Discussion

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Proposed Revisions to Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary – (ACI 318R-95)

Reported by ACI Committee 318

Discussion by: Allen J. Hulshizer, David Arndt, David Darwin, D. Gene Daniel, Donald R. Strand, Edward G. Nawy, Gerry Weiler, Javeed A. Munshi, Joseph J. Messersmith, M. Nasser Darwish, Neil M. Hawkins, Perry Adebar and Jim Mutrie, and Committee 318 closure with proposed amendments and editorial corrections.

By Allen J. Hulshizer
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Basis of Concern
Energy dissipation is an advanced stop in seismic design away from requiring structures to remain elastic throughout their various loading cycles. The inelastic force reduction is achieved in seismic code document by using reduction factors in elastic design methods. Because of the higher demands on members to perform through seismic-induced inelastic cyclic distortions, special criteria is introduced for joint design to ensure their rotation and load-carrying capacity. The integrity of each structural component is needed to maintain the structure's overall stability. As such, spliced reinforcing bars should be capable of maintaining the full capacity of the connected reinforcing after being subjected to inelastic cyclic loading.

The proposed Chapter 21 provisions for mechanical and welded splice criteria, however, do not appear to have advanced into the same domain as that of the other design requirements. Other code bodies have for sometime incorporated requirements for cyclic inelastic prequalification testing of mechanical splices. Chapter 21 has basically adopted the elastic design requirements of Chapter 12. Type 1 splices are restricted in location and are only required to achieve a minimum $1.25f_y$. The only difference being that Type 2 is required to achieve a minimum strength specified tensile capacity of the reinforcement for unrestricted use. No cyclic inelastic prequalification testing is prescribed for either splice Type in Chapter 21. As written in Chapters 12 and 21, it would be assumed that a single, monotonic tensile test would qualify the splices for use.

Code Section 21.2.6 – Mechanical splices
Code Sections 21.2.6.1 and 21.2.6.2

The use of a Type 1 splice is predicated upon the premise that there is change in the reinforcing strain (from inelastic to elastic) that can be accurately defined. It is questionable that members subjected to high strain reversals in frames going through inelastic distortions can have the exact location, extent of yielding, or both precisely determined. There will some behavior, yielding, and reversals not determined by analysis as a result of concrete and material variations, changes in member reinforcing, seismic force/frequency content, live or storage load concentrations, building irregularities, and exterior and interior walls and partitions not accounted for in stiffness determinations.

When yielding of the reinforcement occurs ("tensile stresses in reinforcement may approach the tensile strength" R21.2.6) the bond between the bars and the concrete cannot be counted on to "unload" the bar, especially when subjected to seismic reversals. Therefore, the stress in the bar will be essentially the same in the cracked zones until it is reduced in a member region where the bond is effective. Under the proposed location criteria, a Type 1 splice could inadvertently be called on to perform as a Type 2 splice as a result of yielding in locations not analyzed or envisioned in the design.

Cyclic testing of a number of different manufacture's splices have demonstrated that not all splices that perform well under 1.25 f_y monotonic loading can produce satisfactory results under cyclic inelastic loading.

Additionally, in a meeting held with most of the major suppliers of mechanical reinforcing splices in the USA, it was unanimous that they would prefer to supply one splice that would enveloped all performance requirements for seismic installations. For construction, the potential for using two splice types within a structural frame would require special quality control to ensure that they were not used in the wrong places. The price increase for using a single, prequalified mechanical splice throughout a seismic structure will be negligible for most manufacturer's products.

Recommendation: Designs under Chapter 21 should have only one mechanical splice type that is prequalified by inelastic cyclic testing with prescribed installation inspection for more involved designs and irregular structures.

Commentary Section R21.2.6
Quote from the second paragraph:

> "If the use of mechanical splices in region of potentials yielding cannot be avoided, the designer should have documentation on the actual strength characteristics of the bars to be spliced, in the force-deformation characteristics of the spliced bar, and the ability of the Type 2 splice to be used to meet the specified performance requirements."

This statement is not consistent with Code Section 21.2.6.2, which states that "Type 2 mechanical splices shall be permitted to be used in any location." By definition Type 2 splices are for *"any location"* and therefore are already approved for use without specific prequalification.

Assuming that *"actual"* means the actual bars being used, it is doubtful that anyone could effectively model the actual characteristics of the bar, its splice, or both, into the analysis of a structure. Most, if not all, splices are stiffer than the bars they connect and are very short with respect to member lengths and their influence would be negligible when compared to the rotations and distortions occurring when yielding and cracking occurs during a design seismic event. Apart from the above, it is highly impractical to impossible to have the actual steel and spliced bar force-deformation characteristics at the time the structure is under design. The construction of most projects will lag the design by four to six months or more, so that it would be necessary to select a given splice product and to purchase all the steel up front and stockpile it in order to obtain actual steel and splice properties. Even steel from the same mill, for the same size bar will exhibit some variations in yield, tensile, and elongation properties (all within ASTM specifications) as produced from different heats as purchased throughout a large job.

Recommendation: Eliminate the quoted statement from R21.2.6.

Code Section 21.2.7 – Welded splices
Welded reinforcing splices should not be categorized as being the same as mechanical reinforcing splices.

Mechanical reinforcing splices are made up of components manufactured under controlled processes and assembled with minimal required workman skills. Mechanical splices can be prequalified by testing with a very high degree of proven, repeatable, in-place performance is easily verified by simple visual examination or by sample or sister testing. Mechanical splices can be installed without detrimental influences associated with weather, accessibility, orientation, and workman fatigue.

Welded reinforcing splices are made in-place, using well-understood and dependable processes, but the in-place quality of welded splices is highly dependent on workman skills, accessibility, weather, and workman fatigue and continued attentiveness. Welded splices cannot be prequalified, only the welding techniques and welders basic skills can be prequalified. The repeatability of successful, in-place welded splices is questionable without a consistent nondestructive testing and a field-sampling program.

Section R21.2.6.1, states that "Welded splices are permitted on reinforcement resisting earthquake induced flexural or axial forces when the welding is performed according to a controlled procedure with adequate inspection." While AWS establishes welding procedures that can be used to qualify welders, there is no prescribed inspection and sampling criteria given in AWS or Chapter 21 to identify what is adequate for reinforcing splices.

It is not sufficient to state that there should be "adequate inspection." This shifts the responsibility on individuals who may not have the background, experience, or understanding to determine what is appropriate (adequate). Engineers using ACI 318 rely on it as a code document for direction and support.

Chapter 12 and Chapter 21 recognize the weakness in trying to obtain a fully welded splice that can consistently develop the full capacity of the bar. As such, the useful strength of a welded splice is limited to 1.25 f_y. Since this recognizes the practical limitations of welded splices, welded splices should not be used in Chapter 21, Seismic Designs, for the same reasons given for not allowing Type 1 splices to be used as discussed under Mechanical Splices above.

Recommendation: Do not permit the use of welded splices in Chapter 21 designs.

Summary
ACI 318, Chapter 5, provides sampling and frequency requirements for confirmation of concrete quality. Reinforcing material is accepted on the basis of certified mill test reports. Admixtures are controlled by ASTM's. There are no criteria, however, for qualifying or confirming the quality of installed splices and the vital link to maintaining the integrity of any structural system, especially those designed to remain stable throughout inelastic distortions and rotations caused by seismic loading. There is every good reason to expect a splice to be as good as the bars they join.

The revisions to reinforcing splice provisions in ACI 318, Chapter 21 are long overdue, but the revisions as now proposed are not consistent with the severity, risks, and performance requirements associated with structural frames designed under Chapter 21. The following is a synopsis of my comment on the proposed Chapter 21 changes for reinforcing splices:

- Prequalification testing and installation inspection requirements should be prescribed for all splices used in structural frames and systems.
- Mechanical splices should be prequalified by testing to ensure the specified tensile strength of the reinforcement is achieved after significant cyclic inelastic deformations.
- Only one type of mechanical splice should be permitted for use in Chapter 21 designs.
- Welded splices should not be permitted in any region of structural frames designed under Chapter 21.
- If it is insisted that welded splices be allowed in Chapter 21 designs, there should be a rigid prescribed inspection and sample testing criteria established for installed splices to ensure that the splice can fully develop the capacity of the reinforcement under cyclic loading.

By David Arndt
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Code Section 21.0 – Notation for *c*

1) Add subscript to *c* to clearly distinguish it from general notation for *c* in Section 9.0.

2) Change "consistent with the design displacement δ_u resulting in the largest neutral axis depth" to "consistent with the direction of displacement resulting in the largest neutral axis depth". The factored axial force and nominal moment strength, not δ_u, set the value of *c*. The design displacement reference seems extraneous and confusing.

Code Section 21.1 – Definitions for "Design load combinations" and "Factored loads and forces"

1) At the end of sentence, change "… in 9.2" to "… in 9.2, or as specified by the governing code for earthquake-resistant design." This change would facilitate having ACI 318 as a referenced code in the International Building Code.

Code Section 21.6.6.4(f)

How does the splice location limitation in Sections 21.2.6.2 and 21.2.7.1, "within a distance equal to twice the member depth" apply to mechanical and welded splices in a special boundary element where the member depth is the length of the wall?

Code Section 21.7.7.1

Why not keep α_c in Eq. (21-10) as was the case in previous codes and is still the case for structural walls in Eq. (21-6)?

Code Section 21.9.3.3

Add 21.4.3 to sections to be satisfied.

By David Darwin
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On page S-20, Commentary Section R11.6.3.7 – It should be "$N_i = V_i \cot \theta$" not "$V_i / \cot \theta$".

On page S-25, Code Section 13.3.8.6, line 4 – change "practicable" to "practical".

On pages S-27 and S-28 – It is not clear, but it looks like some of the old notation is being left out of Chapter 14. These may just be additions, but it is confusing since A_g appears in ACI 318-95, but the other ACI 318-95 notations have been left out. It's still needed.

By D. Gene Daniel
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This particular comment is directed to the changes proposed in **Section 5.6 – Evaluation and acceptance of concrete.**

The 318 committee has appropriately decided to include a reference to qualified field and laboratory technicians in Section 5.6. The proposed new Section 5.6.1 leaves the determination of what defines a qualified field testing technician or a qualified laboratory testing technician to the ACI 301 committee or others. As a code, this approach is very much in keeping with the foreword of ACI 318.

The Commentary Section R5.6.1 proposed to accompany the new section does give me cause for concern. The proposal for this commentary includes both the ACI technician certification programs and ASTM C 1077 as proof of qualifications to perform both field and laboratory testing of concrete.

The 318 committee and the ACI membership need to be aware of the major differences in the certification requirements of ACI and ASTM C 1077. Without evaluating the difference between the ACI certification program and the ASTM Standard C 1077, it is difficult to evaluate their compatibility as mutual measuring devices of technician qualifications.

The ASTM standard (C1077) presently accepts the certification of the National Institute for Certification in Engineering Technologies (NICET) via a nonmandatory note. The current proposal of ASTM committee C09.98, which has jurisdiction for C 1077, is to move the NICET certification into the text and give it equal status with the ACI certification program.

From a technical standpoint there are some major differences in the ACI and NICET technician programs. A short list of major differences follows:
ACI Certification

- Closed book written examination;
- One to 1 ½ minutes per exam question;
- Overall score of 70% required for complete written exam;
- Applicant must pass all standards (elements) on exam;
- Minimum score of 60% on each standard on exam; and
- Performance test required under supervision of an unbiased examiner with a score of 100% required for certification.

NICET Certification

- Open book written examination;
- Three to four minutes per exam question;
- No overall score requirements for the complete exam;
- Must pass a specified number of available test elements;

- Minimum score of 60% on each standard that is passed, but there is no requirement to pass all standards on the test; and
- No performance test required; only requires statement of supervisor.

When examined closely on each of these six items, it becomes very clear that the NICET certification program is not equal to the ACI certification program. I believe the 318 Commentary needs to limit recommended certifications to the ACI program or an equivalent program requiring both written and performance examinations.

This will provide every opportunity to use a program advanced by someone other than ACI but sets the high standard needed by the concrete industry. The change to only an ACI reference will also prohibit the lowering of testing standards by outside special interest groups.

When the Commentary refers to "or an equivalent program", there must be a gauge by which to judge a proposed equivalent program. If two widely divergent certification programs are each acceptable, how do we define equivalent? There must be a single gauge by which to judge other programs. That gauge should be the ACI certification program.

Commentary Section R 5.6.1
The suggested proposal is:

> Laboratory and field technicians can establish qualifications by becoming certified through certification programs. Field technicians in charge of sampling of concrete; testing for slump, unit weight, yield, air content, and temperature; and making and curing test specimens should be certified in accordance with the requirements of ACI Concrete Field Testing Technician – Grade 1 Certification Program, ~~or the requirements of ASTM C 1077,~~ or an equivalent program containing both written and performance examinations. Concrete testing laboratory personnel should be certified in accordance with the requirements of ACI Concrete Laboratory Testing Technician, Concrete Strength Technician, or ~~the requirements of ASTM C 1077~~ equivalent certification programs containing both written and performance examinations.

Testing reports should be promptly distributed to the owner, licensed design professional responsible for the design, contractor, appropriate subcontractors, appropriate suppliers, and building official to allow timely identification of compliance or need for corrective action.

