Los Angeles Tall Buildings Structural Design Council

AN ALTERNATIVE PROCEDURE FOR SEISMIC ANALYSIS AND DESIGN OF TALL BUILDINGS

2023 Edition

November 10, 2023

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A consensus document developed by the Council

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ABOUT THE COUNCIL

The Los Angeles Tall Buildings Structural Design Council was formed in 1988 to provide a forum for the discussion of issues relating to the design of tall buildings. The Council seeks to advance state-of-the-art structural design through interaction with other professional organizations, building departments, and university researchers as well as recognize significant contributions to the structural design of tall buildings.

The Council is a nonprofit California corporation whose members are those individuals who have demonstrated exceptional professional accomplishments in the structural design of tall buildings. The annual meeting of the Council represents a program for engineers, architects, contractors, building Official and students. The annual meeting program includes research reports on areas of emerging importance, case studies of current structural designs, and consensus documents by the membership on contemporary design issues.

The Technical Committee of the Council is tasked with continuous development and updating of this document.

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Abbreviations

ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ASCE	American Society of Civil Engineers
CQC	Complete quadratic combination
DE	Design Earthquake, defined by ASCE 7-16 as the earthquake effects that are two-thirds of the corresponding Maximum Considered Earthquake (MCE_R) effects
EOR	Engineer of Record
FEMA	Federal Emergency Management Agency
GMPE	Ground motion prediction equation
IBC	International Building Code
LATBSDC	Los Angeles Tall Buildings Structural Design Council
LRFD	Load and resistance factor design
MCE_R	Risk-targeted Maximum Considered Earthquake
PBD	Performance-based design
PSHA	Probabilistic seismic hazard analysis
$RotD_{D50}$	Ground motions oriented so as to produce geometric mean response
$RotD_{100}$	Ground motions oriented so as to produce maximum response
SLE	Service-Level Earthquake
SPRP	Seismic Peer Review Panel
SSI	Soil–structure interaction
USGS	United States Geological Survey

Glossary

Action – A strain, displacement, rotation or other deformation resulting from the application of design loads.

Deformation-controlled action – An action expected to undergo nonlinear behavior in response to earthquake shaking, and which is evaluated for its ability to sustain such behavior.

Force-controlled action – An action that is not expected to undergo nonlinear behavior in response to earthquake shaking, and which is evaluated on the basis of its available strength.

Critical action – A force-controlled action, the failure of which is likely to lead to partial or total structural collapse.

Ordinary action – A force-controlled action, the failure of which is either unlikely to lead to structural collapse or might lead to local collapse comprising not more than one bay in a single story.

Backstay Effect – The set of lateral forces developing within a podium structure to equilibrate the lateral forces and moment of a tower extending above the podium structure. This condition is common to tall core wall buildings in which the core extends into a stiff basement structure braced by stiff basement walls around the perimeter.

Capacity Design – A design approach wherein the structure is configured and proportioned to restrict yielding and inelastic behavior to specific deformation-controlled actions for which structural detailing enables reliable inelastic response without critical strength decay, and which, through their plastic response, limit the demands on other portions of the structure such that those other parts can be designed with sufficient strength to reliably remain essentially elastic.

Coefficient of Variation – A standardized measure of the dispersion or probability distribution associated with response parameter, calculated as the ratio of the standard deviation to the mean value.

Conditional Mean Spectrum – The expected response spectrum conditioned on occurrence of a target spectral acceleration at a selected period.

Design Earthquake Ground Motion – The level of ground shaking equal to two-thirds of the Risk-Targeted Maximum Considered Earthquake ground motion.

Expected Strength – The probable peak strength of a material, or the probable peak strength of a structural element considering expected material strength and bias in the calculation model, as opposed to nominal or specified strength as commonly used in building codes.

Fault Parallel – Motion along an azimuth parallel to the direction of fault strike.

Fault Normal – Motion along an azimuth perpendicular to the direction of fault strike.

Fling Step – Characteristic of near-fault ground motion associated with elastic rebound of the earth's crust, characterized by large-amplitude velocity pulse and a monotonic step in the displacement history.

Hazard Curve – A plot of the mean annual frequency of exceedance of a ground motion intensity parameter as a function of the ground motion intensity parameter.

Hazard Level – A probability of exceedance within a defined time period (or return period) at which ground shaking intensity is quantified.

Monotonic Loading – Loading of a structural component in which the displacement increases monotonically without unloading or reloading.

Nominal Strength –The strength of an element, calculated using specified material properties and the strength formulation specified by the applicable materials standard, before application of a resistance (strength reduction) factor.

Residual Story Drift Ratio – The value of story drift ratio at a location in a structure at rest, following response to earthquake motion.

Return Period – The average time span between shaking intensity that is equal to or greater than a specified value, also known as the recurrence interval; the annual frequency of exceeding a given intensity is equal to the reciprocal of the return period for that intensity.

Risk-Targeted Maximum Considered Earthquake Ground Motion – The level of ground motion specified by the ASCE 7 standard as a basis for derivation of design ground motions.

Rupture Directivity – Effects associated with the direction of rupture propagation relative to the project site.

Scenario Spectrum – A site-specific response spectrum constructed for a specific magnitude earthquake along a particular fault. The scenario may also include definition of epicentral location, rupture propagation direction, and other parameters.

Service-Level Earthquake Ground Motion – The level of ground motion represented by an elastic, damped, acceleration response spectrum that has a return period of 43 years, approximately equivalent to a 50% exceedance probability in 30 years.

Site Response Analysis – Analysis of wave propagation through a soil medium used to assess the effect of local geology on the ground motion.

Story Drift Ratio – The difference, at a specific instance of time, in lateral deflections at two adjacent horizontal levels divided by the vertical distance between the levels, commonly taken along principal axes of the building.

Transient Story Drift Ratio – The maximum absolute value of story drift ratio that occurs during a single response history analysis.

Uniform Hazard Spectrum – A site-specific, acceleration response spectrum constructed such that the ordinate at each natural period has the same exceedance probability or average return period.

Notation

A_{cv}	Gross area of concrete section bounded by web thickness and length of wall section in the direction of shear force
A_g	Gross area of cross section
A_s	Cross sectional area of steel beam
A_x	Torsional amplification coefficient calculated in accordance with ASCE 7 Section 12.8.4.3
A_{web}	The web area of steel beam
B	Factor to account for conservatism in nominal resistance
C_d	Deflection amplification factor, as defined in ASCE 7
D	Dead loads, or related internal moments, forces, or deformations, including effects of self- weight and permanently attached equipment and fixtures, as defined in ASCE 7
$\mathrm{D}_{5\text{-}95}$	Duration of an earthquake record, during which 90% of the record's energy is expended, computed calculated as an integral of the square of the acceleration
D_u	Ultimate deformation capacity; the largest deformation at which the hysteresis model is deemed valid given available laboratory data or other substantiating evidence
D_{u_VRM}	Same as D_u above
D_{u_LSL}	Ultimate deformation capacity associated with lateral strength loss
d_b	Bar diameter
E	Effect of horizontal and vertical earthquake-induced forces
E_c	Modulus of elasticity of concrete
$(EI)_{tr}$	Transformed EI = $(E_cI_g/5) + E_sI_s$ per ACI 318
E_M	Expected value of the capacity-limited earthquake load on the action, as defined in the applicable material standard (ACI 318, AISC 341)
E_s	Modulus of elasticity of steel, taken as 29,000 ksi (200,000 MPa)
E_X	Earthquake loads, or related internal moments, forces, or deformations, resulting from earthquake shaking applied along the principal axis of building response designated as the X-axis
E_Y	Earthquake loads, or related internal moments, forces, or deformations, resulting from earthquake shaking applied along an axis that is orthogonal to the X-axis
F_a	Short-period site coefficient (at 0.2 s), as defined in ASCE 7
f'_{ce}	Expected compressive strength of concrete
f_c'	Specified compressive strength of concrete

F_{PGA}	Site coefficient for PGA, as defined in ASCE 7
F_r	Post-peak residual yield strength of a component under monotonic loading
f_u	Specified ultimate strength of structural steel or steel reinforcement
f_{ue}	Expected ultimate strength of structural steel or steel reinforcement
F_v	Long-period site coefficient (at 1.0 s), as defined in ASCE 7
f_y	Specified yield strength of structural steel or steel reinforcement
F_y	Effective yield strength of a component under monotonic loading
f_{ye}	Expected yield strength of structural steel or steel reinforcement
f_{yt}	Specified tensile strength of structural steel or steel reinforcement
G_c	Shear modulus of concrete, commonly taken as $0.4E_c$
G_s	Shear modulus of steel, taken as 11,500 ksi (7900 MPa)
h	Story height, or coupling beam depth
H_n	Structural height, which is the vertical distance from the base to the highest level of the seismic force-resisting system of a structure
h_w	Height of entire wall from base to top, or height of wall segment or wall pier considered
H	Height of the roof above the grade plane for damping calculation, or height of basement wall for seismic earth pressure
I_e	Seismic importance factor
I_g	Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement
K_e	Elastic (secant) stiffness up to the yield point of a component
ℓ	Clear span of coupling beam
L	Live load
l_w	Shear wall length
M_{ne}	Expected moment strength of a beam
M_{pe}	Expected plastic moment capacity of a beam
M_W	Moment magnitude, a logarithmic scale for measure of earthquake size, as characterized by the amount of strain energy released by the event
P	Vertical force above a structural level
Q	Characteristic stress resultant (force or moment) in a structural component
Q_{ns}	That portion of the load on an element resulting from dead, live, and effects other than seismic
Q_T	The total demand, including gravity and seismic loading calculated by analysis
R	Response modification coefficient, as defined in ASCE 7

R^*_r	Residual strength from cyclic backbone
R^*_u	Peak strength on a cyclic backbone
R_n	Nominal (or specified) strength of an element, as defined in the applicable materials standards (AISC 341, AISC 360, ACI 318)
R_{ne}	Expected component strength
R_u	Peak strength on a monotonic backbone
R_y	Effective yield strength
S_{MS}	Site-adjusted MCE_R short period spectral acceleration
S_{TV}	Site-adjusted MCE_R vertical spectral acceleration corresponding to period T_V
S_{VA}	Vertical acceleration effect which may be taken as either $0.2S_{MS}$ or $0.3S_{TV}$
T	Fundamental period of vibration of the building
T_V	Building natural period in the vertical direction
u_{FIM}	Foundation input motions that are modified to account for the effects of base-slab averaging and foundation embedment
u_g	Free-field ground motion
V	Seismic base shear at the hazard level under consideration
V_{DE}	Seismic base shear used for design at the Design Earthquake hazard level
V_{SLE}	Seismic base shear at the SLE hazard level
V_{s30}	Average shear wave velocity in the upper 30 meters of soil
V_{s30}^B	Value of V_{s30} at the base of the profile
v_c	Nominal two-way shear strength provided by concrete
v_s	Nominal two-way shear strength provided by steel reinforcement
v_{uv}	factored shear stress on the slab critical section for two-way action, from the controlling load combination, without moment transfer
γ	Load factor from ASCE 7 Chapter 16
δ	Lateral displacement
Δ	Characteristic component displacement
Δ^*_p	Plastic deformation to the peak strength on cyclic backbone
Δ^*_{pc}	Plastic deformation of the descending portion of cyclic backbone
Δ^*_{ult}	Ultimate deformation capacity
Δ_p	Plastic deformation to the peak strength point on monotonic backbone
Δ_{pc}	Plastic deformation of the descending portion of monotonic backbone

ε	Number of logarithmic standard deviations that a spectral response acceleration value lies above (+) or below (-) the median value at a given period
ε_s	Steel yield strain
ζ	Fraction of critical damping
θ	Elastic stability coefficient
Θ	Characteristic component rotation
λ	Modification factor to reflect the reduced mechanical properties for lightweight concrete relative to normal-weight concrete of the same compressive strength
μ	Mean value of a population of values
ρ	Redundancy factor based on the extent of structural redundancy present in a building, as defined in ASCE 7
ρ_t	Ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement
σ	Standard deviation of a population of values
ϕ	Resistance (strength reduction) factor as obtained from appropriate material standard
ϕ_S	Seismic resistance (strength reduction) factor
Ω_0	Overstrength factor, as defined in ASCE 7

1. Introduction

1.1. GENERAL

The intent of the document is to provide a performance-based approach for seismic design and analysis of tall buildings with predictable and safe performance when subjected to earthquake ground motions. These provisions result in more accurate identification of relevant demands on tall buildings. As such, the application of the procedures contained in this document are expected to result in buildings that effectively and reliably resist earthquake forces and are likely to be repairable after major earthquakes. For performance-based wind design please see ASCE Prestandard (ASCE 2019).

Seismic design of buildings in accordance with these guidelines offer a number of advantages including:

- More reliable attainment of intended seismic performance.
- Reduced construction costs.
- Elimination of some prescriptive code design requirements.
- Accommodation of architectural features that may not otherwise be attainable.
- Use of innovative structural systems and materials.

Notwithstanding these potential advantages, engineers contemplating a building design using this document shall give due consideration to the fact that appropriate implementation of these recommendations requires an in-depth understanding of ground shaking hazards, structural materials behavior and nonlinear dynamic structural response.

1.2. DESIGN TEAM QUALIFICATIONS

Appropriate implementation of these procedures requires proficiency in structural and earthquake engineering including knowledge of:

- Seismic hazard analysis and selection and scaling of ground motions.
- Nonlinear dynamic behavior of structures and foundation systems including construction of mathematical models capable of reliable prediction of such behavior using appropriate

software tools.

- Capacity design principles.
- Detailing of elements to resist cyclic inelastic demands, and assessment of element strength, deformation and deterioration characteristics under cyclic inelastic loading.

1.3. SIGNIFICANT CHANGES FROM THE 2020 EDITION

The following is a list of major changes that distinguish this document from the previous edition of the LATBSDC Alternative Analysis and Design Procedure document:

- Incorporation of updates for consistency with relevant provisions of ASCE 7-22, 2021 International Building Code (IBC) and 2022 California Building Code (CBC).
- Revised and updated strength and stiffness properties (Sections 2.5.1 and 2.5.2) and corresponding commentary (Commentary C.2.5);
- Incorporation of guidance for use of DE instead of SLE if the necessary building and seismic hazard conditions are satisfied (Commentary C.3.1).
- Incorporation of updates on selection and modification of ground motion records (Section 3.2.3)
- Updated classification of structural actions (Section 3.3.1)
- Updated provisions for consideration of vertical ground motion effects (Sections 3.4.6 and 3.6.3.2).
- Revised and updated provisions and commentary on foundation modeling and soil-structure interaction (Section 3.4.7 and Commentary 3.4.7.1)
- Revised and updated deformation limits for deformation-controlled actions (Table 6-2).
- Incorporation of new provisions for curtain walls and stairways (Section 3.6.3.2.4
- Updated seismic instrumentation provisions (Chapter 5).
- Incorporation of a new appendix titled Supplement to ACI 318-19 (Appendix D).

2. INTENT, SCOPE, JUSTIFICATION, AND METHODOLOGY

2.1. INTENT

The intent of the document is to provide an alternate, performance-based approach for seismic design and analysis of tall buildings with predictable and safe performance when subjected to earthquake ground motions. These provisions result in more accurate identification of relevant demands on tall buildings. As such, the application of the procedures contained in this document is expected to result in buildings that effectively and reliably resist earthquake forces.

C.2.1. Code provisions are intended to provide a minimum level of safety for engineered buildings. The prescriptive code provisions are intended to produce safe designs for all types of buildings, ranging from small one and two-story dwellings to the tallest structures. As a result of this broad intended applicability, the provisions contain many requirements that are not specifically applicable to tall buildings and which may result in designs that are less than optimal, both from a cost and safety perspective. Advances in performance-based design methodologies and maturity of capacity design principles now permit a more direct, non-prescriptive, and rational approach to analysis and design of tall buildings. This document relies on these advances to provide a rational approach to seismic design of reliable and effective tall building structures. This document addresses only non-prescriptive seismic design of tall buildings.

This document is not intended to cover essential facilities unless acceptance criteria are modified accordingly.

2.2. SCOPE

This document was developed for design of tall buildings although the document may be used to design other building types. For the purposes of this document, tall buildings are defined as those with a height, h_n, greater than 160 feet above average adjacent ground surface.

The height, h_n is the height of Level n above the Base. Level n may be taken as the roof of the structure, excluding mechanical penthouses and other projections above the roof whose mass is small compared with the mass of the roof. The Base is permitted to be taken at the average level of the ground surface adjacent to the structure.

C.2.2. Nothing in this document precludes its applicability to shorter buildings. The focus, however, has been intentionally narrowed to tall buildings. The procedures contained in this document require specialized knowledge and review procedures typically not appropriate for the design of buildings which lend themselves to prescriptive based procedures.

2.3. JUSTIFICATION

The provisions of this document are justified based on Section 104.11 of 2021 edition of International Building Code (2021 IBC) and the same section in the 2022 California Building Code (2022 CBC). These code provisions permit the application of alternative lateral-force procedures using rational analysis based on well-established principles of mechanics in lieu of prescriptive code provisions. The City of Los Angeles Department of Building and Safety has approved this approach in the Information Bulletin P/BC 2017-123, "Alternate Design Procedure for Seismic Analysis and Design of Tall Buildings and Buildings Utilizing Complex Structural Systems."

C.2.3. Codes have traditionally permitted the use of alternative analysis and design methods which can be justified by well-established principles of mechanics and/or supported by convincing laboratory test results.

Section 104.11 of 2021 IBC reads as follows:

"The provisions of this code are not intended to prevent the installation of any material or to prohibit any design or method of construction not specifically prescribed by this code, provided that any such alternative has been approved. An alternative material, design or method of construction shall be approved where the building official finds that the proposed alternative meets all of the following:

1. *The alternative material, design or method of construction is satisfactory and complies with the intent of the provisions of this code,*
2. *The material. Method or work offered is, for the purpose intended, not less than the equivalent of that prescribed in this code as it pertains to the following:*
   - 2.1. *Quality.*
   - 2.2. *Strength.*
   - 2.3. *Effectiveness*
   - 2.4. *Fire resistance.*
   - 2.5. *Durability.*
   - 2.6. *Safety"*

Section 1.3 of ASCE 7-22 also permits the use of alternative performance-based approaches that use analysis, testing, or a combination thereof, as acceptable alternative means.

2.4. METHODOLOGY

The procedures contained in this document are based on capacity design principles followed by a series of performance-based design evaluations. First, capacity design principles shall be applied to design the structure to have a suitable ductile yielding mechanism, or mechanisms, under nonlinear lateral deformations with a clear definition of regions, components, and actions that may behave nonlinearly during seismic response of the structure.

The adequacy of the design and the attainment of acceptable building performance shall be demonstrated using two earthquake ground motion intensities:

A. Serviceable Behavior When Subjected to Frequent Earthquake Ground Motions. The service level design earthquake ground motions shall be taken as the ground motions having a 50% probability of being exceeded in 30 years (43-year return period). Structural models used in the serviceability evaluation shall incorporate realistic estimates of stiffness and damping considering the anticipated levels of excitation and damage. The purpose of this evaluation is to validate that the building's structural and nonstructural components retain their general functionality during and after such an event. Repairs, if necessary, are expected to be minor and could be performed without substantially affecting the normal use and functionality of the building. Subjected to this level of earthquake ground motion, the building structure and nonstructural components associated with the building shall remain essentially elastic. This evaluation shall be performed using three-dimensional linear or nonlinear dynamic analyses. Essentially elastic response may be assumed for elements when force demands generally do not exceed provided strength. When demands exceed provided strength, the exceedance shall not be so large as to affect the residual strength or stability of the structure.

B. Low Probability of Collapse and the Likelihood of Building Repairability when subjected to Extremely Rare Earthquake Ground Motions. The extremely rare earthquake motions shall be taken as the Risk Targeted Maximum Considered Earthquake (MCE_R) ground motions as defined by ASCE 7-22. This evaluation shall be performed using three-dimensional nonlinear dynamic response analyses. This level of evaluation is intended to demonstrate a low probability of collapse for Seismic Risk Category II buildings and low probability of losing the life-safety status for Risk Category III buildings, when the building is subjected to the above-mentioned ground motions. The evaluation of demands includes both structural members of the lateral force resisting system and other structural members. Claddings and their connections to the structure must accommodate MCE_R displacements without failure.

A summary of the basic requirements for each step of analysis is presented in Table 1. More detailed information regarding these steps is contained in the subsequent sections of this document.

Table 1. Summary of Basic Requirements

Design / Evaluation Step	Ground Motion Intensity[1]	Type of Analysis	Type of Mathematical Model	Accidental Torsion Considered?	Material Reduction Factors (ϕ)	Material Strength
1	Nonlinear Behavior Defined / Capacity Design					
2	50/30	LDP[2] or NDP[3]	3D[4]	Evaluated	1.0	Expected properties are used throughout
3	MCE_R[5]	NDP	3D[4]	Yes, if flagged during Step 2. No, otherwise.	See Section 3.6	

1 probability of exceedance in percent / number of years
2 linear dynamic procedure
3 nonlinear dynamic procedure
4 three-dimensional
5 per ASCE 7-22 with modifications and exceptions as noted in this document

2.5. MATERIAL PROPERTIES

2.5.1. Strength and Stiffness Properties

Structural models shall incorporate realistic estimates of stiffness and strength considering the anticipated level of excitation and damage. Expected material properties shall be utilized in all analyses as opposed to nominal or specified properties. The elastic (initial) stiffness of steel members and components shall be modeled using full cross-sectional properties and the elastic modulus of steel (E_s = 29,000 ksi). In lieu of detailed justifications, values provided in Tables 2 and 3 may be used for expected material strengths and estimates of expected strength and stiffness of various materials and structural elements. Effective stiffness values for DE should be based on ACI 318-19 or other recommendations; suggested values are also provided in Table 3.

Table 2. Expected Material Strengths

Material	Expected strength	
Reinforcing Steel	Expected Yield Strength, f_{ye}, ksi	Expected Ultimate Strength, f_{ue}, ksi
A615 Grade 60	70	106
A615 Grade 75	82	114
A706 Grade 60	69	95
A706 Grade 80	85	112
A706 Grade 100	105	To be determined based on tests and documented substantiations
Structural Steel* **		
Hot-rolled structural shapes and bars		
ASTM A36/A36M	1.5 f_y*	1.2 f_u**
ASTM A572/A572M Grade 50	1.1 f_y	1.1 f_u
ASTM A913/A913M Grade 50, 60, 65 or 70	1.1 f_y	1.1 f_u
ASTM A992/A992M	1.1 f_y	1.1 f_u
Plates		
ASTM A36/A36M	1.3 f_y	1.2 f_u
ASTM A572/A572M Grade 50, 55	1.1 f_y	1.2 f_u
Concrete	$f'_{ce} = 1.3 f'_c$	

* f_y is used to designate the specified (nominal) yield strength of steel materials in this Guideline. It is equivalent to f_y or f_{yt} used in ACI 318 and F_y used in AISC (2022) Specifications.

** f_u is used to designate the specified (nominal) ultimate strength of steel materials in this Guideline. It is equivalent to F_u used in AISC (2022) Specifications.

***For steel materials not listed, refer to Table A3.1 of ANSI/AISC 341-22

†f_c' = specified compressive strength per ACI 318 at 28 days. Expected strength f_{ce}' is strength expected at any time between 90 and 365 days. Note that the multiplier on f_c' may be smaller for high-strength concrete, and can also be affected by (1) use of fly ash and other additives, and/or (2) local aggregates.

2.5.2. Use of High-Strength Reinforcing Steel

High strength reinforcing steel (Grade 80 or higher) may be used if such use is consistent with the requirements of the 2019 Edition of ACI 318 and satisfies all of its relevant provisions.

