not be less than 20 ksi for one-way and 24 ksi for two-way members. For the purposes of this section, two-way members are defined as those having an aspect ratio (long span to short span) of less than 2.0.
For simplification, it is permitted to use the value 25 for the term
\[ 4\left(2 + \frac{d_h}{2}\right)^2 \]
and to set the strain gradient amplification fact, \( \beta_g \), equal to
1.2 for \( h \geq 16 \) inches and 1.35 for \( h < 16 \) inches.

Equation 3007-14 can be written in terms of the maximum spacing as follows:

\[
s = \frac{\left(\frac{320}{f_s \beta_g}\right)^2 - 4 \left(2 + \frac{d_s}{2}\right)^2}{4 \left(2 + \frac{d_s}{2}\right)^2} \tag{3007-15}
\]

Table 3007-2 provides the calculated maximum spacing using Equation 3007-15 with the steel tensile stress equal to 20 ksi.

<table>
<thead>
<tr>
<th>Bar Size, no.</th>
<th>Nominal Diameter, in.</th>
<th>Maximum Spacing, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( \beta_g = 1.2 )</td>
</tr>
<tr>
<td>3</td>
<td>0.375</td>
<td>12.5</td>
</tr>
<tr>
<td>4</td>
<td>0.500</td>
<td>12.4</td>
</tr>
<tr>
<td>5</td>
<td>0.625</td>
<td>12.3</td>
</tr>
<tr>
<td>6</td>
<td>0.750</td>
<td>12.1</td>
</tr>
<tr>
<td>7</td>
<td>0.875</td>
<td>12.0</td>
</tr>
<tr>
<td>8</td>
<td>1.000</td>
<td>11.9</td>
</tr>
<tr>
<td>9</td>
<td>1.128</td>
<td>11.8</td>
</tr>
<tr>
<td>10</td>
<td>1.270</td>
<td>11.6</td>
</tr>
<tr>
<td>11</td>
<td>1.410</td>
<td>11.5</td>
</tr>
<tr>
<td>14</td>
<td>1.693</td>
<td>11.1</td>
</tr>
<tr>
<td>18</td>
<td>2.257</td>
<td>10.3</td>
</tr>
</tbody>
</table>
Flexural cracking behavior of two-way slabs is significantly different from that in one-way members. For the same total load, the crack widths in two-way action will usually be less than those experienced in one-way action. The maximum spacing of reinforcement in two-way slabs is limited to the following:

\[ 2(\text{slab thickness}) \leq 12 \text{ in} \quad (3007-16) \]

Refer to ACI 350 for other more specific flexural reinforcement distribution criteria, such as providing skin reinforcement in the webs of beams that are greater than 36 inches deep, but do not meet the requirements of a deep beam.

B. Axial

When designing concrete structures there are various situations that occur in which only a concentric axial load, \( P \), exists on short, symmetrical, non-slender compression member and no moment is present in either end of the compression member. When this situation occurs the compression member must satisfy the following condition as well as be designed by the criteria in Parts 1, 2, and 3. These compression members are similar to standard building columns in how they are designed, but for NRCS structures they typically consist of wall/slab type elements.

\[ \phi P_n \geq P_u \quad (3007-17) \]
in which:

\[ \phi P_n = \text{design strength} \]

\[ P_u = \text{required strength} \]

1. **Design Assumptions**

As stated previously, in order to design a short compression member per this section, there may not be any moment present in the compression member. In concrete construction this is achieved through proper detailing of the connections of the compression member at either end to ensure no moment transfer from connecting elements.

The ACI Code provides the following maximum compressive design strength for members with longitudinal reinforcement with or without tie reinforcement meeting the requirements given in Section 3003.

\[ \phi P_{n(max)} = 0.80 \phi \left[ 0.85 f' \left( A_{sl} - A_x \right) + f_y A_x \right] \]  \hspace{1cm} (3007-18)

in which:

\[ A_{sl} = \text{total area of longitudinal reinforcement, in}^2 \]
Refer to the “Design dimensions for compression members” section of the ACI Code for specific geometric limitations on compression members. Also, if spiral reinforcement is to be used, refer to the ACI Code for a different equation that is used to calculate the maximum compressive design strength.

2. Limits of Reinforcing for Compression Members

Since the majority of compression members used in NRCS structures are wall type elements, the limits provided in this section are based on the wall criteria from the ACI Code. These criteria produce walls that typically have steel areas that are less than 0.01 times the gross concrete area and therefore do not require that the longitudinal reinforcement be enclosed by lateral ties.

a. Other Structures

Minimum ratio of longitudinal reinforcement area to gross concrete area, $\rho$, shall be as follows:

- 0.0012 for deformed bars not larger than No. 5 and $f_y$ not less than 60 ksi; or
- 0.0015 for other deformed bars; or
- 0.0012 for welded wire reinforcement not larger than W31 or D31.

b. Environmental Structures
Minimum ratio of longitudinal reinforcement area to gross concrete area, \( \rho_r \), shall be 0.0030.

b. Ag Waste Structures

Minimum ratio of longitudinal reinforcement area to gross concrete area, \( \rho_r \), shall be 0.0030.

C. Combined Flexure and Axial Loads

Often times a member must be designed that has both flexural and axial loads that need to be resisted. As with flexural members, the design strength must equal or exceed the required strength. Therefore, the necessary moment strength and simultaneously necessary nominal axial force strength must be determined to be greater than the factored moment and axial load as shown below. Most compression members are considered “short” in that they are stocky enough that buckling of the compression member is not likely, as it would be for slender building columns (columns where secondary or magnified moments need to be considered).

\[
\left( \phi P_u, \phi M_u \right) \geq \left( P_u, M_u \right)
\]  

(3007-19)

With this in mind, it is necessary to determine if a particular compression member meets the criteria for being non-slimber (short). This determination is based on whether the column is considered to be part of a non-sway frame or a sway frame, conditions that are otherwise known as braced and unbraced respectively. The following may serve as basic definitions of these two conditions. However, the majority of NRCS structures will be non-slimber and thus this determination will not need to be made. If a situation is encountered where a compression
member is found to be slender, the engineer should reference the criteria in the ACI Code for instructions on the design of slender compression members.

**Braced** = a frame in which relative movement of the ends of a compression member transverse to the axis of the member are prevented.

\[
k \leq 1.0
\]

**Unbraced** = a frame in which relative movement of the ends of a compression member transverse to the axis of the member is possible and restraint is provided only by the rigidity of the joints and the stiffness of interacting beams and columns.

\[
k > 1.0
\]

in which:

\[
k = \text{effective length factor}
\]

1. **Short Compression Member Design**

The basis for the strength design methodology used to develop the interaction diagram given in Figure 3007-2 is provided in Section 3005. This section deals with the development of the equations used to create an interaction diagram for a given compression member geometry and loading conditions.
When designing a compression member the interaction diagram can be developed from a given geometry and then the actual moment and axial load may be plotted to determine if they fall within the curve. If so, then the compression member is adequate to resist the load. If the actual moment and axial load fall on the curve or outside of it, then the compression member properties must be adjusted. To greatly simplify this procedure multiple design aids have been developed. Some of the aids require the engineer to select a trial size and steel ratio and then utilize an interaction diagram for the ratio and reinforcement configuration desired to check the final adequacy of the compression member. Another design aid produced by the Concrete Reinforcing Steel Institute (CRSI) has essentially solved the interaction equations in tabular form for virtually any square, rectangular, or circular shaped...
compression member for given reinforcement sizes. Aside from design aids, many versions of computer software have been developed to design compression members, which perform all of the calculations automatically and can even iterate and find the most efficient cross section for the design. However, as with any software, the engineer’s knowledge and understanding of the program’s assumptions and methods is imperative.