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Section R1.3.1
The last paragraph on page S-5 refers to “inspection independent of the licensed design professional…”. Some building departments (Los Angeles City) allow another professional engineer to take over the construction observation at the owner’s discretion. Thus, the design engineer may not be retained to verify performance and may not want to see inspection reports as defined in this Section because of liability issues.

Section 1.3.5 and R1.3.5
Add special reinforced concrete structural walls and coupling beams under this Section.

Section 2.1
Anchorage Zone – In the third line, change “widely” to “uniformly.” In the last sentence, the meaning of “disturbed regions” needs clarification.

Pedestal – In the second line, change “average at” to “the”.

Section R5.6.1
In the second paragraph, the design professional may not be handling the construction. See comment Section R1.3.1

Section R9.2.3
Service-level and strength-level should be defined and possibly be italicized to differentiate its usage.

Section R9.3.4
In the last paragraph, second line, change “damaged” to “damage.”

Section 10.0
Under (c), change “ration” to “ratio”.

Section R12.15.4
At second paragraph, second line, change “at” to “as”.

Section 21.0
Under ρ_n – Why not delete “perpendicular to that reinforcement,” because it is superfluous and would read similar to ρ_v.

Section 21.1
Please define what is an “ordinary wall”, as used for seismic requirements. It may be advisable to italicize words like “ordinary”, “design basis”, and “design displacement” to differentiate between common usage and special type of procedures.

Section R21.1
In the second line, delete “in design.” Also delete the last sentence since it is repeated better in the first paragraph of R21.2.1.

Section R21.2.1
See top of page S-48 regarding “ordinary” as to definition. See comment under 21.1.

Section 21.2.7.2
In the second line, change “by design” to “for design forces.”

Section 21.4.3.2
A period should be added after “tensions lap splices,” in the third sentence and add “All splices shall…” The reason is to make sure that longitudinal bar buckling does not occur at mechanically spliced or lapped splices. Under 21.2.6.1 mechanical or welded splices are allowed anywhere and could be in the plastic hinge areas as well.

Section 21.6.6.2
(a) Note under equation that Eq (21.x) is “21.8.”
Under (b) change the second line to “…shall extend vertically from the critical section a distance not less than the larger or ℓ_w or $M_u/4V_{u.}$” Delete “along the wall”.

Rather than always having to do a study for conformance to Eq. (21-8), can’t the UBC 1921.6.6.4 exceptions be added to this Section and save the designer unnecessary time of analysis?

Section R21.6.6.2
The second sentence of the first paragraph is very confusing. Can’t a figure be presented on what all this Section means and how Eq. (21-8) was derived. In the fourth line of the second paragraph, add “occurring” after value.

Section 21.6.6.4
Under (d), at a footing it seems that only two or three confinement ties is insufficient. It is suggested that the length be increased to one-half of the embedment length.

Under (f), why not just refer to recommended change to Section 21.4.3.2 or duplicate for this Section.

Section 21.7.5.3
The first sentence refers to 21.4.4.1 through 21.4.4.3. Since this Section only pertains to compressive members, geometry of the cross-section is not critical and it is only necessary to prevent buckling or rebars. Thus, in Section 21.4.4.2, (a) is not necessary since (b) and (c) relate to bar buckling.

Section 21.7.8.2
Same as 21.6.6.4(f).

Section 21.7.8.3
Under (a), it is assumed that the requirements pertain to longitudinal bars.

Section 21.8.2.2
Requiring the tails of the 90 degree hooks to be placed inside the column core is impractical and unnecessary. Development of columns to footings is different than beams to columns since the load taken at the inner radius of the bent bar can be resisted by a thick concrete mass versus a finite column dimension. Also heavy column reinforcement probably could not be made to conform to this provision because of overlapping of bars and lack of space. The referenced tests in the Commentary are primarily for slabs to walls or columns to beams and are correct for these conditions.

Section 21.8.3.3
This section also applies to compressive and tensile loads for grade beams at braces or walls. Add note to include members with compressive stresses exceeding $0.2 f'_c$.

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The proposed section replacing the current z-factor approach of ACI 318-95 Code seems to be essentially the result of taking the basic Gergely-Lutz equation and manipulating it into a reinforcement spacing provision. As clarified below, such an approach does seem to have practical engineering merit when applied to normal beam sections. As founding chairman and continuing member of ACI 224 on Cracking, I have over the years since 1966 when the ACI Board assigned me the task of assembling a committee on cracking, followed the subject of cracking deflection behavior very closely. Hence, I feel a duty to the profession to present this discussion.

One argument advanced for the new proposal and against the current ACI provisions is that the z-factor approach breaks down when a reinforcement cover, d_c, considerably higher than 2 in. is applied to the expression, as some found when using a concrete cover of 3 to 4 in. in the z computation. In such a case, the result is naturally very high z values beyond the permissible level because of the artificially high value of the concrete area in tension, *A,* assumed in the computations. But the increase in the crack width from the level of reinforcing bars to the tensile surface of the member is not linearly proportional to the increase in the cover value. The width increases by a much reduced gradient irrespective of the increase in the cover thickness. Neglecting this hypothesis and applying to the z-expression a code-unintended cover beyond 2 to 2 ½ in. penalizes the bar size and spacing choices that are made in the flexural design. Also, what counts from a corrosion protection viewpoint is the crack width at the steel level and an erroneous use of a larger value of the *A* parameter with a resulting higher z value is not sustainable.

Consequently, this argument for the proposed provisions is not a reason for dumping the present tested provisions of ACI 318-95.

Therefore, any value beyond the standard value of d_c for normal structures in the range of 2 in. should not be used in evaluating the area of concrete in tension, *A,* when computing z for crack control determination. The AASHTO Code (Sec. 8.16.8.4) precisely follows this dictum whereas ACI does not, hence the confusion caused when using the current ACI provisions but exceeding the code-intended concrete cover thickness level. A minor addition to the definition of the term A in the current provisions can easily rectify this problem.

The work in [1] that formed the basis of the proposed provisions is commended for the detailed study of the concrete cover and its interrelationship to flexural cracking. Its thesis is that crack control can be effected through bar spacing control. I do subscribe to this proposition, but only if it is intended for one-way slabs. Normal beams, however, have finite web widths of restrictive ratios of web width to web depth in the range of 1/3 to 2/3 as practical limits. Beams are not usually wide bands of shallow depths and wide webs as are sometime used in a few parking reinforced concrete garages. This observation is equally pertinent to prestressed concrete beams where control of cracking by using the spacing of few mild steel bars as the criteria would have no practical significance.

And what about crack control in structures subjected to severe environmental conditions including sanitary structures and those using high-strength steel reinforcement. The proposed provisions using bar spacing and not crack width as embodied in the present z-factor approach could even be detrimental. Research has demonstrated [2,3,4] that the crack width long-term could double or triple, making it vital to control this aspect through crack width control rather than reinforcement spacing control as bar spacing control for this purpose can often be impractical to apply to normal beams web width whether reinforced or prestressed. This is why the CEB-FIP Model Code Provisions, as extensive as they are, require use of crack width expressions and have not found it logical to control cracking by simply recommending spacing of bars in beams as the answer.

Another argument is sometimes made that the z-factor has litigation risk inherent in its use. I am not aware over all these years of any documented court case on structural distress that resulted from application of the present 318-95 or previous similar ACI 318 Code provision on crack control [2] or the use of ACI 224 Table on tolerable crack widths. Also, consideration needs to be made that in normal beam sizes, as used in most present-day structures, the proposed provisions do not necessarily always give results that conform to the present safe ACI 318-95 provisions that have been successfully used by the engineering profession since 1971. There is a saying “if it ain’t broke don’t fix it.” I believe that this logic equally applies in this instance.

In summary, crack width seriously affects corrosion behavior. Crack width control is essential, particularly when high-strength reinforcement is used. The present provisions should not be discarded with possible detrimental long-term effects on future construction systems. ACI 318 would be wise to keep the present provisions, but amend them by restricting them to beams, using the minimum cover depth, d_c, in the z-factor evaluation, and amending the definition of the term A, similar to the AASHTO practice. For one-way slabs and flanges of T-beams in the negative moment region, I recommend inclusion of the proposed provisions in the forthcoming ACI 318-99 Code as a good criteria to supplement the z-factor provisions, because it limits the bar spacing to a maximum of 12 in. on centers as should be the case in all slabs and plates.

References

1. Oesterle, R.G., 1997, "The Role of Concrete Cover in Crack Control Criteria and Corrosion Protection," PCA R&D Serial No. 2054, Portland Cement Association, Skokie, IL.

2. ACI Committee 224-90 "Control of Cracking in Concrete Structures."

3. Nawy, E.G., 1968, "Crack Control in Reinforced Concrete Structures," Journal Proceedings, *ACI Journal,* Farmington Hills, MI, pp. 825-835.

4. Nawy, E.G. and K.W. Blair, January 1973, "Further Studies on Flexural Crack Control in Structural Slab Systems," ACI SP-30-1, American Concrete Institute, Farmington Hills, MI, 1971, pp. 1-42 and Discussion by ACI 318 Steering Committee and Author's closure, pp. 61-63.

Gerry Weiler
Chairman, ACI Committee 551
Reporting for Committee 551

We would like to summarize some important points regarding proposed Section 14.8 draft as originally prepared by 318 Committee:

1) The term "tilt-up" should be referenced in the text, because it has come into our lexicon and is widely used in the industry to differentiate it from its root term, "precast." The latter is too broad a term, comprising multiple industries with varying applications and structural concrete (such as, ACI Committees 533 and 550). We propose the following definition be added to Chapter 14:

> **Tilt-up** – A variant of precast concrete, where wall elements are cast horizontally on site, in proximity to their final position in the structure, and raised in one operation by a tilting action about a lower edge.

Adding this definition will help to avoid conflicting requirements and give this major segment of the construction industry code identity.

2) Our committee agrees with the concept of converting the UBC Slender Wall method to ACI style format. We believe this should also include adoption of the moment magnifier methodology for evaluation of PΔ slenderness effects, similar to the format for slender column design in ACI 318, Chapter 10. It can be shown that the results obtained by the moment magnifier method essentially agree with both the UBC method and a PΔ iteration. To illustrate, we have included sample calculations comparing the various approaches.

The requirement for Δ_u, as proposed by Committee 318 in Eq. (14-5) of New Code Section 14.8.3, will underestimate the final deflection and resulting maximum moment. It is necessary to use several iteration cycles to converge on the maximum PΔ moment, or use the moment magnifier direct solution.

3) The proposal by Committee 318 for deflection control is based on factored loads instead of service loads. This is contrary to established requirements in most codes and design standards, including ACI 318 Section 9.5, which states in Commentary R9.5.1:

> The provisions of 9.5 are concerned only with deflections or deformations that may occur at service load levels. Where long-term deflections are computed, only the dead load and that portion the live load which is sustained need be considered.

Committee 551 submits that controls for lateral deflections should be based on sustained service loads. Factored load deflections are self-limiting by the PΔ analysis when evaluating the ultimate strength on the wall section.

The relationship between service load deflections and factored load deflections is not simply proportional to the load factor. This is due to the nonlinear effects of PΔ deflections. In other words, a deflection limitation of $\ell_c/1000$ on factored loads is not the same as a $\ell_c/150$ limit on service loads. The majority of tilt-up buildings constructed would not meet the proposed $\Delta_n<\ell_c/100$ criteria, and there is little or no evidence to suggest that the performance of these buildings has been unsatisfactory.

Recommended Changes to Proposed Revisions to ACI 318, Chapter 14 – Walls
Committee 551 recommends revising the following sections to read:

Code Section 14.0 – Notation

A_g = gross area of the wall segment = bh, in.^{2}
A_s = area of vertical tension reinforcement in the wall segment, in.^{2}
A_{se} = area of effective vertical tension reinforcement in the wall segment, in.^{2}, see Eq. (14-9)
b = width of wall segment, in.
c = distance from extreme compression fiber to neutral axis, in.
d = distance from extreme compression fiber to tension reinforcement, in.

E_c = modulus of elasticity of concrete, see 8.5.1
f_c' = specified compression strength of concrete, psi
h = wall thickness, in.
I_e = effective moment of inertia for computation of service load deflections, in.^{4}, see 9.5.2.3
k = effective length (height) factor
K_{bu} = bending stiffness of wall segment due to service loads, see Eq. (14-7)
K_{bs} = bending stiffness of wall segment due to service loads
ℓ_c = vertical distance between supports
M_{cr} = moment causing flexural cracking in the concrete section
M_n = nominal moment strength of wall segment
M_{as} = maximum unfactored "applied moment" due to service loads, not including PΔ effects.
M_s = maximum unfactored moment due to service loads, including PΔ effects
M_{au} = maximum factored “applied moment” at mid height, not including PΔ effects
M_u = maximum factored moment at mid height, including PΔ effects
P_s = unfactored axial load applied at the top of the wall
P_{ms} = unfactored axial load at the design (mid height) section including effects of self weight
P_u = factored axial load at the top of the wall
P_{mu} = factored axial load at the design (mid height) section including effects of self weight
Δ_s = maximum deflection at or near mid height due to service loads
Δ_u = maximum deflection at or near height due to factored loads
ϕ = strength-reduction factor, see 9.3
ϕ_k = stiffness-reduction factor
ρ = ration of tension reinforcement $=A_s/(b\ d)$
ρ_b = reinforcement ratio producing balanced strain conditions, see 10.3.2

Commentary Section R14.0

For b, it is sometimes necessary to divide the length of the wall into segments in order to analyze the effects of openings or concentrated loads.