Table 3. Reinforced Concrete Stiffness Properties*

Component	*Service-Level Earthquake (SLE) Linear Models*			*Design Earthquake (DE) Linear Models*			*MCE Nonlinear Models*		
	Axial	**Flexural**	**Shear**	**Axial**	**Flexural**	**Shear**	**Axial**	**Flexural**	**Shear**
Structural walls[1] (in-plane)	$1.0E_c^{*}A_g$ *or* $0.75\ E_cA_g^{**}$	$0.75E_cI_g$	$1.0G_cA_g$	$1.0E_cA_g$	$0.5\text{-}0.6E_cI_g$	$0.75G_cA_g$	$1.0E_cA_g$	$0.35E_cI_g$	$0.5G_cA_g$
Structural walls (out-of-plane)	--	$0.25E_cI_g$	$1.0G_cA_g$		$0.25E_cI_g$		--	$0.25E_cI_g$	$1.0G_cA_g$
Basement walls (in-plane)	$1.0E_cA_g$	$1.0E_cI_g$	$1.0G_cA_g$	$1.0E_cA_g$	$0.9E_cI_g$	$0.75G_cA_g$	$1.0E_cA_g$	$0.8E_cI_g$	$0.5G_cA_g$
Basement walls (out-of-plane)	--	$0.25E_cI_g$	$1.0G_cA_g$		$0.25E_cI_g$	$1.0G_cA_g$	--	$0.25E_cI_g$	$1.0G_cA_g$
Coupling beams with or without diagonal reinforcement	$1.0E_cA_g$	$0.07\left(\frac{\ell}{h}\right)E_cI_g \leq 0.3E_cI_g$	$1.0G_cA_g$	$1.0E_cA_g$	$0.07\left(\frac{\ell}{h}\right)E_cI_g \leq 0.3E_cI_g$	$1.0G_cA_g$	$1.0E_cA_g$	$0.07\left(\frac{\ell}{h}\right)E_cI_g \leq 0.3E_cI_g$	$1.0G_cA_g$
Coupling beams with steel-fiber reinforcement	$1.0E_cA_g$	$0.07\left(\frac{\ell}{h}\right)E_cI_g \leq 0.3E_cI_g$	$1.0G_cA_g$	$1.0E_cA_g$	$0.07\left(\frac{\ell}{h}\right)E_cI_g \leq 0.3E_cI_g$	$1.0G_cA_g$	$1.0E_cA_g$	$0.07\left(\frac{\ell}{h}\right)E_cI_g \leq 0.3E_cI_g$	$1.0G_cA_g$
Steel Coupling Beams[2]	$1.0E_sA_s$	$0.07\left(\frac{\ell}{h}\right)(EI)_{tr}$	$1.0G_sA_{web}$	$1.0E_sA_s$	$0.07\left(\frac{\ell}{h}\right)(EI)_{tr}$	$1.0G_sA_{web}$	$1.0E_sA_s$	$0.07\left(\frac{\ell}{h}\right)(EI)_{tr}$	$1.0G_sA_{web}$
Non-PT diaphragms (in-plane)[3]	$0.5E_cA_g$	$0.5E_cI_g$	$1.0G_cA_g$	$0.5E_cA_g$	$0.5E_cI_g$	$1.0G_cA_g$	$0.25E_cA_g$	$0.25E_cI_g$	$0.25G_cA_g$
PT diaphragms (in-plane)[3]	$0.8E_cA_g$	$0.8E_cI_g$	$1.0G_cA_g$	$0.8E_cA_g$	$0.8E_cI_g$	$1.0G_cA_g$	$0.5E_cA_g$	$0.5E_cI_g$	$0.5G_cA_g$
Slab-Beam (out-of plane)	$1.0E_cA_g$	***	$1.0G_cA_g$	$1.0E_cA_g$	***	$1.0G_cA_g$	$1.0E_cA_g$	***	$1.0G_cA_g$
Beams	$1.0E_cA_g$	$0.5E_cI_g$	$1.0G_cA_g$	$1.0E_cA_g$	$0.3E_cI_g$	$1.0G_cA_g$	$1.0E_cA_g$	$0.3E_cI_g$	$1.0G_cA_g$
Columns	$1.0E_cA_g$	$0.7E_cI_g$	$1.0G_cA_g$	$1.0E_cA_g$	$0.7E_cI_g$	$1.0G_cA_g$	$1.0E_cA_g$	$0.7E_cI_g$	$1.0G_cA_g$
Mat (in-plane)	$0.8E_cA_g$	$0.8E_cI_g$	$1.0G_cA_g$	$0.5E_cA_g$	$0.5E_cI_g$	$1.0G_cA_g$	$0.5E_cA_g$	$0.5E_cI_g$	$1.0G_cA_g$
Mat[4] (out-of-plane)	--	$0.8E_cI_g$	$1.0G_cA_g$		$0.5E_cI_g$	$1.0G_cA_g$	--	$0.5E_cI_g$	$1.0G_cA_g$

[1]Values are relevant where walls are modeled with non-fiber elements. Where walls are modeled using fiber elements, the model should automatically account for cracking of concrete and the associated effects on member stiffness.

[2] E_s and A_s are modulus of elasticity and cross sectional area of steel beam, respectively. A_{web} is the web area of steel beam. $(EI)_{tr} = (E_c I_g/5) + E_s I_s$ per ACI 318.

[3] Specified stiffness values for diaphragms are intended to represent expected values. Alternative values may be suitable where bounding analyses are used to estimate bounds of force transfers at major transfer levels. For diaphragms that are not associated with major force transfers, common practice is to model the diaphragm as being rigid in its plane. Flexural rigidity of diaphragms out of plane is usually relatively low and is commonly ignored. The exception is where the diaphragm acts as a framing element to engage gravity columns as outrigger elements, in which case out-of-plane modeling may be required.

[4] Specified stiffness values for mat foundations pertain to the general condition of the mat. Where the walls or other vertical members impose large forces, including local force reversals across stacked wall openings, the stiffness values may need to be reduced. The values listed are intended for mat foundations supported directly on soil and not on piles.

* Modulus of elasticity of concrete based on test results or as substantiated by Engineer of Record: E_C used shall not exceed 6000 ksi.

** Since programs like ETABS, which are commonly used for SLE and DE analyses, may not permit using different values for axial and flexural stiffnesses, the use of the lower value of 0.75 E_cA_g for SLE and 0.55 E_cA_g for DE instead of 1.0 E_cA_g is also permitted.

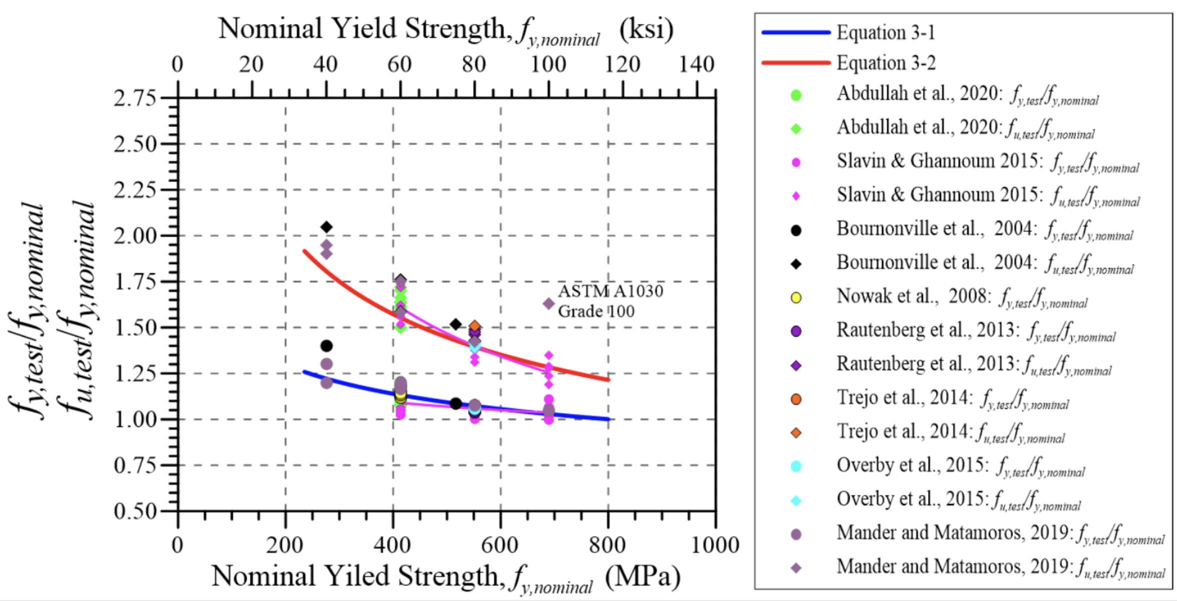
*** See Appendix C, ATC 72 (2010) and ACI 318-19

C.2.5. Table 2 steel properties for Grades 60 and 75 reinforcements are from Bournonville et al. (2004). Properties for Grade 80 are from Overby et al. (2015). The factor 1.3 applied to f_c' is based on experience with concrete mixtures having low to moderate compressive strengths and can vary depending on the factors noted. Where estimates of expected strength are especially critical in evaluating performance, project-specific data or data from projects using similar materials should be used. The values for structural steel are from Table A3.1 of ANSI/AISC 341-16.

Abdullah et al. (2022) studied test results and developed equations relating nominal strength of reinforcement to tested yield and tensile strength for various grades of reinforcement steel (their Equations 3-1 and 3-2) as presented below and compared the results obtained from these Equations to those listed in Table 2 of this document (see Figures below).

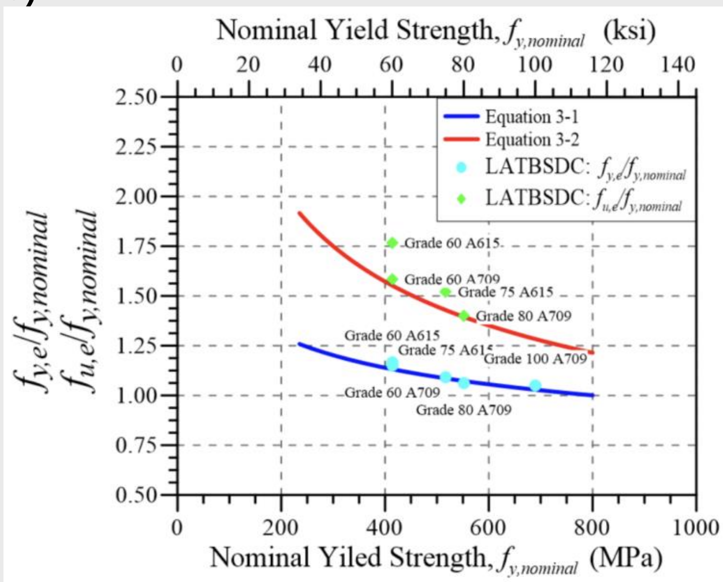
$$\frac{f_{y,test}}{f_{y,nominal}} = \frac{2.80}{\left(f_{y,nominal}, \mathrm{MPA}\right)^{0.15}} = \frac{2.09}{\left(f_{y,nominal}, \mathrm{ksi}\right)^{0.15}} \geq 1.0 \qquad (3-1)$$

$$\frac{f_{u,test}}{f_{y,nominal}} = \frac{13.45}{\left(f_{y,nominal}, \mathrm{MPA}\right)^{0.36}} = \frac{6.71}{\left(f_{y,nominal}, \mathrm{ksi}\right)^{0.36}} \geq 1.0 \qquad (3-2)$$



Expected overstrength ratios for reinforcement yield and tensile strengths from tests reported in the literature and from Eq. 3-1 and Eq. 3-2 of Abdullah et al. (2022).

C.2.5. (continued).



Comparison of expected overstrength ratios for reinforcement yield and tensile strengths from LATBSDC Table 2 with Eq. 3-1 and Eq. 3-2. of Abdullah et al. (2022).

Table 3 effective stiffness values are intended to represent effective stiffness for loading near the onset of material yielding. Where expected stress levels are less than yield, it may be justified to increase the effective stiffness values. Where inelastic structural analysis models are used, the tabulated effective stiffness values can be acceptable as the effective linear branch of the inelastic model.

The value of $1.0E_cA_g$ under the “Axial” column in Table 3 indicates that no stiffness modifiers need to be applied to the elastic axial stiffness term obtained from the use of gross concrete elastic stiffness properties.

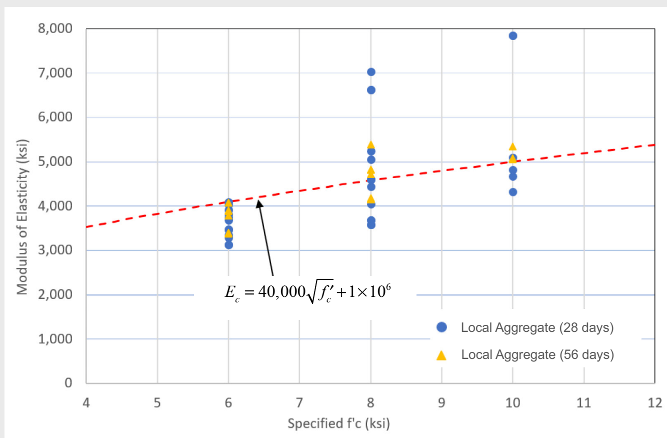
For structural walls, common practice for nonlinear analysis is to use fiber models to represent axial and bending responses, with shear response represented by a linear stiffness. In such cases, the fiber model is used directly to represent axial and flexural stiffness, with the tabulated values not used. The effective shear stiffness would still apply if shear is modeled by a linear spring.

For coupling beams, the effective stiffness values are based on a review of data and analytical solutions of Naish et al. (2013), Son Vu et al. (2014), and Motter et al. (2017), adjusted to account for stiffening effects associated with test specimen scale and presence of the floor diaphragm. Values are intended to be suitable for typical values of beam shear; beams with higher reinforcement ratios tend to be stiffer than typical beams. The effective stiffness values are intended for use in analysis models that explicitly model both flexural and shear deformations.

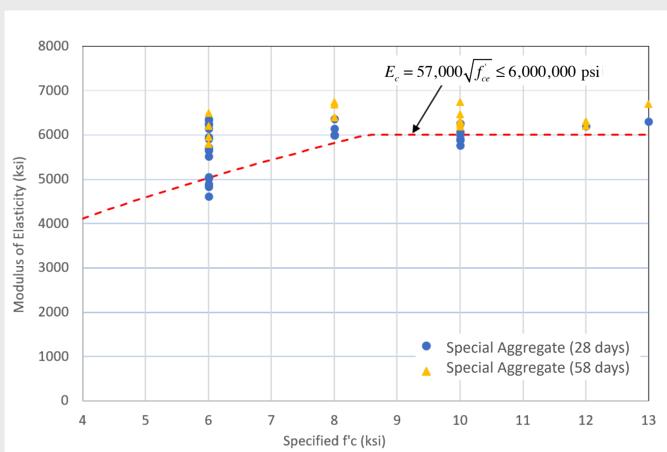
C.2.5. (continued).

The shear stiffness of walls in Table 3 corresponding to MCE_R level evaluation is approximated by a single value (50 % of gross shear stiffness to account for assumed minor shear cracking). As an alternative to this constant stiffness, a bilinear shear stiffness per ASCE 41-17 may be used. In this alternative however, an artificially high shear capacity must be specified in order to capture high shear demand from the nonlinear analysis.

Although results published in ACI 363R-10 indicate that application of $E_c = 57000\sqrt{f_c'}$ produces generally reasonable estimates of concrete modulus of elasticity, our experience in greater Los Angeles area indicates that common local area aggregates tend to exhibit a lower modulus of elasticity while concrete made with special aggregates may exhibit larger values modulus of elasticity. The formulas shown below are based on the evaluation of various aggregates used for multiple tall building projects in the Los Angeles area as illustrated in the graphs below.



Modulus of elasticity for local Los Angeles area aggregates



Modulus of elasticity for special aggregates

3. ANALYSIS AND DESIGN PROCEDURE

3.1. GENERAL

Seismic analysis and design of the building shall be performed in three steps with the intent to provide a building with the following characteristics:

(1) The building has a well-defined inelastic behavior where nonlinear actions and members are clearly defined, and all other members are designed to be stronger than the demand imposed by elements designed to experience nonlinear behavior (Capacity Design Approach).

(2) The building's structural and nonstructural systems and components remain serviceable when subjected to service level earthquake (SLE) defined as an event with a probability of exceedance of 50% in 30 years.

(3) The building has a low probability of collapse during an extremely rare event (on the order of 10% or less, given MCE_R shaking) and the likelihood of being repairable after such event.

A comprehensive and detailed peer review process is an integral part of this design criteria and a Seismic Peer Review Panel (SPRP) shall be established to review and approve the capacity design approach and building performance evaluations. Details of the peer review requirements are contained in Section 4.

C.3.1. The procedure contained in this document is an enhanced embodiment of the philosophy deeply rooted and implicit in most building codes requiring that buildings be able to:

1. Resist minor levels of earthquake ground motion without damage;
2. Resist moderate levels of earthquake ground motion without structural damage, but possibly experience some nonstructural damage;
3. Resist major levels of earthquake ground motion having an intensity equal to strongest either experienced or forecast for the building site, with a low probability of collapse, but possibly with some structural as well as nonstructural damage.

In addition, specifying the maximum residual story drift values, application of reduction factors on the capacities of force-controlled actions and limiting deformations of the deformation-controlled actions to values less than or equal to the capping point of the backbone curves (see Figure 5), are intended to result in the likelihood of building repairability after major events.

These objectives are achieved by requiring serviceability for ground motions having a 50% probability of being exceeded in 30 years and a stable predictable response without excessive deterioration of structural elements for MCE_R response.

This document transitioned from a three-level design in its 2005 Edition to a two-level design in the 2008 Edition which is retained for this 2023 Edition.

The Rationale for Elimination of Explicit Life Safety Evaluation:

The 2022 California Building Code is based on the *2021 International Building Code*, which adopts by reference the ASCE 7-16 seismic provisions. ASCE 7-16 and ASCE 7-22 definition of Risk Targeted Maximum Considered Earthquake (MCE_R) is primarily based on attaining a notional 10% (or lower) probability of collapse given the occurrence of MCE_R shaking, assuming standard structural fragility.

Since its inception, the International Building Code has been intended to provide a low probability of collapse under MCE_R shaking. However, the newer code requirements are intended to provide more explicit and quantitative protection against collapse than did earlier codes. In order to retain *R* coefficients and design procedures familiar to users of the older codes, ASCE 7-16 and ASCE 7-22 have adopted design-level earthquake shaking for purposes of evaluating strength and deformation that is 2/3 of the intensity of MCE_R shaking. This 1/3 reduction in the design earthquake is in recognition that the *R* factors traditionally contained in the older codes incorporated an inherent margin of at least 1.5. That is, buildings designed using these *R* factors are assumed to be able to resist ground shaking at least 150% of the design level without significant risk of collapse.

C.3.1. (continued).

This document adopts a philosophy that is consistent with the philosophy that underlies the 2022 CBC and 2021 IBC. Buildings must be demonstrated, through appropriate nonlinear analyses and the use of appropriate detailing to have a suitably low probability of collapse under MCE_R shaking. In addition, a service-level performance check is incorporated into the procedure to reasonably assure that buildings are not subject to excessive damage under the more frequent, low-intensity shaking, likely to be experienced by the building one or more times during its life. Protection of nonstructural components and systems is reasonably assured by requirements that such components and systems be anchored and braced to the building structure in accordance with the prescriptive criteria of the building code.

Building **performance** must be demonstrated, through appropriate nonlinear analyses and the use of appropriate detailing to have a suitably low probability of collapse under MCE_R shaking.

Some jurisdictions require consideration of the DE in the building design in addition to the SLE and MCE_R out of concern that a building designed without the DE may be damaged by a smaller earthquake than the rest of the building inventory designed with prescriptive procedures based on the DE. Forces from the DE are larger than SLE forces and where consideration of the DE is required, it is possible to determine if the DE will govern over the SLE if the relationships below are satisfied. Where the relationships are satisfied, element demands from the SLE do not need to be considered in the building design, because forces from the DE will govern the design. The equations below are conservative, and more refined relationships specific to building height, location, and Site Class are given in "Comparison of SLE and DE Acceptance Criteria for Tall Buildings" Neuman, et. al. (2023).

Deformation-controlled actions: $V_{DE} \geq 0.85\ V_{SLE}$

Force-controlled actions: $V_{DE} \geq 1.55\ V_{SLE}$

3.2. GROUND MOTION CHARACTERIZATION

3.2.1. Seismic Hazard Analysis

Site-specific seismic hazard analysis shall be used to compute appropriate acceleration response spectra for SLE and MCE_R ground motions. The uniform hazard spectrum for the SLE ground motions shall be calculated using probabilistic seismic hazard analysis with a ratio of critical damping calculated per Equation (1) of this document (see Section 3.4.4). Both probabilistic and deterministic seismic hazard analyses, in accordance with ASCE 7-22 shall be used for determination of MCE_R level shaking.

Appropriate site parameters for ground-surface, free-field conditions shall be used. These site parameters include shear wave velocity in the upper 30 m of the site (i.e., V_{S30} as defined in Chapter 20 of ASCE 7-22) and may also include additional basin depth parameters. V_{S30} values shall be computed using measured shear wave velocities from the ground surface to a depth of 30 m.

MCE_R ordinates derived from site-specific PSHA shall not fall more than 20% below those provided by standard procedures in Chapter 11 of ASCE 7-22. If the combined reductions of MCE_R ordinates relative to those produced by of ASCE 7-22 Chapter 11 procedures exceed 20%, peer-review approval of an appropriate maximum permissible reduction shall be obtained.

C.3.2.1. Ground motion reductions associated with soil–structure interaction effects are independent of those associated with site-specific ground motion analyses. Such reductions should not be applied when checking the 20% (or other) limits described above.

3.2.2. Near Fault Effects

Sites located at close distance to large-magnitude earthquakes can be subject to near-fault rupture directivity effects and fling-step effects. ASCE 7-22 Chapter 11 defines near-fault conditions as fault distances < 15 km for $M_W > 7$ earthquakes and fault distances < 10 km for $M_W > 6$ earthquakes. Ground motions for sites subject to forward rupture directivity effects have an increased likelihood of having pulse-like characteristics in their velocity-time series. When deaggregation results indicate controlling faults meet these criteria, the site shall be considered as a near-fault site.

For near-fault sites, appropriate methods shall be applied to account for rupture directivity effects on target acceleration response spectra used for ground motion selection for the MCE_R ground motions. Uncertainties associated with alternative directivity models, including the option of neglecting directivity shall also be considered.

3.2.3. Selection and Modification of Ground Motion Records

Ground motion records shall be selected and modified following Chapter 16 of ASCE 7-22 (Sections 16.2.2 and 16.2.3), with the limited exceptions noted below.

The minimum number of horizontal record pairs for each considered MCE_R-compatible target spectrum is 11. When ground motions are to be selected for the SLE, a minimum of 3 record pairs shall be used for linear or nonlinear response history analyses.

When vertical components of earthquake ground motion are required, the vertical components that accompany pairs of selected horizontal motions shall be used. For a given three-component record, the same scaling shall be applied to the vertical component as is applied to the horizontal components.

Check for compatibility of the scaled vertical component spectra with the target. Consider alternative methods if there is significant incompatibility between vertical-component motions developed with this procedure and the vertical target.

When using multiple sources, having notably different magnitude/distance combinations and each contributing more than 20% of the relative contribution to hazard at a period of interest, enough records for each of these significantly contributing seismic sources but not less than 11 records total shall be selected for MCE_R level analyses.

The durations of selected and modified ground motions shall also be documented and reported.

For sites where pulse-type motions are considered, not less than five records in the pulse or no-pulse subsets of the ground motion suite shall be used for MCE_R level analyses.

As stated in ASCE 7-16 and 7-22, the selected ground motions shall either be amplitude-scaled or spectrally matched.

Where tight spectral matching is used to individually match each horizontal ground-motion component at near-fault sites, the components shall be matched to correspond to the fault normal and fault-parallel directions.

Where significant fling-step effects are anticipated (typically at rupture distances < 10 km), and judged to be potentially important to the structural response, such affects shall be added to selected time series in the slip-parallel direction (FP for strike–slip earthquakes, FN and vertical for reverse–slip earthquakes).

In the process of ground motion selection and modification for MCE_R level analyses, for checking the period range of applicability of the selected motions, the MCE_R structural period corresponding to the properties listed in Table 3 shall be used.

C.3.2.3. Check for compatibility of the scaled vertical component spectra with the target. Consider alternative methods if there is significant incompatibility between vertical-component motions developed with this procedure and the vertical target.

The intent of multi-source supplementary requirement for ground motion selection is to allow for different characteristics (spectral shape, duration, etc.) to be reflected in the ground motions used for dynamic analysis. It is recommended to use more than five records for events that contribute significantly to the controlling hazard (i.e., >20%). In some cases, this may require the use of a total number of ground motion record pairs that exceeds the ASCE 7-22 minimum of 11. In cases where multiple fault sources contribute to the hazard beyond the 20% level, but the magnitudes identified from deaggregation are similar (i.e., within about 0.5 magnitude units), it is acceptable to combine these sources for the purpose of ground motion selection. The intent of this provision is to allow for separate suites of ground motions when attributes of controlling sources are notably distinct.

Duration is a potentially important characteristic of earthquake ground motions because long-period structures require motions of significant duration to generate response and because of the possible effects of strength or stiffness degradation in some structural components. For this reason, selected ground motions should reasonably reflect the anticipated duration of scenario earthquakes.

Since the orientation of ground motions, even in the near-fault environment, is highly uncertain, it is inadvisable to take advantage of a directional reduction that may or may not develop. These criteria can be met, as needed, by applying necessary modification (direct amplitude scaling or otherwise) to the FP components of ground motions. This may result in a suite of motions where the resultant response is somewhat higher than the typical vector sum of FN and FP components.

Ground motion time series from most databases (including the NGA-West2 database) remove fling effects as part of the data processing. Hence, when such effects are anticipated for a site, they can be added to selected ground motions. Procedures for incorporating the fling step into ground motion time series are given by Burks and Baker (2016) and Kamai et al. (2014). When fling-step effects are to be incorporated into ground motion time series, uncertainties in the key parameters (i.e., pulse period, displacement amplitude) should be considered.

ASCE 7-22 requires that the target spectrum be increased by 10% if spectral matching of the selected seed ground motions is used instead of scaling of the ground motions. In this document, the 10% increase need only to be applied if CMS ground motions are matched and need not be applied if (a) the ground motions are spectrally matched to the uniform hazard spectrum, or (b) when peer review approved hybrid matching and scaling approaches are utilized.

3.2.4. Use of V_{s30} for Site Soil Class Determination

If the total weight of the building, including its subterranean floors, exceed the weight of soil removed for construction of subterranean floors, then the value of V_{s30} determined at the half of the height from the bottom of mat and other shallow foundations to the ground level may be used for site soil class determination. Otherwise, the V_{s30} value determined at the ground level shall be used.

C.3.2.4. See Lew, M. (2020) for justification of this approach.

3.3. CAPACITY DESIGN

The building design shall be based on capacity design principles and analytical procedures described in this document. The capacity design criteria shall be described in the project-specific seismic design criteria. The structural system for the building shall be clearly demonstrated to have well defined inelastic behavior where nonlinear action is limited to the clearly identified members and regions and all other members are stronger than the elements designed to experience nonlinear behavior.

3.3.1. Classification of Structural Actions

All actions (strains, displacements, rotations or other deformations resulting from the application of loads or excitations) shall be classified as either deformation-controlled or force-controlled. Force-controlled actions shall be further classified as critical or ordinary per definitions given below. Table 4 identifies typical force-controlled actions and their recommended categories.

- **Deformation-controlled action** – An action expected to undergo nonlinear behavior in response to earthquake shaking, and which is evaluated for its ability to sustain such behavior.

- **Force-controlled action** – An action that is not expected to undergo nonlinear behavior in response to earthquake shaking, and which is evaluated on the basis of its available strength.

- **Critical action** – A force-controlled action, the failure of which is likely to lead to partial or total structural collapse.