In lieu of software or design aids, the engineer may wish to plot their own interaction diagrams. For a rectangular section this can be done using the following procedure. An understanding of this procedure is important for NRCS structures since most of the compression members that need designed do not lend themselves well to the standard building type column application for computer software. Refer to Figure 3007-3 for determining the location of some of the variables included in the equations.

**Figure 3007-3  Strain and stresses of a compression member with an eccentric load**
a. Calculate the pure axial capacity.

\[ \phi P_{n,\text{max}} = 0.80 \phi \left[ 0.85 f'_c \left(A_g - A_x \right) + f_y A_x \right] \] (3007-20)

This point can be plotted on the vertical (ordinate) axis.

b. Calculate the pure moment capacity with moments taken about the centerline of the section.

\[ M_o = \left[ 0.85 f'_c ab \left( \frac{h}{2} - \frac{a}{2} \right) + A'_s f_y \left( \frac{h}{2} - d' \right) + \left(A_s - A'_s \right) f_y \left(d - \frac{h}{2}\right) \right] \] (3007-21)

Note that ignoring the effects of the compression steel, \( A'_s \), in this step has little effect on the result of the interaction diagram, but for NRCS structures this is common since the majority of compression members are wall type elements without ties and a steel ratio less than 0.01 times the gross area of the concrete section. Therefore, for simplification, it is recommended that \( A'_s \) be set equal to zero in Equation 3007-21.

c. Calculate the balanced failure and compression controls limit where the steel in tension yields and the concrete in compression reaches its strain.
limit of 0.003 simultaneously. For further simplification, assume that the compression steel is at yield strength so that $f'_s = f_y$. A more precise interaction diagram can be obtained by determining if the stress in the compression steel is less than yield strength. If this is the case, then either, the compression steel may be neglected, or the stress in the compression steel may be evaluated by strain proportionality relations and the expression modified accordingly.

$$P_b = 0.85 f'_c a_b b + A'_s f_y - A_s f_y$$  \hspace{1cm} (3007-22)

$$M_b = P_b e = 0.85 f'_c a_b b \left( \frac{h}{2} - \frac{a}{2} \right) + A'_s f_y \left( \frac{h}{2} - d' \right) + A_s f_y \left( d - \frac{h}{2} \right)$$  \hspace{1cm} (3007-23)

in which:

$$a_b = \beta c_b$$  \hspace{1cm} (3007-24)

assuming that $\varepsilon_s = \varepsilon_y$ at balanced failure and the compression controlled limit

$$c_b = \left[ \frac{87000}{87000 + f_y} \right] d$$  \hspace{1cm} (3007-25)
d. Recalculate Equations 3007-22 and 3007-23 with different assumed values of ‘c’. Generally, it is acceptable to select two ‘c’ values, one less than $c_b$ and another greater than $c_b$. 
A. Shear

The relatively abrupt nature of a shear failure in a member, as compared to a ductile flexural failure, makes it desirable to design members so that strength in shear is at least as great as strength in flexure. Most so-called shear failures are really diagonal tension failures and computed shear stresses are really just stress indices, only slightly related to actual stresses. Shear strength is therefore based on the average shear stress on the full effective cross section, and permissible shear stresses are functions of the concrete compressive strength.

In order to ensure a ductile failure, the ACI Code limits the minimum and maximum amount of longitudinal reinforcement as described in Section 3007, and requires a minimum amount of shear reinforcement for certain structural members in which the shear force exceeds one-half the shear strength of the concrete.

1. Design Assumptions

When determining the shear strength of an existing member or designing the shear reinforcement for a new member, the following design assumptions may be made.
a. It is permissible to use the maximum required shear strength, $V_u$, that is present at a distance, $d$, from the face of the support if the following conditions are satisfied:

- The support reaction introduces compression into the end regions of a member. This causes the shear strength to be increased in these regions.
- The applied loads act at or near the top of the member.
- There are no concentrated loads applied within the distance, $d$, from the face of the support.

Note that other situations exist where the critical section must be taken at the face of the support. Refer to the “Shear and Torsion” chapter in the ACI Code and Figure 3008-1 for a description of these situations.

b. No shear reinforcement is required if the factored shear force, $V_u$, is less than or equal to one-half the shear strength provided by the concrete, $\phi V_c$. Additionally, no shear reinforcement is required for slabs, footings, wall type elements, or beams with a height, $h$, no greater than the largest of 10 inches, 2.5 times the thickness of flange, or one-half the width of web.
2. Shear Strength

The nominal shear strength of a concrete member is calculated as the summation of the nominal shear strength provided by the concrete and the nominal shear strength provided by the shear reinforcement as given in Equations 3008-1 and 3008-2.
Other Structures

\[ V_n = V_c + V_s \]  \hspace{1cm} (3008-1)

Environmental Structures

\[ V_n = V_c + \frac{V_s}{S_d} \]  \hspace{1cm} (3008-2)

This total shear strength of a given section must be reduced by the strength reduction factor, \( \phi \), and be greater than or equal to the applied shear at every section under consideration.

\[ \phi V_n \geq V_u \]  \hspace{1cm} (3008-3)

a. Shear Strength Provided by Concrete

The total shear strength of the concrete in a member is actually a combination of three separate shear values, \( V_{cz}, V_{ay}, \) and \( V_d \) which are shown graphically in Figure 3008-2. These values are difficult to calculate with a high degree of certainty, therefore the ACI Code combines them into one value, \( V_c \).
More detailed calculation methods are available from which the nominal shear strength provided by the concrete may be computed; however, for most designs, the following expressions are convenient and acceptable per the ACI Code.

**Members subject to shear and flexure only:**

\[ V_c = 2 \sqrt{f'_{c} b_w d} \]  \hfill (3008-4)

**Members subject to axial compression with shear and flexure:**

\[ V_c = 2 \left(1 + \frac{N_u}{2000 A_g} \right) \sqrt{f'_{c} b_w d} \]  \hfill (3008-5)

\text{in which:}

**\( V_d \) = Component of shear resisted by dowel action**

**\( V_{cz} \) = Component of shear resisted by concrete compression zone**

**\( V_a \) = Component of shear resisted by aggregate interlock**
\[ N_u = \] factored axial load normal to the cross section that occurs simultaneously with \( V_u \) and is taken as positive for compression, lb.

(Note: Do not include the environmental durability factor, \( S_d \), in the calculation of \( N_u \) for environmental structures.)