Code Section 14.8 – Alternate design of slender walls

Commentary Section R14.8

Section 14.8 is based on existing provisions in the Uniform Building Code and experimental research.[14-xx]

The procedure is presented as an alternative to the requirements of 10.10 for design of slender walls as commonly used in precast or tilt-up construction. It is limited to wall elements subjected to combined axial and out-of-plane forces that span vertically and are laterally supported at the top and bottom.

Design requirements for in-plane shear forces are not included in this section.
Many aspects of the design of tilt-up walls and buildings are discussed in Reference 14-ZZ. Research [14-yy] has highlighted problems with the seismic performance of panel connections to other building components such as wood roof structures. This should be addressed by the designer.

Code Section 14.8.1

The provisions of 14.8 shall apply to reinforced concrete walls that do not meet the requirements of 14.5 and 14.6. Due to high slenderness ratios, they shall be designed for second order $P\Delta$ effects to ensure structural stability and satisfactory performance under service loads.

Code Section 14.8.2
When tension in the reinforcement due to flexural bending controls the design of walls, the requirements of 14.8 are considered to satisfy 10.10.

Code Section 14.8.3
Walls designed by the provisions of 14.8 shall satisfy the following Code Sections 14.8.3.1 through 14.8.3.7.

Commentary Section R14.8.3

Maximum moments in load-bearing slender walls usually occur at or slightly above the mid-height section for simply supported conditions.
Applied moments should include the effects of axial load eccentricities.

Effects of end fixity, or continuity of the wall element over multiple supports can be included in the procedure by use of the effective length factor k. The value of k should not be less than 0.9.

The effect of reveals, exposed aggregate or other features should be taken into account when evaluating the properties of the design section.

Wall panels with large openings should be designed taking into account the reduction in cross section. Supporting strips each side of the opening and spanning vertically may be designed to resist tributary loads from the openings. The effective width of the supporting strips should not exceed about 12 times the thickness of the wall.

Code Section 14.8.3.1
Walls shall be designed as simply supported members, spanning vertically between supports. They are subjected to out-of-plane lateral loads and eccentric axial loads with maximum moments occurring at or near mid height.

Code Section 14.8.3.2
The cross section properties shall be assumed constant over the height of the wall segment.
Code Section 14.8.3.3
The ratio of vertical reinforcement A_{se}/bd shall not exceed 0.75 ρ_b, see 10.3.2.

Code Section 14.8.3.4
Where vertical reinforcement is placed in two layers, the effect of compression reinforcement shall be ignored.

Code Section 14.8.3.5
The area of vertical tension reinforcement shall provide a minimum design strength of

$$\phi M_n \geq M_{cr}$$

except where the area provided is at least one-third greater than that required by analysis.

Code Section 14.8.3.6
Effects of wall self-weight shall be included in the analysis. The weight of the wall segment above the design section (mid height) shall be applied as a concentric axial load at the top of the wall.

Code Section 14.8.3.7
Concentrated axial loads applied to the wall above the design section shall be assumed to be distributed over a width:

(a) equal to the bearing width, plus a width on each side that increases at a slope of 2 vertical to 1 horizontal down to the design section; but
(b) not greater than the spacing of the concentrated loads; and
(c) not extending beyond the edges of the wall.

Code Section 14.8.3.8
Vertical stress P_{mu}/A_g at the design section shall not exceed 0.06 f_c'.

Code Section 14.8.4 – Evaluation of PΔ moments
The moment strength ϕM_n of the wall segment shall be:
Where

$$\phi M_n \geq M_u \qquad (14-3)$$

$$M_u = M_{au} + P_{mu}\Delta_u \qquad (14-4)$$

$$\Delta_u = \frac{5 M_u \ell_c^2}{\phi 48 E_c I_{cr}} = \frac{5 M_u}{\phi K_{bu}} \qquad (14-5)$$

M_u is obtained by iteration of deflections or by direct calculation:

$$M_u = \frac{M_{au}}{1 - \frac{P_{mu}}{\phi_k K_{bu}}} \qquad (14\text{-}6)$$

$$K_{bu} = \frac{48\, E_c I_{cr}}{5 \ell_c^{\,2}} \qquad (14\text{-}7)$$

$$I_{cr} = nA_{se}(d - c)^2 + bc^3 / 3 \qquad (14\text{-}8)$$

$$A_{se} = \frac{P_{mu} + A_s f_y}{f_y} \qquad (14\text{-}9)$$

Commentary Section R14.8.4

Moment magnifier Eq. (14-6) provides a direct solution for the iteration of PΔ deflections by combining Eq. (14-4) and (14-5). K_b represents the ratio of maximum moment divided by maximum deflection.

Bending stiffness is reduced by the stiffness-reduction factor ϕ_k. This effectively increases deflections and PΔ effects, see 10.10. ϕ_k should be taken as equal to ϕ.

For evaluating strength using factored loads, the concrete section is to be assumed fully cracked over the height of the wall. The cracked section stiffness I_{cr} is based on an equivalent rectangular concrete stress block.

A_{se} is a modification in the area of reinforcing to partially account for the effects of small axial loads. It is used in the evaluation of both M_n and I_{cr}.

Service load deflections have traditionally not been a problem in slender concrete walls. Maximum moments are often controlled by lateral wind loads that are largely transient in nature.

The deflection limits provided are applicable to industrial applications such as warehouse walls. More stringent limits should be considered where excessive deflection might result in damage to attached components. See 9.5 for further discussion.

Code Section 14.8.5 – Control of deflections

The mid-height deflection Δ_s due to sustained service loads, including PΔ effects shall not exceed $\ell_c/100$, but not greater than can be tolerated by attached structural or nonstructural elements. The mid-height deflection Δ_s shall be determined by:

$$\Delta_s = \frac{5 M_s \ell_c^{\,2}}{48 E_c I_e} = \frac{M_s}{K_{bs}} \qquad (14\text{-}10)$$

$$M_s = \frac{M_{as}}{1 - \frac{P_{ms}}{K_{bs}}} \qquad (14-11)$$

$$K_{bs} = \frac{48 E_c I_e}{5 \ell_c^2} \qquad (14-12)$$

I_e shall be evaluated using the procedure of Section 9.5.2.3, substituting M_s instead for M_a.

I_{cr} shall be evaluated using Eq. (14-8).

Discussion of Proposed Changes to ACI 318, Chapter 14 – Walls

The following comments are provided by ACI Committee 551

14.0 – Notation

a. Reference to tilt-up construction should be included.

b. The parameter "ℓ_w" has been replaced with "*b*", representing the width of the wall segment under consideration. In most calculation procedures for slender wall design, a unit width of 1 ft is used.

c. This definition's section should be expanded to include additional parameters.

d. The bending stiffness parameter K_b is introduced. This is simply the ratio of maximum moment to maximum deflection, and is helpful in illustrating the concept of moment magnifier for PΔ calculations.

e. The term "vertical load" should be replaced with "axial load." Note that axial load is frequently applied at an eccentricity to the centerline axis of the wall.

New Code Section 14.8 – Alternate Design of Slender Walls

f. This new code section is intended only for design of simply supported slender walls spanning vertically. Effects of end fixity and panel continuity are not specifically included at this time.

g. The procedure in UBC 1997 is based on factored load design, not working stress.

h. Section 10.3.3 recommends a limit of the reinforcement ratio to 0.75 ρ_b in order to provide assurance against brittle failure (concrete crushing). Committee 551 sees no justification for a further reduction to 0.6 ρ_b. The limit should, however, be based on the effective area of reinforcement A_{se} because this includes an allowance for axial load.

i. Effects of wall self weight are an important consideration in the PΔ analysis and should be included as a code requirement.

Section 14.8.4 – Evaluation of PΔ Moments

j. This section provides procedures for evaluating the strength of the wall, including PΔ effects. Eq. (14-5) uses the parameter M_u instead of M_{au} as proposed by Committee 318. Otherwise, the bending moments may be seriously underestimated since M_{au} would limit the iterative process to only once cycle.

k. In order to bypass the iteration procedure for moments and deflections, a direct calculation (moment magnifier) is proposed. The moment magnifier Eq. (14-6) will give the same results as a complete iterative process and is in keeping with the methodology in Section10-10 for the design of slender columns.

l. Bending stiffness should be reduced by the stiffness reduction factor ϕ_k, as required in Section 10-10. The value of ϕ_k should be made equal to the stiffness reduction factor ϕ. For most tilt-up wall panels, ϕ will be in the range of 0.85 to 0.88. Note that a stiffness reduction of 0.80, as suggested for slender columns Section 10-10, is considered to be too severe for slender walls since axial loads are small and the failure mechanisms is flexural yielding of tension reinforcement, not concrete compression. The final results obtained by the proposed method will be the same as those based on the current procedure of UBC.

Section 14.8.5 – Control of Deflections

m. Committee 318 has proposed a limitation for factored load deflections of $\ell_c/100$. This can be significantly more conservative than a limit on service load deflections of $\ell_c/150$ and is inconsistent with current requirements in both ACI 318 and UBC. Factored load deflections are important for evaluating PΔ effects at ultimate strength. The procedures outlined in Section 14.8.4 are self-limiting for factored out-of-plane deflections.

n. Deflection limits based on service loads are more appropriate although there is some question as to whether any controls at all are necessary for industrial buildings. The ACI/SEASOC "Green Book" recommends a service load deflection limit of $\ell_c/100$. This has traditionally been adequate for warehouse walls where deflections have not been a problem. Greater limits may be required for nonindustrial applications, but these should not be arbitrarily set without further study. Committee 551 will expand on this in a design guide for slender walls.

o. The revised equations for Section 14.8.5 employ the same procedures as 14.8.4 for evaluating PΔ deflections, using service loads instead of factored loads. It is noted that a reduction in bending stiffness (by factor ϕ_k) has not been included in the calculations for service load deflections. This is consistent with the procedures outlined in Section 9.5.2.3 and UBC.

p. Design requirements for panel lifting and in-plane shear forces have not been included at this time.

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The committee needs to be commended for simplifying Section 10.6.4. The proposed Eq. (10-5) is simple, direct, and easy to use. I have the following comments/questions, however, on 10.6.4 published in November 1998 *Concrete International.*
The new provisions significantly relax the spacing requirements for covers between 2 in. and 4 in. [Fig. 1] when compared to the current provisions. The new method, however, does not distinguish between different exposure conditions. The required spacing for crack control is independent of the exposure condition. This results in significant difference in spacing between the current and the new procedure for elements exposed to severe exposure (see Fig. 1, z = 145 kips/in). This is a rather drastic change. In essence, the new provisions suggest that exposure condition has no bearing on the required reinforcement spacing for crack control. Since corrosion should be somewhat exposure sensitive, does this also mean that spacing of reinforcement has no relation to corrosion?

The question also arises whether the new provisions are geared for corrosion resistance. The 1995 provisions were somewhat explicit in this by including the terms such as inside (mild) exposure (z =175 kips/in.) and outside (severe) exposure (z = 145 kips/in.) that gave a sense that corrosion was being taken care of. The new provisions make no mention of the corrosion aspect at all. The reasons for change and the commentary 10.6.4 indicate that there is no relation between crack widths of 0.016 in. and 0.013 in. used previously for the two exposure conditions and corrosion. The new provisions, however, are still based on the crack width of 0.016 in. as stated in Reason 2. It follows that by limiting the crack width to 0.016 in., perhaps, the new provisions are geared more for preserving the appearance of concrete [Reference 1]. The background literature [References 2, 3] suggest that if crack widths are limited to 0.016 in., it should result in good corrosion protection. If this crack width is to be maintained despite the cover thickness, the concept is essentially the same as used previously. Why is it then said that corrosion has no relation to the crack width?

The new Eq. (10-5) has a term $2.5c_c$ that reduces the reinforcement spacing for larger covers (presumably to limit the crack width to 0.016 in.?). For Grade 60 steel (f_s = 36 ksi) and c_c varying from 2 in. to 6 in., the required reinforcement spacing drops linearly from 10 in. to 0 in., respectively (Fig. 1). There is no lower limit on reinforcement spacing in the new provisions. After studying the background material [Reference 3] related to this, it seems that there is no advantage of increasing cover beyond 3 in. for corrosion resistance. If one were to use this 3 in. limit and back substitute it in Eq. (10-5), this would result in spacing of 7.5 in., which could be used as a practical lower limit on the spacing. This would remove the unnecessary tightening of spacing required by Eq. (10-5) for covers larger than 3 in. Note this is similar to the concept of assuming that cover in excess of 3 in. is sacrificial [see Reference 4] and spacing computations should be based on clear cover of 3 in. when cover exceeds 3 in.

The Commentary Section R10.6.4 states that "new provisions for spacing are intended to control surface cracks to a width that is generally acceptable in practice but may widely vary in a given structure." How is it rationalized that predicted crack width will be different when the equation used for crack control is based on a predicted crack width of 0.016 in. ? Why use this particular value of 0.016 in. when the predicted crack widths vary by as much as "± 50%"? It seems that while a reason is given not to get hung up on crack width of 0.016 in., yet the new method uses the same value for predicted crack width. Why?

