SEISMIC DETAILING OF CAST-IN-PLACE CONCRETE GRAVITY FRAMING

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1. INTRODUCTION

Building structures generally comprise a three-dimensional framework of structural elements configured to support gravity and lateral loads. Although the complete three-dimensional system acts integrally to resist loads, structural engineers commonly conceive of both the gravity and lateral force-resisting systems as being composed of vertical elements, horizontal elements, and the foundation. The applicable building codes permit separate design considerations with respect to gravity and lateral force-resistance systems. This Guide is written to describe the use, analysis, design, and construction of cast-in-place reinforced concrete gravity framing elements. Specifically, this Guide will focus on columns, beams, slab-column joints, and slab-wall framing considered exclusively as part of the gravity-resisting system. The Guide focuses on the analysis and design of non-prestressed elements with limited commentary with regards to prestressed gravity framing elements. Finally, this Guide is accompanied by a Design Example to further illustrate the design procedure for gravity framing.

![Figure 1-1 Basic building structural system (Moehle et al., 2016)](image)

Gravity framing members are designed to support gravity loads and the load effects of vertical ground motion, while subjected to the lateral design displacement. While gravity framing elements are not considered as contributing to the lateral force resistance of a structure, they are subjected to displacements induced by lateral loads. Additionally, the modelling of gravity framing elements may provide more comprehensive representation of a structure’s behaviour and their inclusion is common practice in the creation of linear and nonlinear models.
The recognition of gravity framing as separate from the lateral-force-resisting system has several benefits including potential economic savings and increasing the inherent redundancy of lateral-force resistance. However, code provisions for gravity framing are frequently overlooked or misinterpreted in design, which can result in significant structural damage.

The design requirements for gravity framing elements—referred to as “members not designated as part of the seismic-force-resisting system”—are presented in the American Concrete Institute 318, Building Code Requirements for Structural Concrete (ACI 318 §18.14, 2019). The requirements relate to materials, strength, detailing, and construction inspection for diaphragms in any building in addition to requirements for buildings assigned to Seismic Design Category D, E, or F per the Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7-16) (ASCE 2016).

This Guide follows the 2019 edition of ACI 318, along with the pertinent requirements of the International Building Code (IBC) (IBC 2021). Additionally, this Guide utilizes ASCE 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures and ASCE 41-17 Seismic Evaluation and Retrofit of Existing Buildings (ASCE 2016, ASCE 2017). This Guide also draws from Seismic Design of Reinforced Concrete Building (Moehle, 2015). All combined, these texts contain the latest information on design of cast-in-place gravity framing elements at the time of writing. Because these editions may not yet be adopted in many jurisdictions, not all provisions described herein will necessarily apply in every area.
2. The Roles of Gravity Framing

2.1. HISTORIC DEVELOPMENT AND OBSERVED PERFORMANCE

While the practice of assigning all seismic forces to limited parts of the structure existed earlier within the United States, the formal recognition of gravity framing as separate from a structure’s lateral-force-resisting system occurred in 1967 as seen in the Uniform Building Code (UBC, 1967). Detailing requirements of gravity framing were introduced to ACI 318 in 1983. Notable updates to gravity detailing requirements of ACI 318 occurred in the 1995 and 2005. The former created more stringent requirements due to the poor performance observed in some gravity frames after the 1994 Northridge earthquake and the latter introduced provisions for slab-column frames.

The discussion and detailing for gravity framing focuses on preventing previously observed failures. Noting instances of poor performance guides the development of more comprehensive design.

The Kaiser parking garage located in Los Angeles, CA experienced a total collapse during the 1994 Northridge earthquake. The lateral-force-resisting system was core shear walls with perimeter moment frames. The failure was traced to the loss of axial capacity in interior gravity columns while undergoing earthquake-induced displacements. The failure of interior columns caused a progressive collapse when slabs then pulled inward on the exterior frames. The resulting failure was an inward collapse as opposed to the expected sidesway mechanism. This failure represents the need to assess a structure’s expected drifts as an input to confinement detailing of gravity columns.