Members subject to significant axial tension with shear and flexure:

For this condition it is acceptable per the ACI Code to assume the concrete has no shear strength, and therefore, the total shear is resisted by the shear reinforcement. In lieu of this assumption the actual shear strength of the concrete may be calculated using Equation 3008-6, with \( N_u \) being negative for tension. (Note: Do not include the environmental durability factor, \( S_d \), in the calculation of \( N_u \) for environmental structures.)

\[ V_c = 2 \left( 1 + \frac{N_u}{500A_g} \right) \sqrt{f'c' b_w d} \geq 0 \] (3008-6)

b. Shear Strength Provided by Shear Reinforcement

Shear reinforcement is required for added design strength when the factored shear force, \( V_u \), is greater than the shear strength provided by the concrete, \( \phi V_c \). When shear reinforcement is provided using vertical stirrups that
are perpendicular to the axis of the member, the nominal shear strength is calculated as follows:

Other Structures

\[
V_s = \frac{A_v f_y d}{s} \leq 8\sqrt{f'_c b_w d}
\]  \hspace{1cm} (3008-7)

Environmental Structures

\[
V_s = \frac{A_v f_y d}{S_d s} \leq 8\sqrt{f'_c b_w d}
\]  \hspace{1cm} (3008-8)

in which:

\[
s = \text{spacing of shear reinforcement measured in a direction parallel to the longitudinal reinforcement, in.}
\]

\[
A_v = \text{area of shear reinforcement within a distance, } s, \text{ in}^2. \text{ Note that this includes the total number of legs that exist in that plane.}
\]

When Equations 3008-1 and 3008-2 are combined with Equation 3008-3, the area of shear reinforcement required may be calculated in the following manner.
When shear reinforcement is provided using inclined stirrups, the nominal shear strength is calculated as follows:

\[
A_v = \frac{(V_u - \phi V_c)s}{\phi f_y d}
\]  

(3008-9)

\[
A_v = \frac{S_d(V_u - \phi V_c)s}{\phi f_y d}
\]  

(3008-10)

When shear reinforcement is provided using inclined stirrups, the nominal shear strength is calculated as follows:

\[
V_s = \frac{A_v f_y (\sin \alpha + \cos \alpha)d}{s} \leq 8\sqrt{f_c} b_w d
\]  

(3008-11)

\[
V_s = \frac{A_v f_y (\sin \alpha + \cos \alpha)d}{S_d s} \leq 8\sqrt{f_c} b_w d
\]  

(3008-12)

which can be reformulated as:
Other Structures

\[ A_v = \frac{(V_u - \phi V_c) s}{\phi f_y (\sin \alpha + \cos \alpha) d} \]  

(3008-13)

Environmental Structures

\[ A_v = \frac{S_d (V_u - \phi V_c) s}{\phi f_y (\sin \alpha + \cos \alpha) d} \]  

(3008-14)

Refer to Figure 3008-3 for examples of vertical and inclined shear reinforcement.

Figure 3008-3  Shear reinforcement alternatives

Vertical Stirrups

Inclined Stirrups
3. Shear Reinforcement Limitations

In order to control crack widths and ensure ductility, thus preventing sudden failure, limitations have been established for shear reinforcement. The ACI Code provisions are given in the area of spacing restrictions and minimum area of steel required. In general, however, the yield strength of shear reinforcement should be limited to 60 ksi unless welded deformed wire fabric is being used; in which case yield strength of 80 ksi is permitted.

a. Spacing of Shear Reinforcement

The maximum spacing of shear reinforcement is dependent on the actual factored shear force and its relationship to the shear strength of the concrete as can be seen in Table 3008-1 and illustrated in Figure 3008-4.
### Table 3008-1 Maximum spacing of shear reinforcement

<table>
<thead>
<tr>
<th>Condition</th>
<th>No Shear Reinforcement Required</th>
<th>$d/2 \leq 12$ in</th>
<th>$d/4 \leq 12$ in</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Environmental Structures</strong></td>
<td>$V_u \leq \frac{\phi V_c}{2}$</td>
<td>$\phi V_c \geq V_u &gt; \frac{\phi V_c}{2}$</td>
<td>$V_u &gt; \phi V_c$ *</td>
</tr>
<tr>
<td><strong>Other Structures</strong></td>
<td>$\frac{d}{2} \leq 24$ in</td>
<td>$\frac{d}{4} \leq 12$ in</td>
<td>$\phi V_s &gt; \phi 4\sqrt{f'c b_w d}$</td>
</tr>
</tbody>
</table>

* When calculating $V_s$ for environmental structures, the equations that include the environmental durability factor, $S_d$, must be used.

For sections requiring reinforcement by design, the actual spacing of vertical shear reinforcement may be calculated directly by rearranging Equations 3008-9 and 3008-10 to solve for $s$, as shown below. Note, that the spacing may be increased along a member to economize the design, but changing the spacing more than three times is not recommended.

**Other Structures**

\[
s = \frac{\phi A_s f_y d}{(V_u - \phi V_c)} \quad (3008-15)
\]
Environmental Structures

\[ s = \frac{\phi A_v f_v d}{S_d (V_u - \phi V_c)} \]  

(3008-16)

Figure 3008-4 Shear strength requirements

b. Minimum Shear Reinforcement

Per the ACI Code a minimum area of shear reinforcement must be used when the factored shear force, \( V_u \), is greater than one-half the shear strength of the concrete, \( \phi V_c \). This is true for all reinforced concrete flexural members except:

- Slabs and footings
• Concrete joists
• Beams with total depth, h less than the larger of:
  o 10 in
  o 2.5 times the thickness of the flange
  o \( \frac{1}{2} \) the width of the web

When shear reinforcement is required per the above criteria and when torsion may be neglected, the minimum area of shear reinforcement may be calculated as:

\[
A_v = 0.75 \sqrt{f'_c \frac{b_w s}{f_y}} \geq 50 \frac{b_w s}{f_y}
\]  \hspace{1cm} (3008-17)

In order to neglect torsion and use the minimum area of shear reinforcement given in Equation 3008-17, the torsional moment must satisfy the following conditions.

Members subject to shear and flexure only:

\[
T_n < \phi \sqrt{f'_c \left( \frac{A_{cp}^2}{P_{cp}} \right)}
\]  \hspace{1cm} (3008-18)

Members subject to axial load (tensile or compressive) with shear and flexure:
\[ T_u < \phi \sqrt{f_c^\prime} \left( \frac{A_{cp}^2}{P_{cp}} \right) \sqrt{1 + \frac{N_u}{4A_g \sqrt{f_c^\prime}}} \]  \hspace{1cm} (3008-19)

in which:

\[ P_{cp} = \text{outside perimeter of the concrete cross-section, inches.} \]

\[ A_{cp} = \text{area included by the outside perimeter of the concrete cross-section, inches}^2. \]
A. Development Length

The development length concept is based on the attainable average bond resistance over the length of embedment of the reinforcement. If a reinforcing bar in a member has enough embedment in concrete, it cannot be pulled out of the concrete before the bar fails by yielding of the steel. Additionally, the calculated tension or compression in any bar at any section must be developed on each side of that section by proper embedment length, end anchorage, or hooks. The criteria presented in this section apply to both Other and Environmental Structures.