References

1. Mast, R.F., "ACI 318 - Crack Control Provisions" (unpublished report)

2. Darwin, D. et al., May 1985, "Debate: Crack width, cover, and Corrosion" *Concrete International*, V.7, No. 5.

3. Osterle, R.G., 1997, "The Role of Concrete Cover in Crack Control Criteria and Corrosion Protection," RD Serial No. 2054, Portland Cement Association, Skokie, IL.

4. Rice, P.F., October 1984, "Structural Design of Concrete Sanitary Structures," *Concrete International*.

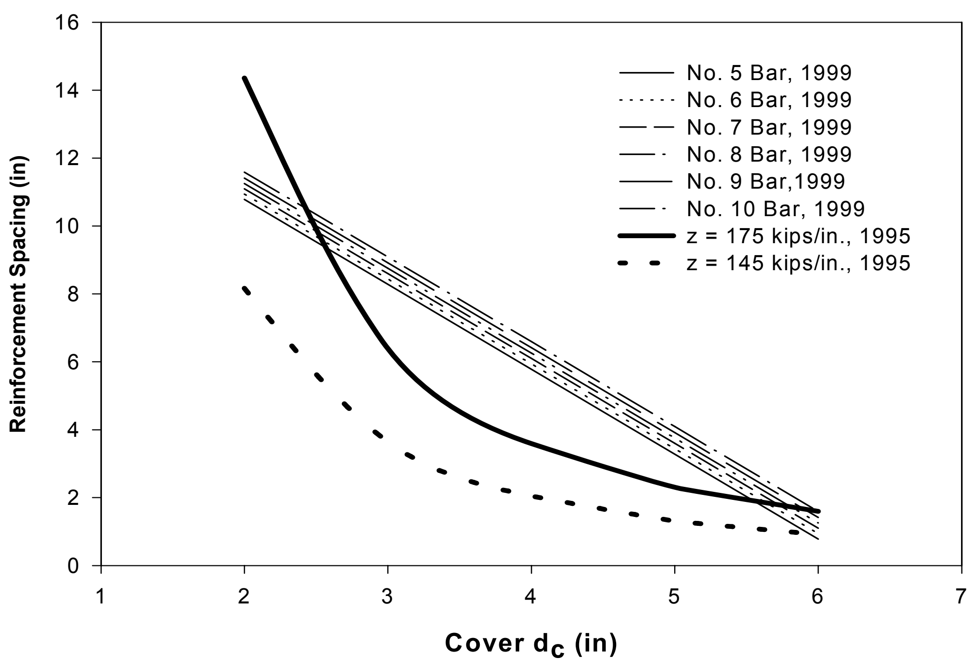

Fig. 1 Comparison of reinforcement spacing for slabs per 10.6.4 of 1995 and 1999 codes

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The following are suggested changes to the proposed revisions, followed by reasons for the suggested changes.

Commentary Section R1.1

Also add a new paragraph to the end of R1.1 as follows: "Appendix C of the Code allows the use of the ~~load~~ factored load combinations in Section ~~2.4~~ 2.3 of ASCE 7, 'Minimum Design Loads for Buildings and Other Structures' if structural framing includes primary members of materials other than concrete."

Reason: ACI 318-99 references ASCE 7-95 and the section of ASCE 7-95 that has the strength design load combinations is 2.3. The other change is editorial.

Commentary Section R1.1.8.1

For ~~buildings~~ structures located in regions of low seismic risk, or for structures assigned to low seismic performance or design categories, no special design or detailing is required; the general requirements of the main body of code apply for proportioning and detailing of reinforced concrete ~~buildings~~ structures. It is the intent of Committee 318 that concrete structures proportioned by the main body of the code will provide a level of toughness adequate for low earthquake intensity.

Reason: For consistency in terminology (see 1.1.1).

Commentary Section R1.1.8.2

For ~~buildings~~ structures located in regions of moderate seismic risk, or for structures assigned to intermediate seismic performance or design categories, reinforced concrete moment frames proportioned to resist earthquake effects require some special reinforcement details, as specified in 21.~~9~~10 of Chapter 21 for intermediate moment frames. The special details apply only to ~~frames (~~beams, columns, and slabs~~)~~ to which the earthquake-induced forces have been assigned in design. The special details are intended principally for unbraced concrete frames where the frame is required to resist not only normal load effects, but also the lateral load effects of earthquake. The special reinforcement details will serve to provide a suitable level of inelastic behavior if the frame is subjected to an earthquake of such intensity as to require it to perform inelastically. There are no special requirements for structural walls provided to resist lateral-force effects of wind and earthquake, or other ~~non~~structural components that are not part of the lateral-force-resisting system of ~~buildings~~ structures located in regions of moderate seismic risk, or for structures assigned to intermediate seismic performance or design categories. Structural walls proportioned by the main body of the Code are considered to have sufficient toughness at anticipated drift levels ~~in regions of moderate seismicity~~for these structures.

For ~~buildings~~ structures located in regions of high seismic risk, or for structures assigned to high seismic performance or design categories, all ~~building~~ components~~, structural and~~

~~nonstructural~~ that are part of the lateral-force-resisting system, including foundations, must satisfy requirements of 21.2 through 21.~~7~~8 of Chapter 21. In addition, frame members that are not assumed in the design to be part of the lateral-force-resisting system must comply with 21.9 of Chapter 21. The special proportioning and detailing requirements of Chapter 21 are intended to provide a monolithic reinforced concrete structure with adequate "toughness" to respond inelastically under severe earthquake motions, see also R21.2.1.

Reason: Many of the changes are intended to maintain consistency of terminology. Another change clarifies that in structures at intermediate seismic risk there are no special requirements for structural members other than those in intermediate moment frames. Changes being suggested to the last paragraph more clearly portray the intent by distinguishing between components that are part of the lateral-force-resisting system and frame members that are not. In two places section numbers are changed for consistency with the new numbering scheme of Chapter 21.

Code Section 1.1.8.3

The ~~S~~seismic risk level of a region, or seismic performance or design category of a structure, shall be regulated by the legally adopted general building code of which this Code forms a part, or determined by local authority.

Commentary Section R1.1.8.3

Seismic risk levels (Seismic Zone Maps) and seismic performance or design categories are under the jurisdiction of a general building code rather than ACI 318. Subtle changes in terminology were made to the 1999 edition of the Code to make it compatible with the latest editions of model building codes in use in the U.S. For example, the phrase "seismic performance or design categories" was introduced. Over the past decade, the manner in which seismic risk levels have been expressed in U.S. building codes has changed. They have been represented in terms of seismic zones. Recent editions of the BOCA *National Building Code* (NBC) and *Standard Building Code* (SBC), which are based on Reference 1.X (91 NEHRP), have expressed risk not only as a function of expected intensity of ground shaking on solid rock, but also on the nature of the occupancy and use of the structure. These two items are considered in assigning the structure to a *Seismic Performance Category* (SPC), which in turn is used to trigger different levels of detailing requirements for the structure. The *International Building Code* (IBC), to be published in early 2000, also uses the two criteria of the NBC and SBC. It goes one step further, however, and also considers the effects of soil amplification on the ground motion when assigning seismic risk. Under the IBC, each structure is assigned a *Seismic Design Category* (SDC). Among its several uses, it triggers different levels of detailing requirements. Table R1.1.8.3 correlates low, moderate/intermediate, and high seismic risk, which has been the terminology used in this code for several editions, to the various methods of assigning risk in use in the U.S. under the various model building codes, the ASCE 7 standard, and the NEHRP Recommended Provisions.

In the absence of a general building code…local authorities (engineers, geologists, and building officials) should decide on proper need and application of the special provisions for seismic design. Seismic risk or zoning maps, such as recommended References 1.10 and 1.11, are suitable for correlating seismic risk.

Note: Replace existing reference 1.10 (ASCE 7-95) with the 1997 edition of the NEHRP Recommended Provisions.

Note: Reference 1.X, the 1991 NEHRP Recommended Provisions will need to be added if it isn't referenced already.

Reference 1.X
"NEHRP Recommended Provisions for Seismic Regulations for New Buildings," 1991 Edition, Part 1: Provisions (FEMA 222, 199 pp.) and Part 2: Commentary (FEMA 223, 237 pp.), Building Seismic Safety Council, Washington, D.C., 1992.

Note: Insert new Table R1.1.8.3

Table R1.1.8.3

Code, Standard, or Resource Document and Edition	Level of Seismic Risk or Assigned Seismic Performance or Design Categories as Defined in Code Section		
	Low (21.2.1.2)	Moderate/Intermediate (21.2.1.3)	High (21.2.1.4)
BOCA National Building Code 1993, 1996, 1999	SPC[1] A,B	SPC C	SPC D, E
Standard Building Code 1994, 1997, 1999	SPC A, B	SPC C	SPC D,E
Uniform Building Code 1991, 1994, 1997	Seismic Zone 0,1	Seismic Zone 2	Seismic Zone 3,4
International Building Code	SDC[2] A, B	SDC C	SDC D, E, F
ASCE[3] 7-93, 7-95	SPC[1] A,B	SPC C	SPC D, E
NEHRP[4] 1991, 1994	SPC[1] A,B	SPC C	SPC D,E
NEHRP[5] 1997	SDC[2] A,B	SDC C	SDC D, E, F
Note: Underlining in table has been omitted for clarity.			
[1]SPC = *Seismic Performance Category* as defined in code, standard, or resource document			
[2]SDC = *Seismic Design Category as defined in code, standard, or resource document*			
[3]Minimum Design Loads for Buildings and Other Structures, ASCE			
[4]NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings			
[5]NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings and Other Structures			

In the absence of a general building code that addresses earthquake loads and seismic zoning, it is the intent of Committee 318 that the local authorities (engineers, geologists, and building code officials) should decide on proper need and application of the special provisions for seismic design. Expected ground-motion maps and ~~S~~seismic zoning maps, such as recommended in References 1.10 and 1.11, are suitable for correlating seismic risk.

Note: Replace existing Reference 1.10 (i.e., ASCE 7-95) with the 97 NEHRP.

Reason: The additional text and table that are being suggested will give the user some background on the basis of how model codes assign seismic risk and how the terminology used by the model codes can be related to that of ACI 318. Suggested Table R1.1.8.3 will provide a much needed road map of how to implement the intent of the general building code with respect to the detailing required by ACI 318.

Commentary Section R1.3

~~The quality of concrete structures depends largely on workmanship in construction. The best of materials and design practices will not be effective unless the construction is performed well.~~ Proper performance of a concrete structure must begin with a design that complies with this code and is clearly portrayed in construction documents. This must be followed by construction that accurately represents the documents, within the tolerances allowed. Inspection by qualified inspectors is necessary to confirm that the construction is in accordance with the ~~design drawings and project specifications~~ construction documents. ~~Proper performance of the structure depends on construction that accurately represents the design and meets code requirements, within the tolerances allowed. Qualifications of~~ An inspectors ~~can be obtained~~ should be required to demonstrate that he or she is qualified by obtaining certification from a recognized certification program. One such ~~as the~~ certification program for reinforced concrete special inspectors is cosponsored by ACI, International Conference of Building Officials (ICBO), Building Officials and Code Administrators International (BOCA), and Southern Building Code Congress International (SBCCI).

Reason: It is being suggested that the first sentence be deleted because "workmanship" is a subjective term and has been dropped from building codes. What is excellent workmanship to one person may be average to another. In addition, the second sentence should be deleted because construction may be performed "well," whatever that means, and still not comply with the plans and specifications. It is suggested that the text be rearranged so it flows in a more logical sequence. The suggested change to "construction documents" is for consistency with the terminology used in the *International Building Code* and with new verbiage being added in R1.3.1. Adding the word "Special" in the last sentence is for consistency with similar wording in R1.3.1. Other changes are editorial in nature and more clearly convey what I interpret to be the intent of R1.3.1.

Commentary Section R1.3.1

Revise the proposed second paragraph to read: ~~Qualified i~~Inspectors should ~~establish their qualification~~ be required to demonstrate that they are qualified by becoming certified to inspect and record the results of concrete construction, including preplacement, placement and postplacement operations through the Reinforced Concrete Special Inspector program sponsored by ACI, ICBO, BOCA, and SBCCI or equivalent.

Reason: An inspector should not be considered qualified until it is demonstrated through certification by a recognized certification program.

Code Section 1.3.5
For special moment frames resisting seismic loads in regions of high seismic risk, or in structures assigned to high seismic performance or design categories, continuous inspection of the placement of the reinforcement and concrete shall be made by a qualified inspector. The inspector shall be under the supervision of the engineer responsible for the structural design or under the supervision of an engineer with demonstrated capability for supervising inspection of special moment frames resisting seismic loads in regions of high seismic risk, or in structures assigned to high seismic performance or design categories.

Reason: For consistency with new terminology added elsewhere in Chapter 1. Also, the extremely long sentence has been broken into two sentences for better readability.

Code Section 21.1 – Definitions
Moment frame – Space frame in which members and joints resist forces through flexure, shear, and axial force. Moment frames shall be categorized as follows:

> **Ordinary moment frame –** A frame complying with the applicable requirements of Chapters 1 through 18.

Structural walls – Walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake motions. A "shearwall" is a "structural wall." Structural walls shall be categorized as follows:

> **Ordinary reinforced concrete structural wall –** A wall complying with the applicable requirements of Chapters 1 through 18.
> **Ordinary structural plain concrete wall –** A wall complying with the applicable requirements of Chapter 22.