Also from the 1994 Northridge earthquake, both the Bullock department store and the Four Seasons office building demonstrated punching failures between gravity columns and their respective flooring systems. This failure type highlights the column-slab connection detailing of gravity framing. Seismically induced punching shear failure indicates the slab-column connection reinforcement was not adequate for the rotational demands induced by building drift.
2.2. GRAVITY FRAMING BENEFITS AND APPLICATIONS

As previously discussed, gravity framing has been an explicit part of structural design for several decades. Gravity framing is found in structures of a wide variety of shapes and sizes. The complementary lateral-force-resisting system varies from moment-resisting frames to shear walls to hybrid systems. The delineation between the lateral-force-resisting system and gravity framing has several notable benefits. These advantages have led gravity framing to be a common aspect of building design in the United States.

First, in high seismic regions, the two-system approach can be economically appealing. The design and construction of seismic systems is intensive in terms of knowledge, skillset, and material. Gravity-only components generally have lower design forces, which result in a lighter reinforcement schedule. The typically lighter reinforcement scheduling for gravity-only components increases the ease of construction onsite. This is especially relevant for connections between elements, which can exhibit high levels of congestion making both the placement of reinforcement and the subsequent pouring of concrete laborious. While a two-system approach does not guarantee lighter reinforcement schedules—the detailing of gravity-only components is ultimately a function of load requirements and expected displacements—it is likely for typical structures.

Second, assigning specific areas of the structure for lateral resistance can facilitate a more open floor plan resulting in an aesthetic benefit. This is especially true in high rise buildings, which increasingly feature a shear-wall core with perimeter gravity columns—a configuration that often reduces the amount or sizing of perimeter elements thereby improving building views.

Finally, separating the lateral-force-resisting system from gravity framing may improve the building performance. Gravity framing increases the inherent redundancy of a structure’s lateral-force-resistance capability upon being built. While gravity framing is not considered to contribute to lateral resistance in analysis, it will ultimately increase the stiffness of the built structure. Separating the
lateral-force-resisting system from gravity framing can improve the performance of the building beyond increasing stiffness. A specific example occurs in the use moment-resisting frames along the perimeter. In this case, excluding the corner columns from the moment resisting frames—consider them gravity-only elements—helps to reduce biaxial bending and overturning actions at the corners.
3. Gravity Framing Components and Principles

3.1. COMPONENTS AND CONFIGURATION

Gravity framing consists of columns, beams, and connection joints that are not part of the lateral-force-resisting system. These components are intended to support gravity loads defined within ASCE 7-16, *Minimum Design Loads and Associated Criteria for Buildings* (ASCE 2016). Additionally, gravity framing should consider ultimate loads calculated per load combinations related to earthquake loads. Particularly in exterior columns, axial loads can increase substantially during a seismic event. As explained in ACI 318 §15.2, if lateral forces cause transfer of moment at beam-column/slab-column joints, the shear resulting from moment transfer shall be considered. The detailing required to resist such forces must be accommodated by properly sized gravity framing components.

A typical configuration for the gravity framing of a structure is columns supporting a flat slab. The span between columns can vary significantly and depends largely on architectural intent. Beams may or may not be necessary to transfer slab forces to columns. For gravity framing, the relative strength between column and beam or column and slab is a lesser concern than when considering connections within the lateral-force-resisting system where hinging in horizontal elements is much preferred to hinging in vertical components. However, gravity columns proportioned to be stronger than inframing beams or slabs is generally preferred as this configuration represents a residual margin against collapse. Plastic behaviour in gravity columns is acceptable so much as strength reduction in columns has only a localized effect on performance and does not compromise vertical load carrying capacity.

In a finite element model (i.e. SAP, ETABS), gravity loads can be distributed based on element stiffness. Tributary area is another way to distribute loads. In addition to gravity loads, it is important to consider the drift-induced forces within the gravity framing system. While gravity framing is not intended for lateral resistance at a global level, high forces caused by induced drifts and rotations may
result in forces in individual components beyond the originally conceived gravity loads. Failure to appropriately detail for seismically induced forces and displacements in gravity framing is the root cause of the previously discussed structural failures.