- **Ordinary action** – A force-controlled action, the failure of which the failure of which is unlikely to lead to structural collapse or it might lead to local collapse comprising not more than one bay in a single story.

Table 4 Force-controlled actions and their categories

	Component	Seismic Action	Category: Critical	Category: Ordinary
Reinforced Concrete	Below Grade Perimeter Retaining Walls	Moment		X
		Shear		X
	Below Grade Non-Perimeter / Non-Core Walls	Shear	X	
	Core Walls Above and Below Grade and All Above Grade Walls	Shear	X	
	Diaphragms with Major Shear Transfer	Axial	X	
		Flexure	X**	
		Shear	X	
	Coupling beams without special diagonal reinforcing including steel-fiber reinforced coupling beams*	Shear	X	
	Typical (non-transfer slab) Diaphragm Forces (excludes collectors and shear transfer to vertical element)	Axial		X
		Flexure		X
		Shear		X
	All Drag (Collector) Members	Compression	X	
		Tension	X	
	Vertical Element-to-Diaphragm Connection	Bearing	X	
		Shear Transfer (Shear Friction)	X	
	Gravity Columns and Special Moment Frames (Columns, Beam-Column joints) excluding, Intentional Outrigger Columns, & Columns Supporting Discontinuous Vertical Elements)	Axial	X	
		Shear	X	
		Flexure (in P-M)	***	***
	Special Moment Frame Beams	Shear	X	
	Intentional Outrigger Columns & Columns Supporting Discontinuous Vertical Elements****	Axial	X	
		Shear	X	
		Flexure (in P-M)		X
	Transfer Girders****	Flexure	X	
		Shear	X	
	Strut and Tie in strut and tie formulation	Compression	X	
		Tension		X

* Coupling beam shear may be considered an ordinary action only if the consequence of element failure is minimal.
** See the footnote on page 69 regarding different factors for application of Equations 5a to 5f for this case.
*** Classification should depend on axial load and modeling approach used (e.g., elastic, hinge model, fiber model)
**** These actions are sensitive to vertical ground acceleration when using Equations 5a to 5f.

Table 4 Force-controlled actions and their categories (continued)

Component		Seismic Action	Category	
			Critical	**Ordinary**
Reinforced Concrete	Foundations	Flexure		X
		Shear	X	
	Foundation Piles (Structural Capacity)	Compression	X	
		Tension		X
		Flexure		X
		Shear	X	
Structural Steel	Braces in Eccentric Braced Frame	Axial	X	
	Columns in braced frame and moment frame	Compression	X	
		Tension		X
		Shear	X	
		Flexure (in P-M)		X
	Beams in braced frame	Axial		X
		Shear	X	
		Flexure (in P-M)		X
	Connections of buckling restrained braces (BRBs)	All	**X**	
	Vertical boundary elements of steel plate shear walls	Compression	X	
	Horizontal boundary elements of steel plate shear walls	Compression		X
	Moment frame connections	Flexure	X	
		Shear	X	
	Gusset plate connection in braced frames	Axial	X	
	Transfer Trusses	Axial	X	
		Flexure	X	
		Shear	X	
	All other force-controlled actions including (1) column splice forces and (2) connections of braces to beams, columns and walls		X	
Composite Plate Shear	Shear wall, CPSW	Shear	X	
	Coupling beam, CPSW	Shear *	X	
	Coupling beam-to-wall connections, CPSW	All *	X	

* Action is limited by a well-defined yield mechanism (see Section C.3.6.3.2.1.)

3.3.2. Limitations on Nonlinear Behavior

Nonlinear action shall be permitted only in clearly delineated zones. These zones shall be designed and detailed as ductile and protected zones so that the displacements, rotations, and strains imposed by the MCE_R ground motions can be accommodated with sufficient reserve capacity to avoid collapse.

C.3.3.2 Limiting occurrence of nonlinear behavior to limited and clearly identified areas of the building that are designed to dissipate energy and exhibit significant ductility is the essence of Capacity Design. Typical zones and actions commonly designated for nonlinear behavior are identified in the following table. This table is not meant to be conclusive. Other zones may be included into the design based on sufficient justification.

Table C.3.3.2 Zones and actions commonly designated for nonlinear behavior

Structural System	***Zones and Actions***
Special Moment Resisting Frames (steel , concrete, or composite)	• *Flexural yielding of Beam ends (except for transfer girders)* • *Shear in Steel Frame Beam-Column Panel Zones* • *P-M-M* yielding at the base of columns (top of foundation or basement podiums)*
Special Concentrically Braced Steel Frames	• *Braces (yielding in tension and buckling in compression)* • *P-M-M yielding at the base of columns (top of foundation or basement podiums)*
Eccentrically Braced Steel Frames	• *Shear Link portion of the beams (shear yielding preferred but combined shear and flexural yielding permitted).* • *P-M-M yielding at the base of columns (top of foundation or basement podiums)*
Buckling Restrained Steel Braced Frames	• *Unbonded brace cores (yielding in tension and compression)* • *P-M-M yielding at the base of columns (top of foundation or basement podiums)*
Special Steel-Plate Shear Walls	• *Shear yielding of web plates* • *Flexural yielding of Beam ends*
R/C Shear Walls and Composite Plate Shear Walls	• *P-M-M yielding at the base of the walls (top of foundation or basement podiums) and other clearly defined locations throughout the height of the wall.* • *Flexural yielding and/or shear yielding of link beams*
Foundations	• *Controlled rocking* • *Controlled settlement*

* yielding caused by combined axial force and uniaxial or biaxial flexure

3.4. MODELING REQUIREMENTS

3.4.1. Mathematical Model

Three-dimensional mathematical models of the physical structure shall represent the spatial distribution of the mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of the building's dynamic response. Structural models shall incorporate realistic estimates of stiffness and damping considering the anticipated levels of excitation and damage.

Expected material properties shall be used for all evaluations.

C.3.4.1. Three-dimensional mathematical models of the structure are required for all analyses and evaluations. Given the current state of modeling capabilities and available software systems, the representation of the actual three-dimensional behavior of tall buildings no longer needs to rely on approximate two-dimensional models. The accuracy obtained by using three-dimensional models substantially outweighs the advantage of the simplicity offered by two-dimensional models.

3.4.2. Modeling Floor Diaphragms

Floor diaphragms shall be modeled to accurately simulate the distribution of inertial forces to the vertical members of the seismic-force-resisting system as well as transfer forces acting between these members. In general, diaphragms may be modeled with finite elements using stiffness parameters based on the anticipated level of cracking in the concrete or concrete-filled steel deck floor system.

Diaphragm chord and drag forces shall be established in a manner consistent with the floor characteristics, geometry, and well-established principles of structural mechanics. Consider shear, axial, and bending stresses in diaphragms. The dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm at (a) diaphragm discontinuities, such as openings and re-entrant corners, and (b) around the podium level diaphragm and other levels where significant discontinuities exist in vertical elements of the seismic-force-resisting system shall be evaluated.

Where diaphragms are designed to remain essentially elastic, they may be modeled using elastic finite elements with the effective stiffness values specified in Table 3. The finite-element mesh shall be constructed with sufficient refinement to model the distribution of stresses within the diaphragm and transfers into chords, collectors, intersecting walls, and other members.

Diaphragms consisting of concrete slabs or concrete-filled metal decks may be modeled as rigid in-plane elements where:

1. There are no significant changes or discontinuities in the vertical elements of the gravity or seismic-force-resisting system above and below the diaphragm;
2. There are no re-entrant corners, large openings, or other horizontal irregularities in the diaphragm as defined in ASCE 7-22 Table 12.3.1; and,
3. The horizontal span-to-depth ratio of the diaphragm is less than 3.

Regardless of the relative rigidity or flexibility of floor diaphragms, flexibility of diaphragms with significant force transfer (e.g., podium levels and other setback levels) shall be explicitly included in the mathematical model.

C.3.4.2. Modeling of floor diaphragms as rigid in-plane elements may result in unrealistically large transfer forces at levels having significant discontinuities in vertical elements of the seismic-force-resisting system. Such levels include the podium, where shear forces from the superstructure transfer through the podium diaphragms to basement walls, and other setback levels. More realistic estimates of the transfer forces at such discontinuities can be obtained by modeling diaphragm flexibility at the level of the discontinuity and, perhaps, for a few levels above and below the discontinuity level.

To adequately model diaphragm flexibility, the finite-element mesh will typically need to have at least three to five elements within each bay, although a finer mesh may be needed in transfer regions with high stress gradients. Moehle et al. (2016) provides further information on diaphragm design and modeling.

C.3.4.2. (continued). At podium levels it is particularly important to model the interaction among stiff vertical elements, the diaphragms, and the basement walls. The so-called "backstay effect" can result in very large transfer forces and may produce a drastic change in the distribution of shear force and overturning moment below the podium-level diaphragm. The backstay effect will depend strongly on the in-plane stiffness and strength of the diaphragm and its supporting elements. Realizing that these stiffness values depend on the extent of cracking, and that such extent is difficult to accurately calculate, it may be necessary to make bounding assumptions on stiffness properties to envelope the forces for which the various components of the podium structure should be designed. Appendix A of ATC 72 (2010) and Moehle et al. (2016) provide further discussion and guidance on design and modeling considerations to address the backstay effect.

3.4.3. Modeling Seismic Mass and Torsion

The seismic mass shall be determined based on the expected seismic weight of the building, including the dead load, superimposed dead load, and storage live loads. The mass shall be distributed in plan to represent the translational and torsional inertial effects. Inherent eccentricities resulting from the distribution of mass and stiffness shall be included. Where vertical ground motions are included in the analysis, the vertical component of mass with sufficient horizontal distribution to compute the important vertical modes of response shall be included.

The seismic mass of the entire building shall be included in the model, including both the superstructure and below grade structure with the following exceptions:

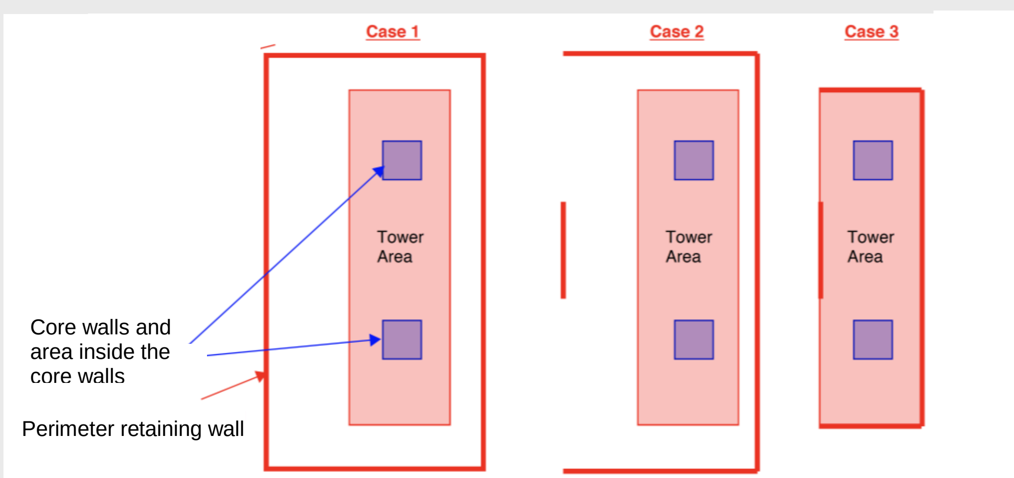
1. In response spectrum analyses, where inclusion of the below grade mass may overestimate floor accelerations if the lateral stiffness of the below grade soil may be ignored; and,

2. In response history analyses where it can be demonstrated that the inertial mass below grade will either (a) not exert forces on the structural components that are modeled in the analysis or (b) be incorporated through other means in determining required member forces that are consistent with the system behavior.

The mass of the ground floor shall be considered in analysis. Consideration of the mass corresponding to the footprint of the areas within core walls of the tower for subterranean floors which are surrounded by soil on all sides is an acceptable option.

The torsional amplification factor, A_x, shall be calculated per Provisions of Sections 12.8.4.2 and 12.8.4.3 of ASCE 7-22 using equivalent static lateral loads and documented during the SLE evaluation. For the MCE evaluation accidental torsion need only be considered if the value of A_x as calculated during the SLE evaluation exceeds 1.50 for any floor (see Section 3.5.4).

C.3.4.3. In general, there are four possible mechanisms to resist inertial forces of below grade mass: (1) passive soil bearing pressures on basement walls and mat foundation, (2) side friction on basement walls, (3) friction below the foundation, or (4) shear resistance of drilled shafts, piles or other deep foundation elements. The extent to which one or more of these will resist inertial loads depends primarily on the relative stiffness of each, which in turn depends on the site conditions, basement depth, and type of foundation (deep versus shallow). In cases with shallow foundations (e.g., mat foundations without piles), the primary resistance is likely to be due to friction below the foundation and side friction on basement walls. On the other hand, in cases with deep foundations, the deep foundations and side friction on the basement wall are likely to resist most of the inertial force from the superstructure and substructure. In any case, provisions should be made for resisting the inertial forces due to the below grade mass either in the nonlinear analysis model or through separate design checks. In the examples shown below, the mass corresponding to the area inside core walls may be used for Case 1 and mass corresponding to tower area may be used for Cases 2 and 3.



3.4.4. Equivalent Viscous Damping

A small amount of equivalent viscous damping may be included in both linear response spectrum analyses and in linear and nonlinear response history analyses to account for energy dissipation that is not otherwise represented by the analysis model. Unless evidence is provided to justify larger values, effective additional modal or viscous damping for the primary modes of response for SLE evaluation shall not exceed the fraction of critical damping given below:

$$\zeta_{critical} = 0.36 / \sqrt{H} \leq 0.05 \quad (1)$$

where H is the height of the roof, excluding mechanical penthouses, above the grade plane, in feet. Figure 1 plots the above equation for buildings of differing heights. For MCE analysis the same equation may be used but $z_{critical}$ need not be taken less than 0.025. Where viscous damping is explicitly modeled in the soil–foundation interface, an analysis of the total viscous damping shall be conducted to determine whether the equivalent viscous damping applied through modal or Rayleigh models should be reduced.

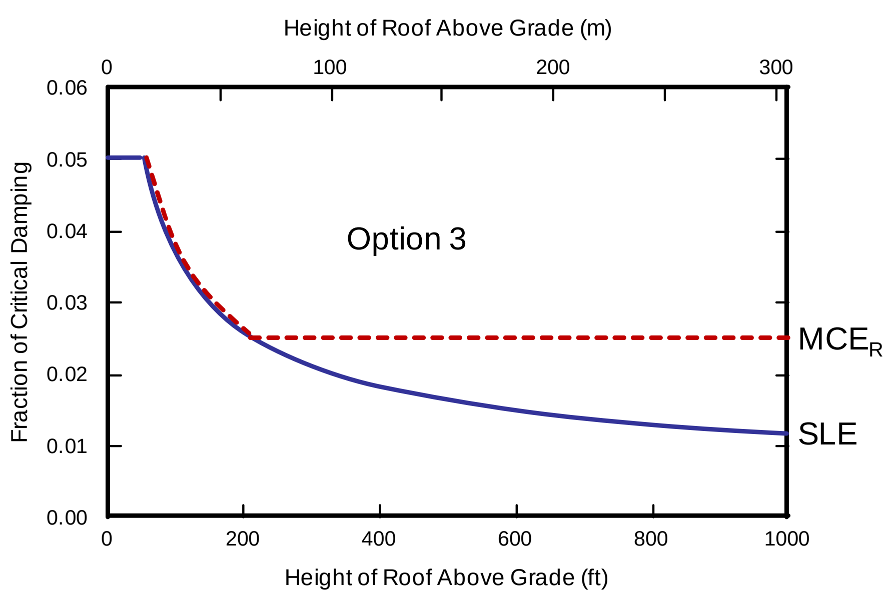


Figure 1. Equivalent viscous damping versus building height

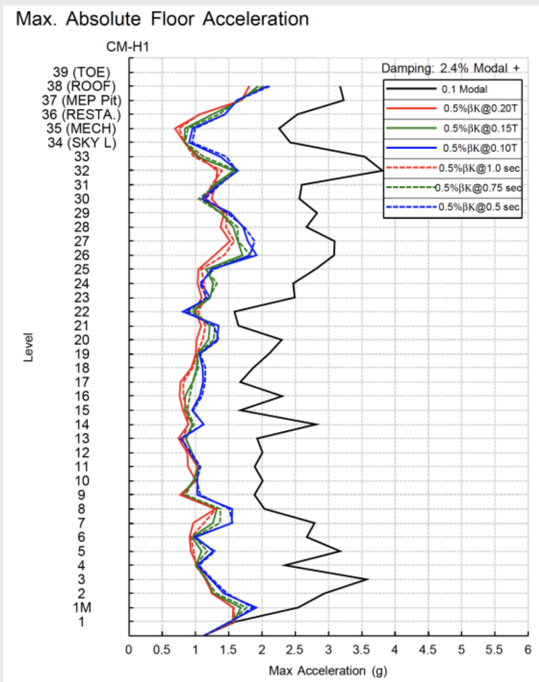
C.3.4.4. Damping effects of structural members, soil–foundation interaction, and nonstructural components that are not otherwise modeled in the analysis can be incorporated through equivalent viscous damping. The amount of viscous damping should be adjusted based on specific features of the building design. The equivalent viscous damping can be represented through modal damping, including the fundamental mode up through higher modes with periods greater than 0.2 times the fundamental period. Alternatively, mass and stiffness proportional Rayleigh damping may be used, where checks are made to ensure that modes of response significant to the calculated demand parameters are not overdamped. ATC 72 (2010). Both ATC 72 and more recent research publications (e.g., Cruz and Miranda, 2016; Bernal et al., 2015) provide evidence from measured building data that damping in tall buildings is less than that in low-rise buildings. Hence, as illustrated in Figure 1, viscous damping is limited. Reasons for the lower damping are mainly attributed to smaller relative damping contributions from foundations in tall buildings. If soil–foundation damping is modeled explicitly in the analysis, then low-amplitude shaking (below the elastic limit) can be applied to establish the total amount of viscous damping in the structure. The total amount of viscous damping should be evaluated based on measured damping in buildings (see ATC 72, 2010). Equation 1 is based on a study of a large number of buildings subjected to excitations that are generally below the SLE level excitations in California (Cruz and Miranda, 2016). Therefore, a floor of 0.025 is applied when damping at MCE_R level is considered.

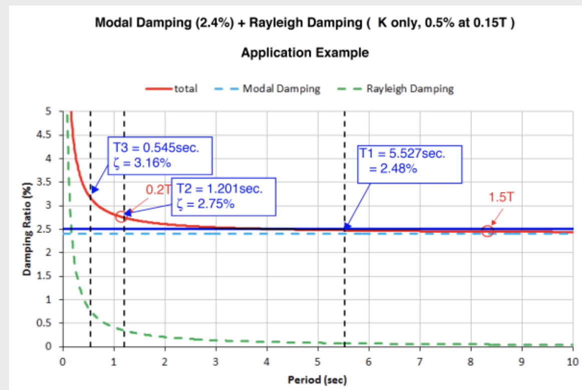
3.4.4.1 Modeling Viscous Damping for Nonlinear Analysis

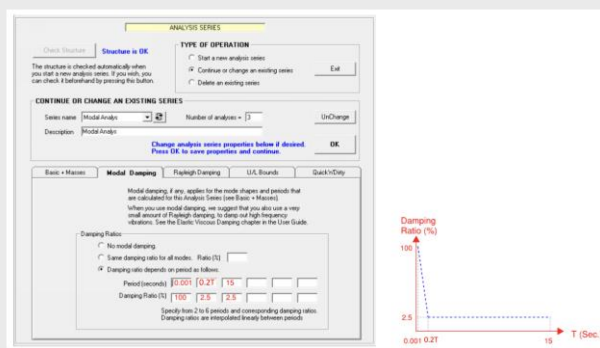
The following approaches to modeling viscous damping for nonlinear analysis are acceptable:

1. Use of modal damping values equal or less than the values specified in Section 3.4.4 for all modes considered.
2. Use of Rayleigh damping with anchor points selected such that during the entire period range of interest (0.2*T* to 2.0*T*) damping values are less than or equal to the values specified in Section 3.4.4.
3. Use of a combination of the same modal damping value for all periods longer than 0.2*T* and stiffness-proportioned only Rayleigh damping or linearly increasing damping from 0.2*T* to the period of 0.0 seconds in a way that in the entire period range of interest (0.2*T* to 2.0*T*) damping values are less than or equal to the values specified in Section 3.4.4. except for the periods in the range of 0.2*T* to 0.3*T*, the damping values may slightly exceed 2.5% (see the commentary below).

C.3.4.4.1. LATBSDC recent investigations of application of various viscous damping modeling approaches for nonlinear analysis has shown that use of the same small modal damping for all periods results in a serious overestimation of floor accelerations. Therefore, to alleviate this problem, Option 3 of this section is added to the list of available options for modeling viscous damping in nonlinear analysis.







3.4.5. P-Delta Effects

P-Delta effects shall be included in nonlinear analysis, regardless of whether elastic analysis design checks indicate that such effects are important. The P-Delta effects shall include the destabilizing gravity loads for the entire building, where the gravity loads are spatially distributed to capture both building translation and twist.

3.4.6. Vertical Ground Motion Effects

Explicit simulation of vertical earthquake response shall be performed where there are significant discontinuities in the vertical-load-carrying elements. In these cases, vertical masses (based on the effective seismic weight) shall be included with sufficient model discretization to represent the primary vertical modes of vibration in the analysis model used to simulate vertical response.

C.3.4.6. For most structural elements, the effect of vertical response is only of moderate influence, given that gravity-load-resisting elements have substantial reserve capacity associated with the dead- and live-load combinations specified by the building code. In typical cases, the effect of vertical response can be approximated through use of the terms $0.2S_{MS}D$ or $0.3S_{TV}D$. However, where significant discontinuities occur in the vertical-load-resisting system (e.g., where building columns supporting several stories and significant floor area terminate on transfer girders, or major load-bearing walls terminate on columns), vertical response can significantly amplify demands. Additionally, columns having significant inclinations can result in coupling between vertical and horizontal responses, leading to large increases in column axial forces. In such cases, these Guidelines recommend explicit simulation of vertical response. It is not the intent to require such analyses where relatively minor columns supporting only a few stories or where columns are sloped less than 8 degrees from vertical.

If site-specific vertical spectrum is developed based on the ratio of vertical to horizontal spectra (V/H), it is noted that in most V/H models "H" is RotD50. Also, in some provisions, there is somewhat conservative lower limit of 0.5 for V/H. This lower limit may be taken as 0.4 at long periods.

3.4.7. Foundation Modeling and Soil-Structure Interaction

3.4.7.1 Foundation Modeling

Modeling of foundation elements for performance based seismic design can be accomplished with two general approaches, illustrated in Figure 2 below: (1) an explicit modeling approach where the foundation system is included directly in the full building model; or (2) an uncoupled modeling approach where reactions from the superstructure analysis model are applied to a separate foundation model.

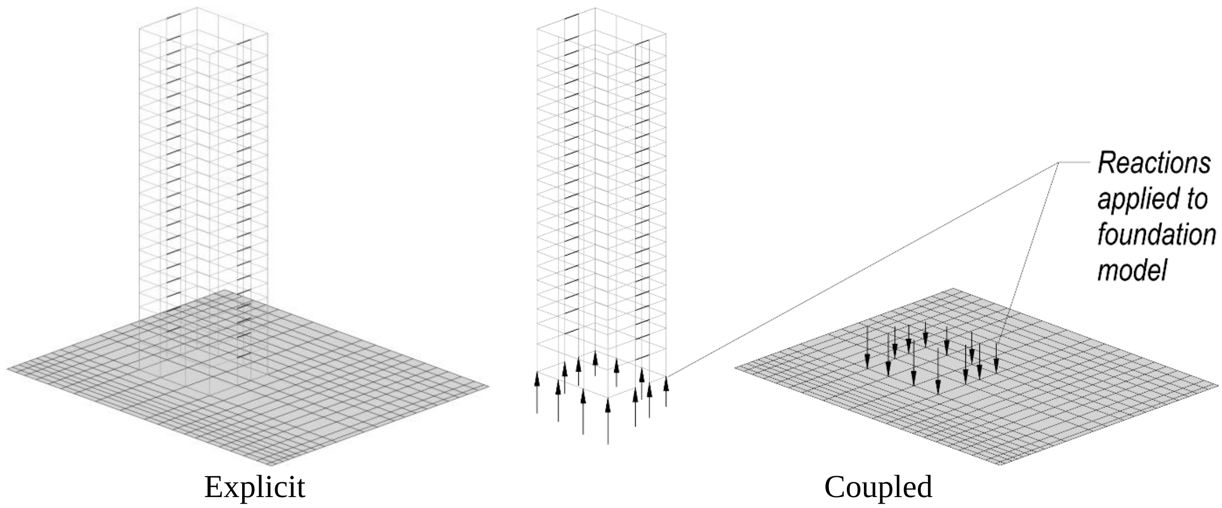


Figure 2. Illustrative examples of explicit and coupled foundation modeling approaches.

Explicit Approach: The explicit approach most accurately captures the effects of foundation flexibility on the superstructure but can add a significant computational burden to the nonlinear analysis model. Foundation elements can be modeled with varying degrees of complexity depending on the element and its action and criticality classifications. Demands on foundation elements can be directly evaluated at each step of the response history analysis.

Uncoupled Approach: The uncoupled approach requires a separate analysis model from the superstructure nonlinear analysis model, reducing computation time in the analysis model and allowing for foundation design iterations separate from the building analysis. Response history foundation demands can be simulated in the foundation model with static loads scaled to match the superstructure analysis model on a step-by-step basis. Alternatively, reactions from the superstructure analysis model can be enveloped and applied as a limited number of load cases in the foundation model. See Figure 3 for an illustration of one such enveloping procedure and C.3.4.7.1 for more details.