Reason: Not all the provisions of Chapters 1 through 18 apply to ordinary moment frames and to ordinary reinforced concrete structural walls; therefore, it is being suggested that the word "applicable" be inserted in the definitions. Not all the provisions of Chapter 22 apply to ordinary structural plain concrete walls; therefore, it is being suggested that the word "applicable" be inserted in the definition.

Comment: Code Section 21.1 includes a new definition of "moment frame" and a revised definition of "structural walls". Included as a part of "moment frame" is a definition of "ordinary moment frame", which is defined as "a frame complying with the [applicable] requirements of Chapters 1 through 18." Included as a part of “structural walls” is a definition of “ordinary reinforced concrete structural wall”, which is defined as “a wall complying with the [applicable] requirements of Chapters 1 through 18.” Also included as part of “structural walls” is a definition of “ordinary structural plain concrete wall”, which is defined as a “wall complying with the [applicable] requirements of Chapter 22.”

Since these definitions include requirements that are contained outside of Chapter 21, it is recommended that the two definitions (moment frame and structural walls) be repeated in Chapter 2, Section 2.1, where one normally expects to find terms that are defined in the Code. ACI Committee 318 and the Portland Cement Association have jointly sponsored a code change proposal to the *International Building Code* (IBC) to indicate that:

> Ordinary reinforced concrete shear walls [terminology used in the IBC] are walls conforming to the requirements of ACI 318 for ordinary reinforced concrete structural walls.

Therefore, users of the IBC will be directed to look in ACI 318 to determine the requirements for ordinary reinforced concrete structural walls. The first place they are likely to look is the definitions section in Chapter 2.

Commentary Section R21.2.1 – Scope

Revise the third paragraph of proposed R21.2.1 to read:

The provisions of Chapter 21 relate detailing requirements to type of structural framing, earthquake risk level at the site, level of energy dissipation planned in structural design, and the anticipated use and occupancy of the structure. Earthquake risk levels ~~are~~ have traditionally been classified as low, moderate, and high. ~~These risk levles are defined in the Uniform Building Code.[21.8] Regions of low, moderate, and high seismic risk correspond approximately to Zones 0 and 1; Zone 2; and Zones 3 and 4, respectively, of the Uniform Building Code. The 1994 NEHRP Provisions, ASCE 7 95 (formerly ANSI A58.1), the BOCA National Building Code, and the Standard Building Code combine the seismic risk at the site of a structure and the occupancy of a structure into Seismic Performance Categories (SPC). Low, Intermediate, and High Sesimic Performance Categories of 21.2.1.2; 21.2.1.3; and 21.2.1.4 refer to SPC A and B; SPC C; and SPC D and E, respectively. In the 1997 NEHRP Provisions, Seismic Performance Categories have been renamed Seismic Design Categories (SDC). Low, Intermediate, and High Seismic Design categories of 21.2.1.2; 21.2.1.3; and 21.2.1.4 refer to SDC A and B; SDC C; and SDC D, E, and F, respectively.~~ The seismic risk level of a region, or the seismic performance or design category of a structure is regulated by the legally adopted general building code or determined by local authority (see 1.1.8.3, R1.1.8.3, and Table R21.2.1a).

Insert new Table R21.2.1a

Table R21.2.1a

Code, Standard, or Resource Document and Edition	Level of Seismic Risk or Assigned Seismic Performance or Design Categories as Defined in Code Section		
	Low (21.2.1.2)	Moderate/Intermediate (21.2.1.3)	High (21.2.1.4)
BOCA National Building Code 1993, 1996, 1999	SPC[1] A, B	SPC C	SPC D, E
Standard Building Code 1994, 1997, 1999	SPC A, B	SPC C	SPC D, E
Uniform Building Code 1991, 1994, 1997	Seismic Zone 0,1	Seismic Zone 2	Seismic Zone 3,4
International Building Code 2000	SDC[2] A, B	SDC C	SDC D, E, F
ASCE[3] 7-93, 7-95	SPC[1] A, B	SPC C	SPC D, E
NEHRP[4] 1991, 1994	SPC [1] A, B	SPC C	SPC D, E
NEHRP[5] 1997	SDC[2] A, B	SDC C	SDC D, E, F
[1]SPC = *Seismic Performance Category* as defined in building code, standard, or resource document			
[2]SDC = *Seismic Performance Category* as defined in building code, standard, or resource document			
[3]Minimum Design Loads for Buildings and Other Structures, ASCE			
[4]NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings			
[5]NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings and Other Structures			

Reason: The verbiage suggested for deletion has been expressed in tabular form. The typical code-user would prefer to have a table rather than have to read the long text in order to determine detailing requirements for a structure being designed. It is being suggested that the reader be referred to R1.1.8.3 (see suggested change to R1.1.8.3) for an in-depth explanation of how model codes and standards assign seismic risk. Even though proposed Table R21.2.1a is identical to proposed Table R1.1.8.3, it is strongly recommended that it be repeated here so the user does not have to turn back to the front of the code.

Note: Revise the fourth, fifth, sixth, and seventh paragraphs of R21.2.1, and add a new eighth paragraph and table as follows. The existing eighth and ninth paragraphs remain unchanged and follow the new eighth paragraph.

The design and detailing requirements must be compatible with the level of energy dissipation (or toughness) assumed in the computation of the design seismic ~~loads~~ forces. The terms ordinary, intermediate, and special are specifically used to facilitate this compatibility. The degree of required toughness, and therefore, the level of required detailing, increases for structures progressing from ordinary through intermediate to special categories. It is required that structures in regions of higher seismic ~~zones~~ risk, or assigned to higher seismic performance or design categories possess a higher degree of toughness~~,~~. ~~But~~ It is possible, however, to design and construct structures ~~for higher toughness~~ in ~~the~~ regions of lower seismic ~~zones~~ risk, or assigned to lower performance or design categories for higher toughness and take advantage of the lower design force levels.

The provisions of Chapters 1 through 18 and 22 are intended to provide adequate toughness for structures in regions of low seismic risk, or assigned to ~~ordinary~~ low performance or design categories. Therefore, it is not required to apply the provisions of Chapter 21 ~~for~~ to lateral-force resisting systems consisting of ordinary moment frames or ordinary reinforced concrete structural walls~~structures~~.

Chapter 21 requires some details for reinforced concrete structures in regions of moderate seismic risk, or assigned to intermediate seismic performance or design categories. These requirements are contained in 21.2.~~2~~1.3 and 21.~~8~~10.

Structures ~~I~~in regions of high seismic risk, or assigned to high seismic performance or design categories~~, structures~~ may be subjected to strong ground shaking. Structures designed using ~~loads~~ seismic forces based upon response modification factors for special moment frames or special reinforced concrete structural walls are likely to experience multiple cycles of lateral displacements well beyond the point where reinforcement yields should the design earthquake ground shaking occur. The provisions of Sections 21.2 through 21.~~7~~9 have been developed to provide the structure with adequate toughness for this special response.

The requirements of Chapter 21, as they apply to various components of structures in regions of intermediate or high seismic risk, or assigned to intermediate or high seismic performance or design categories, are summarized in Table R21.2.1b.

Note: Insert new Table R21.2.1b

Table R21.2.1b – Sections of Chapter 21 to be satisfied*

Component Resisting Earthquake Effect, Unless Otherwise Noted	Level of Seismic Risk or Assigned Seismic Performance or Design Categories as Defined in Code Section	
	Intermediate (21.2.1.3)	High (21.2.1.4)
Frame members	10	2,3,4,5
Structural walls and coupling beams	None	2,6

Structural diaphragms and trusses	None	2,7
Foundations	None	2,8
Frame members not proportioned to resist forces induced by earthquake motions	None	9
* In addition to the requirements of Chapters 1 through 18 for structures at immediate seismic risk (21.2.1.3), and Chapters 1 through 17 for structures at high seismic risk (21.2.1.4).		

Reason: Changes suggested above are basically editorial in nature and are intended to ensure consistency in terminology, use terms that are defined for the first time in this edition, and to improve readability. It is also being suggested that existing Table R21.2.1 be retained but in a revised format to accommodate the changes to the 1995 edition of the Standard. The table is a user-friendly way of providing guidance to infrequent users of the chapter.

New Code Section 21.8.2.3
Columns or boundary elements of special reinforced concrete structural walls that have an edge within one-half the footing depth from an edge of the footing shall have transverse reinforcement in accordance with 21.4.4 provided below the top of the footing. This reinforcement shall extend into the footing a distance equal to the smaller of the ~~full~~ depth of the footing, mat, or pile cap, minus the cover required by Chapter 7, or the development length in tension of the longitudinal reinforcement.

Comment: The first sentence requires that transverse reinforcement must enclose longitudinal reinforcement where columns or structural wall boundary elements are located near the edge of a footing. The second sentence requires that the transverse reinforcement extend into the footing of the smaller of the footing thickness or the tensile development length of the longitudinal reinforcement. When the footing thickness controls (such as, the development length of longitudinal reinforcement is achieved with a hook), the requirement cannot be met because of cover requirements of Chapter 7. In other words, the hoop or crosstie at the bottom of the footing cannot be located at the footing thickness below the top of the footing because adequate cover must be provided (usually three inches) between the bottom-most transverse reinforcement and the bottom of the footing. The suggested revision will correct the problem.

New Code Section 21.8.2.4
~~Footings beneath all~~ Where earthquake effects create uplift forces in boundary elements of special reinforced concrete structural walls ~~and benath all~~ or columns ~~resisting tension forces induced by earthquake effects shall have.~~, flexural reinforcement shall be provided in the top of the footing, mat, or pile cap to resist the design load combinations, and shall not be less than required by 10.5.

Reason: The wording of the provisions suggests that flexural reinforcement is required in the top of all footings supporting boundary elements of special reinforced concrete structural walls, regardless of whether the boundary element is resisting uplift loads or not. On the other hand, if the footing is supporting a column, flexural reinforcement is only required in the top of the footing where the column is resisting tensile forces (uplift loads). This does not appear to be the intent since it makes no sense to require the top reinforcement in a footing supporting a boundary element if the boundary element is not subject to uplift. The suggested revision will clarify the provisions.

Comment: Because this new provision is located in Section 21.8, it only applies to structures in regions of high seismic risk or assigned to high seismic performance or design categories. There is nothing contained in these provisions, however, that is also not applicable to structures in lower seismic risk areas or assigned to lower seismic performance or design categories. In fact, the same situation envisioned by this provision may occur when the structure is subjected to code-required design wind forces. If the engineer must be reminded that a column experiencing uplift due to seismic load may subject the top of a footing to flexural tensile stresses, it is questionable if he or she is qualified to design the structure. It is recommended that the section be deleted since it has no requirement that is unique to structures at high seismic risk. Section 10.5 will still apply with this deleted.

Commentary References – Revise "New Reference 21.2" to read:

"NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures," 1997 Edition, Part 1: Provisions (FEMA 302, 353 pp.) and Part 2: Commentary (FEMA 303, 335 ~~pages~~pp.), Building Seismic Safety Council., Washington, DC., 1998.

Reason: To correct the title and add the edition that is on the cover of the document.

By M. Nasser Darwish
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Regarding the proposed revision to ACI 318-95 as stated repeatedly in the code:

- "The code is intended to cover all buildings of the usual types, both large and small" (introduction to 318/318R-2, ACI Standard/Committee Report);
- "Code provides…for design and construction of structural concrete elements of any structure…" (Code Section 1.1.1); and
- "…for any structural concrete design or construction…", (Commentary Section R1.1).

Still in the category of usual structures, however, the code excludes some structural elements depending upon their capability to carry certain loads (in the case of slabs on grade; New Commentary Section R21.8.3) or the type of loads they are subject to (piers, piles, caissons not in seismic regions; New Code Section 21.8.4.1). Although one can understand that piles and piers may be elements of special nature.

For example, Code Section 1.1.6 states “this Code does not govern design…of soil-supported slabs unless the slab transmits vertical loads or lateral forces from other portions of the structure to the soil,” and no code commentary section is provided. What is the Code definition of slabs on grade or ground supported slabs or slabs that transmit vertical loads. A clear definition should be included for such structural elements, which although encountered commonly, unfortunately may not be well designed.

The writer is familiar with some ACI Committees’ pertinent definitions, such as, the definitions for slabs on grade by ACI Committee 360 (ACI 360R-92), which may not include concrete pavements, concrete slabs for parking lots, and industrial slabs. This is mainly because the former and other slabs although are slabs on grade/soil-supported slabs, however, there are other ACI committees to deal with them (such as, ACI 325), and therefore, there is no unified definition available among ACI Committees. Acknowledging the inherent differences that might exist between such systems, however, from a reinforced concrete structural design point of view, that of ACI Committee 318, and that of a code for structural concrete, all these slabs are structural elements and should be structurally designed following the same procedure, however, under their pertinent loads and including any inherent special features.

Moreover, the code provides no guidelines for the design of slabs under vertical loads and there is no code commentary. In contrast to that, the treatment of such slabs under lateral forces (where they are supposed to act as structural diaphragms in seismic areas) is mentioned in more detail (new Code Section 21.8.3.4 and New Commentary Section R21.8.3.4). One must note that soil-supported slabs could be treated as reinforced concrete structural elements (to transfer vertical as well as other loads). Structurally active slabs on grade/actively reinforced concrete slabs on grade (ground supported slabs)[1,2,3] and sometimes referred to as Type F slabs[4], are not uncommon. They are designed and reinforced as structural reinforced concrete slab elements making full use of the principles of reinforced concrete strength design, with the concrete slab acknowledged to be cracked and the reinforcement designed and positioned to carry the tensile forces and to increase the load-carrying capacity, as done in most reinforced concrete elements, instead of relying on the slab concrete tensile strength and treating such concrete slabs as plain and unreinforced. Practical design curves for the direct estimation of the load-carrying capacity of such slabs could be found in some references (such as, 1 and 2). Hence, a new Commentary Section R1.1.6 may be needed with some design guidelines and references for such structurally active slabs.