3.2. DESIGN PRINCIPLES FOR INTENDED BEHAVIOR

An important step in the design process is to assign elements as deformation-controlled or force-controlled. While this delineation is only required for nonlinear design and analysis, it remains a beneficial exercise for other code-based analyses. The intent of U.S. building codes is that significant inelastic response will be limited to framing elements of the seismic force-resisting system. These elements are designed to have a ductile response and detailed to control hinge formation locations. Conversely, diaphragms, foundations, and connections are intended to have a dominantly elastic response during a seismic event. Largely absent in current literature is clear and detailed recommendations on what the performance intent of reinforced-concrete gravity framing components should be and how to design these elements to achieve it. This Guide intends to provide a basis of performance intent and design recommendations for gravity framing.

The primary concern for gravity framing is damage that leads to the loss of component gravity load carrying capacity. Especially in gravity columns, loss of axial-force capacity can result in a progressive collapse of the structure. An additional concern for gravity framing is shear failure at connection locations. In the case of slab-column connections, rotational demands caused by building drift can result in a punching shear failure. This failure mode can result in a progressive collapse similar to when gravity columns fail axially. Therefore, the three following principles act to guide gravity framing design:

1. Control deformation demands on gravity framing.
2. Confine elements where yielding is expected.
3. Avoid shear and axial failures.
3.2.1. CONTROL DEFORMATION DEMANDS

Failures in gravity framing are predominantly related to high lateral drifts during a seismic event. Large drifts and the subsequent element rotations lead to high shear forces, spalling, and other inelastic behaviour. Where drifts are minimal, failure in gravity framing is rare. Therefore, designing to either limit the drifts or designing gravity framing with consideration to drift is crucial. Examples of building systems where drift could have a more pronounced effect include frame systems which tend to have high flexibility and tall buildings that exhibit stories with large inter-story drifts at higher modes. Additionally, drift demands can be amplified at the interface of structural components with non-structural components like stairways and car ramps.

3.2.2. CONFINE ELEMENTS WHERE YIELDING IS EXPECTED

Gravity columns are often weak relative to the flooring system framing into them. The weak-column strong-slab configuration results in column rotations. These rotations accommodate the building drift and can lead to inelastic behaviour especially at the column ends. Therefore, it is important to provide confinement to enhance column ductility at these locations. Confinement is also required where splicing occurs. As detailed later, if gravity columns are expected to undergo minimal drift, evenly spaced transverse reinforcement without special consideration to column ends or splices is acceptable. Additionally, in situations where the column is large compared to the surrounding flooring system, rotations would occur in the flooring system and connection reinforcement becomes critical in preventing shear punching failure.

3.2.3. AVOID SHEAR AND AXIAL FAILURE

The primary objective of confinement is to ensure a more ductile response that prevents shear or axial failure in columns. In addition to achieving greater ductility, transverse reinforcement acts as the direct resistance to shear failure. It is recommended to design transverse reinforcement to achieve shear strengths corresponding the probable moments developed in the columns, where probable moment
strength is typically calculated from conventional flexural theory in which reinforcement yielding strength is $1.25f_y$. Shear strength in beams and slabs should exceed the shear demands induced by lateral drifts.

The detailing of slab-column connections should carefully consider the potential of punching failure. Bottom reinforcement for the slab should pass through the column cage or be fully developed within exterior columns. As shown in Figure 3-1, if the slab-column connection fails, progressive collapse can be guarded against by ensuring reinforcement remains intact to support the slab via catenary action. Notably, top reinforcement is less effective in this situation due to spalling of the concrete cover as shown in Figure 3-1.