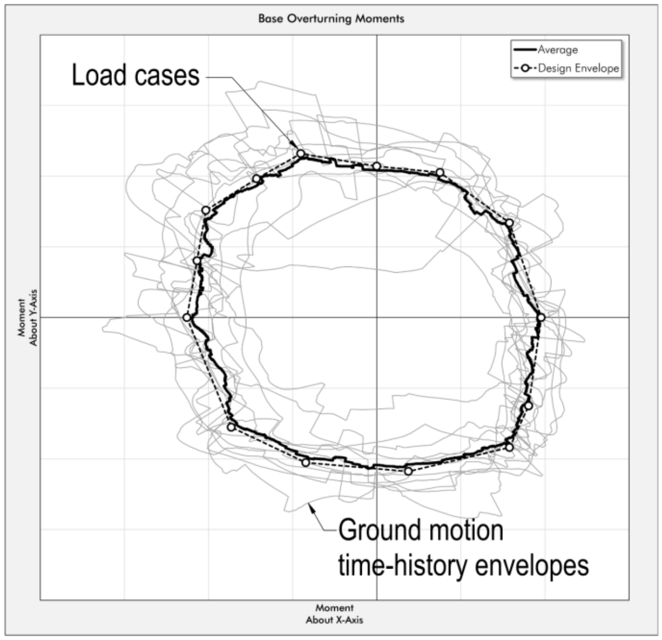


Figure 3. Example of superstructure analysis model reaction enveloping

Foundation elements shall be modeled consistent with their action classification and criticality defined by ACI 318-19 Appendix A. The following guidance is provided for each category of foundation elements:

Shallow Foundations: Mat foundations or spread footings may be modeled as elastic shell elements. Recommended effective stiffness values are provided in Table 3. Shallow foundations with demands exceeding the elastic range for flexure and axial load can be modeled using shell elements with nonlinear flexural behavior modeled.

Deep Foundations: Pile caps or mats supported by deep foundation elements may be modeled similar to shallow foundations. Deep foundation elements can be modeled as elastic frame elements. Where a plastic hinge is permitted to be formed in the deep foundation elements, nonlinear hinges akin to moment frame column hinges may be used. Alternatively, deep foundations may be modeled as discrete springs. The spring stiffness shall be calibrated to account for stiffness of both the deep foundation element and settlement of the surrounding soil.

Soil Resistance: Soil resistance may be modeled using distributed or discrete springs developed in coordination with the Geotechnical Engineer and having stiffness and strength consistent with the analysis demand level. Vertical distributed springs may be used to model continuous soil resistance associated with shallow foundation bearing behavior (Figure 4(a)), whereas discrete vertical springs may be used to model the region of the foundation spanning behavior between deep foundation elements (Figure 4(b)).

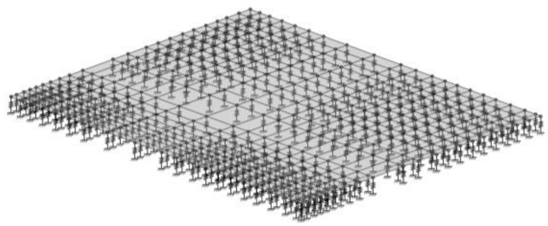
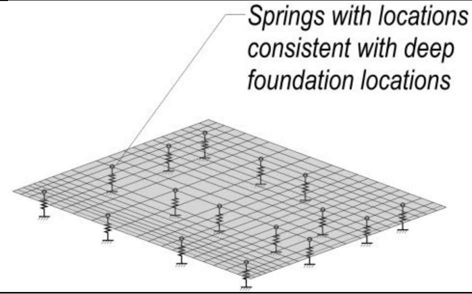
	
(a) Distributed	(b) Discrete

Figure 4. Distributed versus discrete vertical soil springs

C.3.4.7.1. The design of the foundation system shall be based on gravity demands, seismic demands, and any other applicable demands (e.g., unbalanced soil pressure, etc.) excreted on the foundation system. Seismic demands shall be calculated based on the average maximum demands obtained from nonlinear response history analyses (NLRHA) recorded at the base of all lateral load-resisting elements. Any seismic forces induced on the foundation through gravity elements (e.g., unintentional outrigger columns) shall also be considered.

The average of maximum seismic forces obtained from NLRHA shall be calculated for a number of loading directions to reasonably represent multidirectional demands on the foundation. A minimum of four loading directions shall be considered (i.e., positive and negative loading directions along two principal axes of the building). In this case, demands can be combined using an appropriate directional load combinations (e.g., 100%/30% rule). Directional load combinations can be also derived directly based on the selected loading directions.

Maximum seismic demand for a desired loading direction for an individual ground motion record shall be calculated as a point on a demand trace envelope (see Figure C.3.4.7.1.1 for an example). The validity of the assumed directional load combinations shall be verified with an orbit plot that represents the average of maximum seismic demands for all radial directions between 0 and 360 degrees (see Figure C.3.4.7.1.2 for an example).

A sufficient number of loading directions shall be considered to envelop reasonably well the maximum demands from the ground motion record. Any alternative method to calculate maximum demand for a single earthquake record and a given earthquake direction that will result in more conservative seismic demands (e.g., using the maximum projection of the demand to the desired loading direction) is acceptable.

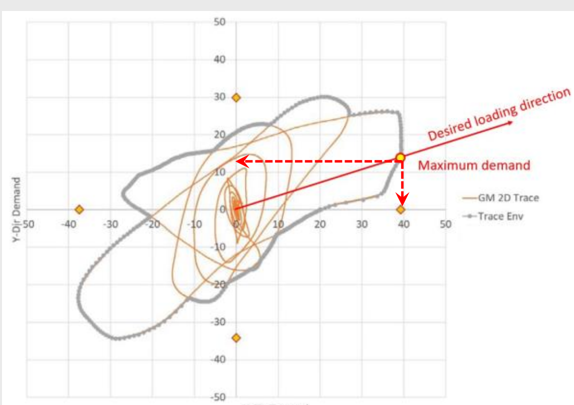


Figure C.3.4.7.1.1. Procedure to find maximum demands along a desired loading direction

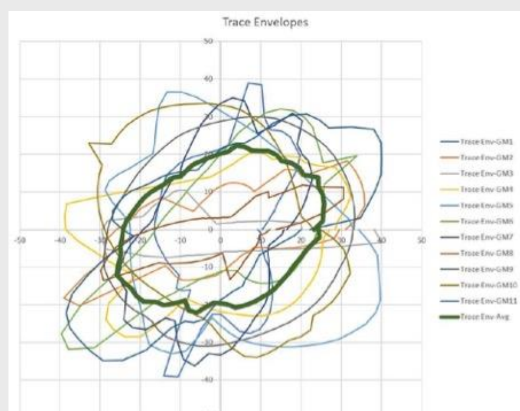


Figure C.3.4.7.1.2. Average orbit plot based on 11 ground motion records

3.4.7.2 Soil-Foundation-Structure Interaction

Explicit consideration of soil–structure interaction (SSI) effects in the structural model is optional. The simplified procedure explained in this section may be used to include subterranean levels in the structural models used for dynamic response analyses. In this approach (Figure 5 (b), soil springs need not be included in the model, but floor slab strength and stiffness characteristics shall be reasonably included.

The most rigorous approach considers spatially variable ground motions around the structure due to wave propagation effects as driving the foundation and structural response. Such approaches are too complex for most projects and are not shown in Figure 5. Figures 5(c)-(d) present two simplified approaches that represent many of the principal SSI effects while maintaining practicality given the capabilities of most current structural engineering analysis software. Figure 5(c) shows the "bathtub model," which includes elements to simulate soil–foundation interaction (shown with springs and dashpots) and allows for ground motion change from the free-field (u_g) to the foundation (u_{FIM}). The bathtub model introduces one simplification relative to the complete system, which is depth-invariant ground motions. Figure 5(d) shows an option in which soil–foundation interaction elements are included at the foundation level only, which avoids the use of input motions applied to the ends of interaction elements along basement walls.

The bathtub model (Figure 5c) is a simplified representation of the structure–foundation–soil system. It was developed to avoid the need for multi-support excitation in response history analyses that incorporate SSI (most structural engineering software for such analyses do not allow for multi-support excitation). The bathtub model has been shown to accurately simulate most structural responses relative to more complete modeling that includes multi-support excitation, the principal exception being subterranean parameters such as soil pressures (Naeim et al., 2008). An option that can be considered to overcome this problem is to use interaction elements at the foundation level only (Figure 5d). When interaction elements are used only at the foundation level, they should in aggregate reproduce the cumulative stiffness of the embedded foundation as given in NIST (2012) or similar documents (this stiffness is higher than that of the base slab alone as a result of embedment effects). Application of any of the three simplified options indicated in Figure 5b, 52c, or 5d is acceptable if their suitability for the project is sufficiently demonstrated.

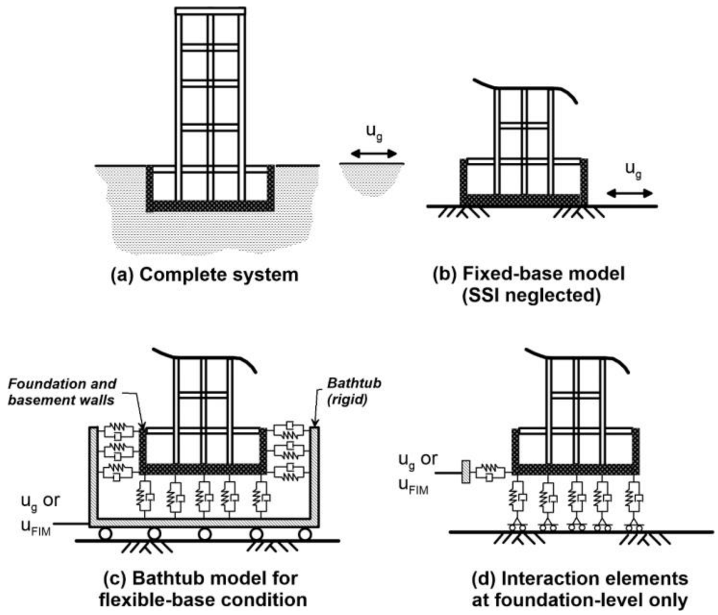


Figure 5. Schematic illustration of alternative models of buildings with basements.

Motion shall be applied at the base of the structure and can consist either of free-field motion (u_g) or the foundation input motion (u_{FIM}), which is modified for kinematic interaction effects.

C.3.4.7.2. The simplified approaches presented in Figure 5b and 5d can be readily adopted in structural analysis and design practice. More complicated methods may require substantially more effort and still may not necessarily result in more accurate results as shown by Naeim et al (2010). In the short-term future advances in practical computing software are anticipated which would make more sophisticated and realistic modeling of soil-foundation-structure-interaction more useable in a design office environment.

Special care should be afforded before the simplified approach presented here is used in special circumstances. For example, application of this approach to buildings where expansion joints run all the way down to the foundation is not justified and should be avoided.

3.4.8. Modeling Subterranean Components

For all cases, structural analysis model shall be extended down to the base of the structure, as shown in Figures 5a-d. Subterranean levels in the structural model used for dynamic response analysis, shall include appropriate mass, stiffness, and strengths of the subterranean structural members including walls, columns, and slabs.

If SSI effects are to be considered, flexibility and (if desired,) damping at the soil–foundation interface either using soil springs, springs combined with dashpots, or a continuum (finite-element or finite-difference) model of the soil-foundation-structure system shall be explicitly modeled. If a springs-only approach is used, the beneficial effects of radiation damping are not included in the model.

If SSI is considered (Figures 5-d), seismic excitation shall be applied using the foundation input motion (u_{FIM}), which is modified for kinematic interaction effects. As an alternative, the free-field motion (u_g) may be used. As recommended in ASCE 7-22 Chapter 19, consideration of reduced ground motions from embedment and base-slab averaging is not permitted without also considering the flexibility and damping at the foundation-soil interface.

3.4.9. Backstay Effects

Where applicable, for MCE_R evaluations, two sets of analyses shall be conducted to evaluate backstay effects:

1. A model which uses upper-bound (UB) stiffness assumptions for floor diaphragms at the podium and below. Drift evaluation is not required.

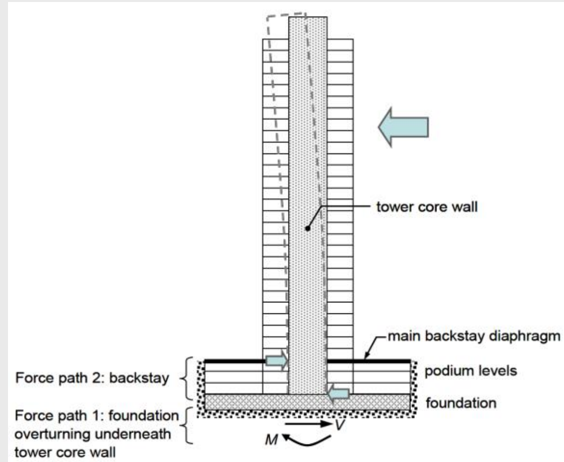
2. A model which uses lower-bound (LB) stiffness assumptions for floor diaphragms at the podium and below. Drift evaluation is not required.

These backstay LB and UB analyses results are to be used for the design of the transfer diaphragms, shear walls and basement walls at or below the transfer levels, and the foundation. Table 5 contains recommendations for UB and LB Stiffness parameters for backstay sensitivity analyses. The sensitivity analyses, where applicable, shall be performed in addition to the analyses performed using stiffness properties provided in Table 3.

Table 5. Stiffness parameters for Upper Bound and Lower Bound Models

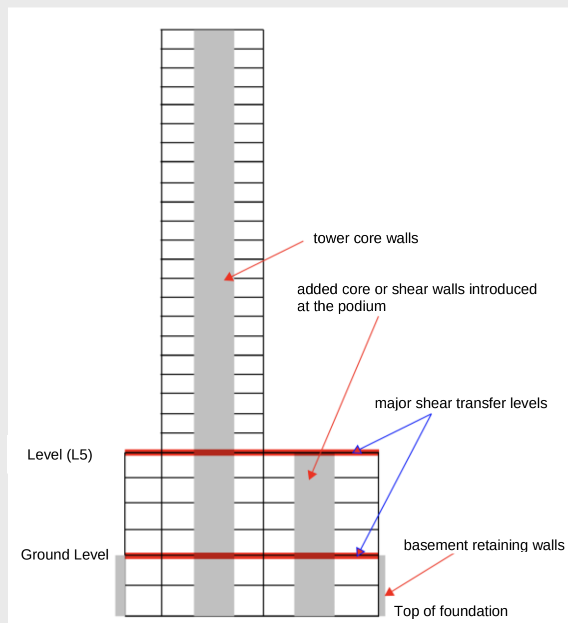
Stiffness Parameters	UB	LB
R/C Diaphragms at the podium and below		
$E_c I_g$	0.35	0.10
$G_c A$	0.35	0.10
PT Diaphragms at the podium and below		
$E_c I_g$	0.60	0.20
$G_c A$	0.60	0.20

C.3.4.9. Any lower part of a tall building structure that is larger in floor plate, and contains substantially increased seismic-force resistance in comparison to the tower above, can be considered a *podium*. Backstay effects are the transfer of lateral forces from the seismic force resisting elements in the tower into additional elements that exist within the podium, typically through one or more floor diaphragms. The lateral force resistance in the podium levels, and force transfer through floor diaphragms at these levels, helps a tall building resist seismic overturning forces.



(Illustration from ATC-72)

This component of overturning resistance is referred to as the backstay effect, based on its similarity to the backspan of a cantilever beam. It is also sometimes called "shear reversal" because the shear in the seismic force resisting elements can change direction within the podium levels. Since the stiffness properties of the elements, particularly diaphragms, are both influential on the seismic design and uncertain, a sensitivity analysis is required. The UB analysis provides an upper-bound estimate of forces in the backstay load path and a lower bound estimate of forces in the foundation below the tower. This case will govern the design forces for the podium floor diaphragms and perimeter walls, and the associated connections.



Example Configuration

The LB analysis provides a lower-bound estimate of forces in the backstay load path and an upper-bound estimate of forces in the foundation below the tower. This case will govern the design forces for the tower foundation elements. In the example configuration shown above, the sensitivity analyses using UB and LB stiffness parameters should be applied to Level (L5) and below.

3.4.10. Beam-column Joints

Modeling of joints in moment-resisting frames shall account for the flexibility of the joint, including the panel zone. In lieu of explicit modeling of beam-column panel zone behavior, center-to-center beam dimensions may be used.

C.3.4.10. Additional guidance as to appropriate stiffness assumptions for concrete and steel framing may be derived from appropriate test data or found in Moehle et al. (2008) and Hamburger et al. (2009), respectively. Additional guidance for concrete frames is provided in Elwood et al. (2007) and Elwood and Eberhard (2009).

3.4.11. Component Analytical Models

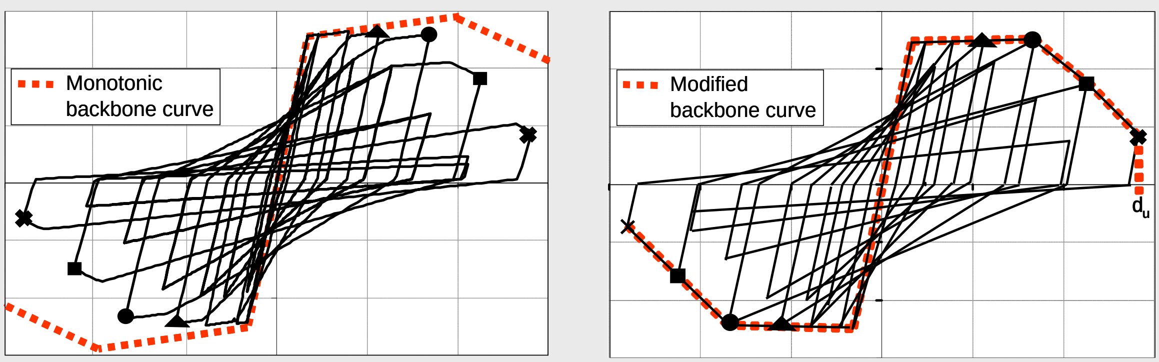
When applicable, the ASCE 41 component force versus deformation curves may be used as modified backbone curves, with the exception that the drop in the resistance following the point of peak strength need not be as rapid as indicated in some ASCE 41 curves. Alternatively, the modeling options presented in ATC (2010) may be employed.

C.3.4.11. The rapid post-peak drop in resistance indicated in the ASCE-41 curves is not realistic (unless fracture occurs) and is likely to cause numerical instabilities in the analysis process. Section 2.2.5 of ATC (2010) proposes four options for component analytical models. In this commentary two of these options which are considered more appropriate are discussed.

<u>Option 1 – explicit incorporation of cyclic deterioration in analytical model.</u> This option explicitly incorporates post-capping strength deterioration and cyclic deterioration in the analytical model, by using the monotonic backbone curve as a reference boundary surface that moves "inward" (towards the origin) as a function of the loading history. This option is more rational, and potentially more accurate. However, at this time, such modeling options are not commonly available in commercially available computer programs used for analysis and design of buildings.

<u>Option 2 – use of a cyclic envelope curve as a modified backbone curve; cyclic deterioration is not considered explicitly.</u> If the cyclic envelope curve is known (e.g., from a cyclic test that follows a generally accepted loading protocol) then this envelope curve may be used as the modified backbone curve for analytical modeling and ignore additional cyclic deterioration - provided that no credit is given in the analysis to undefined strength characteristics beyond the bounds established by the cyclic envelope curve, i.e., the ultimate deformation δ_u in any analysis should be limited to the maximum deformation recorded in the cyclic test. Modeling parameters in ASCE 41 were determined using this option. When using this approximation, the negative tangent stiffness portion of the backbone curve must be included except in cases where no component deforms beyond the point where degradation begins.

Figure C.3.4.11 illustrates the two options discussed above.



(a) Option 1 – with cyclic deterioration

(b) Option 2 – modified backbone curve = envelope curve

Figure C.3.4.11. Illustration of implementation of two options for analytical component modeling (Courtesy of Helmut Krawinkler).

3.4.12. Column Bases

Realistic assumptions shall be used to represent the fixity of column bases. A column base may be considered fixed if the column base connection to the foundation is capable of transferring columns forces and deformations to the foundation with negligible joint rotation, considering the flexibility of the foundation itself.

3.4.13. Response Modification Devices

Response modification devices (such as seismic isolation, damping, and energy dissipation devices) shall be modeled based on data from laboratory tests representing the severe conditions anticipated in Maximum Considered Earthquake shaking. If the properties of these devices vary significantly, the structure response simulations shall use alternative models incorporating upper and lower bound properties. If the devices have a functional limit beyond which the devices cease to operate (for example, a displacement limit), this functional limit must be represented in the analytical model. The consequences of attaining this limit must be demonstrated to be tolerable to the structure, or the functional limit will not be attained under 1.5 times the mean demand obtained from Maximum Considered Earthquake response analysis.

3.4.14. Flexural Behavior of Concrete Elements using Fiber Models

Concrete stress-strain behavior for members modeled using fiber-element sections shall comply with ASCE 41backbone curves or shall be based on suitable laboratory test data. Approximations fitted to analytical curves defined by Collins and Mitchell (1997), and adjustments made to allow for confinement effects as described by Mander et al. (1988) and Saatcioglu and Razvi (1992) are acceptable (see Figure 6). High-strength concrete may have stress-strain relationships that are different from those for regular strength concrete.

Reasonable bilinear approximation of steel stress-strain curve is acceptable (see Figure 7).

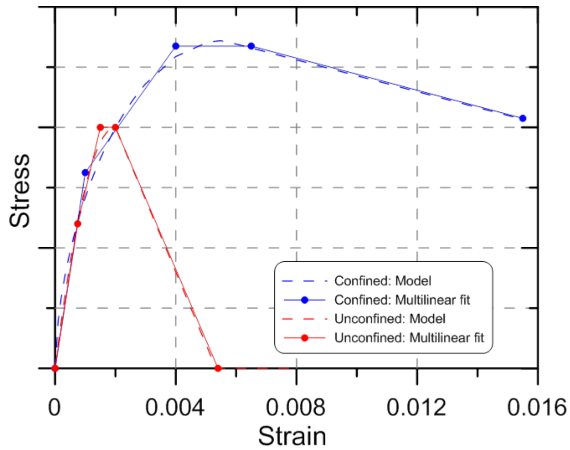


Figure 6. Examples of acceptable stress-strain models for concrete

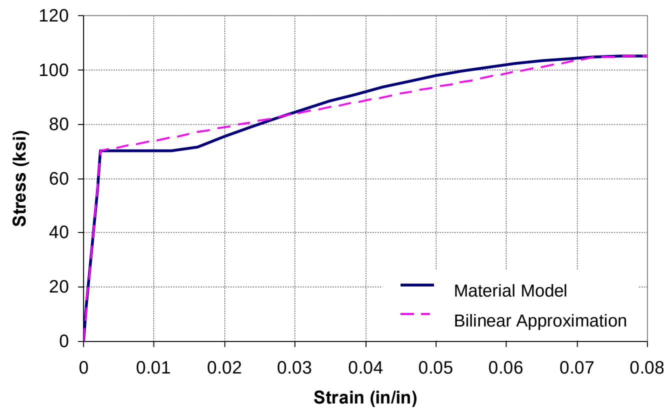


Figure 7. Example of an acceptable bilinear approximation of expected reinforcing steel stress strain curve

The effective plastic hinge length shall be used to monitor the compressive strain and ascertain the maximum dimensions of the beam, column, and wall elements in the analytical model.

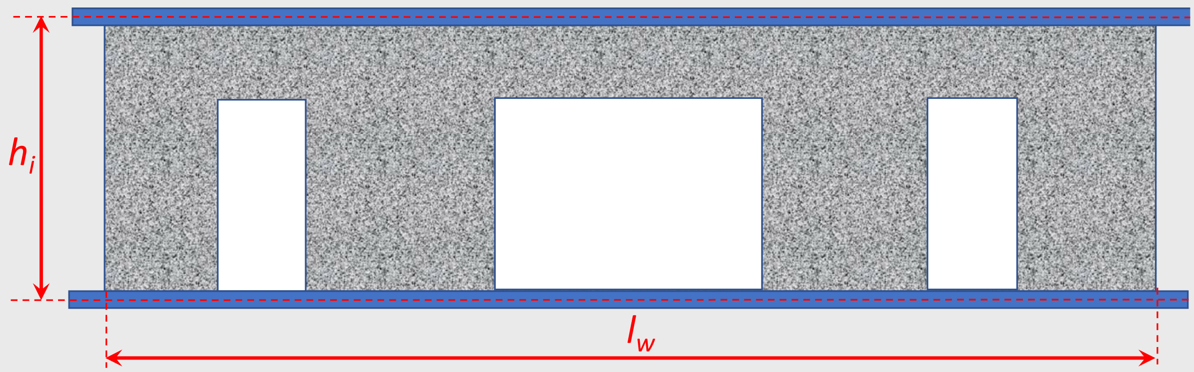
For beam or column elements where fiber-type models are used, plastic hinge length shall be 0.5h to 1.0h, depending on the location of yielding, where h is the member total depth. In cases where a concentrated plastic hinge is used, it should be located at the beam-joint or column-joint interface.

For walls, the maximum vertical dimension of the fiber elements in the regions that plastic deformations may occur (l_p) in nonlinear analytical models may be taken as:

$$l_p \leq \text{minimum of} \left\{ \begin{array}{c} 0.5 l_w \\ h_i \\ 0.1 h_n \end{array} \right\} \qquad (2)$$

where l_w is the wall horizontal length in the direction under consideration, h_i is the story height, and h_n is the total height of the building from the shear base to the roof.

C.3.4.14. The value of l_p obtained from Equation (2) is intended to guide the analytical modeling of fiber elements and assignment of proper length for strain gauges so that the strains are not improperly averaged between zones that may experience plastic deformations with zones that remain elastic. The definitions of l_w and h_i in Equation (2) are illustrated in the figure below.