1. Tension

The development length for deformed straight bars in tension, \( \ell_d \), may be determined using Equation 3009-1 for both environmental and non-environmental structures.

\[
\ell_d = \frac{3}{40} \frac{f_y}{f'_c} \sqrt{\frac{c_b + K_d}{d_b}} \psi_c \psi_s \psi_x \lambda d_b \quad (3009-1)
\]
The variables contained within the Equation 3009-1 can be determined as follows:

\[ \psi_t = \text{reinforcement location factor.} \]

- if more than 12 in. of fresh concrete is below the development length or splice, top bar \[ \psi_t = 1.3 \]
- other locations \[ \psi_t = 1.0 \]

\[ \psi_e = \text{coating factor.} \]

- epoxy-coated bars or wires with cover less than \( 3d_b \)
  or clear spacing less than \( 6d_b \) \[ \psi_e = 1.5 \]
- all other epoxy-coated bars or wires \[ \psi_e = 1.2 \]
- uncoated reinforcement \[ \psi_e = 1.0 \]

Note that the product of \( \psi_t \) and \( \psi_e \) does not need to be taken as greater than 1.7

\[ \psi_s = \text{reinforcement size factor.} \]

- No. 6 and smaller bars and deformed wires \[ \psi_s = 0.8 \]
- No. 7 and larger bars \[ \psi_s = 1.0 \]

\[ \lambda = \text{lightweight aggregate concrete factor.} \]

- when lightweight aggregate concrete is used \[ \lambda = 1.3 \]
- when normal weight concrete is used \[ \lambda = 1.0 \]
Note that if $f_{ct}$ is specified, $\lambda$ may be taken as follows:

$$\lambda = 6.7 \frac{\sqrt{f_c'}}{f_{ct}} \geq 1.0 \quad (3009-2)$$

$$\left( \frac{c_b + K_{tr}}{d_b} \right) \leq 2.5 \quad (3009-3)$$

in which:

$c_b =$ smallest side cover measured from the edge of concrete to center of bar or the center-to-center spacing of bars.

$K_{tr} =$ transverse reinforcement factor; may be taken as zero for design simplification, otherwise:

$$K_{tr} = \frac{A_{tr} f_{tr}}{1500 sn} \quad (3009-4)$$
Should there be a situation where there is more reinforcement provided in a flexural member than what is required by design, the development length may be reduced by the following ratio.

\[
\frac{A_{\text{required}}}{A_{\text{provided}}}
\]  

(3009-5)

a. Standard Hooks in Tension

When there is inadequate length for full embedment of a bar, it is permissible to use standard hooks, which allow for a shorter embedment length. The development length for standard hooks in tension, \( \ell_{dh} \), may be determined by using Equation 30-99 and the subsequent modification factors. It should be noted that standard hooks are not useful for developing reinforcement in compression.

\[
\ell_{dh} = \left( \frac{0.02 \psi_e \lambda f_y}{\sqrt{f'_c}} \right) d_b \geq \text{larger}(8d_b \text{ or } 6 \text{ in})
\]

(3009-6)

in which:

\( \psi_e \) = coating factor.

- epoxy-coated bars .......................1.2
- uncoated reinforcement ..................1.0

\( \lambda \) = lightweight aggregate concrete factor.

- when lightweight aggregate concrete is used .........................1.3
• when normal weight concrete is used..............................1.0

Additionally, for the following conditions, multiplying the length calculated in Equation 3009-7 by the factors given below may reduce the standard hook length. However, it should be noted that the first reduction factor is readily achievable on NRCS structures, but the remaining three are rarely utilized for NRCS structures. Figures 3009-1, 3009-2, and 3009-3 graphically represent the conditions described.

• For No. 11 bars and smaller hooks with side cover (normal to the plane of the hook) not less than 2-½”, and for 90 deg hooks with cover on the bar extension beyond the hook not less than 2 in ..................0.7

**Figure 3009-1  Concrete covers for standard hooks**

![Concrete covers for standard hooks](image-url)
• For 90 deg hooks of No. 11 and smaller bars that are either enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along the development length $\ell_{dh}$ of the hook; or enclosed within ties or stirrups parallel to the bar being developed, spaced not greater than $3d_b$ along the length of the tail extension of the hook plus bend.................................0.8

Figure 3009-2 Spacing of ties for reduction in standard hook development length
• For 180 deg hooks of No. 11 and smaller bars that are enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than \(3d_b\) along the development length \(\ell_{dh}\), of the hook .................0.8

• Where anchorage or development for \(f_y\) is not specifically required, reinforcement in excess of that required by analysis, the development length may be reduced by the following ratio:

\[
\frac{A_{\text{required}}}{A_{\text{provided}}} = \frac{\ell_{dc}}{\ell_{\text{indf}}}
\]  (3009-7)

2. Compression

The development length for deformed straight bars in compression, \(\ell_{dc}\), may be determined by using the larger of Equations 3009-8 and 3009-9 and the subsequent modification factors.

\[
\ell_{dc} = \left( \frac{0.02 f_y}{\sqrt{f'_c}} \right) d_b \geq 8 \text{ in} \quad (3009-8)
\]

\[
\ell_{dc} = \left( 0.0003 f_y \right) d_b \geq 8 \text{ in} \quad (3009-9)
\]
• If there is more reinforcement provided than what is required by analysis, then the development length may be reduced by the following ratio:

\[
\frac{A_s \text{ required}}{A_s \text{ provided}} = (3009-10)
\]

• Reinforcement enclosed within spiral reinforcement not less than \(\frac{1}{4}\) in. diameter and not more than 4 in. pitch or within No. 4 ties in conformance with standard tie requirements and spaced at not more than 4 in. on center..............................................................0.75

Tables 3009-2 and 3009-3 provide recommended development and splice lengths for various situations for \(f'c\) equal to 3 ksi and 4 ksi respectively. Note that the basic development length, \(\ell_d\), calculated in Part A.1 of this section is the same as the Class A lap splice for an “other” bar provided in the tables.
Table 3009-2  Minimum development length and splice length of reinforcing bars, $f'c = 3$ ksi

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Tension Bar Development Length (Other Bars)</th>
<th>Tension Bar Development Length (Top Bars)</th>
<th>Tension Bar Lap Splice 'A' (Other Bars)</th>
<th>Tension Bar Lap Splice 'B' (Other Bars)</th>
<th>Tension Bar Lap Splice 'A' (Top Bars)</th>
<th>Tension Bar Lap Splice 'B' (Top Bars)</th>
<th>Compression Bar Development Length</th>
<th>Compression Bar Splice</th>
<th>Hook Development Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
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</tbody>
</table>

Notes:
1) Above table has been developed for un-coated reinforcing bars which meet the requirements of ACI 318 Section 12.2.3.
2) The excess reinforcement factor from ACI 318 for reducing the basic development length has been ignored.
3) The side cover is assumed to be 2.5".
4) The cover on the tail of the hook is assumed to be 2 in.
5) The Transverse Reinforcement Factor is assumed to be 0.

Table 3009-3  Minimum development length and splice length of reinforcing bars, $f'c = 4$ ksi

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Tension Bar Development Length (Other Bars)</th>
<th>Tension Bar Development Length (Top Bars)</th>
<th>Tension Bar Lap Splice 'A' (Other Bars)</th>
<th>Tension Bar Lap Splice 'B' (Other Bars)</th>
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Notes:
1) Above table has been developed for un-coated reinforcing bars which meet the requirements of ACI 318 Section 12.2.3.
2) The excess reinforcement factor from ACI 318 for reducing the basic development length has been ignored.
3) The side cover is assumed to be 2.5".
4) The cover on the tail of the hook is assumed to be 2 in.
5) The Transverse Reinforcement Factor is assumed to be 0.
3. **General Requirements of Flexural Reinforcement**

Development of flexural reinforcement is divided into two different sets of criteria: Positive Moment Reinforcement and Negative Moment Reinforcement. In addition to the specific requirements that will be given for both situations, the following are criteria that are applicable to all flexural reinforcement for both Other and Environmental structures.