![Figure 3-1: Punching failure diagram (After Pan and Moehle, 1992, courtesy of American Concrete Institute)](image)
4. Building Analysis Guidance for Gravity Framing

4.1. GRAVITY FRAME MODELING OPTIONS

When the lateral-force-resisting system is assigned to specific building segments, the entire structure’s design capability to resist lateral loads should be limited to those areas in the model. This is captured in the model by “disabling” the lateral resistance of gravity framing when analyzing the response to lateral loads. Within the context of a linear model undergoing a linear analysis, there are two primary ways to build the structure within a three-dimensional model. First is to model the lateral-force-resisting system and gravity framing with separated vertical elements and connect them via pin-pin beams at each floor level. Second is to model the entire structure as architecturally intended and to modify gravity framing properties when running the model for lateral-load resistance. These approaches are meant for extracting design loads and drifts for linear models undergoing elastic analysis. These results are then modified for nonlinear response by code recommended factors (i.e. deflection amplification factor, $C_d$). If the building is designed using performance-based design, a nonlinear model and analysis should be conducted. Additionally, if investigating the true response of a structure, the model should be built and analysed without these isolation techniques.

4.1.1. SINGLE MODEL APPROACH FOR LATERAL ANALYSIS

The procedure of analyzing a single model in the determination of design forces and drifts can vary significantly between engineers being largely dependent on building intent or designer preference. In addition to design loads, the primary concern of the analysis—with respect to gravity framing—is drift magnitude. In pursuit of drift values, there are a variety of ways to isolate the lateral-force-resisting system within a model. To model this separation, gravity beam connections, base connections of gravity columns, stiffness coefficients of the gravity system, and stiffness coefficients of the lateral-force-resisting system must be considered.
Typically, gravity beams are modeled as pin-pin connections. This is important to isolate the lateral-force-resisting system because a rigid connection between beams and columns would create unintended moment frames thereby reducing forces on lateral force-resisting system and the extracted drift values. The alternative approach is to model gravity connections as fixed and for determining the forces in lateral force-resisting elements, reduce the flexural stiffness of gravity elements close to 0.

It is important to realize that the intent of both of these approaches is to minimize contribution of gravity elements to lateral resistance. However, for determining the seismic induced forces in the gravity elements, the gravity system should be modelled with its true connectivity and stiffness.

When modeling base connections of gravity columns, drift and base shear forces must both be adequately considered. Modeling gravity framing base connections as pinned would result in more conservative design drift values. However, the reality of the gravity column within the structure should not be neglected in the decision-making process. RC connections are dominantly moment connections. By modeling columns as pinned, shear and moment values at the column base might be underrepresented.

Recommendations for modelling assumptions using second approach with low stiffness of gravity elements are summarized in Table 4-1.

<table>
<thead>
<tr>
<th>Elements</th>
<th>Lateral force-resisting system</th>
<th>Gravity system</th>
</tr>
</thead>
<tbody>
<tr>
<td>LRFS stiffness</td>
<td>Effective flexural stiffness</td>
<td>Effective flexural stiffness</td>
</tr>
<tr>
<td>LRFS boundary conditions</td>
<td>Fixed</td>
<td>Fixed</td>
</tr>
<tr>
<td>Gravity stiffness</td>
<td>Zeroed (≤0.1I₉) flexural stiffness&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Effective flexural stiffness</td>
</tr>
<tr>
<td>Gravity boundary conditions</td>
<td>Fixed</td>
<td>Fixed</td>
</tr>
<tr>
<td>Gravity base conditions&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Pinned</td>
<td>Fixed</td>
</tr>
<tr>
<td>Force level</td>
<td>Strength (divided by R)</td>
<td>Displacement (Strength x C₄)</td>
</tr>
</tbody>
</table>

<sup>1</sup> I₉ = 1.33I₀

<sup>2</sup> I₀ = I₉ + I₈
Engineer should exercise their judgement. For stiff structures such as walls, the stiffness may not need to be lowered as the relative stiffness difference between lateral and gravity system is large.

These are general recommendations and may vary for specific configuration.

4.2. ANALYSIS PROCEDURES

The previous discussion focused on modelling buildings within the context of a linear model. This approach is satisfactory for most building structures. Different analysis procedures are permitted by ASCE 7 and some are required for certain building designs. ASCE 7 Table 12.6-1 outlines the permissibility of each with respect to structural characteristics. Each analysis procedure is briefly discussed predominantly with respect to expected drifts because expected drift constitutes a primary parameter by which gravity framing reinforcement is designed.