3.5. SERVICEABILITY EVALUATION

3.5.1. General

The purpose of this evaluation is to demonstrate that the building's structural systems and nonstructural components and attachments will retain their general functionality during and after such an event. Repairs, if necessary, are expected to be minor and could be performed without substantially affecting the normal use and functionality of the building.

C.3.5.1. The intent of this evaluation is not to require that a structure remain within the idealized elastic behavior range if subjected to a serviceability level of ground motion. Minor post-yield deformations of ductile elements are allowed provided such behavior does not suggest appreciable permanent deformation in the elements, or damage that will require more than minor repair.

In typical cases a linear response spectrum analysis may be utilized, with appropriate stiffness and damping, and with the earthquake demands represented by a linear response spectrum corresponding to the serviceability ground motion. Where dynamic response analysis is used, the selection and scaling of ground motion time series should comply with the requirements of Section 3.2 of this document.

3.5.2. Service Level Design Earthquake

The service level design earthquake shall be taken as an event having a 50% probability of being exceeded in 30 years (43-year return period). The Service Level Design Earthquake is defined in the form of a site-specific, linear, uniform hazard acceleration response spectrum with the damping level determined using Equation (1).

3.5.3. Description of Analysis Procedure

Either linear response spectrum analyses or nonlinear dynamic response analyses may be utilized for serviceability evaluations. The analysis shall account for P-Δ effects. Effects of inherent and accidental torsion are considered to establish whether accidental torsion needs to be included in the Collapse Prevention evaluation (see Section 3.6). The structure shall be evaluated for the following load combinations:

(a) Response Spectrum Analysis

$$1.0D + L_{\mathrm{exp}} \pm 1.0E_{\mathrm{x}} \pm 0.3E_{\mathrm{y}}$$

$$1.0D + L_{\mathrm{exp}} \pm 1.0E_{\mathrm{y}} \pm 0.3E_{\mathrm{x}}$$

(b) Nonlinear Dynamic Response Analysis

$$1.0D + L_{exp} + 1.0E$$

where D is the dead load and L_{exp} is the expected live load. L_{exp} may be taken as 25% of the unreduced live load unless otherwise substantiated and shall be included in all gravity calculations and P-Δ analyses.

C.3.5.3. Building Code response modification factors do not apply (that is, R, Ω_0, ρ, and C_d, are all taken as unity).

3.5.3.1. Elastic Response Spectrum Analyses

At least 90 percent of the participating mass of the structure shall be included in the calculation of response for each principal horizontal direction. Modal responses shall be combined using the Complete Quadratic Combination (CQC) method.

The corresponding response parameters, including forces, moments and displacements, shall be denoted as Elastic Response Parameters (ERP) and shall not be reduced.

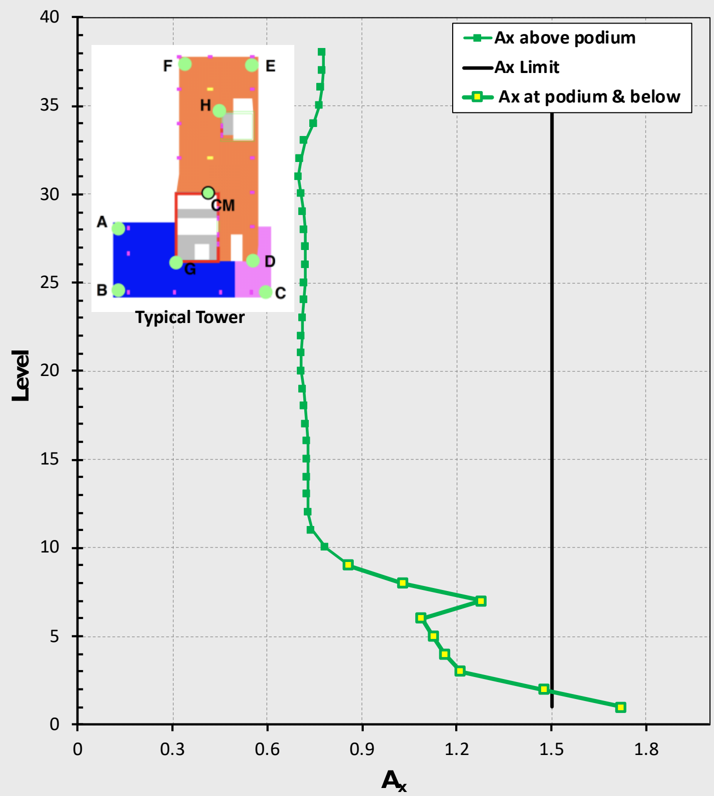
3.5.3.2. Nonlinear Dynamic Response Analyses

The mathematical model used for serviceability evaluation shall be the same mathematical model utilized for collapse prevention evaluation under MCE_R ground motions with the exception that different stiffness modifiers for serviceability and MCE_R models may be used per Table 3.

3.5.4. Evaluation of Effects of Accidental Torsion

Accidental eccentricities need not be considered for serviceability design. However, the torsional amplification factor, A_x, as defined in Section 12.8.4.3 of ASCE 7-22 shall be calculated for each floor, x, during serviceability evaluations using equivalent static lateral force procedure and accidental torsion as defined in Section 12.8.4.2 of ASCE 7-22. If the value of A_x exceeds 1.50 for any floor where the maximum average MCE_R drift ratio at any location of the floor exceeds 0.010, then accidental eccentricity shall be considered during MCE_R evaluations.

C.3.5.4. Large values of A_x are inconsequential if the maximum story drifts at all locations on the floor are very small. Value of A_x shall be calculated for each floor in each direction. If the maximum value of A_x exceeds 1.50 for any floor, then the maximum average MCE_R drift ratio at location of maximum drift ratio for the floor shall be compared to the limit of 0.010. For the example building shown below, the maximum story drift ratios occur in the *x* direction and the maximum value of A_x only at Level 1 exceeds 1.50 and is equal to 1.73 at points A and C. If the maximum average of MCE_R drift ratios at points A and C is larger than 0.010, then accidental torsion must be addressed in MCE_R evaluations. Otherwise, accidental torsion may be ignored. When consideration of accidental torsion is necessary, it may be applied as static moments at each floor.



3.5.5. Acceptability Criteria

Regardless of the analysis method used, maximum story drift at any location of the floor along either of the two principal axes of the building shall not exceed 0.5% of story height in any story.

3.5.5.1. Elastic Response Spectrum Analyses

The structure shall be deemed to have satisfied the acceptability criteria if none of the elastic demand to capacity ratios (ratio of ERP to the applicable expected strength limits for steel members or expected strength limits for concrete members using $\phi = 1.0$) exceed:

a) 1.50 for deformation-controlled actions for Risk Category I and II Buildings (ASCE 7-22 Table 1.5-1); 1.20 for deformation-controlled actions for Risk Category III Buildings; and a factor smaller than 1.20 as determined by the SPRP (see Section 4) for Risk Category IV Buildings.

b) 0.70 for force-controlled actions.

3.5.5.2. Linear or Nonlinear Dynamic Response Analyses

A minimum of three pairs of ground motion time series scaled per provisions of Section 3 shall be utilized (eleven or more pairs are recommended). Ground motion time series shall be scaled to the serviceability design spectrum with a damping level specified in Eq. (1). If less than 11 pairs are used the maximum response values shall be used for evaluation, otherwise, the average of the maximum values may be used.

The requirements of Section 3.5.5.1 (a) and (b) apply for linear analysis and the requirements of Section 3.5.5.1 (b) apply for nonlinear analysis for force-controlled actions. For nonlinear analysis, for deformation-controlled actions, deformation demands shall not exceed a value at which sustained damage requires repair, either due to strength deterioration or permanent (residual) deformation, as demonstrated by appropriate laboratory testing. Repair, if required, generally should not require removal and replacement of structural concrete, other than cover, or removal or replacement of reinforcing steel or structural steel. In lieu of the use of laboratory test data, the acceptance criteria for Immediate Occupancy performance as contained in ASCE 41 may be utilized.

3.6. MCE$_R$ EVALUATION

3.6.1. General

Three-dimensional nonlinear dynamic response analyses are required. P-Delta effects shall be included in all dynamic response analyses. P-Delta effects that include all the building dead load plus expected live load shall be included explicitly in the nonlinear dynamic response analyses. Expected live load need not be considered in building mass calculations. The structure shall be analyzed for the following load condition:

$$1.0D + L_{exp} + 1.0E_x + 1.0E_y$$

In addition to the designated elements and components of the lateral force resisting system, all other elements and components that in combination significantly contribute to or affect the total or local stiffness of the building shall be included in the mathematical model.

Expected material properties shall be used throughout.

All structural elements for which demands for any of the nonlinear dynamic response analyses are within a range for which significant strength degradation could occur, shall be identified and the corresponding effects appropriately considered in the dynamic analysis.

Ramps above the ground level shall be explicitly included in the nonlinear model of the building.

Coupling between the core walls and gravity columns, or between two columns, shall be explicitly modeled in the nonlinear analysis according to the provisions of Appendix C of this document.

3.6.2. Accidental Torsion

If serviceability evaluation indicates that accidental torsion must be included (see Section 3.5.4), a pair of ground motion time series that results in above mean demand values on critical actions shall be selected and substantiated. This pair shall be applied once with centers of mass at the original locations and once at locations corresponding to a minimum accidental eccentricity in one or both horizontal directions, or in the direction that amplifies the building's natural tendency to rotate.

The ratio of maximum demands computed from the model with accidental eccentricity over the maximum demands computed from the model without accidental eccentricity shall be noted for various actions. If this ratio (γ) exceeds 1.20, the permissible force and deformation limits for corresponding actions shall be divided by the corresponding (γ) value.

Alternatively, all ground motion time series may be included in the analyses with the minimum eccentricity (in addition to the original analyses) without changing permitted capacities.

3.6.2.1. Sensitivity Analyses

In lieu of accidental torsion analysis of Section 3.6.2 or as an additional measure, a program of sensitivity analyses may be utilized by varying material properties and/or configurations at various locations of the building to demonstrate the vitality of the building.

> **C.3.6.1.1.** The implemented procedure flags importance or insignificance of accidental eccentricity issue during the less cumbersome, serviceability evaluation. If during the serviceability evaluation, accidental eccentricities are established to be significant, then the accidental eccentricities must be included in collapse prevention evaluations. Even then, a set of sensitivity analyses may be performed in lieu of considering the traditional notion of accidental eccentricities.

3.6.3. Acceptance Criteria

3.6.3.1 Global Acceptance Criteria

Global acceptance criteria include the validity of the response, peak transient and residual story drifts, and loss of story strength.

(a) Unacceptable Response

Unacceptable response to ground motion shall not be permitted. Unacceptable response to ground motion shall consist of any of the following:

1. Analytical solution fails to converge,
2. Predicted demands on deformation-controlled elements exceed the valid range of modeling, or
3. Predicted deformation demands on elements not explicitly modeled exceed the deformation limits at which the members are no longer able to carry their gravity loads.

C.3.6.3.1(a). ASCE 7-16 and ASCE 7-22 Chapter 16 permit not more than one unacceptable response in a suite of eleven ground motions for Risk Category II buildings and no unacceptable response for buildings assigned to higher risk categories. PEER-TBI Guideline (PEER, 2016) adopts ASCE 7-16 provisions with an exception permitting one unacceptable response if a suite of not less than 20 ground motions is used for Risk Category III structures.

This document does not permit any unacceptable response. It should be noted, however, that the definition of unacceptable response in this document is different from that of PEER-TBI Guideline. Per common interpretation of PEER-TBI, the acceptability criteria in that document are evaluated one ground motion pair at a time. In contrast, most of the acceptability criteria for this document are evaluated either for maximum average of results obtained from all ground motions or a factor (i.e., 1.3 or 1.5) times the maximum average values. For Example, if demands on a critical or ordinary force-controlled element exceeds the element capacity in response to a single ground motion pair, then according to PEER-TBI, an unacceptable response is obtained. According to the provisions of this document, however, it is acceptable to exceed this capacity (without the amplification factor of 1.3 or 1.5) in response to one or two ground motion pairs if the acceptance criteria as described in Section 3.6.3.2 as applied to maximum average of the responses obtained from all ground motions are satisfied. In this document, only global acceptability criteria such as peak transient drift and peak residual drift have limits imposed on the results obtained from a single ground motion pair result.

The valid range of modeling for deformation-controlled elements is defined in ACI 318-19 Appendix A as D_u; in this document, this term is referred to as D_{u_VRM}. Acceptance criteria in this document are typically associated with lateral strength loss (D_{u_LSL}), e.g., for diagonally reinforced coupling beams, the rotation value of $0.06/I_e$ in Table 6-2 is based on the rotation at lateral strength loss based on the tests reported by Naish et al. (2013). In this document, D_{u_LSL} is compared against the average demands from the suite of ground motions to assess acceptance. Although this limit may be exceeded in one or two ground motions, in any ground motion where the acceptance criterion associated with D_{u_LSL} is exceeded, strength loss must be included in the model and the maximum rotation value must not exceed the rotation value associated with D_{u_VRM}.

(b) Peak Transient Drift

In each story, the mean of the absolute values of the peak transient drift ratios from the suite of analyses, regardless of the building Risk Category, shall not exceed 0.030. In each story, the absolute value of the maximum story drift ratio from the suite of analyses, regardless of the building Risk Category, shall not exceed 0.045.

Drifts shall be assessed within the plane of the seismic-force-resisting element or gravity-framing element being evaluated. For structural systems without primary planes, the principal axes shall be determined for the overall structural system or an alternate assessment method. Cladding systems, including the cladding and cladding connections to the structure, shall be capable of accommodating the mean of the absolute values of the peak transient story drifts in each story.

C.3.6.3.1(b). The use of a story drift limit ratio of 0.030 has resulted in efficient designs that have been judged effective by review panels in recent tall building projects. There is general consensus that, up to this story drift, structures with proper yielding mechanisms and good detailing will perform well (without significant loss of strength), and that properly attached nonstructural components will not pose a major life safety hazard. The drift limit should be applied to the "total" story drift (caused by story shear and story flexural rotation) because it is intended to protect all components of the structure including the gravity system components that are surrounding shear walls or braced frames and are subjected mostly to a story shear (racking) mode of deformations. A story drift ratio limit of 0.030 also provides P-Δ control in stories with large vertical loads.

The 0.045 story drift ratio limit is imposed to ensure that the variations in ground motions do not produce results that may invalidate the performance objectives of this document.

Exceeding the 0.045 drift ratio limit for one ground motion pair out of each 11 pairs only, may be justified for cases where exceedance is very limited or local in nature or when a new structural system with larger drift capacity is introduced. In such cases the engineer of record must clearly state and substantiate the justification for exceeding this limit.

(c) Residual Drift

In each story, the mean of the absolute values of residual drift ratios from the suite of analyses, regardless of the building Risk Category, shall not exceed 0.010. In each story, the maximum residual story drift ratio in any analysis, regardless of the building Risk Category, shall not exceed 0.015 unless proper justification is provided.

C.3.6.3.1(c). The residual story drift ratio of 0.010 is intended to protect against excessive post-earthquake deformations that likely will cause condemnation or excessive downtime for a building. This criterion is added to provide enhanced performance for tall buildings. The limits on residual drifts also are based on concern that tall buildings with large residual drifts may pose substantial hazards to surrounding construction in the event of strong aftershocks. Repair or demolition of tall buildings with large residual drifts also may pose community risks. In each case, these limits are to be evaluated against the maximum responses calculated in any of the response histories. Larger residual drifts may be acceptable if the large residual is due to peculiarities in the ground motion characterization, that may not be fully appropriate, or it can be demonstrated that the response is reliably simulated and acceptable, even given the large residual drifts.

3.6.3.2. Acceptance Criteria at the Component Level

3.6.3.2.1 Force-Controlled Actions

(a) Critical Actions Not Sensitive to Vertical Accelerations

Critical force-controlled actions not sensitive to vertical accelerations shall satisfy either Equation (5a) or Equation (5b):

$$1.0Q_{ns} + 1.3I_e\left(Q_T - Q_{ns}\right) \le \phi_s BR_n \qquad \text{(5a)}^1$$

$$1.0Q_{ns} + 1.5I_e\left(Q_T - Q_{ns}\right) \le \phi_s BR_{nem} \qquad \text{(5b)}^1$$

(b) Critical Actions Sensitive to Vertical Accelerations

Critical force-controlled actions sensitive to vertical accelerations shall satisfy either Equations (5c) **and** (5d), **or** Equations (5e) **and** 5(f):

$$\left\{\begin{aligned} (1.2 + S_{VA})D + 1.0L_{exp} + 1.3I_e(Q_T - Q_{ns}) &\le \phi_s BR_n \qquad \text{(5c)} \\ (0.9 - S_{VA})D + 1.3I_e(Q_T - Q_{ns}) &\le \phi s BR_n \qquad \text{(5d)} \end{aligned}\right.$$

$$\left\{\begin{aligned} (1.2 + S_{VA})D + 1.0L_{exp} + 1.5I_e(Q_T - Q_{ns}) &\le \phi_s BR_{nem} \qquad \text{(5e)} \\ (0.9 - S_{VA})D + 1.5I_e(Q_T - Q_{ns}) &\le \phi s BR_{nem} \qquad \text{(5f)} \end{aligned}\right.$$

[1] For in-plane flexure of transfer diaphragms , the factors 1.3 and 1.5 in equations 5a to 5f may be replaced with 1.10 and 1.25, respectively. This is because although "Flexural" action in a transfer diaphragm is classified as a "Critical" action, it is important to ensure that the designed transfer slab has a flexural limit state (as opposed to shear limit state).

EXCEPTION: When explicit vertical response analysis is performed, Equations (5a) or (5b) may be used instead of Equations (5c)-5(d) or (5e)-5(f).

where:

I_e	is the seismic importance factor appropriate to the Risk Category as defined in ASCE 7,
S_{VA}	Vertical acceleration effect which may be taken as either $0.2S_{MS}$ or $0.3S_{TV}$
Q_T	is the mean of the maximum values of the action calculated for each ground motion,
Q_{ns}	is the non-seismic portion of Q_T determined using appropriate load factors,
B	is a factor to account for conservatism in nominal resistance R_n, normally taken as having a value of 1.0. Alternatively, it can be taken as $B = 0.9(R_{ne} / R_{nem})$ for Eqs. 5. B values are listed in Appendix B.
R_n	is the nominal strength for the action, determined in accordance with the applicable material standard,
ϕ_s	is the resistance factor defined in Table 6-1,
R_{nem}	is the nominal strength for the action, determined in accordance with the applicable material standard using expected material properties,
R_{ne}	is the expected value of component resistance determined from test results using expected material properties as provided in Appendix A.

Table 6-1 Seismic resistance factors, ϕ_s.

Action Type	ϕ_s
Critical force-controlled element	ϕ as specified in the applicable material standard (ACI 318, AISC 360, AISC 341, AISC 358)
Ordinary force-controlled element	0.9

(c) Ordinary Actions

Ordinary force-controlled actions shall satisfy either Equation (6a) or Equation (6b):

$$Q_{ns} + 0.9I_e(Q_T - Q_{ns}) \le \phi_s B R_n \quad (6a)$$

$$Q_{ns} + I_e(Q_T - Q_{ns}) \le \phi_s B R_{nem} \quad (6b)$$

C.3.6.3.2.1.

(a) Force-controlled actions sensitive to vertical accelerations include force-controlled actions associated with transfer beams and girders, discontinuous columns and walls, long span cantilevers and elements that support such components.

(b) Special attention must be paid to use of equations that contain the ($Q_T - Q_{ns}$) term. Since the superposition rules do not apply to nonlinear analysis, in cases where gravity force distribution is highly unsymmetrical and/or in cases where strong directionality exists in building response where forces in one direction along an axis are significantly larger than the same forces in the other direction of the same axis, orbital plots or contours should be plotted to make sure that straight use of the ($Q_T - Q_{ns}$) term does not produce unconservative results.

(c) A recent study (Chen and Moehle, 2022) indicates that for tall core-wall buildings $0.2S_{DS}D$ ($0.12\ S_{MS}D$) appears to be an adequate approximation for the vertical contribution in response history analyses. The study suggests, however, that for critical force-controlled elements such as columns, a higher margin of safety is desirable. As such, this document has adopted the use of the higher value of $0.2S_{MS}D$ factor in this edition. Alternatively, 0.3 times the vertical MCE response spectrum acceleration value at the fundamental vertical period of the building may be used ($0.3S_{TV}D$).

(d) Where force-controlled actions (critical or ordinary) are limited by a well-defined yield mechanism, the adequacy may be evaluated using the following equations:

$$(1.2 + S_{VA})D + 1.0L + E_M \leq \phi_s R_n \qquad \text{(C-1)}$$

$$(0.9 - S_{VA})D + E_M \leq \phi_s R_n \qquad \text{(C-2)}$$

Equations (C-1) and (C-2) are applicable only where the yielding mechanism fully limits the force-controlled action to which the equations are being applied. Example applications include: beam or column shear as limited by development of probable moment strengths at the ends of the beams or columns, respectively; axial force in columns in moment frames and braced frames where axial force is limited by the sum of probable strengths of beams or braces; forces on braces and their connections in eccentric braced frames; and forces on connections in concentric and buckling-restrained braced frames. Equations (C-1) and (C-2) are not applicable to shear in wall piers because the mechanism is not uniquely defined; therefore, apparent higher-mode effects can result in substantially larger shears in these elements. When calculating the shear demands in columns of moment frames, the shear should be calculated considering that flexural yielding occurs at the ends of the column rather than in the adjacent beams or joint, because P-Delta effects and apparent higher-mode effects can increase column shears beyond those limited by yielding of the beams and joints.

3.6.3.2.2 Deformation-Controlled Actions

The demand values (member total deformations) shall be permitted to be taken respectively as the average of the values determined from the eleven or more pairs of records used in the analyses. Collector elements shall be provided and must be capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces. Every structural component not included in the seismic force–resisting system shall be able to resist the gravity load effects, seismic forces, and seismic deformation demands identified in this section.

Acceptance criterion may be assumed to be equal to $1/I_e$ times the "*a*" values for plastic deformations or $1/I_e$ times the "*d*" values for deformation ratios published in ASCE 41 tables for nonlinear response procedures (see Figure 8).

Table 6-2 also lists some recommended deformation limits for deformation-controlled actions. These limits apply to the maximum average results obtained from the suite of MCE_R ground motion pairs.

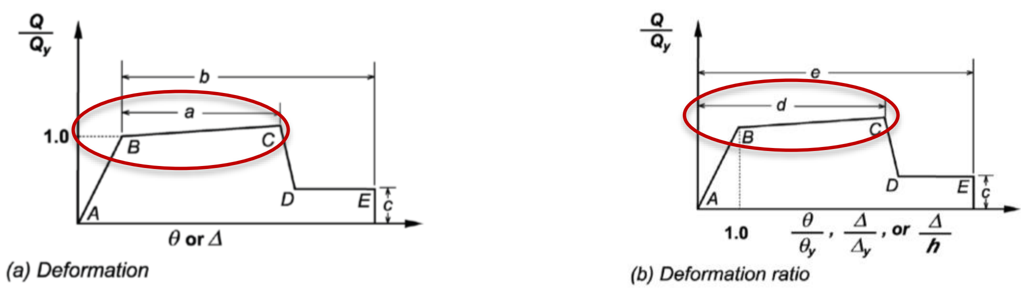


Figure 8. ASCE 41 Generalized Component Force-Deformation Relations for Depicting Modeling and Acceptance Criteria.

<u>Exception:</u> Larger values for acceptance criteria than given in ASCE 41 or in Table 6-2, may be used only if substantiated by appropriate laboratory tests or approved by the peer review process. If larger values are used, the deformation demands in some ground motions are likely to exceed the value associated with strength loss (e.g., the a- and d-values in ASCE 41); therefore, for any ground motion, the following must be satisfied: (a) strength degradation, stiffness degradation and hysteretic energy dissipation appropriate for the range of deformation demands for that ground motion shall be considered, (b) base shear capacity of the structure that considers element strength degradation shall not fall below 90% of the base shear capacity prior to the initiation of strength degradation in any element, and (c) the maximum deformation shall not exceed the valid range of modeling. Coupling beams in special reinforced concrete shear walls provide an example of where this exception may be applied.

Table 6-2. Recommended deformation limits for deformation-controlled actions

Item		Engineering Demand Parameter	Acceptance Limit
Reinforced concrete walls (outside of primary hinge zone)	No confinement	Concrete compression strain over gage length [1]	0.001/I_e
		Steel tension strain over gage length [1]	$2\varepsilon_y/I_e$
	Intermediate confinement per ACI 318-19 18.10.6.5	Concrete compression strain over gage length [1]	0.003/I_e
		Steel tension strain over gage length [1]	0.01/I_e
	Full confinement per ACI 318-19 18.10.6.4 except provisions of Section 18.10.6.4(i) need not be satisfied [2]	Concrete compression strain over gage length [1]	0.005/I_e (0.01/I_e [3])
		Steel tension strain over gage length[1]	0.01/I_e (0.05/I_e [3])
Reinforced concrete walls (primary hinge zone)	Full confinement of the entire cross section per ACI 318-19 18.10.6.4 [2]	Concrete compression strain over gage length [1]	0.005/I_e (0.01/I_e [3])
		Steel tension strain over gage length [1]	0.01/I_e (0.05/I_e [3])
Coupling beams	Conventionally-reinforced [4]	Total chord rotation	0.04/I_e
	Diagonally-reinforced [4]	Total chord rotation	0.06/I_e
	Fiber-reinforced [5]	Total chord rotation	0.04/I_e
	Steel-reinforced	Total chord rotation	0.06/I_e
Slab outrigger beams	At wall end [6]	Total rotation	0.05/I_e
	At column end [7], with shear reinforcement, $v_{uv}/(v_c+v_s) \le 0.7$	Total rotation	0.05/I_e
	At column end [7], with shear reinforcement, $v_{uv}/(v_c+v_s) > 0.7$	Total rotation	0.03/I_e
	At column end, without shear reinforcement	Total rotation	refer to ACI 318-19 18.14.5

Table 6-2. Continued

Composite Plate Shear Walls	Concrete filled composite plate coupling beam rotations	Plastic rotation	$0.03/I_e$ [8]
	CPSW plate tensile strain	Total	$0.025/I_e$ [9, 10]
	CPSW plate compressive strain	Total	$0.0045/I_e$ [9, 10]
	CPSW concrete axial compressive strain	Total	$0.0045/I_e$ [9, 10]
Soil bearing	Where soil springs are used	Movement at soil-structure interface	Per Geotechnical Engineer

[1] Strain gage length should be taken per Equation (2) on page 48. The limits in this table are selected to reflect the gage length concerns discussed in commentary C.3.6.3.2.2 below.