- Critical sections for development of reinforcement are at points of maximum stress and at points where adjacent reinforcement terminates.
- Except at supports of simple spans and at the free end of cantilevers, every reinforcing bar must be extended beyond the point at which it is no longer needed to resist flexural stress, for a distance equal to the effective depth of the member or $12d_{n}$, whichever is greater.
- Flexural reinforcement must not be terminated in a tension zone unless the following conditions are satisfied:
  - The factored shear at the cutoff point does not exceed $\frac{2}{3}$ of the design shear strength, $\phi V_{n}$.
  - Stirrup area in excess of that required for shear is provided along each terminated bar over a distance from the termination point equal to $\frac{3}{4}$ of the effective depth. The excess stirrup area must not exceed $60b_{w}s/f_{y}$. The spacing, $s$, must not exceed $d/8\beta_{b}$ where $\beta_{b}$ is the ratio of area terminated to total area of tension reinforcement at the section.
For No. 11 bars and smaller, continuing reinforcement provides at least double the area required for flexure at the cutoff point and shear does not exceed \( \frac{3}{4} \) of that permitted.

Figure 3009-3 graphically represents the conditions concerning critical section locations and location of bar cutoffs for a section of a typical continuous structural slab or beam with positive and negative moments. Figure 30-4 depicts similar information for dowel bars extending out of a slab into a wall.

**Figure 3009-3** Development of positive and negative moment reinforcement – slab/beam element
a. Development of Positive Moment Reinforcement

When changes in loading, settlement of supports, or lateral loads occur, the actual moments in a member may change from those obtained during analysis. This possibility is why the ACI Code requires at least $\frac{1}{3}$ the total area of positive moment reinforcement in simple members and $\frac{1}{4}$ in continuous members, be extended into the support. For example, in Figure 3009-3, Bar D must be at least $\frac{1}{4}$ the area of reinforcement required at the point of maximum positive moment. The ACI Code stipulates that for beams, that extension into the support is required to be 6 in. minimum.
At simple supports and at points of inflection, positive moment tension reinforcement must be limited to a bar diameter such that the development length, $\ell_d$, satisfies the following criteria:

$$\ell_d \leq \frac{M_n}{V_u} + \ell_a$$

(3009-11)

in which

- $M_n$ = nominal moment strength assuming all reinforcement at the section to be stress to $f_y$, in-lbs.
- $V_u$ = required shear strength at the section, lbs.
- $\ell_a$ = at a support, the sum of the embedment length beyond the center of the support and the equivalent embedment length of any hook; at a point of inflection, the effective depth of the member or $12\,d_b$, which ever is greater.

Note that if the reinforcement is confined by a compressive reaction, such as is the case for a beam on a column, the value of $M_n/V_u$ may be increased by 30%.

b. Development of Negative Moment Reinforcement
At least \( \frac{1}{3} \) the negative moment reinforcement at a support must extend beyond the extreme position of the point of inflection a distance not less than the effective depth of the member, \( 12d_b \) or \( \frac{1}{16} \) the clear span, whichever is greater.

c. Development of Web Reinforcement

The ends of single leg, simple or multiple U-stirrups must be anchored by one of the following means:

- For No. 5 bars or smaller, it is acceptable to provide a standard stirrup hook.
- For No. 6, 7 or 8 stirrups with \( f_y \) greater than 40,000 psi a standard stirrup hook may still be used, but the embedment, \( \ell_e \), as shown in Figure 3009-5 must be equal to or greater than:

\[
\ell_e \geq \frac{0.014d_b f_y}{\sqrt{f_c}} \quad (3009-12)
\]
Additionally, between the ends of a simple or multiple U-stirrup, every bend must enclose a longitudinal bar, as shown for the bottom longitudinal bars of Figure 3009-5.

When stirrups are placed in deeper members, they are often comprised of pairs of U-stirrups or ties that create a closed unit as depicted in Figure 3009-6. The splices must be at least $1.3 \ell_d$ long. When a member is at least 18 in. deep, the splices may extend the full height of the member as shown in Figure 3009-6 as long as the product $A_{bfy}$ is less than or equal to 9000 lb.
B. Lap Splices

To ensure ductile behavior, lap splices should be adequate to develop more than the yield strength of the reinforcement. Splices should, if possible, be located close to points of inflection and away from points of maximum tensile stress. It is permissible to use welded or mechanical splices as long as they have sufficient strength of at least 125 % of $f_y$ for bars larger than No. 5. Welded or mechanical splices in No. 5 and smaller bars do not need to meet the 125 % strength requirement for tension splices if they meet the requirements given in the “Splices of deformed bars and deformed wire intension” section in the ACI Code.

In all cases except for compression splices in Part B.2, no bars larger than No. 11 may be spliced. Also, splices are not required to be in contact, but must be within $\frac{1}{5}$ the required lap splice length, but not more than 6 in.
Note that the ACI 350 code provides additional structure-specific lap splice criteria for environmental structures, such as circular tanks designed for hoop tension.

1. Tension

Lap distances for tension splices are determined by applying the appropriate multiplier given in Table 3009-4 to the tensile development length, without the reduction factor for excess reinforcement. In order to select the multiplier, the splice must first be defined as either Class A or Class B.

Table 3009-4 Lap splices in tension

<table>
<thead>
<tr>
<th>Splice Classification</th>
<th>Multiplier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class A splice</td>
<td>1.0(l_d)</td>
</tr>
<tr>
<td>Class B splice</td>
<td>1.3(l_d)</td>
</tr>
</tbody>
</table>

a. Class A Splice

Class A splices are defined as having at least two times more reinforcement provided than what is required by analysis over the entire length of the splice and at most 50% of the bars within the lap length are spliced. Both these conditions must be met for a splice to be defined as Class A. Therefore, if the splices are not staggered then more than 50% of the reinforcement is being spliced and the splice may not be considered Class A.
b. Class B Splice

Class B splices are defined as any splice that does not meet the requirements of Class A splices.

Given the class definitions and the increased lap length for Class B splices it is apparent that the ACI Code recommendation is to provide staggered splices located in areas of low tensile stress.

2. Compression

When \( f_y \) is less than or equal to 60,000 psi the compression lap splice length is given by the following expression:

\[
0.0005 f_y d_b \geq 12 \text{ in}
\]  

(3009-13)

If \( f'_c \) is less than 3000 psi then the lap splice distance must be increased by \( \frac{1}{3} \). Also, in the case of different size bars being spliced in compression, the length must be the larger of the development length of the larger bar or the splice length of the smaller bar.
A. Limitations of Use

Before an explanation of the design processes for structural plain concrete are given, it is important to understand the applications where structural plain concrete may or may not be used. The major limitation of use for structural plain concrete is that it may not be used in the design of any structure that falls under the classifications of environmental or hazardous material containment. The basic reason for this limitation is that structural plain concrete depends solely on the properties of the concrete for strength and stability and therefore, since concrete inherently cracks, it would be susceptible to leaking and deterioration from infiltrated liquids and chemicals.