1. Equivalent Lateral Force Analysis (ELF) (ASCE 7 §12.8)
2. Modal Response Spectrum Analysis (MRSA) (ASCE 7 §12.9.1)
3. Linear Response History Analysis (LRHA) (ASCE 7 §12.9.2)
4. Nonlinear Response History Analysis (ASCE 7 §16)

4.2.1. LINEAR STRUCTURAL MODEL AND ANALYSIS

The ELF, MRSA, and LRHA are elastic analysis procedures that are based on seismic forces obtained via a design response spectrum as defined in ASCE 7-16 §12.6-12.9. These forces are input into a mathematical model of the structural system to determine drift values, which are factored to determine design drift values per ASCE 7-16 §12.8.6. Drift values are factored by coefficients \( C_d/I_e \), where \( C_d \) is the deflection amplification factor found in ASCE 7 Table 12.2-1 and \( I_e \) is the importance factor per ASCE 7-16 §11.5.1. Design forces and drifts are further modified to account, when necessary, for torsion and P-delta effects. Requirements for torsion and P-delta can be found in ASCE 7-16 §12.8.4 and §12.8.4, respectively. These amplifications are the same for equivalent lateral force design, modal response spectrum design, and linear response history design.
Determining the building base shear is highly dependent on the structure’s period. For ELF analysis, the approximate fundamental period or a period via modal analysis may be used. As period lengthens, a decrease in associated peak response for calculations related to base shear occurs. Therefore, an upper limit on the period \((C_uT_a)\) is considered. This becomes especially relevant in tall structures. ASCE 7 §12.7-§12.8 provides guidance for determining base shear calculations with respect to period length.

The ELF procedure is well-known and simple to implement. However, ASCE 7 limits its use to largely regular structures that align with the procedure’s inherent assumptions. For example, ELF procedures are not permitted for long period structures or structures with certain irregularities. Conversely, ASCE 7 places no explicit limits on the use of the MRSA procedure, which—with current software capabilities—requires minimal additional time to conduct.

4.2.2. NON-LINEAR STRUCTURAL MODEL AND ANALYSIS

For structural design in which prescriptive building code recommendations are not satisfactory, a nonlinear model and analysis may provide a viable alternative. It is important to recognize that nonlinear analysis requires more effort and technical expertise. Additionally, it is an approach that is rapidly evolving. As described in NEHRP Seismic Design Technical Brief No. 4, “Typical instances where nonlinear analysis is applied in structural earthquake engineering practice are to: (1) assess and design seismic retrofit solutions for existing buildings; (2) design new buildings that employ structural materials, systems, or other features that do not conform to current building code requirements; (3) assess the performance of buildings for specific owner/stakeholder requirements” (Deierlein et al., 2010). Gravity framing design would likely not provide a foundational reason to conduct a nonlinear analysis. However, a nonlinear analysis generally provides a more comprehensive understanding of how a structure responds to ground shaking, allowing increased confidence in assigning the appropriate reinforcement schedules for the structure’s gravity framing element.
4.3. **STIFFNESS RECOMMENDATIONS**

Modeling software packages typically allow the designer to modify material properties. These modifications can be used to control element stiffnesses. As discussed in Section 4.1, the modification of stiffness properties is an important step in extracting information (e.g. drift values) from a linear model. Stiffness is a critical input parameter because it heavily influences the building period, base shear, story drift, and internal force distribution. The safety of the structural design requires the designer to consider an envelope of responses, many of which respond directly to element stiffness.