[2] Full confinement is required for the entire cross section of the core at the wall critical section and shall be extended vertically above and below per ACI 318-19 18.10.6.2, but need not exceed two levels above and below the critical sections. A tall building may have a number of critical sections at different heights.

[3] For compressive strains larger than 0.005 or tensions strains larger than 0.01 refer to Appendix A.1, which requires a reduction in wall shear strength.

[4] Limit is valid for coupling beams with aspect ratio larger than 2.0.

[5] Limit is derived from limited test data (Pérez-Irizarry and Parra-Montesinos, 2016) and it applies only to conditions consistent with corresponding tests.

[6] Limit is derived from limited test data (Klemencic, Fry, Hurtado, and Moehle, 2006), at strength loss, and it applies only to conditions consistent with corresponding tests. If strength loss is modeled beyond the 0.05 limit, the acceptance limit for rotation at the slab-wall interface can be increased to as high as 0.08.

[7] Limit is derived from limited test data (Hueste, Browning, Lepage, and Wallace, 2007), and it applies only to conditions consistent with corresponding tests. Limits may be exceeded if supported by test results approved by the peer review team.

[8] Ahmad, M., Shafaei, S., Bradt, T., Varma, A. (2021).

[9] Shafaei, S., Varma, A.H., Seo, J., Klemencic, R. (2021).

[10] Shafaei, S., Varma, A.H., Broberg, M., Klemencic, R. (2021).

C.3.6.3.2.2.

(a) Wall Compression strains are usually underestimated by the commonly used fiber-based models (e.g., shear wall element in Perform 3D) due to the following reasons:

1. Fiber-based model formulation typically does not consider interaction between axial/flexural and shear behavior in RC walls and is based on the plane-sections remain plane (Bernoulli-Euler) assumption. These assumptions can lead to an underestimation of compressive strains (Kolozvari et al., 2018). As illustrated in the Figure C.3.6.3.2.2-1, comparison between vertical strains obtained from LVDTs located at the base of the test specimen RW2 (Thomsen and Wallace, 1995) and predicted using Perform 3D shear wall element suggests that compressive strains obtained from the analytical model are approximately 50% of the compression strains measured during the test for all three considered drift levels applied at the top of the wall (Kolozvari et al., 2018).

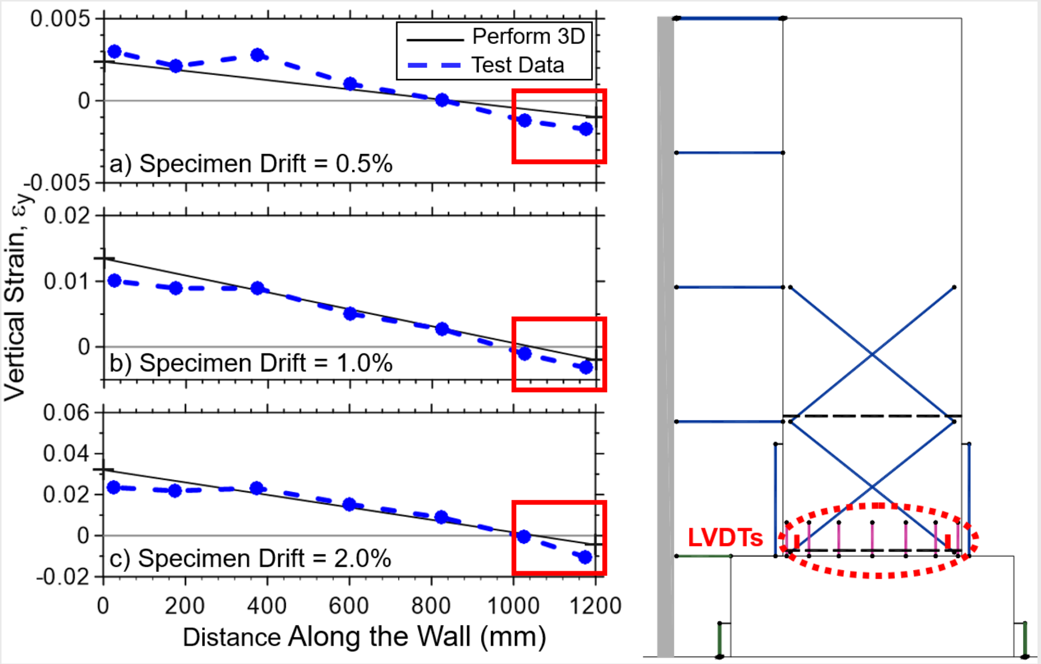
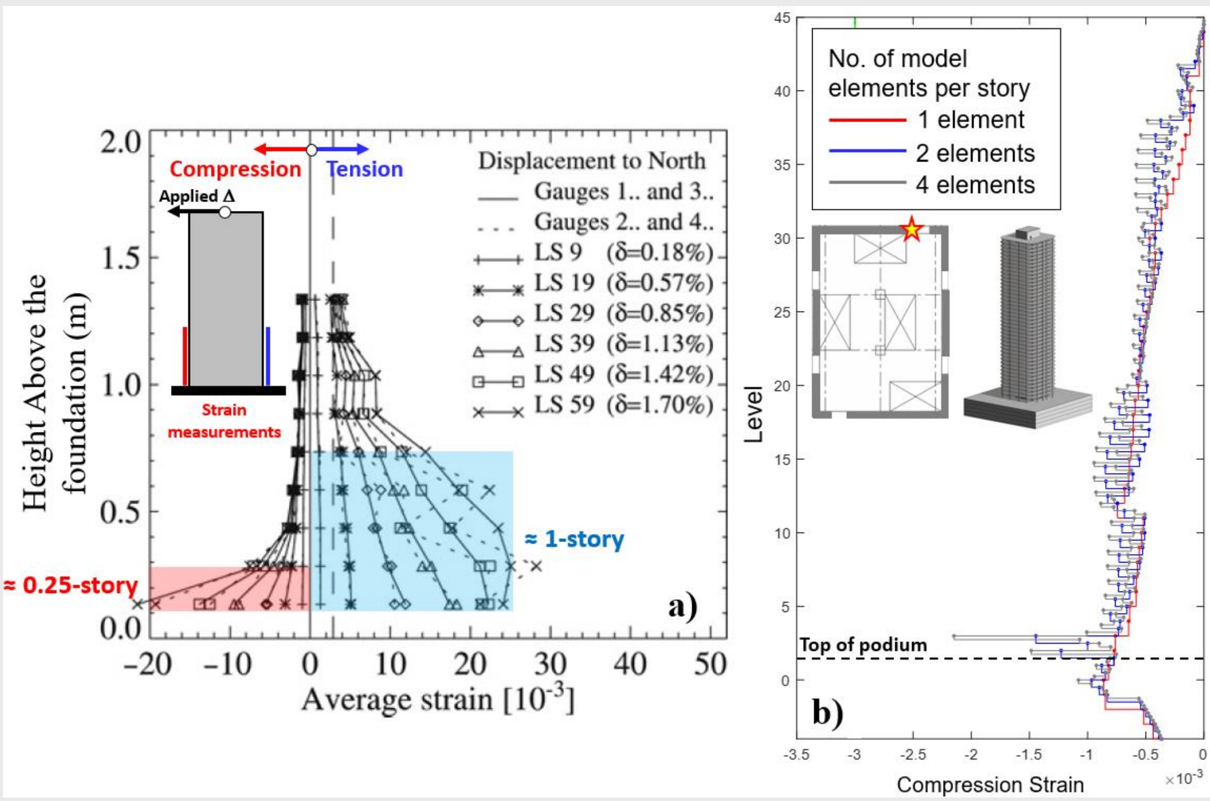


Figure C.3.6.3.2.2-1. Comparison of predicted (Perform 3D, data extracted from Kolozvari et al., 2018) and measured vertical strain profiles along the base of the wall specimen RW2 (Thomsen and Wallace, 1995)

C.3.6.3.2.2. (continued)

2. Element height (and corresponding strain gage length) used in engineering practice is typically equal to one story height (Table 6-2). However, experimental evidence show that compression strains tend to localize over much shorter length, as illustrated in Figure C.3.6.3.2.2-2(a) for test specimen WSH6 (Dazio et al., 2009), suggesting that using wall element height of approximately 25% of the story height would be more appropriate. Figure C.3.6.3.2.2-2(b) further compares profiles of predicted compression strains at the wall boundary of a 42-story RC core wall building (Moehle et al., 2011) for a representative ground motion, showing that using two and four wall elements per story height could increase predicted compressive strains approximately 2.0 and 3.0 times, respectively, particularly in the plastic hinge region.



(a) Wall test specimen WSH6 (Dazio et al., 2009)

(b) Analysis results of a tall RC core wall building (Moehle et al., 2011)

Figure C.3.6.3.2.2-2. Distribution of vertical strains over the height of the boundary element

(b) Given the very specific acceptance criteria provided in Table 6-2, the authors are of the opinion that extending the full confinement for the entire cross section of the core at the wall critical section per ACI 318-19 18.10.6.2 for a height not exceeding two levels above and below the primary plastic hinge zone is sufficient.

3.6.3.2.3 Multiple Towers on a Common Podium or Basement

Risk category and corresponding importance factor, I_e, for the building shall be determined by the building officials.

C.3.6.3.2.3. Appropriate risk category or categories of the building(s) shall be determined by the Building Officials and reported to the design team and the seismic peer review panel. This commentary contains recommendations that apply only to cases where each of the towers have their complete and separate means of egress to outside of the building complex above the area of the building common to more than one tower and a complete and separate means of egress for the areas below the towers.

Where multiple towers on a common podium or base create a situation in which the number of occupants at or below the podium or ground level may exceed 5,000 persons, then the value of $I_e = 1.25$ may be used in conjunction with application of Equations 5a, 5b, 6a, and 6b for all force-controlled actions including those of the podium diaphragm and below, including the foundations placed under the Risk Category III portion of the structure and all force-controlled elements of the tower passing through the Risk Category III portion of the project.

Acceptance criterion for all deformation-controlled elements of the tower passing through the Risk Category III portion of the project as well as all deformation-controlled actions of the first level of each tower immediately above the common podium may be considered as 1/Ie times the acceptance limit for a similarly situated the Risk Category II building. In addition, the story drift limit of all tower floors may be considered as $1/I_e$ times the acceptance limit for a similarly situated the Risk Category II building.

3.6.3.2.4 Curtain Walls and Stairways

Curtain walls and stairways shall be designed to be able to resist the average of maximum MCE_R story drifts, accelerations, and displacements at the location of these elements in the building without failure or creating life-safety hazards inside or outside of the building. If sliding connections are used, such connections shall be able to accommodate maximum average plus one standard deviation of MCE_R story drifts, accelerations, and displacements without failure.

4. PEER REVIEW REQUIREMENTS

For each project, a Seismic Peer Review Panel (SPRP) shall be convened. The SPRP is to provide an independent, objective, technical review of those aspects of the structural design of the building that relate to seismic performance, according to the requirements and guidelines described in this document, and to advise the Building Official whether the design generally conforms to the intent of this document and other requirements set forth by the Building Official.

The SPRP participation is not intended to replace quality assurance measures ordinarily exercised by the Engineer of Record (EOR) in the structural design of a building. Responsibility for the structural design remains solely with the EOR, and the burden to demonstrate conformance of the structural design to the intent of this document and other requirements set forth by the Building Official resides with the EOR. The responsibility for conducting Structural Plan Review resides with the Building Official and any Plan Review consultants.

4.1. QUALIFICATIONS AND SELECTION OF SPRP MEMBERS

Except when determined otherwise by the Building Official, the SPRP shall include a minimum of three members with recognized expertise in relevant fields, such as structural engineering, earthquake engineering research, performance-based earthquake engineering, nonlinear dynamic response analysis, tall building design, earthquake ground motion, geotechnical engineering, geological engineering, and other such areas of knowledge and experience relevant to the issues the project poses. The SPRP members shall be selected by the Building Official based on their qualifications applicable to the Seismic Peer Review of the project. The Building Official may request the opinion of the Project Sponsor and EOR on proposed SPRP members, with the Building Official making the final decision on the SPRP membership. SPRP members shall bear no conflict of interest with respect to the project and shall not be part of the design team for the project. The SPRP provides their professional opinion to and acts under the instructions of the Building Official.

4.2. PEER REVIEW SCOPE

The general scope of services for the SPRP shall be indicated by the Building Official. The SPRP, either individually or as a team, shall include a written scope of work in their contract to provide engineering services. The scope of services shall include review of the following: earthquake hazard determination, ground motion characterizations, seismic design methodology, seismic performance goals, acceptance criteria, mathematical modeling and simulation, seismic design and results, drawings and specifications.

The SPRP shall be convened as early in the structural design phase as practicable to afford the SPRP opportunity to evaluate fundamental design decisions that could disrupt design development if addressed later in the design phase. Early in the design phase, the EOR, Building Official, and the SPRP shall jointly establish the frequency and timing of SPRP review milestones, and the degree to which the EOR anticipates the design will be developed for each milestone. The SPRP shall provide written comments to the EOR and to the Building Official, and the EOR shall prepare written responses thereto. The SPRP shall maintain a log that summarizes SPRP comments, EOR responses to comments, and resolution of comments. The SPRP shall make the log available to the EOR and to the Building Official as requested. At the conclusion of the review the SPRP shall submit to the Building Official a written report that references the scope of the review, includes the comment log, and indicates the professional opinions of the SPRP regarding the design's general conformance to the requirements and guidelines in this document. The Building Official may request interim reports from the SPRP at the time of interim permit reviews.

C.4. Formation of an advisory board appointed by the Building Official is strongly recommended. This advisory board shall consist of experts who are widely respected and recognized for their expertise in relevant fields, including but not limited to, structural engineering, performance-based design, nonlinear analysis techniques, geotechnical engineering. The advisory board members may be elected to serve for a predetermined period of time on a staggered basis. The advisory board shall oversee the design review process across multiple projects periodically; assist the Building Official in developing criteria and procedures spanning similar design conditions, and resolve disputes arising under peer review.

5. SEISMIC INSTRUMENTATION

Buildings analyzed and designed according to the provisions of this document shall be furnished with seismic instrumentation according to the provisions of this section.

5.1. OVERVIEW

The primary objective of structural monitoring is to improve safety and reliability of building systems by providing data to improve computer modeling and enable damage detection for post-event condition assessment. Given the spectrum of structural systems used and response quantities of interest (acceleration, displacement, strain, rotation, pressure), the goal of these provisions is to provide practical and flexible requirements for instrumentation to facilitate achieving these broad objectives. The instrumentation used on a given building shall be selected to provide the most useful data for post-event condition assessment.

The recent advances in real-time structural health monitoring and near real-time damage detection may be extremely useful in rapid evaluation of status of the building after an event and deciding whether the building is fit for continued occupancy or not (Naeim 2011). To facilitate this, response data (and any resulting information) shall be made immediately available after an earthquake triggered event to the Building Official, owner, and/or other approved agency. See Section 5.5 for more details.

5.2. INSTRUMENTATION PLAN AND REVIEW

An instrumentation plan shall be prepared by the EOR and submitted to SPRP and Building Official for review and approval. SPRP Approved instrumentation plans shall be marked accordingly on the structural drawings.

> **C.5.2.** For projects in the state of California, if the building is intended to be included in the inventory of buildings monitored by the California Geologic Survey's Strong Motion Instrumentation Program (CGS/CSMIP) then the recorders, accelerometers and wiring must be of a type approved by the CSMIP and described in the CSMIP Technical Specifications Letter for instrumentation. In this case, the instrumentation plan may also come from CGS instead of the EOR.

5.3. MINIMUM NUMBER OF CHANNELS

The building shall be provided with minimum instrumentation as specified in the Table 8. The minimum number of required channels maybe increased at the discretion of SPRP and Building Official. Please note that for reliable real-time structural health monitoring and performance evaluations a substantially larger number of channels may be necessary (Naeim 2011).

Each channel corresponds to a single response quantity of interest (e.g., unidirectional floor acceleration, story displacement, etc.).

Table 8. Minimum Number of Channels of Instrumentation

Number of Stories Above Ground	Minimum Number of Channels
6-10	12
11 – 20	15
21 – 30	21
31 – 50	24
> 50	30

C.5.3. For example, a 34-story building shall have at least 24 sensors. Three horizontal sensors would be located at the roof level and six other levels, plus two vertical sensors at the base, and one placed either to measure special conditions at the roof, or at the base, near a third wall to get rocking in a second direction. In general, the seven levels would be chosen where there are changes in stiffness or mass or offsets in the structural system, if any, otherwise they would be evenly distributed over the height.

5.4. DISTRIBUTION

The distribution or layout of the proposed instrumentation shall be logically designed to monitor the most meaningful quantities.

The sensors shall be located at key measurement locations in the building as appropriate for the measurement objectives and sensor types. The sensors shall be connected by dedicated cabling to one or more central recorders, interconnected for common time and triggering, located in an accessible, protected location with provision for communication.

Strong motion instrumentation should be located strategically in a building in order to learn as much as possible about the response of the building during an earthquake and to confirm/verify design and analysis assumptions.

1. It is important to measure the horizontal and torsional motion on each of a series of floors, from the base to the roof. This requires (at least) three uniaxial horizontal accelerometers on each chosen floor. These shall be located near the perimeter of the building along walls on opposing sides of the building (as distant as practical from the core) to get the best torsional signal. The sensors placed along the walls shall be at the same relative position (e.g., at mid length). They shall be oriented with their sensing directions parallel to the walls. A third accelerometer shall be placed near the center of the floor, oriented perpendicular to the other two, to measure horizontal motion in that direction.

2. Another goal is to measure rocking at the base of the building, especially for a stiff building founded on soft soils, to determine any rocking contribution to the drift. At least two vertical accelerometers are needed, placed near walls on the opposing sides of the building. To measure rocking in both directions, a third is needed near one of the other walls. In general, the upper floors do not need vertical accelerometers.

3. In general, for easy interpretation and analysis of the recorded data, sensors on different floors shall be stacked vertically if possible, that is, placed at the same relative position on each floor, so that the same location in the response is measured.

4. If there are special features near the roof, such as mechanical equipment in the penthouse or architectural features with mass, it may be important to place additional sensors there.

5. It is often effective to install the sensors in the interstitial space above the false ceiling, if

present. This keeps the sensors out of the way of the occupants and the normal building activities, reducing likelihood of damage to the sensors. For example, the sensors planned to measure the motion of the 8th floor could actually be located on the underside, above the ceiling on the 7th floor.

6. The central recorder shall be located in a utility or electrical room with AC available, on one of the lower floors of the building, for convenience. A communication line (Internet) shall be provided at the recorder location(s). Internet service shall be provided and maintained by building owner or approved agency.

7. Cabling from the accelerometers to the recorder shall be continuous runs (i.e., no splices). A pathway will need to be established for the vertical run from the sensors on the upper floors to the recorder location. Depending on local ordinances and fire codes, plenum rated cable may be required.

5.5. INSTALLATION AND MAINTENANCE

Prior to installation, a site walkthrough shall be performed to finalize and mark exact instrument locations. Typical participants for this walkthrough include:

- Contractor to facilitate access (keys, ladders, etc) to all potential locations.
- Electrical subcontractor in charge of cable installation.
- Instrument service provider to document final locations and installation plan.
- EOR or SPRP representative to approve final physical locations and/or any changes required due to unforeseen circumstances.

The building owner shall be responsible for installation and commissioning of the seismic monitoring system. A service provider authorized by SPRP or CGS shall be contracted to perform installation and commissioning per EOR specifications and instrument manufacturer requirements. Supporting trade services such as electrical shall be contracted by the building owner as needed. A commissioning report that meets all documentation requirements in section 5.6 shall be submitted to the SPRP and/or EOR for final approval.

For the instrumentations done under this document, the building owner shall provide for annual preventative maintenance per instrument manufacturer recommendations and as approved by the SPRP. Annual maintenance reports shall be submitted to the Building Official.

If the system is not under CGS/CSMIP monitoring program, then event data, reports and all other documentation shall be stored indefinitely in an secure offsite repository hosted and maintained by an approved agency and made accessible to Building Official, owner, and other approved agencies.

5.6. DOCUMENTATION

The sensor locations shall be well documented for reference during analysis of the motions after an earthquake is recorded. Strong shaking is infrequent in a building, and it is possible that by the time an earthquake occurs the activities in a building have resulted in certain sensors being moved for construction work and not returned with the same orientation or location. Digital photos shall be taken to document location and orientation of the installed sensors at initial installation and whenever changes are made. A sensor layout showing the sensor locations and key structural elements on plan and typical sections shall be prepared. A tag shall be attached at each sensor location to underscore its importance.

C.5.6. A sensor layout will facilitate rapid visual interpretation of the recorded data. It is valuable to archive design plans, especially structural plans, to allow thorough analysis of the data and finite-element modeling of the building when earthquake motion has been recorded.

The tag attached at each sensor location to underscore its importance can read, for example, “Seismic sensor - Do not remove without notifying Building Official.” The documentation is particularly important to be maintained since after an earthquake, depending on the level of shaking, it may not be possible to access certain areas in the building until building officials have been able to schedule a visit. With good documentation, analysis of the recorded data and assessment of the structural response can occur without accessing the building.

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APPENDIX A -- Reinforced Concrete Elements

A.1. Expected Component Strengths

The calculation of expected strengths based on the nominal strength equations and expected material properties follows the accepted practice of ASCE 41-17, ASCE 7-22 (Chapter 16), and other standards. Adjustment is made in Equation (5) to allow for the use of nominal material properties and (6) to allow for the use of expected material properties. However, there are instances, some of which are described in this section, where the expected strengths should be adjusted to correct for biases in the nominal strength equations using mean measured values R_{ne} from tests. The test specimens considered should have details and materials that are representative of those used in the structure to determine a B-value to use in the acceptance criteria for force-controlled elements in Section 3.6.

The subsections below described the supporting information used to determine B-values recommended in Appendix B for wall shear (A.1.1) and diaphragm shear (A.1.2), and wall panel zone shear strength and detailing requirements (A.1.3). Test results are used to determine B-values relative to ACI 318-14 shear strength equations to adjust for the bias in Equations (5) and (6). The R_{ne} values (equivalent to V_{ne} in these cases) are used to determine B-values, they are not intended to replace R_n and R_{nem} in Equations (5) and (6). Applicable stress limits also apply, i.e., $10\sqrt{f_c'}$ and $8\sqrt{f_c'}$ for individual wall segments and wall segments resisting shear ACI 318-19 18.10.2.3 and $8\sqrt{f_c'}$ for diaphragms resisting shear (18.12.9.2).

A.1.1. Reinforced Concrete Structural Walls – Shear Strength

A.1.1.1 Wall piers having height-to-length ratio $h_w/\ell_w \geq 2$

Where calculated concrete compressive strains $\bar{\varepsilon}_c \leq 0.005$ and calculated longitudinal reinforcement tensile strains $\bar{\varepsilon}_s \leq 0.01$ at all points along a cross section, V_n and V_{ne} of the wall pier can be determined as:

$$V_n = A_{cv}\left(2\lambda\sqrt{f'_{ce}} + \rho_t f_{ye}\right) \le 10A_{cv}\sqrt{f'_{ce}} \quad \text{(A-1a)}$$

$$V_{ne} = 1.5A_{cv}\left(2\lambda\sqrt{f'_{ce}} + \rho_t f_{ye}\right) \le 15A_{cv}\sqrt{f'_{ce}} \quad \text{(A-1b)}$$

Where f'_{ce} and f_{ye} are the expected material strengths, the 1.5 factor in Eq. (A-1b) represents a slightly conservative estimate of the mean overstrength determined from evaluation of test data (Wallace et al, 2013; Kim 2016), where $\bar{\varepsilon}_c$ and $\bar{\varepsilon}_s$ are the mean values of the maximum strain values determined from the 11 or more ground motions considered. Therefore,

$$\begin{aligned} &\frac{V_{ne}}{V_n} = 1.5 \\ &B = 0.9\frac{V_{ne}}{V_n} = 1.35 \end{aligned} \quad \text{(A-1c)}$$

Where $0.005 < \bar{\varepsilon}_c < 0.01$ or $\bar{\varepsilon}_s > 0.01$ at any point along the cross section, the ratio of V_{ne}/V_n for the wall pier shall be taken as:

$$\begin{aligned} &\frac{V_{ne}}{V_n} = \min\left\{1.5 - \left(\frac{\bar{\varepsilon}_s - 0.01}{0.02}\right);\ 1.5 - \left(\frac{\bar{\varepsilon}_c - 0.005}{0.005}\right)\right\} \\ &B = 0.9\frac{V_{ne}}{V_n} \end{aligned} \quad \text{(A-1d)}$$

The ratio of V_{ne}/V_n in (A-1d) need not be taken less than 1.0. In no case shall $\bar{\varepsilon}_c$ be taken greater than 0.01.