Since, structural plain concrete is not permissible for structures designed per ACI 350, all code requirements are taken from ACI 318. In ACI 318, structural plain concrete may be used for cast-in-place or precast members, but the code provisions should not be used for the design of soil supported slabs, such as sidewalks or slabs on grade, unless direct vertical or lateral loads from other portions of the structure are transferred to the slab.

If the component to be designed is considered structural, then the limitations for use of this section are that the member must satisfy one of the following criteria:

- There is continuous vertical support provided by soil or other structural members.
• There is arching action present that ensures complete compression in all loading conditions.
• The members being designed are walls or pedestals.

Aside from the above criteria, the provisions of this section or ACI 318 for structural plain concrete may not be used for columns or concrete piles and piers embedded in ground.

B. Design Procedure

Structural plain concrete members are those in which no reinforcement is provided, or at least the code minimum amount of reinforcement is not provided. This means that all strength or stability requirements will be based on the properties of the concrete alone. With this in mind, the minimum specified compressive strength must 2500 psi.

1. Joints

In order to prevent the build up of tensile stress in structural plain concrete members, it is necessary to provide contraction or isolation joints regularly. Typically, internal forces such as creep, shrinkage, or temperature effects cause these stresses. The number and location of these contraction or isolation joints is dependent on many factors, including: climatic conditions, proportioning of materials, mixing, placing, curing, or construction techniques, to name a few.
2. **Strength Design Equations**

The basic premise for design is the same as that for Other Structures given throughout this handbook. This includes the use of the same load factors and combinations that are provided in Section 3005, Part B.1. One major difference from other reinforced concrete design is the use of a strength reduction factor, $\phi$, of 0.55 for all aspects of design (flexure, compression, shear, and bearing).

In any case where the design strength of a structural plain concrete member is not greater than or equal to the required strength as given in Equation 30-1, the member must be designed as a reinforced concrete member per the requirements of the other sections of this handbook and ACI 318. Additionally, when steel reinforcement is used that does not meet the minimum requirements; it may not be assigned any strength in resisting the applied loads and forces.

As a general rule, for the purposes of design, the entire depth of a section may be used for determining strength. The only exception to this rule is the case where concrete is cast against soil, which is cause for reducing the depth by 2 inches.

a. **Flexure**

For all cross sections that must be designed for flexure, the following condition must be satisfied:

$$\phi M_\mu \geq M_\mu$$

(3010-1)
In the case of structural plain concrete, the design strength is simply calculated using one of the following equations:

Tension Controls:

\[ M_n = 5 \sqrt{f_c' S_m} \]  

(3010-2)

Compression Controls:

\[ M_n = 0.85 f_c' S_m \]  

(3010-3)

in which:

\[ S_m = \text{elastic section modulus, inches}^3. \]

b. Compression

For members that must resist compression, but are not considered columns, the design strength must meet the general condition following:

\[ \phi P_n \geq P_u \]  

(3010-4)

The nominal compression strength is then calculated as follows:

\[ P_n = 0.60 f_c' \left[ 1 - \left( \frac{f_c'}{32h} \right)^2 \right] A_t \]  

(3010-5)
in which:

\[ \ell_c = \text{length between supports, inches.} \]
\[ H = \text{overall thickness of member, inches.} \]
\[ A_f = \text{area loaded in compression, inches}^2. \]

c. **Flexure and Compression**

Any structural plain concrete member that is subject to flexure and axial load producing compression must satisfy the following interaction equations:

**Compression Face:**

\[
\frac{P_u}{\phi P_n} + \frac{M_u}{\phi M_n} \leq 1
\]  

(3010-6)

**Tension Face:**

\[
\frac{M_u}{S_m} - \frac{P_u}{A_g} \leq 5\phi \sqrt{f'_c}
\]  

(3010-7)

d. **Shear**

Any shear present in rectangular sections must meet the following condition:
The nominal shear strength is then calculated as follows:

**Beam Action:**

\[ V_n = \frac{4}{3} \sqrt{f'_e b h} \]  

(3010-9)

**Two-way Action/Punching Shear:**

\[ V_n = \left[ \frac{4}{3} + \frac{8}{3 \beta_c} \right] \sqrt{f'_e b_o h} \leq 2.66 \sqrt{f'_e b_o h} \]  

(3010-10)

e. **Bearing Areas**

When structural plain concrete resists compressive bearing forces it must meet the following condition:

\[ \phi B_n \geq P'_n \]  

(3010-11)

The nominal bearing strength is then calculated as follows:

\[ B_n = 0.85 f'_e A_t \]  

(3010-12)
If the concrete supporting surface is larger on all sides than the bearing area, then the bearing strength calculated in Equation 30-134 may be multiplied by:

$$\sqrt{\frac{A_1}{A_2}} \leq 2 \quad (3010-13)$$

C. Other Requirements

There are various additional parameters for structural plain concrete that are given in ACI 318 that should be referred to when designing the specific structural components that are listed below.

- Walls
- Footings
- Pedestals
- Earthquake resisting structures
636.3011 – Special Topics

A. Seismic Design

Seismic design is a very complex and ever-evolving science, one that gets updated after every seismic event based on how well existing structures performed during the earthquake. Unlike other design loads, earthquake loads require a thorough understanding of the site characteristics to determine the design load. Once the load is included in the design combinations, many specific detailing practices must be incorporated to ensure the proper response of the structure to the design earthquake. The engineer should thoroughly investigate the necessity of seismic design for a given structure before ruling it out, especially since the newer seismic design procedures are becoming broader in their application and more stringent in the detailing requirements.

A thorough explanation of seismic design is not warranted due to the complexity of the procedures; therefore, the engineer is referred to the ACI Code for detailing and to other standards such as the American Society of Civil Engineers’ “Minimum Design Loads for Buildings and Other Structures” (ASCE 7) or the International Building Code (IBC) for assistance in determining the design loads and how they may be applied to a given structure.

B. Torsion Design

It is not the intention of this section to provide an exhaustive explanation of all aspects of torsional design since NRCS structures do not typically induce torsional loads into concrete.
members. For this reason the engineer is referred to the ACI Code as it will provide the engineer with comprehensive procedures for checking and designing for torsional stresses.

The basic premise of designing for torsion is that the torsional moment must be resisted by the torsional strength of the member as depicted in the following relationship:

\[ \phi T_a \geq T_u \]  

(3011-1)

Torsional strength in a member is a function of geometry of the section and the amount of longitudinal and transverse reinforcement present. Figure 3011-1 depicts the difference between torsional and shear stresses. Notice that the torsional stresses assume the center of a solid section does not contribute to the torsional strength and therefore, can be ignored, thus the member is treated as if it is a hollow section.

Figure 3011-1  Torsional stress versus shear stress
In Section 3008, Part A.3.b of this handbook, the threshold limits for torsional stress considerations were given so the engineer may determine whether torsional stresses may be ignored when determining the minimum shear reinforcement. This is required since, as shown in Figure 3011-1, the torsional and shear stresses may be additive and therefore the presence of too much torsion reduces the shear capacity of a member.