Code Section 1.1.5

This section does not govern design for (concrete piles, drilled piers, and caissons) the usual cases. The same elements, however, are included if they are in regions of high seismic risk (New Code Section 21.8.4). Such differentiation in treatment seems inconsistent and should be alleviated if possible.

Commentary Section R1.3
"Qualifications of inspectors can…and Southern Building Code Congress International (SBCCI) or equivalent."

Reason: Do not limit qualified inspector to those qualification. To match the same qualifications as proposed in Commentary Section R1.3.1.

Code Section 1.3.1
"…or by a qualified inspector or competent representative responsible to that engineer."

Reason: Having a highly qualified design professional on site permanently in certain cases is expensive and impractical. His or her qualified representatives should represent/act on his or her behalf still under his or her responsibility.

New Code Section 5.6.1
"…Qualified laboratory technicians or equivalent shall perform…"

New Commentary Section R5.6.1
"…or the requirement of ASTM C 1077 or equivalent program."

New Section Code 21.8.2.1 and Commentary R21.8.2.2
An illustrative drawing would be helpful.

New Code Section 21.8.2.4
"Footings …forces induced by earthquake effects shall have in addition to other (bottom) reinforcement, flexural reinforcement in the top…"

New Commentary Section R21.8.2.4
"…top reinforcement is required to resist those loads in addition to the required reinforcement under other loads."

Reason: To emphasize the preexistence of flexural and other reinforcement under usual loads, other than the needed top reinforcement.

Clarifications may be required regarding some cases, such as combined footings, strap footings, where the main reinforcement in usual cases is predominantly top steel.

References

1) Losberg, A., December 1978, "Pavements and Slabs on Grade with Structurally Active Reinforcement," *ACI Journal,* pp. 647-657.
2) Darwish, M. N., Nilson, A., August 1988, "Structural Analysis and Design of Reinforced Concrete Slabs on Grade," *Report 88-7,* Department of Structural Engineering, Cornell University, New York, p. 371.
3) Darwish, M.N., 1991, "Nonlinear Response of Ground Supported Concrete Slabs," *Mechanics Computing in the 1990s and Beyond,* American Society of Civil Engineers, pp. 912-917.

4) ACI 360R-92, Reapproved 1997, “Design of Slabs on Grade.”

By Neil M. Hawkins
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Section 18.13 of ACI 318-95 on “Post-Tensioned Anchorage Zones” provided little guidance to the designer on the specifics of when and how to provide reinforcement in anchorage zones. The proposed revisions correct that situation, and in particular, the references in the Commentary provide more detailed design information. Figures in the Commentary, however, do not match well with the text of both Code and Commentary. Several changes are desirable:

- Fig. R18.13.1 has three subfigures (a), (b), and (c). Only subfigure (b) is given a title and unfortunately subfigure (c) seems to be titled “longitudinal edge tension force.” Subfigures (a) and (c) should be given titles and what corresponds to bursting, spalling, and forces on subfigure (c) should be more clearly designated.
- It is confusing for the reader that neither Fig. R18.13.1 or Fig. R18.13.3 show information that explains the symbols of Eq. (18A) and (18B) the reader needs to refer to references 18X and 18Y. That situation is undesirable. A new Fig. R18.13.5 should be added to the Commentary. That figure should show all the quantities of Eq. (18A) and (18B) for an eccentric loading case.
- To accommodate the inclusion of Fig. R18.13.5, Commentary Section R18.13.5 would need to be revised so that after the first sentence of the second paragraph the following sentence is added: “Meanings for the terms of Eq. (18A) and (18B) should be shown in Fig. R18.13.5 for a loading with small eccentricity.”
- Fig. R18.13.3 should be given a title: “Section Change Effect” is appropriate.

By Perry Adebar
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and
Jim Mutrie
Jones Kwong Kishi
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The proposed new Section 21.6.7, which deals with the design of coupling beams connecting structural walls used to resist earthquake forces, is a welcomed addition to ACI 318. There are, however, two problems with the proposed requirements for coupling beams reinforced with diagonally-placed bars.

Design rules for diagonally reinforced coupling beams have existed in New Zealand

(Reference 1) and Canada (Reference 2) for decades. In the seismically active west-coast region of Canada, coupled structural walls with diagonally reinforced coupling beams is the most common ductile earthquake resisting system used for high-rise buildings. From the experience gained, it is known that efficient and cost-effective coupled-wall structures usually involve walls that are relatively thick – 30 in. walls are common. When the proposed Section 21.6.7.4 is applied to such walls, two problems arise.

In sentence (1) the requirement that the diagonal reinforcement be assembled in a core having "sides" no smaller than $b_w/2$ is reasonable for narrow coupling beams such as is shown in the proposed new Fig. 21.6.7, but is not appropriate for wide coupling beams. As correctly explained in the proposed commentary R21.6.7, the objective is to place the diagonal bars with as large an inclination as possible. Requiring a minimum core dimension of 30 in./2 = 15 in. in both directions of 30 in. wide coupling beams will unnecessarily reduce the inclination of the diagonal bars. Fig. 5.55 of Reference 3 illustrates an appropriate arrangement for diagonally placed bars in a coupling beam. The minimum spacing of the diagonal reinforcement that defines the vertical dimension of the core should be as is given in Section 7.6 of ACI 318. The proposed requirement for the minimum dimension of the core in the horizontal direction, however, is appropriate. Thus the phrase in sentence 21.6.7.4 (1) should be changed from "a core having sides ... no smaller than $b_w/2$" to "a core having a horizontal dimension ... no smaller than $b_w/2$."

The second problem that occurs when designing wide coupling beams with large groups of bars results from the proposed sentence 21.6.7.4 (3) that requires groups of diagonally placed bars to be enclosed in transverse reinforcement satisfying the confinement provisions 21.4.4.1 through 21.4.4.3. As given in Eq. (21-5) of the proposed 21.6.7.4 (2), diagonally placed reinforcement is provided to resist the entire shear force (and associated bending moments). The diagonal reinforcement resists all of the diagonal tension in one direction of the earthquake and resists all of the diagonal compression in the opposite direction of the earthquake. The concrete's role is to stabilize the diagonal bars. Thus, the reason for providing transverse reinforcement around diagonally placed bars in a coupling beam is to prevent buckling of the bars – not to confine the concrete within the group of bars. When the proposed confinement requirement is applied to a wide coupling beam with diagonal reinforcement spread across the beam, the required amount of transverse reinforcement cannot possibly be provided. Sentence 21.6.7.4 (3) should be replaced with a requirement for an appropriate amount and arrangement of antibuckling ties spaced at no more than six times the diameter of the diagonal reinforcement as given in References 1, 2, or 3.

References

1. *New Zealand Standard Code of Practice for the Design of Concrete Structures,* 1995, NZS 3101: Part 1, Standard Association of New Zealand, Wellington, 256 pp.

2. CSA Committee A23.3, 1994, "Design of concrete structures: Structures (Design) - A National Standard of Canada," Canadian Standards

Association, Rexdale, 199 pp.

3. Paulay, T., and Priestley, M.J.N., 1992, "Seismic design of reinforced concrete and masonry buildings," John Wiley.

Committee Closure

ACI Committee 318 is grateful to all those who took the time to critically evaluate the proposed 1999 changes to the Building Code, published in the November 1998 issue of *Concrete International*. During the ACI spring convention held in Chicago, IL, in March 1999, Committee 318 carefully reviewed all discussions. In accordance with the Institute's standardization and technical committee procedures, further modifications were made to incorporate suggestions and respond to concerns. These changes, as well as a response to all of the major points raised in the discussions, are contained in this closure.

After publication of the proposed changes in November 1998, Committee 318 members found areas requiring editorial clarification. Many of the amendments printed after this closure reflect this review.

The ACI Building Code is a continually changing document. Committee 318 invites further criticism and suggestions for improvement from Building Code users as experience is gained after adoption of these provisions.

The ACI 318-99 Section 1.3.1 requirements apply where the local jurisdiction's adopted code does not address inspection requirements. Thus, local requirements such as those of the City of Los Angeles supercede the inspection requirements in ACI 318-99 in the example given in Mr. Strand's comment. Committee 318 felt that it is desirable for the engineer of record to be involved in the inspection and worded the commentary accordingly. The Committee did not mandate in R5.6.1 that the "design engineer" referred to in the comment perform the inspections. One reason it was not mandated is the concern that the engineer may not be compensated for the service as pointed out by Mr. Strand.

Mr. Gene Daniel commented on 5.6 regarding evaluation and acceptance of concrete. Committee 318 feels that there are a number of ways technicians can establish qualifications. It is felt that the ACI Certification Program should not constitute a minimum program content and technique for establishing qualification. The Commentary gives guidance but does not mandate how qualification may be established.

Dr. Darwish offered comments on several of the proposed changes. Committee 318 will take as new business the clarity of the definition of structural elements excluded under ACI 318-99. The scope of the Code in 1.1.5 intentionally excludes some structural elements or portions thereof, unless special circumstances occur, as the design criteria are dealt with by other documents. There are a number of ways inspectors can establish qualifications. Section R1.3 gives guidance as to how qualification may be established. It does not limit how qualification may be established. The proposed wording does not add clarity and is redundant with "such as". The proposed language for 1.3.1 does not add clarity. The suggested revision to 5.6.1 creates ambiguity. Committee 318 feels that there are a number of ways technicians can establish qualifications. Section R5.6.1 gives guidance but does not mandate how qualifications may be established.

Professor Nawy and Dr. Munshi discussed the new 10.6.4. Some of their comments may be answered by the reason for change statement published with the proposed changes. Committee 318 believes that it is misleading to purport to calculate crack widths, given the inherent variability in cracking. The three important parameters in flexural cracking are steel stress, cover, and bar spacing. Steel stress is the most important of the three, and most of the correlation between test results and crack width prediction formulas is correlated with steel stress. When the steel stress is held constant, there is very little correlation with the $\sqrt[3]{d_c A}$ parameter. This may be seen in Fig. 4b of Professor Nawy's Reference 3. Although the Gergely-Lutz regression line is drawn through the scattered data, any other line would fit almost as well (actually, as poorly).

For this reason, Committee 318 preferred to specify bar spacing directly as a function of cover, without implying a calculated crack width. Predictions by Beeby [10.z], Frosch [10.zz], and Gergely-Lutz (ACI 318-95) for a crack width of 0.016 in. were plotted on a diagram of bar spacing versus clear cover, as shown in Fig. 10.6. The three curves were approximated by a simple straight line relating bar spacing to clear cover. Similar figures were done for other steel stresses.

Professor Nawy and Dr. Munshi suggested the use of a constant bar spacing for covers greater than 2 or 3 in. The Committee believes that bar spacing should be reduced for larger covers for appearance reasons.

The distinction between interior and exterior exposure was based on a difference in predicted crack width of 0.003 in. Even if such a difference could be accurately predicted (which it cannot be), research cited in References 10[xx] and 10[yy] indicates that this would not have a material effect on corrosion. Therefore, the distinction between interior and exterior exposure was eliminated.

References:

10.z. Beeby, A.W. "The Prediction of Crack Widths in Hardened Concrete," *The Structural Engineer,* V. 57A, No. 1, United Kingdom, January 1979, pp. 9-17.

10.zz. Frosch, R.J. "Another Look at Cracking and Crack Control in Reinforced Concrete," accepted for publication in the May/June 1999 *ACI Structural Journal*.

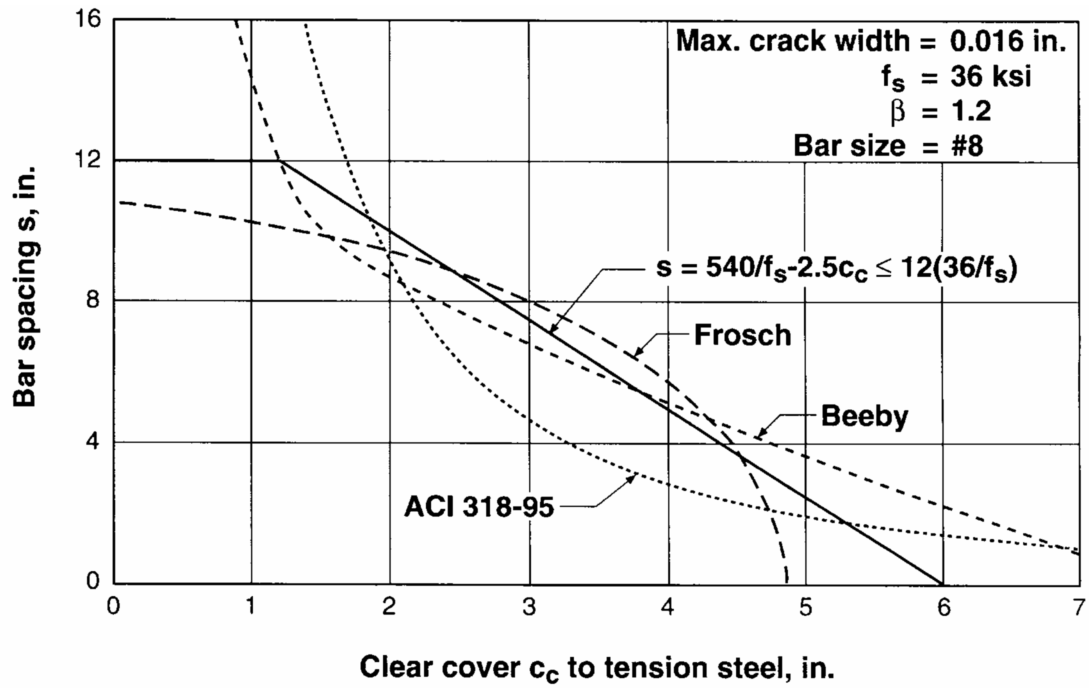


Figure 10.6 Bar Spacing Versus Clear Cover

Committee 318 thanks Committee 551 for their proposal for a rewrite of 14.8. A revision of this scope will be taken up as new business for the 2002 code. Committee 551 has pointed out two specific items that need to be taken care of now.