A.1.1.2 Wall piers having height-to-length ratio $h_w/\ell_w < 2.0$:

V_n and V_{ne} of the wall pier can be determined as:

$$V_n = A_{cv}\left(\alpha_c\lambda\sqrt{f'_{ce}} + \rho_t f_{ye}\right) \le 10A_{cv}\sqrt{f'_{ce}} \quad \text{(A-1e)}$$

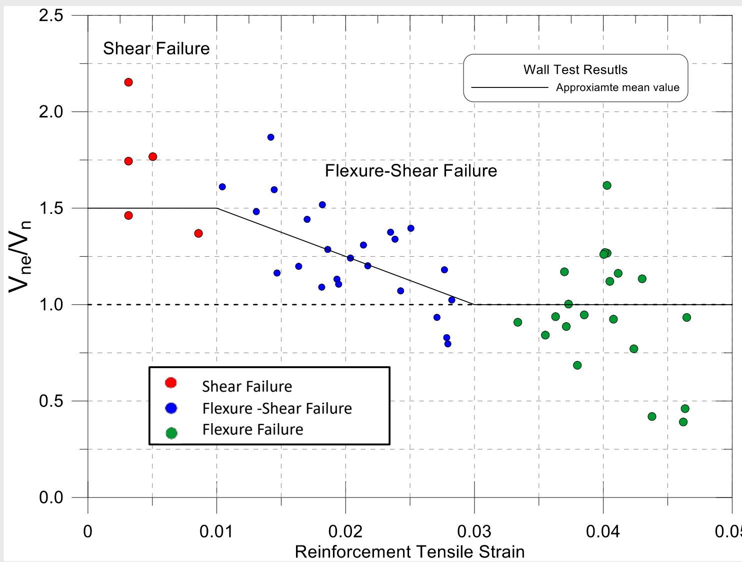
$$V_{ne} = 1.15A_{cv}\left(\alpha_c\lambda\sqrt{f'_{ce}} + \rho_t f_{ye}\right) \le 11.5A_{cv}\sqrt{f'_{ce}} \quad \text{(A-1f)}$$

Where f'_{ce} and f_{ye} are the expected material strengths, α_c is a factor that depends on the ratio of h_w/ℓ_w and varies between 2 and 3, and the factor 1.15 in (A-1f) represents the mean overstrength determined from evaluation of approximately 200 tests of walls with rectangular cross sections (with coefficient of variation of about 0.30). Therefore,

$$\frac{V_{ne}}{V_n} = 1.15$$
$$B = 0.9\frac{V_{ne}}{V_n} = 1.05 \qquad \text{(A-1g)}$$

Given the fact, however, that walls at podium levels and below are subjected to sensitivity analyses for backstay effects per Section 3.4.9 of this document, the value of *B* per Eq. (A-1g) is increased by 20% in Appendix B to 1.25.

C.A.1.1. Tests on slender walls failing in shear show that shear strength decreases with increasing inelastic flexure (Wallace et al, 2013; Moehle, 2014; Kim 2016). The shear strength determined from Equation (A-1b) is applicable to walls with relatively low flexural ductility demands. Equation (A-1d) is based on an evaluation of test data for walls demonstrating tensile yielding of boundary longitudinal reinforcement prior to shear failure (Figure A-1), as reported by Abdullah and Wallace (2017). Comparison of compressive strain values obtained from fiber models with uncoupled axial-flexural and shear responses indicate that peak compressive strains at wall boundaries may be underestimated by a factor of two (Wallace, 2007; Kolozvari and Wallace, 2016); the limit on mean concrete compressive strain of 0.01 is intended to protect against compression failures at wall boundaries due to bias in modeling results.



For low-aspect ratio walls, less overstrength is observed, in part due to the α_c factor applied in ACI 318 which linearly increases from 2.0 to 3.0 between h_w/ℓ_w values of 2.0 and 1.5. Although larger overstrength was observed for walls with substantial flanges or barbells at both ends, this condition is unlikely to exist in typical tall buildings.

A.1.2 Reinforced Concrete Diaphragms

Due to a lack of test data for structural concrete diaphragms, the B-value is determined from the wall tests described in A.1.1, which are assumed to be reasonably representative of diaphragms.

$$V_n = A_{cv}\left(\alpha_v \lambda \sqrt{f'_{ce}} + \rho_t f_{ye}\right) \leq 8A_{cv}\sqrt{f'_{ce}} \tag{A-2a}$$

$$V_{ne} = 1.5A_{cv}\left(2\lambda \sqrt{f'_{ce}} + \rho_t f_{ye}\right) \leq 12A_{cv}\sqrt{f'_{ce}} \tag{A-2b}$$

$$\frac{V_{ne}}{V_n} = 1.5$$

$$B = 0.9\frac{V_{ne}}{V_n} = 1.35 \qquad \text{(A-2c)}$$

C.A.1.2. The B-value for diaphragms is based on test results for wall segments. It is noted that the $8\sqrt{f_c'}$ stress limit for diaphragms resisting shear in ACI 318-19 Section 18.12.9.2 applies where evaluating acceptance criterion for shear strength as noted in Equations (A-2a).

A.1.3. Reinforced Concrete Structural Wall Panel Zones – Shear Strength

For structural wall panel zones, V_n and V_{ne} can be determined as:

$$V_n = A_{cv}\left(\alpha_c \lambda \sqrt{f'_{ce}} + \rho f_{ye}\right) \le 10 A_{cv}\sqrt{f'_{ce}} \qquad \text{(A-3a)}$$

$$V_{ne} = A_{cv}\left(3\lambda \sqrt{f'_{ce}} + \rho f_{ye}\right) \le 25 A_{cv}\sqrt{f'_{ce}} \qquad \text{(A-3b)}$$

Therefore,

$$B = 0.9\frac{V_{ne}}{V_n} \qquad \text{(A-3c)}$$

Where the design panel zone shear force V_u exceeds $\phi 10 b_w h_{wp}\sqrt{f'_c}$, the panel zone should be confined by transverse reinforcement as required in confined boundary elements of special structural walls, where b_w = thickness of the panel zone, and h_{wp} = designer-defined height of the panel zone. In general, h_{wp} should be selected to satisfy the design requirement that $V_u \le \phi V_n$, but h_{wp} should be larger than the story height h_s.

C.A.1.3. Panel zones are regions of structural walls that act as connections between intersecting wall segments or between walls and other structural elements. Figure A-2 illustrates four examples of panel zones. Such regions can be subjected to relatively high shear stresses resulting from force transfers under lateral loads. Villalobos et al. (2016) showed that shear strength of wall panels can be expressed by Equation (A-3b).

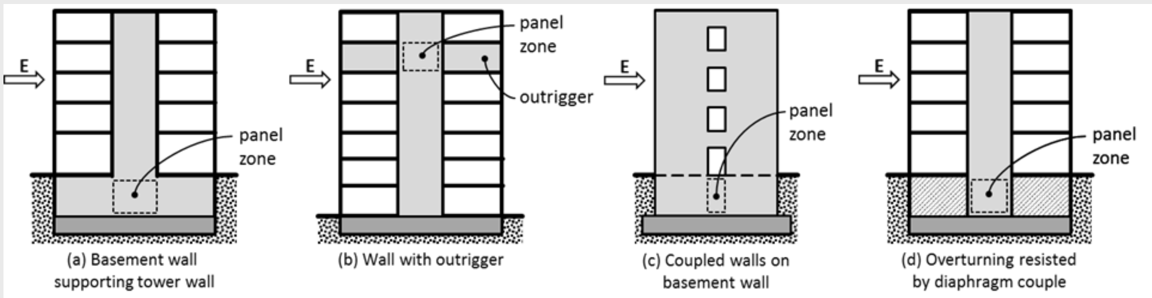
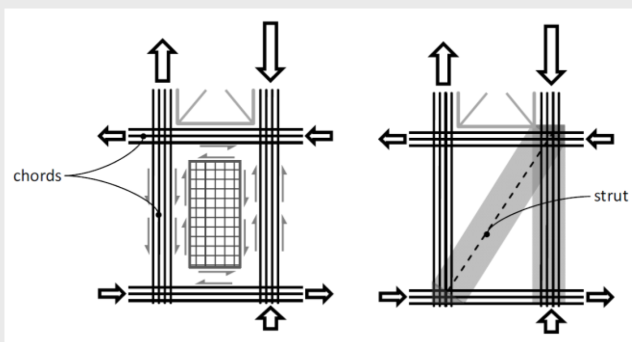


Figure A-2 Examples of panel zones in structural walls.

Making h_{wp} larger than h_s ensures that floor diaphragms above and below the pane zone can act as the chords shown in A-3.



(a) Chords and shear panel(s) (b) Strut-and-tie

Fig. A-3 Concepts for internal force resistance (Villalobos et al, 2016)

A.2. Concrete Shear Strength of Reinforced Concrete Mat Foundations

One way concrete shear strength, for mat foundations without minimum shear reinforcement shall be determined as:

$$V_c = 8\lambda_s \lambda \left(\rho_w\right)^{1/3} \sqrt{f_c^{'}} b_w d \qquad \text{(A-4a).}$$

But need not be taken less than $1.0\lambda\sqrt{f_c^{'}} b_w d$.

If minimum shear reinforcement per ACI 318-19 Section 9.6.3.4 (for beams) is provided, one-way concrete shear strength shall be determined as:

$$V_c = 2.0\lambda\sqrt{f_c^{'}} b_w d \qquad \text{(A-4b)}$$

C.A.2. One-way shear strength relations were updated in ACI 318-19 to reduce the number of equations, improve safety, improve ease of use, and remove discontinuities (Kuchma et al, 2019). The two primary issues related to safety involved adding a factor to account for size effect for members without minimum shear reinforcement and address unconservative shear strength estimates for members with low longitudinal reinforcing ratios. The 318-19 concrete shear strength equations are summarized in Table A1.

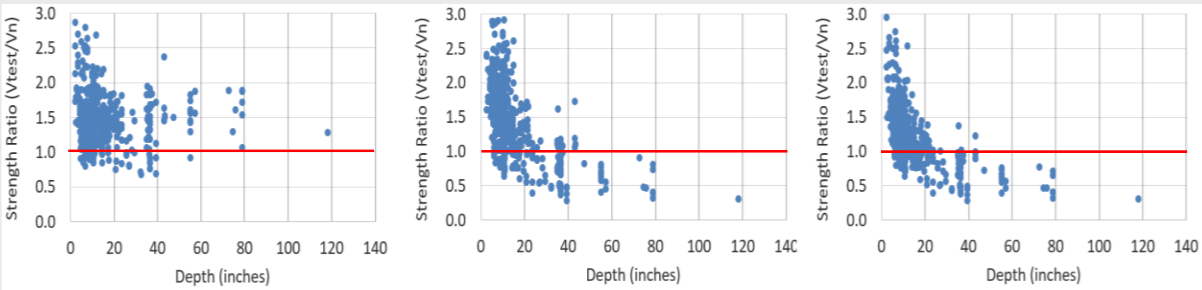
Table A1 Concrete Shear Strength

Criteria	V_c		
$A_v \geq A_{v,min}$	Either of:	$\left[2\lambda\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d$	(a)
		$\left[8\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d$	(b)
$A_v < A_{v,min}$	$\left[8\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d$		(c)

C.A.2. Continued

where axial load N_u is positive for compression and negative for tension, V_c shall not be taken less than zero or greater than $5\lambda\sqrt{f_c'}b_w d$, the value $N_u/6A_g$ shall not be taken greater than $0.05f_c'$, $\lambda_s = \sqrt{2/(1+d/10)}$, λ is a modification factor applied when lightweight concrete is used, and ρ_w is the ratio of tension reinforcement A_s to $b_w d$.

Comparison of strength values obtained in tests to values determined using Equation (c) In Table A.1 are shown in Fig. A-4. Results shown In Fig A-4(a) indicates that the provisions in ACI 318-19 provide Improved safety, especially as depth of member increases.



(a) ACI 318-19 (b) ACI 318-14 Simplified (c) ACI 318-14 Detailed

Figure A-4 ACI 318 Concrete shear strength variation with member depth.

In cases where the lateral system consists of a core wall, the integrity of the mat foundation is essential for the system to achieve the desired performance; therefore, these Guidelines reduce concrete shear strength to address size effect even though the size effect factor in 318-19 does not apply to mat foundations. However, a lower limit is applied.

Minimum shear reinforcement for mat foundations is typically based on the requirements of 9.6.3.4 for beams, because ACI 318-19 does not include minimum shear reinforcement for one-way or two-way slabs.

A.3. Diaphragm-to-wall connections

Introduction

The anchorage of collectors in shear walls of tall buildings is a topic that is not well addressed in building codes or other publications, and that is not implemented consistently in design practice. Some designers anchor the bars into the wall boundaries a distance equal to the bar development length without further consideration; this practice leaves open the question of whether that length is sufficient to transfer the collector force into the wall without overstressing it locally. Other designers use anchoring-to-concrete provisions of ACI 318; this solution is questionable because reinforcing bars are not cast-in anchors and because the wall into which they are anchored is not merely a block of concrete but is instead an element that is heavily stressed by other actions. Still other designers routinely extend the tension collector bars the full length of the wall; this alleviates most concerns about whether the forces can be transferred adequately into the wall, but leaves open the question of whether more efficient design might be feasible.

The following material was developed as an aid to thinking about force transfer from collectors into shear walls and suggests design solutions.

A.3.1 Anchorage of Diaphragms at Typical Elevated Floor Levels

Figure A-5 illustrates conditions for a typical floor level that is not within an intended wall plastic hinge zone and where inertial forces are transferred from a floor diaphragm into the wall through a collector. Only a tension collector is shown, but the discussion applies equally with minor modifications if collector forces also enter through compression and/or through shear-friction. In typical designs, the wall is subjected to high shear stresses that are likely to result in inclined cracks, as shown by the blue lines.

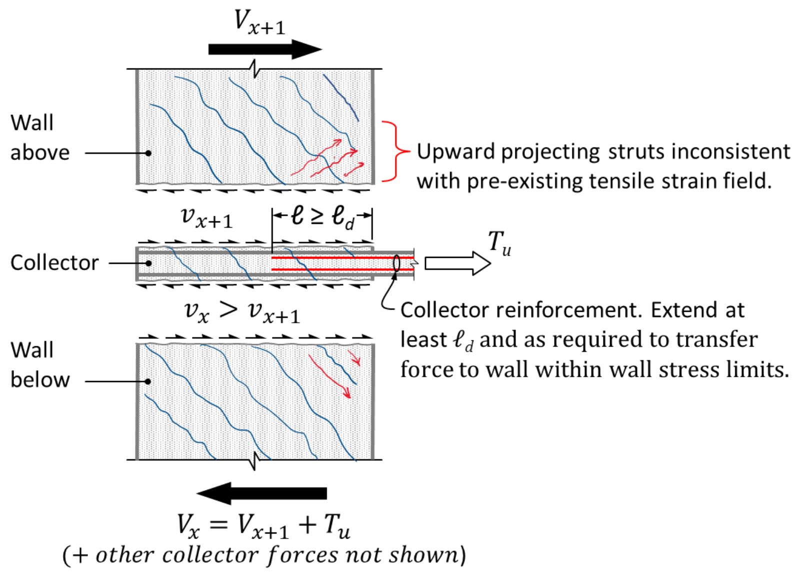


Figure A-5. Partial free-body diagrams showing forces associated with collector-to-wall transfer away from plastic hinge. (Adapted from Moehle, Hooper, and Meyer, NIST Tech Brief No. 3, 2nd ed.)

Design of the collector reinforcement embedment into the wall requires two considerations.

- First, the embedment length ℓ must be at least the development length of the collector bars, ℓ_d, calculated for the expected tensile force, which is typically on the order of $1.25f_y$.
- Second, the embedment length must be sufficient such that it does not overstress the wall into which it is embedded. For this second consideration, some designers envision the tension force T_u spreading into the wall above and below the collector through a series of struts (red arrows in Figure A-5). It is highly unlikely, however, that the upper wall will participate significantly in resisting the tension force T_u because this would require the development of struts across a tension field caused by wall shear, resulting in an incompatibility of strains. (This would also result in overlapping struts, which ACI 318 permits only at nodes.) Rather, the collector force must be transmitted to the wall below. This creates an additional local wall shear T_u/ℓ per unit length acting along the length ℓ. Assuming that the wall shear from the wall above the collector is uniformly distributed along the wall length, the resulting unit wall shear acting along the embedment length is $V_{x+1}/\ell_w + T_u/\ell$. This unit shear should not exceed the provided unit shear strength of the wall at that location and should not exceed the permissible maximum unit shear for a shear wall.

In general, it is preferable to extend the collector tension reinforcement into the wall as required above without splices. In some cases, the designer may choose to lap splice the collector tension reinforcement with added reinforcement in the wall. This added reinforcement can be within the thickness of the collector or below the collector; reinforcement placed above the collector should not be considered effective because of incompatibility of strains as discussed previously. Design of the added reinforcement can be based on reasonable interpretations of provisions in ACI 318-19 Chapter 17 or conservative application of strut-and-tie provisions in Chapter 23. For example:

- Figure A-6 illustrates an interpretation of the provision suggested in Figure R17.4.2.9 of ACI 318. In this approach, it is assumed that the collector reinforcement is fully anchored at a point located ℓ_d from the far end of that reinforcement – this point is equivalent to the head of a cast-in anchor. Consistent with the provisions and associated commentary of ACI 318, added "anchor" reinforcement within the depth of the collector or within $h_{ef}/2$ below the collector can be included to drag the collector force farther along the wall length, provided that the added reinforcement is fully developed on both sides of an assumed failure cone. The added reinforcement (shown blue) could be horizontal web reinforcement in excess of that required to resist wall shear or it could be reinforcement specifically added to lap with the collector reinforcement.
- Figure A-7 illustrates a simplified interpretation of strut-and-tie modeling. This simplified interpretation assumes a nodal point at the mid-length of the collector reinforcement within the wall, with strut action to engage added reinforcement within a "reasonable" strut angle projecting from the node. Alternatives to using the mid-length might be acceptable, but this might require detailed strut-and-tie modeling. The strut angle should be defined with due consideration of this being earthquake loading with multiple loading cycles occurring within a shear wall that is already highly stressed. An angle exceeding 45 degrees would seem to require justification that includes consideration of these conditions. Bars acting as added reinforcement need to be fully developed.

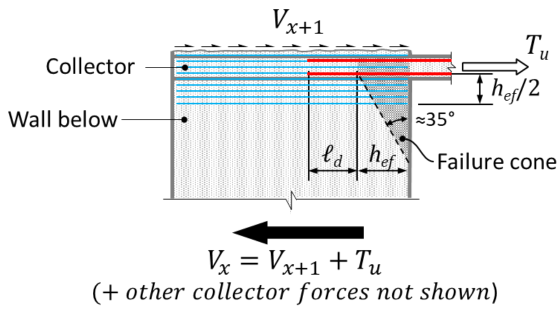


Figure A-6. An interpretation of Figure R17.4.2.9 applied to added anchor reinforcement.

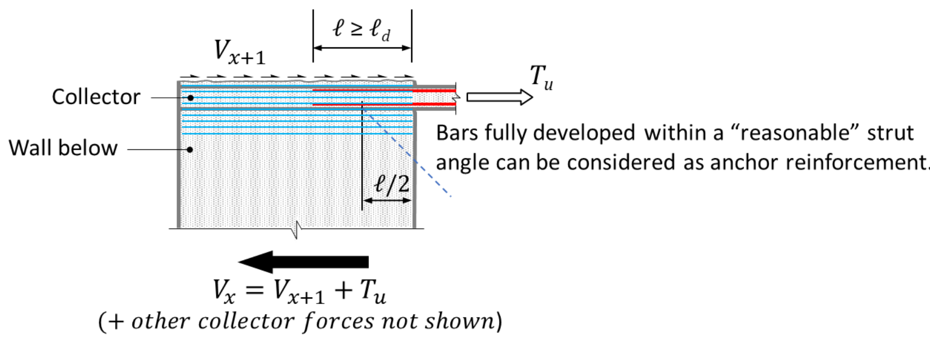


Figure A-7. A simplification of the strut-and-tie method applied to added anchor reinforcement.

A.3.2 Anchorage into Wall Plastic Hinge Zones at Transfer Levels

The conditions of the preceding discussion were for a typical floor level that is not within an intended wall plastic hinge zone and where inertial forces are transferred from a floor diaphragm into the wall through a collector. In contrast, the following discussion applies where the wall is expected to develop a plastic hinge, including core walls at transfer levels. At such locations, the collector tension reinforcement is likely to enter the flexural tension side of the wall, as shown in Figure A-8. Only a tension collector is shown, but the discussion applies equally with minor modifications if collector forces also enter through compression and/or through shear-friction.

In this case, the natural force path is for shear from the wall above the transfer level being transferred out of the wall through the collector tension reinforcement. The transverse tension in the flexural tension zone of the wall reduces the bond capacity of the collector reinforcement, such that lap splicing with adjacent anchor reinforcement is likely to be ineffective. At these locations, collector tension reinforcement should be extended along the full length of the wall and should be developed within the flexural compression zone of the wall. Collector forces that enter the side of the wall through shear-friction likewise should be dragged toward the flexural compression zone with added reinforcement. The vertical positioning of the added reinforcement will depend on the force conditions within the wall above and below the transfer level.

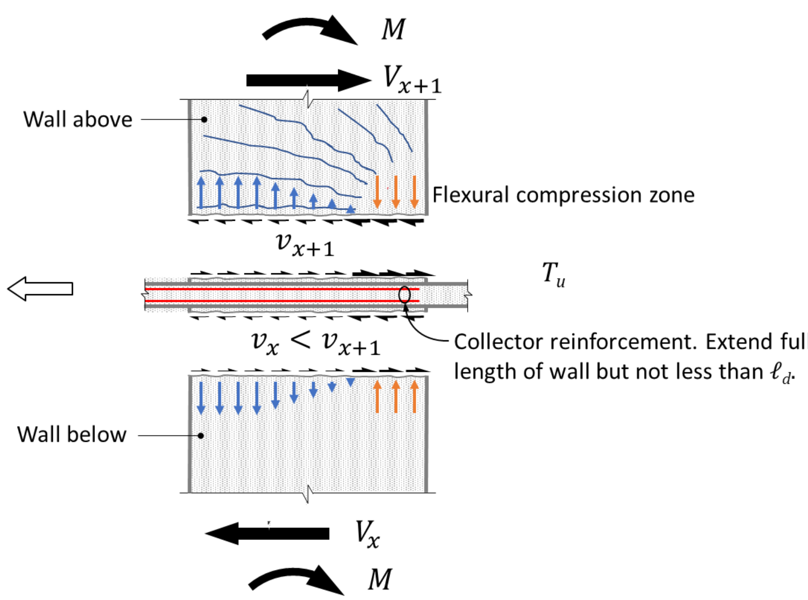


Figure A-8. Partial free-body diagrams showing forces associated with collector-to-wall transfer at wall plastic hinge.

APPENDIX B -- Recommended B and ϕs Values to be used in conjunction with Equations 5a, 5b, 5c, 5d, 5e, 5f, 6a, and 6b.

Component		Seismic Action	Classification	ϕ_s	*B*
Below Grade Perimeter Walls		Flexure	Ordinary	0.9	1.0
		Shear	Ordinary	0.9	1.0
Other walls (below and above grade)	$h_w/\ell_w < 2.0$	Shear	Critical	0.75	1.25
	$h_w/\ell_w \geq 2.0$	Shear	Critical	0.75	1.35
Core Walls		Shear	Critical	0.75	1.35
Diaphragm with Major Shear Transfer		Flexure	Critical***	0.9	1.0
		Shear	Critical	0.75	1.35
Typical (non-transfer slab) Diaphragm Forces (excludes collectors and shear transfer to vertical element)		Axial (includes chord forces)	Ordinary	0.9	1.0
		Flexure	Ordinary	0.9	1.0
		Shear	Ordinary	0.9	1.0
Drag (Collector) Members		Compression	Critical	0.65	1.0
		Tension	Critical	0.9	1.0
Vertical Element-to-Diaphragm Connection		Bearing	Critical	0.65	1.0
		Shear Transfer (Shear Friction)	Critical	0.75	1.0
Gravity Columns and Special Moment Frames (Beams, Columns, Beam-Column joints) excluding, Intentional Outrigger Columns, & Columns Supporting Discontinuous Vertical Elements)		Axial	Critical	0.65	1.0
		Shear	Critical	0.75	1.0
		Flexure (in Axial – Flexure Combinations)	Ordinary	0.9	1.0
Intentional Outrigger Columns & Columns Supporting Discontinuous Vertical Elements*		Axial	Critical	0.65	1.0
		Shear	Critical	0.75	1.0
		Flexure (in Axial – Flexure Combinations)	Ordinary	0.9	1.0
Transfer Girders*		Flexure	Critical	0.9	1.0
		Shear	Critical	0.75	1.0
Foundations		Flexure	Ordinary	0.9	1.0
		Shear (with A_v**)	Critical	0.75	1.35
		Shear (w/o A_v**)	Critical	0.75	1.00
Foundation Piles (Structural Capacity)		Compression	Critical	0.65	1.0
		Tension	Ordinary	0.9	1.0
		Flexure	Ordinary	0.9	1.0
		Shear	Critical	0.75	1.0

* Effects of vertical acceleration shall be considered.

** A_v = Shear reinforcement

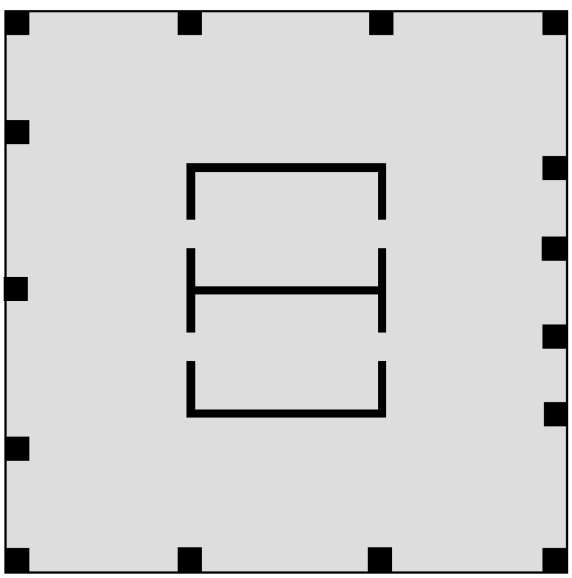
*** See the footnote on page 58 regarding different factors for application of Equations 5a to 5f for this case.