C. Reinforcement at Openings in Slabs and Walls

In general there is no limit on the size or shape of an opening in a slab or wall as long as all of the design requirements for strength and serviceability are met for the requisite design strips. Particular attention must be given to the deflection calculations, since the stiffness of a concrete panel may be significantly influenced by the presence of a large opening. In lieu of arduous hand calculations, the engineer may want to take advantage of using finite element software to model the slab or wall with the opening to determine the redistribution of forces and subsequent reinforcement requirements.

Additional attention must be given to the amount of reinforcement required around any opening. In general the amount of reinforcement required for a slab or wall without an opening must be maintained even after an opening is added. This is accomplished by placing \( \frac{1}{2} \) the amount of steel, which is interrupted by the opening, on either side of the opening. Additional steel may be required at geometric discontinuities as shown in Figure 3004-8 in Section 3004.
D. Deep Flexural Members

The design of deep beams has become more clearly defined with specific approaches and requirements that are now consistent throughout the ACI Code. Research has shown that deep beams may be designed using the approach of the Strut-and-Tie model; however, NRCS still allows deep beams to be designed using a nonlinear strain distribution. Various references are available that provide guidance on how to perform a nonlinear stress/strain analysis.

Designing by the Strut-and-Tie model approach introduces new concepts and may appear complex at first, but once the basic idea of visualizing truss action through compression struts and tension ties is understood, the application of the design equations may become routine. A detailed description of the Strut-and-Tie model is not given here due to the length of introduction to the concept that would be required; therefore the user is encouraged to investigate the ACI Code and other references should they desire to use this technique. Figure 3011-2 below reflects a simple strut-and-tie model for a deep beam. Depending on the loading, different models will be possible.
1. Definition

Deep beams are defined as having load applied on one face and being supported on the opposite face (as shown in Figure 3011-2) and meet either of the following criteria:

1. \( \ell_n \leq 4h \)  \hspace{1cm} (3011-2)

in which:

\[ \ell_n = \text{clear span between supports.} \]

\[ h = \text{overall member depth.} \]
2. Concentrated loads within $2h$ from the face of support

2. **Flexure**

Minimum flexural reinforcement for deep beams is the same as given in Section 3007, Part A.4. However, deep beams may require additional side face longitudinal (skin) reinforcement due to the possibility of large portions of concrete that would be unreinforced and therefore would form locations for potentially significant cracking in the web.

3. **Shear**

   a. **Design Strength**

   Deep beams may be designed for shear using either Strut-and-Tie model approach or by using the nonlinear analysis method represented by the following equations. Note that either method must be used exclusively and not combined for portions of a deep beam; this means that it is not possible to perform flexural design per the Strut-and-Tie model and shear design using a nonlinear analysis.

   \[
   \phi V_n = (\phi V_c + \phi V_s) \leq \phi 10\sqrt{f'_c b_w d} \tag{3011-5}
   \]

   \[
   \phi V_c = \phi 2\sqrt{f'_c b_w d} \tag{3011-6}
   \]
b. Minimum Reinforcement

Research has shown that vertical shear reinforcement is more effective than horizontal shear reinforcement; therefore, the values for the minimum reinforcement in each direction are given in Equations 3011-7 and 3011-8 for vertical and horizontal shear reinforcement respectively.

\[
A_v = 0.0025 b_w s \quad (3011-7)
\]

\[
A_{hb} = 0.0015 b_w s_2 \quad (3011-8)
\]

in which:

\[
s \text{ or } s_2 \leq \frac{d}{5} \text{ or } 12 \text{ in.} \quad (3011-9)
\]

Note that the minimum shear reinforcement limits will be different if the beam is designed according to the strut-and-tie model.

E. D-Regions

Deep beams constitute one example of what is formally called a disturbed region (D-Region). Other examples of D-Regions that could be designed by the Strut-and-Tie model approach are shown in Figure 3011-3 below.
D-regions typically occur in flexural members that meet the following condition:

\[
\left( \frac{a}{d} \right) \text{ or } \left( \frac{M}{Vd} \right) \leq 2 \text{ to } 2.5 \tag{3011-10}
\]

in which:
a = shear span, or the distance between a concentrated load and the face of the support.

D-Regions can not be accurately analyzed using elastic beam theory, thus traditionally, reinforcement in these regions relied on lessons learned from past experience. The remaining regions of a beam are considered B-Regions (since plane strain theory applies).
Appendix 30A – Working Stress Design

A. Design Methodology for Working Stress Design

1. Historical Background

The title of “Working Stress Design” for this design method has not been officially used by the ACI 318 code since the 1963 edition. Since then, the official title has been the “Alternate Design Method”. However, due to familiarity, the working stress title has continued to be used and will be used in this document as well, despite the fact that the procedures given are taken from the “Alternate Design Method” of Appendix A in ACI 350.

With the “Ultimate Design Method” (strength design) becoming the focus of research and teaching, the “Working Stress Design Method” was simplified with built in conservativeness. Therefore, the “Alternate Design Method” is similar to the original “Working Stress Design Method” with the major differences occurring in the design of compression members with or without flexure and the development of reinforcement. Design for flexure without axial load has remained the same.
2. Basic Design Methodology

The use of this section is limited to nonprestressed, reinforced concrete members. Therefore, any members not meeting those criteria must be designed using the strength design procedures given elsewhere in this handbook or in the ACI Code. Except for the provisions in the ACI Code dealing with redistribution of negative moments and requirements that are specific to strength design, all provisions in the ACI Code must be applied to members designed by this section, particularly those pertaining to the control of deflections.

The basic premise for designing by this method is that service (unfactored) loads are applied to the structure and analyzed, resulting in stresses that are compared to allowable stress levels given in this section. In strength design terminology, this is similar to setting load factors and strength reduction factors equal to 1.0.

This section is not meant to encompass all aspects of ACI 350 relating to the Alternate Design Method; therefore, the engineer must refer to the ACI Code for additional design criteria for this method.

B. Permissible Service Load Stresses

The allowable concrete stresses for design by this section are given in Table 30A-1. Additionally, the allowable tensile stress in reinforcement, $f_t$, is given in Table 30A-2.
Table 30A-1  Permissible service load stresses for NRCS concrete structures

<table>
<thead>
<tr>
<th>Design Condition</th>
<th>Location/Member Type</th>
<th>Allowable Stress, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexure</td>
<td>Extreme fiber stress in compression</td>
<td>$0.4 f_c'$</td>
</tr>
<tr>
<td></td>
<td>Beams and one-way slabs and footings</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Shear carried by concrete, $v_c$</td>
<td>$1.1 \sqrt{f_c'}$</td>
</tr>
<tr>
<td></td>
<td>Maximum shear carried by concrete plus shear reinforcement</td>
<td>$v_c + 4.4 \sqrt{f_c'}$</td>
</tr>
<tr>
<td></td>
<td>Joists</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Shear carried by concrete, $v_c$</td>
<td>$1.2 \sqrt{f_c'}$</td>
</tr>
<tr>
<td></td>
<td>Two-way slabs and footings</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Shear carried by concrete, $v_c$</td>
<td>$\left(1 + \frac{2}{\beta_c}\right) \sqrt{f_c'} \leq 2 \sqrt{f_c'}$</td>
</tr>
<tr>
<td>Bearing on loaded area</td>
<td></td>
<td>$0.3 f_c'$</td>
</tr>
</tbody>
</table>
Table 30A-2  Permissible service load stresses for reinforcement in NRCS structures