1. Section 14.8.3 requires only one iteration for determining the PΔ effect, and does not include a ϕ factor. This underestimates M_u, and is on the unsafe side.

2. The new deflection limit of $\ell_c/100$ at factored load is more severe than the former limit of $\ell_c/150$ at service load, because of the nonlinear nature of PΔ effects. Committee 551 believes that many existing structures designed under previous codes would not qualify under the new deflection limit. Committee 551 believes that the deflection limit should be computed at service load.

Revisions to Chapter 14 are proposed, in response to these considerations. Other items suggested by Committee 551 will be taken up as new business for 318-2002.

Dr. Darwin reports some editorial matters. The new notation for Chapter 14 is in addition to the current notation, but the definitions of A_g, f_c', ℓ_c and ϕ should not be duplicated. Dr. Darwin is correct about an error in the printed version of changes to R11.6.3.7 (refer to *Concrete International*, November 1998, pg. S-20, fourth line of new wording for Commentary section R11.6.3.7). The axial force should be $N_i = V_i \cot \phi$. It is incorrectly shown as $N_i = V_i/\cot \phi$. Regarding changing the word “practicable” to “practical,” the Committee discussed this comment and agreed not to change the word, as it reflects the Committee’s intent.

The Committee thanks Professor Hawkins for his excellent comments. The Committee has redrawn Fig. R18.13.1, retitled Fig. R18.13.3 and added a new Fig. R18.13.5 for a tendon load with small eccentricity, all basically as suggested for improved clarity. Amendments are made accordingly.

Mr. Strand raises several important points related to seismic design, and has suggested several editorial improvements that will be adopted. The Committee will consider as new business the addition of special reinforced concrete structural walls and coupling beams in 1.3.5. Regarding the definition of ρ_n in 21.0, the Committee believes the definition is clearer as written. In 21.1, the terminology “ordinary wall” is used to be consistent with the draft IBC provisions. In R21.1, the words “in design” are necessary to the intent of the commentary. Similarly, in 21.2.7.2, the Committee believes the original wording is correct.

Mr. Strand is correct to note that transverse reinforcement is required to restrain buckling of mechanical splices; such transverse reinforcement is already required in plastic hinge regions of beams and columns where buckling tendencies are greatest. The UBC requirements related to 21.6.6.2 were considered for 318-99 but were not thought to be appropriate because of the different basis for those provisions. Section R21.6.6.2 has been simplified to clarify the intent. For 21.6.6.4(d), the Committee will consider as new business whether to reduce the transverse reinforcement extension into the footing. Regarding 21.7.5.3, the reference to 21.4.4.2(a) is carried over from previous editions of the code; the recommendation to consider eliminating this reference will be taken up as new business. Regarding the hooks of column bars in footings (21.8.2.2), the Committee agrees that confinement provided by large footings is beneficial but has concerns that in many cases the confinement will be inadequate to resist the forces transmitted from the column. The current wording requires the hooks to be bent inward only in cases where small footing depth requires the use of hooks for development. When the footing depth is greater the hooks can be bent outward. The recommendation regarding 21.8.3.3 will be taken up as new business.

Committee 318 appreciates the well considered and detailed comments by Mr. Hulshizer. Subcommittees of ACI Committee 318 have been in contact with ACI Committee 439 asking them to provide technical guidance on many of the topics raised by Mr. Hulshizer. As he suggests, the nonlinear response of the structural components of a reinforced concrete building is not well understood. The specific demands on the reinforcement cannot be expressed with precision. That is a very good reason for keeping the specified requirements to maintain simplicity in the code as well as safety in the structure. If field or laboratory data are obtained to suggest that welding or mechanical splicing of reinforcing bars executed under the current requirements will lead to a lack of safety, the Code requirements will be reconsidered.

Mr. Messersmith is thanked for many editorial comments that will improve the code and Commentary. Some of these have been included in the proposed amendments. Others will require detailed consideration by the Committee as new business. The wording suggested for 1.1.8.3 does not clarify or improve the code. Regarding R1.3.1, Committee 318 feels that there are a number of ways inspectors can establish qualifications. The commentary gives guidance as to how qualifications may be established. A number of practicing engineers on the Committee felt that engineering licensing also established qualification.

Dr. Darwish points out in Reference to 21.8.2.4 that reinforcement other than top reinforcement may be required in footings, and that the code provision might be misleading by emphasizing only the top reinforcement. The Commentary has been modified to clarify the intent.

Mr. Arndt brings up several points in relation to Chapter 21. Regarding the notation for c, the definitions here and in Chapter 9 are referring to the same quantity, so no changes will be made in the code. New commentary has been added to R.21.6.6.2 to clarify the procedure for calculating c. The Committee considered the question about mechanical and welded splice location in a special boundary element of a structural wall, and decided that the current wording of 21.2.6.2 and 21.2.7.1 was sufficiently clear in prescribing that those splices should not be located closer than twice the depth from the critical section. The Committee will consider more specific language for walls as future business. Eq. (21-10), defining nominal shear strength of diaphragms, was written without the term α to simplify the code. The Committee will consider as new business the proposal to refer to 21.4.3 in the provisions of 21.9.3.3.

Dr. Adebar and Mr. Mutrie raise important issues for relatively thick diagonally reinforced coupling beams. The Committee agrees that for thick beams the vertical dimension of the diagonal reinforcement cage need not be as large as $b_w/2$, and has developed an amendment to allow some reduction in the vertical dimension. Further reduction of the vertical dimension seems inadvisable because of the role of concrete within the diagonal reinforcement cage to restrain the diagonal bars and to resist part of the diagonal compressive force. Section 21.6.7 refers to 21.4.4.1 through 21.4.4.2 because of the view that confinement of the concrete within the diagonal reinforcement cage improves behavior of the beam. The required dimensions and detailing of diagonally reinforced coupling beams will be considered further as new business.

AMENDMENTS TO ACI 318-99

Commentary Section R1.1

Revise the new paragraph added at the end of R1.1 to read:

Appendix C of the Code allows the use of the <u>factored</u> load ~~factor~~ combinations in section ~~2.4~~ <u>2.3</u> of ASCE 7, ‘Minimum Design Loads for Buildings and Other Structures’ if structural framing includes primary members of materials other than concrete.

Code Section 2.1
Revise to read:
Anchorage Zone -- In post-tensioned members, the portion of the member through which the concentrated prestressing force is transferred to the concrete and distributed more ~~widely~~ uniformly across the section. Its extent is equal to the largest dimension of the cross section. For intermediate anchorage devices, the anchorage zone includes the disturbed regions ahead of and behind the anchorage devices.

Code Section 3.8.1
Update ASTM A 706-96b to A 706-98

Commentary Section R9.3.4
Replace first sentence of last paragraph with the following:
Short structural walls were the primary vertical elements of the lateral-force-resisting system in many of the parking structures that sustained damage during the 1994 Northridge earthquake.

Code Section 10.0 - Notation
Revise to read:
s = center-to-center spacing of flexural tension reinforcement nearest to the extreme tension face, in. Where there is only one bar or wire nearest to the extreme tension face, s is the width of the extreme tension face.

β_d = (a) for nonsway frames, β_d is the ratio of the maximum factored axial sustained load to the maximum factored axial load associated with the same load combination

New Code Section 10.6.4
Reformat equation to:

$$s = \frac{540}{f_s} - 2.5c_C \qquad (10\text{-}5)$$

Commentary Section R10.6.4
Add the following reference footnotes after the second sentence of the first paragraph of R10.6.4:

… directly.[10.UU, 10.VV, 10WW]

Add the following references:

10.UU Beeby, A.W., January 1979, "The Prediction of Crack Widths in Hardened Concrete," *The Structural Engineer*, V. 57A, No. 1, United Kingdom, pp. 9-17.

10.VV Frosch, R.J., "Another Look at Cracking and Crack Control in Reinforced Concrete," accepted for publication in the May – June 1999 *ACI Structural Journal*.

10WW ACI Committee 318, May 1999, "Closure to Public Comments on ACI 318-99," *Concrete International,* pp. 318-1 to 318-49.

Code Section 11.0 – Notation
Revise to read:
f_{yh} = specified yield strength of circular tie, hoop, or spiral reinforcement.

Commentary Section R11.1.3
Change R11.1.3 (third paragraph) as follows:
Support conditions where this provision should not be applied include: (1) Members framing into a supporting member in tension, such as shown in Fig. R11.1.3.1(c). For this case, the critical section for shear should be taken at the face of the support~~,~~. ~~s~~Shear within the connection should also be investigated~~,~~ and special corner reinforcement should be provided. (2) Members for which loads are not applied at or near the top of the member. This is the condition referred to in Fig. 11.1.3.1(d). For such cases the critical section is taken at the face of the support. Loads acting near the support must be transferred across the inclined crack extending upward from the support face. The shear force acting on the critical section must include all loads applied below the potential inclined crack. (3) Members loaded such that the shear at sections between the support and a distance d from the support differs radically from the shear at distance d. This commonly occurs in brackets and in beams where a concentrated load is located close to the support, as shown in Fig. R11.1.3.1(e) or in footings supported on piles. In this case the shear at the face of the support should be used.

Code Section 11.5.6.3
Revise to read:
When circular ties, hoops, or spirals are used as shear reinforcement, V_s shall be computed using Eq. (11-15) where d shall be taken as the effective depth defined in 11.3.3. A_v shall be taken as two times the area of the bar in a circular tie, hoop or spiral at a spacing s, and f_{yh} is the specified yield strength of circular tie, hoop or spiral reinforcement.

Commentary Section R11.1.2
Revise to read in part:
R11.1.2 – A limited number of tests [11.7,11.8] of reinforced concrete beams made with high-strength concrete (f'_c greater than about 8000 psi) suggest that the inclined cracking load increases less rapidly than Eq. (11-13) or (11-5) would suggest. This was offset by an increased effectiveness of the stirrups compared to strength predicted by Eq. (11-15), (11-16), and (11-17). Other ~~unpublished~~ tests [11.nn] of high-strength concrete girders with minimum web reinforcement indicated that this amount of web reinforcement was inadequate to prevent brittle shear failures when inclined cracking occurs. There are no test data on the two-way shear strength of high-strength concrete slabs or torsional strength. Until more practical experience is obtained …

New Reference 11.nn
Add reference:

Roller, J. J. and Russell, H. G., March-April, 1990, "Shear Strength of High-Strength Concrete Beams with Web Reinforcement," *ACI Structural Journal*, V. 87, No. 2, pp. 191-198.

Commentary Section R14.3

Revise R14.3, second sentence to:

These apply to walls designed according to 14.4~~,~~ ~~or~~ 14.5, or 14.8.

Code Section 14.0 – Notation

Add

M = maximum unfactored moment due to service loads, including PΔ effects

M_{sa} = maximum unfactored "applied moment" due to service loads, not including PΔ effects

P_s = unfactored axial load at the design (midheight) section including effects of self weight

Δ_s = maximum deflection at or near midheight due to service loads

Delete

Δ_n = (not used)

Revise Eq. (14-5) as follows.

$$\Delta_u = \frac{5 M_u \ell_c^2}{\phi 48 E_c I_{cr}} \qquad (14\text{-}5)$$

Add to New Code Section 14.8.3, immediately after Eq. (14-5):

M_u shall be obtained by iteration of deflections, or by direct calculation using Eq. (14-6).

$$M_u = \frac{M_{ua}}{1 - \dfrac{5 P_u \ell_c^{\,2}}{\phi 48 E_c I_{cr}}} \qquad (14\text{-}6)$$

Renumber Eq. (14-6) and (14-7) as (14-7) and (14-8), respectively.

Replace New Code Section 14.8.4 with the following.

Code Section 14.8.4 – Control of deflections

The maximum deflection Δ_s due to service loads, including PΔ effects, shall not exceed $\ell_c/150$. The midheight deflection Δ_s shall be determined by:

$$\Delta_s = \frac{5 M \ell_c^2}{48 E_c I_e} \qquad (14\text{-}9)$$

$$M = \frac{M_{sa}}{1 - \dfrac{5 P_s \ell_c^2}{48 E_c I_e}} \qquad (14\text{-}10)$$

I_e shall be evaluated using the procedure of 9.5.2.3, substituting M for M_a

I_{cr} shall be evaluated using Eq. (14-7)

New Commentary Section R14.8 — Alternate design of slender walls

Add new commentary:

Section 14.8 is based on the corresponding requirements in the Uniform Building Code and is based on experimental research.[14-XX] The procedure is presented as an alternate to the requirements of 10.10 for the out-of-plane design of precast wall panels, where the panels are restrained against overturning at the top.