APPENDIX C – Outrigger Modeling

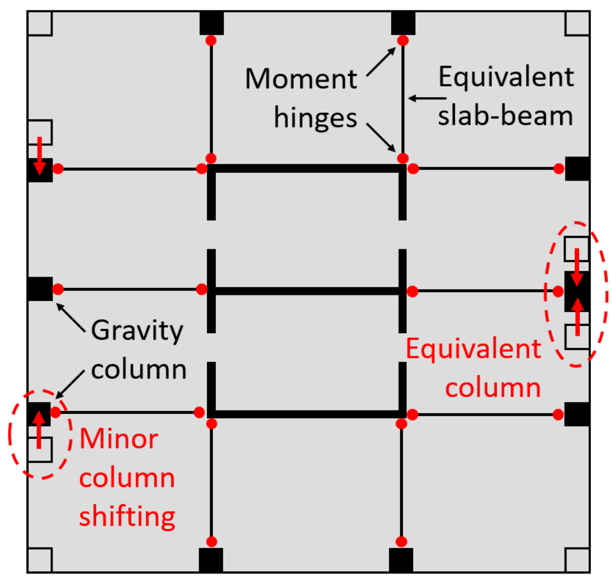
Coupling between the core wall and the gravity columns, or between two columns, shall be explicitly modeled in the nonlinear analysis using an equivalent slab-beam (outrigger beam) if any of the following two conditions apply:

1. Column-to-core distance is less than 20 feet.
2. Column-to-column distance is less than 10 feet.

The floor plan and model of a typical combined slab-column frame and core wall system are shown in Figure C-1 to demonstrate the acceptable configuration of outrigger beams. Minor shifting of columns, as well as combining two columns into one equivalent column, is acceptable in order to simplify model geometry.



a) Floor plan



b) Model

Figure C-1. Floor plan and model of the combined slab-column frame and core wall system.

Slab-column system (outrigger system) shall be modeled using elastic frame elements with moment hinges at both ends (Figure C-2). Effective stiffness of the frame elements shall be calculated per recommendations by Hwang and Moehle (2000). Figure C-3 illustrates the application of effective width model to the core wall. Moment hinge capacity shall be calculated considering reinforcement and post-tensioning (if applicable) within the effective slab width calculated based on Hwang and Moehle (2000) and shall consider effect of gravity load.

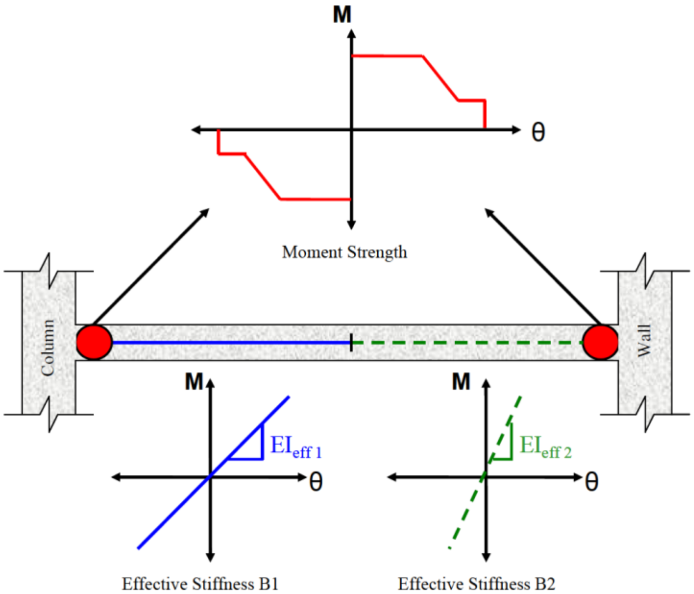


Figure C-2. Schematic of the slab model.

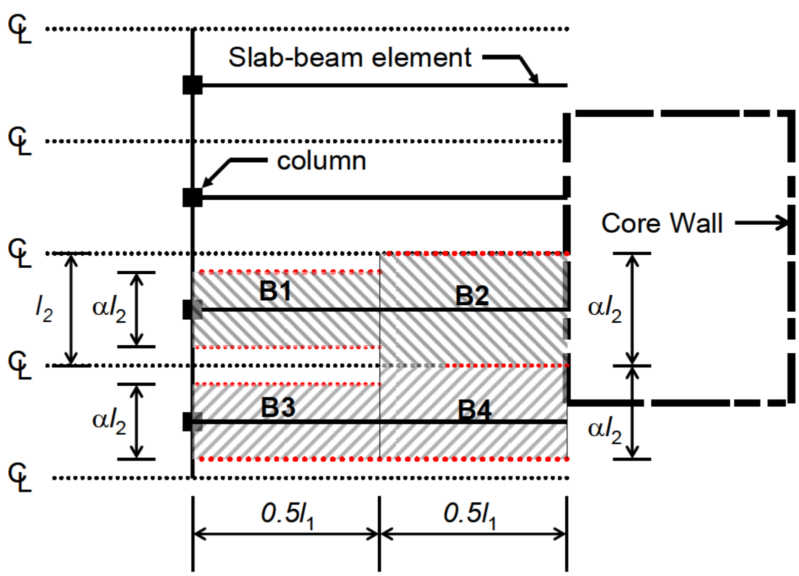


Figure C-3. Application of effective width model to core wall.

The following design checks shall be performed based on analysis results obtained from the model with slab-column outrigger system:

1. Slab rotations should satisfy rotations limits prescribed in Table 6-2.
2. Axial force in columns (force-controlled critical action) shall be less than axial capacity of the columns.
3. Shear force in columns (force-controlled critical action) shall be less than shear capacity of the columns.
4. Shear force in core walls (force-controlled critical action) shall be less than shear capacity of the core walls.

C.1. Axial Shortening of Columns in Outrigger Models.

A common issue in modeling of the outrigger system in the nonlinear analysis is the presence of rotations in the outrigger beams after the application of gravity loads. These rotations typically occur in analysis results due to differential shortening of the outrigger columns relative to the reinforced concrete core, and foundation settlement under gravity loads, which are not realistic behavior of the slab outrigger system because some differential deformations between the columns and the core are corrected during construction. Therefore, when deemed necessary, it is recommended to minimize the fictitious rotations of slab outriggers either in the modeling stage (e.g., by adjusting the axial stiffness of the outrigger columns or other reasonable means) or in the post-processing stage (e.g., by removing the unrealistic part of outrigger rotations) of nonlinear analysis.

APPENDIX D – Supplement to ACI 318-19

D.1. General

Recent changes adopted in ACI 318-19 have resulted in important issues that are being addressed by proposed changes to ACI 318-25 or future updates. The intent of this Appendix is to identify some of these important issues and whether they are likely to be addressed by updates to ACI 318-25 or a subsequent version (e.g., ACI 318-28), and to recommend approaches that can be used in the interim until these updates are adopted and published in ACI 318. Because ACI 318 code changes are not public documents, the approach used in this appendix is to present alternatives that are consistent with those being considered for adoption in 318-25. It is noted that some clarifications to ACI 318-19 were published in the June 2023 issue of *Concrete International* by ACI Committee 318 that also provide relevant information and that the version of 318-25 adopted by ACI Committee 318 will be available for public comment in early 2024.

As noted above, most of the issues identified in this supplement are currently being studied by ACI Committee 318 members and others and code changes to address these issues have been proposed and are being balloted for adoption in ACI 318-25 (or possibly subsequent versions). However, because adoption of ACI 318-25 in California and many areas will not occur until January 2029, it is essential that an alternative approach be provided to:

- Clarify the intent of the 318-19 provisions so that they are appropriately applied,
- Incorporate changes proposed for 318-25 that are expected to be approved,
- Identify alternative approaches that can be applied to reduce the economic impact of the new 318-19 provisions that do not reduce safety,
- Avoid drastic changes to construction practice in 2023 that would only be in effect until ACI 318-25 is approved and adopted.

The specific issues addressed in this supplement include: (D2.1) wall shear amplification, (D2.2) wall shear strength, (D2.3) specifying a limit of f_c' for calculation of wall shear strength, (D2.4) coupling beam demand redistribution, (D2.5) limits on penetrations in coupling beams, (D2.6) one-way shear strength for slabs, (D2.7) out-of-plane (one-way) shear strength for basement walls, (D2.8) wall detailing, and (D2.9) column development length requirements. The goal of this appendix is to supplement the ACI 318-19 provisions for code-based design of reinforced concrete buildings of all heights, with an emphasis on buildings with special structural walls, to produce buildings with predictable and safe performance when subjected to Design Earthquake ground motions as defined by the general building code. The application of the recommendations contained in this appendix are intended to be used in conjunction with ASCE 7-16 linear design approaches and ACI 318-19 and produce buildings that meet the target performance requirements of ASCE 7-16 without the need for independent peer review. However, in some cases, the information provided also applies to alternative design approaches based on nonlinear response history analysis (as outlined in this document and in ACI 318-19 Appendix A).

The supplement to ACI 318-19 contained in this appendix is based on a review of the ACI 318-19 provisions and relevant research that supported the changes adopted in ACI 318-19 and code changes approved for ACI 318-25, or a conservative assessment of the proposed changes being considered for adoption in 318-25 and available research.

This appendix does not include any modifications to the ASCE 7-16 (or ASCE 7-22) provisions. Supplements to ACI 318-19 are included in Section D.2. No other supplements beyond those specifically identified in Section D.2 are envisioned or allowed unless approved by the jurisdiction responsible for enforcing the general building code.

D.2. ACI 318-19 Supplement

Each of the subsections provided below are intended to supplement specific sections of ACI 318-19.

D.2.1. Section 18.10.3

The following items are expected to be clarified in ACI 318-25.

1. Design shear forces for horizontal wall segments, including coupling beams, are covered in 18.10.7. This update will clarify that the wall shear amplification approach in 318-19

18.10.3 does not apply to coupling beams. This issue also is clarified in the June 2023 Concrete International article.

2. Design shear forces for wall piers are covered in 18.10.8. This update will clarify that the wall shear amplification approach in 318-19 18.10.3 does not apply to wall piers. This issue also is clarified in the June 2023 Concrete International article.

3. For other cases, wall shear amplification of ACI 318-19 applies, but with the following changes, which were implemented to simplify the approach and to clarify various issues.

    a. If nonlinear analysis is used, ACI 318-19 Appendix A is used (and ASCE 7-16, Chapter 16), or alternative recommendations are used, such as contained in this document. This clarifies that shear amplification of 18.10.3 does not apply in this case, as shear amplification is directly considered if nonlinear analysis is used.

    b. If the wall factored shear force $\boldsymbol{V_{uEh}}$ is determined by linear analysis procedures of the general building code, then only the portion of the design shear force due to $\boldsymbol{E_h}$ is amplified by the product $\boldsymbol{\Omega_v \omega_v}$, which may be calculated or determined using Table 18.10.3.1.2, modified as:

Table D1 – Wall Shear Amplification

Condition	$\boldsymbol{\Omega_v}$	$\boldsymbol{\omega_v}$
$\boldsymbol{h_{wcs}/\ell_w} \leq 1.0$	1.0	1.0
$1.0 < \boldsymbol{h_{wcs}/\ell_w} < 2.0$	Linear interpolation permitted between 1.0 and 1.5	
$\boldsymbol{h_{wcs}/\ell_w} \geq 2.0$	1.5	$0.8 + 0.09h_n^{1/3}$

The modified approach clarifies that any wall shear due to gravity load cases does not need to be amplified and simplifies the determination of $\boldsymbol{\Omega_v \omega_v}$. The simplifications include that, although it will be allowed to calculate $\boldsymbol{\Omega_v}$ in 318-25, a value of 1.5 may be used without the need to do any calculations. One other simplification that may be adopted in 318-19 is related to the load combinations that are used if overstrength is calculated. In 318-19, consideration of load cases 5.3.1(e) and 5.3.1(g) of Table 5.3.1 is required whereas 318-25 may adopt use of a single load case, based on expected axial load, as defined in ASCE 7, Section 16.3.2.

The notation used in Table D1 is the same as used in ACI 318-19. The term h_n is defined in the June 2023 Concrete International article as:

> Where $\boldsymbol{h_n}$ is the structural height from the base to the highest level of the seismic force-resisting system of the structure, in ft, where the base is the level at which the horizontal seismic ground motions are considered to be imparted to the structure.

Also, it is expected that it will be permitted to take $\boldsymbol{\Omega_v \omega_v}$ equal to $\boldsymbol{\Omega_o}$, in which case, the redundancy factor of ASCE 7-16 may be taken equal to 1.0 for determination of $\boldsymbol{V_{uEh}}$. This change clarifies that $\boldsymbol{\Omega_v \omega_v}$ need not exceed $\boldsymbol{\Omega_o}$ and how the redundancy factor is treated. The treatment of the redundancy factor is clarified in the June 2023 CI issue, the limit of $\boldsymbol{\Omega_o}$ is likely to be adopted in ACI 318-25.

c. Section 21.2.4.1 should be modified to note that it does not apply if $\boldsymbol{\Omega_v} \geq 1.5$ in Section 18.10.3. This change clarifies that $\phi = 0.6$ does not apply when $\boldsymbol{\Omega_v} \geq 1.5$, because shear demand has been amplified to account for flexural overstrength. This change is also likely to be adopted in ACI 318-25.

D.2.2 ACI 318-19: Section 18.10.4

The following changes are being considered for ACI 318-25, but balloting and approval may not be completed in time for changes to appear in 318-25. Therefore, the recommendations summarized below are based on a conservative assessment of the available research (Rojas Leon, 2022; Rojas Leon et al., 2023a, 2023b).

In Section 18.10.4.4, for walls sharing a common lateral force, $\boldsymbol{V_n}$ shall not be taken greater than $8\sqrt{f_c'}A_{cv}$. For any one of the individual vertical wall segments, $\boldsymbol{V_n}$ shall not be taken greater than

$10\sqrt{f_c'}A_{cw}$, where $\boldsymbol{A_{cw}}$ is the area of concrete section of the individual vertical wall segment considered. These limits have been shown to be conservative for walls with compression flanges. Therefore, based on the study by Rojas-Leon et al. (2023b), the following is recommended. For all vertical wall segments sharing a common lateral force, $\boldsymbol{V_n}$ shall not be taken greater than the sum of $\alpha_{sh}\,8\sqrt{f_c'}A_{cv}$ for these wall segments, where α_{sh} for each vertical wall segment is determined as:

$$\alpha_{sh} = 0.7\left(1 + \frac{b_{fc}t_{fc}}{A_{cv}}\right)^2 \leq 1.2 \qquad \text{Eq. (D1)}$$

Where $b_{fc}$is the effective compression flange width determined according to 18.10.5.2. The value of α_{sh} in Eq. (D1) need not be taken less than 1.0. It is noted that use of α_{sh}= 1.0 for all case is acceptable. For any one of the individual vertical wall segments, V_n shall not be taken greater than $\alpha_{\mathrm{sh}}\ 10\sqrt{f_c'}A_{cw}$, where $\boldsymbol{A_{cw}}$ is the area of concrete section of the individual vertical wall segment considered.

Changes in the acceptance criteria for wall shear strength in since the 2014 version of the LATBSDC Guidelines are summarized below for expected material properties to clarify these changes.

LATBSDC 2014: $\phi V_{ne} \geq 1.5\bar{V}_u$
$\phi = 1.0$
V_{ne}= ACI 318-08 Equation (21-7) for expected material properties
$\bar{V}_u$ = mean of maximum absolute values for each ground motion

$(1.0)V_{ne} \geq 1.5\bar{V}_u$ $\qquad V_{ne} \geq 1.5\bar{V}_u$
$V_{ne} \leq 10\sqrt{f_{ce}'}A_{cv}$

LATBSDC 2020: $\phi BV_{ne} \geq 1.5\bar{V}_u$
$\phi = 0.75$
B = ACI 318-14 Equation (18.10.4.1) strength bias (=1.35)
V_{ne}= ACI 318-14 Equation (18.10.4.1) for expected material properties
$\bar{V}_u$= mean of maximum absolute values for each ground motion

$(0.75)(1.35)V_{ne} \geq 1.5\bar{V}_u$; $\qquad V_{ne} \geq 1.5\bar{V}_u$
$V_{ne} \leq 10\sqrt{f_{ce}'}A_{cv}$

LATBSDC 2023: $\phi BV_{ne} \geq 1.5\bar{V}_u$
B = ACI 318-19 Equation (18.10.4.1) strength bias (=1.35)

V_{ne}= ACI 318-19 Equation (18.10.4.1) for expected material properties
$\bar{V}_u$= mean of maximum absolute values for each ground motion

$(0.75)(1.35)V_{ne} \geq 1.5\bar{V}_u$; $V_{ne} \geq 1.5\bar{V}_u$
$V_{ne} \leq 10\sqrt{f'_{ce}}A_{cv}$ Rectangular wall cross sections
$V_{ne} \leq \alpha_{sh}10\sqrt{f'_{ce}}A_{cv}$ Flanged wall cross sections (max)

For vertical wall segments sharing a common lateral force, the limiting stress should be taken as an appropriate weighted average:

$$8\left(\sum_{i=1}^{n} \alpha_{sh,i}\, A_{cv,i}\sqrt{f'_{c,i}}\right) \quad \text{Eq. (D2)}$$

Where n is the number of walls resisting the shared common lateral force and $\alpha_{sh,i}$, $A_{cv,i}$, and $f'_{c,i}$ are the values for wall i, respectively.

References:

Rojas-Leon, M., 2022, “Framework to Set Model Performance Requirements Applied to the RC Wall Shear Strength Problem and Proposition of New Code-Oriented Equation,” Ph.D. Dissertation, University of California Los Angeles, Los Angeles, CA, 260 pp.

Rojas-Leon, M.; Abdullah, S.; Kolozvari, K.; and Wallace, J., 2023a, “Framework to Set Performance Requirements for Structural Component Models: Application to Reinforced Concrete Wall Shear Strength,” *ACI Structural Journal*, accepted for publication.

Rojas-Leon M.; Wallace JW; Abdullah SA; Kolozvari K (2023b), “New Equations to Estimate Reinforced Concrete Wall Shear Strength Derived from Machine Learning and Statistical Methods,” *ACI Structural Journal*, accepted for publication.

D.2.3.ACI 318-19: Section 18.10.4

Modify ACI 318-19 Section 18.10.4 to specify a limit on f'_c.

1. The value of f'_c in Section 18.10.4 shall be permitted to be taken equal to the value of f'_{ce}, and the value of f'_{ce} shall not be taken greater than 15,000 psi.

   This change is based on the review of test data by Rojas Leon (2022) and Rojas Leon et al. (2023b) that indicates that the strength bias B (see Appendix A and B of this Guideline) for the ACI 318-19 equation for wall shear strength tends to be greater as concrete strength increases from 10,000 psi and 15,000 psi then it is for values of concrete strength less than 10,000 psi. Although some wall tests were conducted that had concrete strength values greater than 15,000 psi, the data are limited; therefore, use of values greater than 15,000 psi is not recommended until additional data are available. It is noted that, in the study by Rojas-Leon (2022), the value of concrete strength was taken as average value of f_c' recorded from cylinder tests typically conducted at or near the date of the wall test was used. For this appendix, f_{ce}' should be based on the expected strength as defined in Table 2 of this Guideline and should be verified by project specific material testing.

D.2.4. ACI 318-19: Section 18.10.7

Add a new subsection to 18.10.7 that allows redistribution of coupling beam demands if certain conditions are satisfied. It is noted that this approach is common in current practice, but that it has not been adopted (or included) in ACI 318. The approach summarized below is expected to be approved in ACI 318-25.

1. Design shear force V_e of coupling beams shall be permitted to be redistributed to coupling beams in adjacent floors provided (a) through (d) are satisfied:
    a. Coupling beams sharing redistributed forces shall be vertically aligned within a special structural wall.
    b. Coupling beams sharing redistributed forces shall have $l_n/h \geq 2$.
    c. The maximum redistribution of V_e from any beam shall not exceed 20% of the value determined from analysis.
    d. The sum of ϕV_n of coupling beams sharing redistributed demands shall be equal to or greater than the sum of V_e in those beams.

   It is noted that, for coupling beams designed in accordance with 18.6, as allowed by 18.10.7.1 and 18.10.7.3, the redistribution of seismic beam moments in proportion to the redistributed shears is necessary to maintain internal equilibrium. In addition, redistribution should only be considered for vertically aligned beams near to each other (e.g., over a limited number of stories).

D.2.5. ACI 318-19: Section 18.10.7

Limitation on penetrations in diagonally reinforced special coupling beams will be included in ACI 318-25. The limitations that are likely to be adopted are summarized in the paper by Abduallah et al. (2023).

Reference:

Abdullah, Saman A; Rafiq, Serwan K; Fields, David; and Wallace, John W., “Seismic Performance of Diagonally Reinforced Concrete Coupling Beams with Penetrations,” ACI Structural Journal, V. 120, No. 1, January 2023. doi: 10.14359/51736118.

D.2.6. ACI 318-19: Section 8.5.3.1.1

This section requires a one-way shear check in accordance with Section 22.5. Changes were adopted in 22.5 to include a size effect ($\boldsymbol{\lambda_s}$), which reduces concrete shear strength if the slab thickness exceeds 10 inches, and to consider the longitudinal reinforcing ratio ($\boldsymbol{\rho_w}$) if minimum shear reinforcement is not provided. These changes were adopted based on a review of test data for one-way shear tests (e.g., beam and slab tests).

If this provision is applied to a two-way slab, e.g., a transfer (or podium) slab, in some cases, the concrete contribution to shear strength is about 50% of what it was for ACI 318-14. This can result in substantially thicker slabs for cases where significant out-of-plane loading exists (e.g., a wood building supported on a podium slab, which is very common in some areas of the US). However, there is no evidence that there are issues with gravity failures of two-way slabs based on a long history of buildings being constructed with provisions in ACI 318-14 and prior ACI 318 versions, which included identical shear strength equations going back many years.

It is noted that the size effect factor ($\boldsymbol{\lambda_s}$) is included in the two-way shear strength equations in Section 22.6.5.2. Although it is debatable whether it should be included for two-way slabs, this issue requires additional study. However, based on long standing practice for two-way podium slabs, which have been observed to perform adequately in a large number of buildings for many years, reducing concrete shear strength further based on longitudinal reinforcing ratio ($\boldsymbol{\rho_w}$) if minimum shear reinforcement is not provided, does not appear to be justified. Therefore, it is recommended to base the one-way shear check of 8.5.3.1.1 on Table 22.5.5.1, Equation (a), including the size effect factor ($\boldsymbol{\lambda_s}$), i.e.:

$$\boldsymbol{V_c = \left[2\lambda\lambda_s\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d}$$

This supplement is intended to apply only to slabs with substantial out-of-plane loading (e.g., as noted above). For a more typical podium slab for a tall building, out-of-plane slab loading should be relatively modest such that the one-way shear check of ACI 318-19 Section 8.5.3.1.1 does not control.

D.2.7. ACI 318-19: Section 11.5.5.1

This section requires a one-way shear check in accordance with Section 22.5 for out-of-plane loads on walls. Similar to Section D2.6, the changes in ACI 318-19 either require a significantly thicker basement wall or a significant increase in (shear) reinforcement for the same wall thickness. Basement wall out-of-plane loading is typically governed by gravity (or soil) loading and there is a long history of adequate performance of basement walls (i.e., no observed shear failures). Therefore, it is recommended to base the one-way shear check of 11.5.5.1 on Table 22.5.5.1, Equation (a), excluding the size effect factor ($\boldsymbol{\lambda_s}$), i.e.:

$$V_c = \left[2\lambda\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d$$

D.2.8. ACI 318 Section 18.10.6.4(f)

The response published in the June 2023 issue of Concrete International clarifies that overlapping hoops are not required at locations where a wall web and flange intersect. Therefore, overlapping hoops may not be required for core walls to meet the detailing requirements for a special wall in 18.10.6.4.

It is also noted that a code change proposal is likely to be approved for ACI 318-25 to modify 1810.6.4(f) to only require overlapping hoops if $b < \sqrt{c\ell_w/40}$ and $\boldsymbol{\delta_u/h_{wcs}}$ > 0.012 (also see 1810.6.2) based on the study by Abdullah and Wallace (2019; 2020).

References:

Abdullah S and Wallace JW (2019), "Drift Capacity of RC Structural Walls with Special Boundary Elements," ACI Structural Journal, 116(1), pp 183-194. doi: 10.14359/51710864

Abdullah S and Wallace JW (2020), A Reliability-Based Design Methodology for RC Structural Walls with Special Boundary Elements," ACI Structural Journal, 117(3), 14 pp. doi: 10.14359/51721375

D.2.9. ACI 318-19 Section 18.7.4.3

Changes were adopted in ACI 318-19 to require that: "*Over the column height, longitudinal reinforcement shall be selected such that* $1.25l_d \leq l_u/2$." This requirement is intended to limit bond splitting failures. Recent evaluation of this requirement indicates that it is impractical and will significantly increase the cost of moment frame buildings and result in rebar congestion (a very large number of smaller diameter columns bars). The 1.25 multiplier on l_d is typically used (e.g., see 18.10.2.3(b)) if significant yielding is expected in the longitudinal bars, which is generally not the case for moment frame columns, due to the requirements of ACI 318-19 Section 18.7.3. In addition, it is not clear that available column tests data for special moment frame columns indicates that columns that do not satisfy this requirement have lower deformation capacity at strength loss than columns that do satisfy this requirement. For example, does the K_{tr}expression in ACI 318-19 Equation 25.4.2.4(b) adequately represent the conditions that exist for special moment frame columns that have substantial transverse reinforcement over the entire column height. Until additional studies are undertaken to address these issues, it is recommended that (1) this provision be ignored, or (2) the provision be modified to require $l_d \leq l_u/2$ over the column height.

Reference:

ACI 318 (2019). Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary, American Concrete Institute, Farmington Hills, MI.

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