<table>
<thead>
<tr>
<th>Design Condition</th>
<th>Allowable Stress, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Members in axial tension</td>
<td>20,000</td>
</tr>
<tr>
<td>Flexural and shear reinforcement</td>
<td></td>
</tr>
<tr>
<td>Bar size</td>
<td>Exposure condition</td>
</tr>
<tr>
<td>#3 - 5</td>
<td>Severe</td>
</tr>
<tr>
<td></td>
<td>Normal</td>
</tr>
<tr>
<td>#6 - 8</td>
<td>Severe</td>
</tr>
<tr>
<td></td>
<td>Normal</td>
</tr>
<tr>
<td>#9 - 11</td>
<td>Severe</td>
</tr>
<tr>
<td></td>
<td>Normal</td>
</tr>
</tbody>
</table>

There is one unique design situation that supersedes the limits given in Table 30A-2 and that is when a one-way slab is designed for a clear span of 12 feet or less and is reinforced with No. 3 bars or welded wire fabric with a diameter less than or equal to 3/8 in. In this case the allowable tensile stress may be increased to the lesser of 0.5 $f_y$ or 30,000 psi.

C. Bar Spacing for Flexural Crack Control

When designing by the Alternate Design Method for NRCS structures, the serviceability requirements of Sections 3006 and 3007 must be followed. The bar spacing given in Figures 30A-1, 30A-2, and 30A-3 can be used with the absolute maximum spacing being no more than 12 inches when designing by the Alternate Design Method. The engineer should use caution in determining the maximum spacing by keeping in mind that closer spacing provides for better crack control.
Figure 30A-1 Maximum bar spacing chart 1

$$s = \frac{(Z / f_s)^3}{10.70}$$

#3 through #5 Bars with 2 in. cover

Allowable service load stress, $f_s$, ksi

Bar spacing, $s$, in.

Figure 30A-2 Maximum bar spacing chart 2

$$s = \frac{(Z / f_s)^3}{12.50}$$

#6 through #8 Bars with 2 in. cover

Allowable service load stress, $f_s$, ksi

Bar spacing, $s$, in.

Figure 30A-3 Maximum bar spacing chart 3

$$s = \frac{(Z / f_s)^3}{14.63}$$

#9 through #11 Bars with 2 in. cover

Allowable service load stress, $f_s$, ksi

Bar spacing, $s$, in.
D. Flexure

For design scenarios in which there is no axial load or very small axial load, the member may be designed for flexure using the straight-line theory of mechanics, which assumes a linear relationship between stress and strain. If it is not known whether the axial load present may be considered small, the member must be checked according to the provisions given in Part E of this section.

Additional criteria that must be taken into consideration during design includes the following:

- Members classified as deep flexural members per the definitions in Section 3011, Part D may not be designed according to the methods of straight-line theory.
- Concrete may not be relied upon to resist tensile forces.
- The modular ratio may be taken as the nearest whole number greater than or equal to 6 for the expression given in Equation 30A-1, except for stress computations of doubly reinforced flexural members. For the latter case, the value calculated in Equation 30A-1 may be doubled.

\[ n = \frac{E_s}{E_c} \]  

(30A-1)
E. Compression Members With or Without Flexure

In general, all compression members must be designed by the strength design provisions given in other parts of this handbook and the ACI Code, due to the inability of the previous working stress methods to consistently provide an adequate factor of safety for the load-moment interaction. The comparable working stress limit is obtained by taking 40% of the strength design capacity calculated. It is important to check slenderness effects with the procedures given in Section 3007, Part C.

F. Shear

The working stress provisions given in ACI 350 for shear capacity are more in depth than those given for flexure or axial load. Thus, only the major points will be discussed here, and the engineer should refer to ACI 350 for a more complete understanding of the provisions.

1. General

In order to determine the design shear stress, $v$, the actual shear force, $V$, at a given section being considered must be entered into the following equation. Similar to strength design, the shear stress to design for need only be that calculated at a distance, $d$ from the end of a member that has induced compression.

$$v = \frac{V}{b_w d}$$

(30A-2)
2. **Shear Stress Carried by Concrete**

When members are to resist shear and flexure only, or axial compression, the shear stress carried by the concrete, $v_c$, must meet the limits given in Table 30A-1, unless a more detailed calculation is performed as described in ACI 350. When a member must resist significant axial tension, the shear reinforcement must be designed to resist all of the shear stress unless a more detailed calculation is performed using Equation 30A-3.

\[
v_c = 1.1 \left( 1 + 0.004 \frac{N}{A_g} \right) \sqrt{f'_c}
\]  

(30A-3)

3. **Shear Stress Carried by Shear Reinforcement**

All shear reinforcement must meet one of the following classifications in order to be considered adequate for resisting shear stresses in a member as well as be limited to the maximum design yield strength of 60,000 psi.

- Stirrups perpendicular to the axis of the member
- Welded wire fabric perpendicular to the axis of the member and having an angle of $45^\circ$ or more with the longitudinal tension reinforcement.
- Bent longitudinal reinforcement having an angle of $30^\circ$ or more with the longitudinal tension reinforcement.
• Combinations of stirrups and bent longitudinal reinforcement

• Spirals

a. Spacing limits for shear reinforcement

The predominant form of shear reinforcement is that of stirrups placed perpendicular to the axis of the member, therefore, the maximum spacing of such reinforcement must not exceed the lesser of \( \frac{d}{2} \) or 12 inches unless the following is true:

\[
(\nu - \nu_c) > 2\sqrt{f_c}
\]  
(30A-4)

in which case the spacing must not exceed \( \frac{d}{4} \) or 12 inches. The engineer must refer to ACI 350 for spacing limits of other forms of shear reinforcement.

b. Minimum shear reinforcement

Shear reinforcement is required in all flexural members in which the following condition is true, except for slabs, footings, joists, or beams with total depth less than the greater of 10 inches, 2 \( \frac{1}{2} \) times the flange thickness, or \( \frac{1}{2} \) the web width.

\[
\nu > \frac{1}{2} \nu_c
\]  
(30A-5)
When the above conditions are satisfied, the minimum area of shear reinforcement must be computed by:

\[ A_v = 50 \frac{b_w s}{f_y} \]  \hspace{1cm} (30A-6)

c. Design of shear reinforcement

In the case where the design shear stress, \( v \), is greater than the shear stress carried by the concrete, \( v_c \), shear reinforcement must be provided that meets the following requirements.

When shear reinforcement is perpendicular to the axis of the member, the required area of shear reinforcement may be calculated as:

\[ A_v = \frac{(v - v_c) b_w s}{f_s} \]  \hspace{1cm} (30A-7)

When inclined stirrups are used for shear reinforcement, the required area may be calculated as:

\[ A_v = \frac{(v - v_c) b_w s}{f_s (\sin \alpha + \cos \alpha)} \]  \hspace{1cm} (30A-8)

In no case may the value of \((v - v_c)\) exceed \(4.4 \sqrt{f_c}\).