The procedure, as prescribed in the UBC, has been converted from working stress to factored load design.

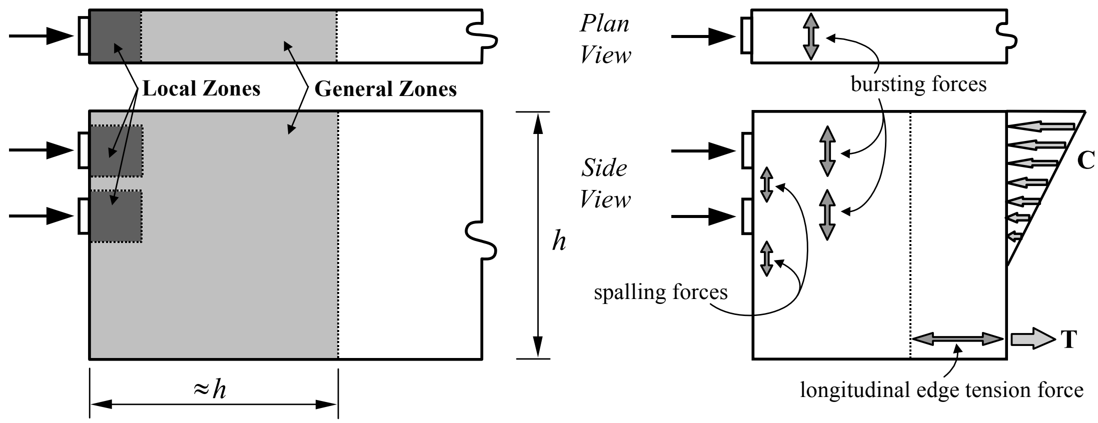
Panels that have windows or other large openings are not considered to have constant cross section over the height of the panel. Such walls must be designed taking into account the effects of openings.

Many aspects of the design of tilt-up walls and buildings are discussed in Reference 14-ZZ and 14YY. ~~Research[14-YY] has highlighted problems with the seismic performance of tilt-up buildings that should be addressed by the designer.~~

Commentary Section R18.13.5 Design Methods

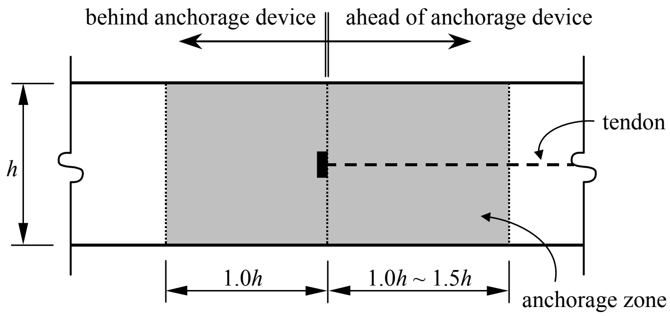
Revise second paragraph to read:

For many cases, simplified equations based on References 18[X] and 18[Y] can be used. Values for the magnitude of the bursting force, T_{burst}, and for its centroidal distance from the major bearing surface of the anchorage, d_{burst}, may be estimated from Eq. (18A) and 18B, respectively. Meanings for the terms of Eq. (18A) and (18B) are shown in Fig. R18.13.5 for a tendon load with small eccentricity. In the applications of Eq. (18A) and (18B) the specified stressing sequence should be considered if more than one tendon is present.



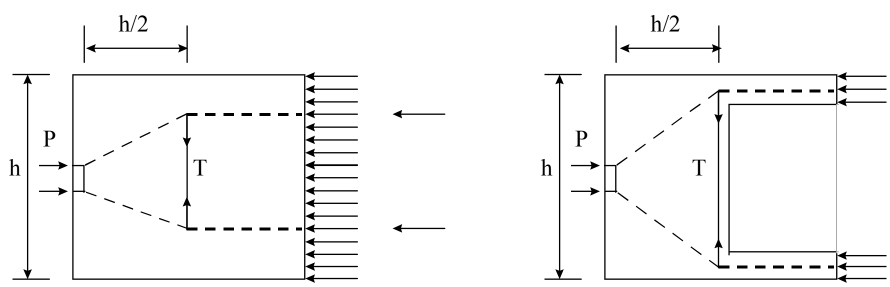
(a) Local and general zones concept

(c) Tensile stress zones



(b) General zone for intermediate anchorage device

Figure R18.13.1 Anchorage zones



(a) Rectangular section

T ~ 0.25 P

(b) Flanged section with end diaphragm

T ~ 0.50 P

Figure R18.13.3 Effect of cross-section change

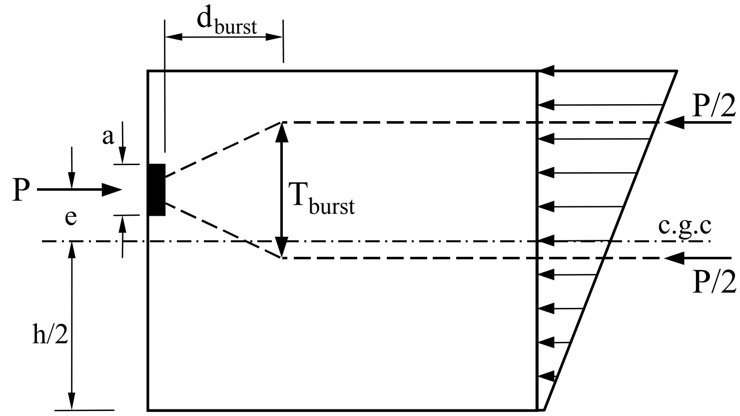


Figure R18.13.5 Strut and tie model example

New Code Section 18.13.5.6
Revise to read:
Where curved tendons are used in the general zone, except for monostrand tendons in slabs or where analysis shows reinforcement is not required, bonded reinforcement shall be provided to resist radial and splitting forces.

Code Section 21.6.6.2
In (a), change "21.x" to "21.8".

Modify (b) as follows:
(b) Where special boundary elements are required by 21.6.6.2(a), the special boundary element reinforcement shall extend vertically from the critical section ~~along the wall~~ a distance not less than the larger of l_w or $M_u/4V_u$ ~~from the critical section~~.

Commentary Section R21.6.6.2
Modify the first paragraph as follows:
This section is based on the assumption that wall inelastic response is dominated by flexural action at a critical yielding section. ~~To be applicable, the wall should be effectively continuous over its height without significant changes in cross section, and the nominal moment strength at all sections except the critical section should exceed the moments that develop when the wall reaches the probable moment strength M_{pr} at the critical section under the design lateral forces amplified to achieve M_{pr}.~~ The wall should be proportioned so that the critical section occurs where intended.

Add a third paragraph following the second paragraph, as follows:
The neutral axis depth c in Eq. (21-8) is the depth calculated according to 10.2, except the nonlinear strain requirements of 10.2.2 need not apply, corresponding to development of nominal flexural strength of the wall when displaced in the same direction as δ_u. The axial load is the factored axial load that is consistent with the design load combination that produces the displacement δ_u.

Code Section 21.6.7.4

Modify (1) as follows:

(1) Each group of diagonally placed bars shall consist of a minimum of four bars assembled in a core having sides measured to the outside of the transverse reinforcement no smaller than $b_w/2$ perpendicular to the plane of the beam and $b_w/5$ in the plane of the beam and perpendicular to the diagonal bars.

Code Section 21.7.8.3(a)

Modify as follows:

(a) a minimum spacing of three longitudinal bar diameters, but not less than 1-1/2 in., and a minimum concrete cover of two and one-half longitudinal bar diameters, ….

Code Section 21.8.2.3

Modify as follows:

Columns or boundary elements of special reinforced concrete structural walls that have an edge within one-half the footing depth from an edge of the footing shall have transverse reinforcement in accordance with 21.4.4 provided below the top of the footing. This reinforcement shall extend into the footing a distance ~~equal to~~ no less than the smaller of the ~~full~~ depth of the footing, mat or pile cap or the development length in tension of the longitudinal reinforcement.

Code Section 21.8.2.4

Modify as follows:

~~Footings beneath all~~ Where earthquake effects create uplift forces in boundary elements of special reinforced concrete structural walls ~~and beneath all~~ or columns ~~resisting tension forces induced by earthquake effects shall have~~, flexural reinforcement shall be provided in the top of the footing, mat or pile cap to resist the design load combinations, and shall not be less than required by 10.5.

Commentary Section R21.8.2.4

Replace as follows:

The purpose of 21.8.2.4 is to alert the designer to provide top reinforcement as well as other required reinforcement.

Commentary References

Revise "New Reference 21.2" to read:

"NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures," 1997 Edition, Part 1: Provisions (FEMA 302, 353 pp.) and Part 2: Commentary (FEMA 303, 335 ~~pages~~pp.), Building Seismic Safety Council, Washington, DC., 1998.

EDITORIAL CORRECTIONS

Commentary Section R1.1.8.2

Modify the first sentence of each of the first and second paragraphs to correct the section reference, as follows:

For buildings in regions of moderate seismic risk, or for structures assigned to intermediate seismic performance or design categories, reinforced concrete moment frames proportioned to resist earthquake effects require some special reinforcement details, as specified in ~~21.9~~ 21.10 of Chapter 21.

For buildings located in regions of high seismic risk, or for structures assigned to high seismic performance or design categories, all building components, structural and nonstructural, must satisfy requirements of 21.2 through ~~21.7~~ 21.8 of Chapter 21.

Commentary Section R1.3
Revise fifth sentence to read in part:
~~Qualifications~~ Qualification of inspectors can be obtained from a certification program

Code Section 2.1
Revise to read
Pedestal – Upright compression member with a ratio of unsupported height to average ~~at~~ least lateral dimension not exceeding three.

Code Section 7.6.7.1
Revise to read:
~~Clear~~ Center-to-center spacing of pretensioning tendons at each …

Code Section 10.0 — Notation
Revise Item (c) to read:
β_d = (c) for stability checks of sway frames carried out in accordance with ~~10.13.16~~ 10.13.6, β_d is the ~~ration~~ ratio of the maximum factored sustained axial load to the maximum factored axial load.

Commentary Section R10.6.4
Revise the second sentence of the third paragraph to read:
"Research (10^{xx}, 10^{yy}) shows that corrosion is not clearly correlated with surface crack widths in the range normally found with reinforcement stresses at service ~~lead~~ load levels."

Commentary Section R12.15.4
Revise the second sentence of the second paragraph to read:
"Such welds are rated ~~at~~ as the product of total weld length…"

Code Section 18.13
On Page S-34, the reason statement was fragmented, and paragraphs printed in the wrong sequence. Revise contents of that page in part as follows:

Code Section 18.13 — Post-Tensioned Tendon Anchorage Zones
Reason. This proposal is a modernization and expansion of the present Section 18.13. It applies to post-tensioned anchorages. There have not been many serious problems with

monostrand tendons meeting the PTI Specifications so that the requirements of that Specification are included by reference.

The proposal is mainly directed at the proper anchorage of large multistrand post-tensioning tendons using the very compact proprietary anchorage systems now widespread in usage. Such tendons have been widely used in bridge systems and their use resulted in a number of failures, many incidences of severe cracking requiring repair, and a number of major claims, delays and lawsuits. A comprehensive study was summarized in NCHRP Report 356, and design and construction provisions were adopted in the 1994 AASHTO Standard Specifications for Highway Bridges. Previous AASHTO Specifications and ACI Building Codes gave little guidance to control of bearing, bursting and spalling stresses in the tendon anchorage areas. Most of the modern compact tendon anchorage devices for multiple strands require substantial confining reinforcement immediately around the anchor device and many require supplemental face steel in the anchor region to fully develop strength and control cracking.

This growing usage of multistrand tendons in building-type structures and the experience of AASHTO indicates it is timely to expand ACI 318 coverage in this area. It was decided by Committee 318 to not use the full AASHTO provisions but to condense them for building-type applications. The AASHTO acceptance test procedures for special anchorage devices are adopted by reference. Detailed design requirements given in AASHTO are cited in the ACI 318 Commentary for guidance. Adoption of this proposal should result in more consistent, safer, and better detailed tendon anchorage zones.

Replace present Section 18.13 with the following:

New Commentary Section R18.13
Replace present Commentary R18.13 with the following and renumber existing R18.14 to R18.15, R18.16 to R18.18, R18.18 to R18.20 , and R18.19 to R18.21 …..

New Commentary Section R18.22 – External post-tensioning
External attachment of tendons is a versatile method of providing additional strength, or ~~improvising~~ improving serviceability, or both, in existing structures. It is well …

Commentary Section R21.2.1
Revise the second sentence of the sixth paragraph to read:
"These requirements are contained in ~~21.2.2.3~~ 21.2.1.3 and ~~21.8~~ 21.10."

Revise the last sentence of the seventh paragraph to read:
"The provisions of Sections 21.2 through ~~21.7~~ 21.9 have been developed to provide the structure with adequate toughness for this special response."

Metric Edition

Code Section 3.8.1
Update ASTM A 706M-96b to A 706M-98