ACI 318 Tables 6.6.3.1.1 (a) and (b) outline the recommended stiffness coefficients as they relate to element type. In these tables, the moment of inertia is adjusted for elastic analysis at factored load level. For beams and columns this factor ranges from 0.25-0.50 and 0.35-0.875, respectively. With respect to punching failures in slabs, careful consideration is required when assigning property modifications for slab elements. For conventionally reinforced concrete slabs, Hwang and Moehle (2000b) determined the maximum effective stiffness modification factor can be approximated per Equation 4-1, for slabs in which \( c_1 = c_2, l_1 = l_2 = 1 \). Here, \( c_1 \) and \( c_2 \) are column cross-section dimensions, \( l_1 \) is the column-to-column span by which effective width is being determined, and \( l_2 \) is the column-to-column span running perpendicular. For prestressed slabs, a factor of 0.50 is appropriate because prestressed strands reduce cracking (Kang and Wallace, 2005; Elwood et. al., 2007).

\[
\beta_{cr} = 4 \frac{c}{l} \geq \frac{1}{3} \quad \text{(Equation 4-1)}
\]

The stiffness adjustments per ACI 318 Tables 6.6.3.1.1 relate to determining design forces. However, when analyzing for design drifts, the stiffnesses of gravity framing should not contribute to the global stiffness of the structure. As discussed, the structure should comply with maximum drift regulations without stiffness contribution from gravity framing. This can be ensured by minimizing the stiffness of gravity framing. Equating the multiplier to zero may result in numerical errors. Therefore, when determining design drifts, a multiplier of 0.01-0.05 for gravity framing components can be used. Unless coefficients are built into the software, the extracted design drifts should be multiplied by \( C_d/I_e \).
When analysing gravity framing, the stiffness of gravity elements is considered in the model and will decrease the calculated drifts. Engineers should use judgement on whether an additional scale factor beyond $C_d$ is needed when determining induced forces on gravity framing.
5. Gravity Framing Strength and Design Guidance

5.1. LOAD COMBINATIONS AND REDUCTION FACTORS

The load combination requirements for gravity framing systems can be found in ASCE 7, Chapter 2. While gravity framing is not part of the lateral-force-resisting system, it still must maintain its integrity under earthquake loading. Therefore, in addition to the standard load combinations for gravity loads per ASCE 7-16 §2.3.1, the strength of gravity framing elements must also be checked for gravity loads defined as part of seismic load combinations:

\[ 1.2D + E_v \pm E_h + 1.0L \]

\[ 0.9D - E_v \pm E_h \]

ASCE 7 §12.4.2 defines the horizontal seismic effect as \( E_h = \rho Q_E \) and the vertical seismic effect as \( E_v = 0.2S_D S_D \) where \( E_h \) is to be applied in both positive and negative directions and \( E_v \) is to contribute to gravity as shown. The live load factor is permitted to equal 0.5 when unreduced design live load is less than or equal to 100 psf, with the exception of garages or areas designated for public assembly.

During a seismic event, gravity framing will experience induced moment and shear forces. To ensure the integrity of gravity framing members during ground shaking, ACI 318-19 §18.14 permits two options. First, the engineer can neglect the calculation of induced forces and opt to design gravity framing elements more conservatively per ACI 318-19 §18.14.3.3. Second, the engineer can calculate induced forces (force demands under design drifts) then compare these forces to the capacity of respective gravity framing elements. If induced forces exceed capacity, then §18.14.3.3 is again followed. If induced forces are less than capacity, then §18.14.3.2 standards are permitted. The design differences are detailed in the following sections.

As shown in ACI 318 Table 21.2.1, strength reduction factors (\( \Phi \)) are dependent upon the action and/or structural element type. In general, \( \Phi = 0.75 \) for shear actions. Per ACI 318-19 Table 21.2.2, the \( \Phi \)
factor ranges for moment and axial forces depending on whether the section is compression- or tension-controlled. A comparatively lower factor is used for compression-controlled elements because these sections have less ductility and are more sensitive to variation in concrete strength (ACI 318 §R21.2.2, 2019).

5.2. COMPONENT STRENGTH AND DEFORMATION

Designing components for both gravity framing and lateral force resistance is aided by first identifying components as force-controlled actions or deformation-controlled actions. Force-controlled actions are meant to remain within a dominantly elastic zone during a seismic event. Force-controlled actions can be further delineated based upon failure significance: (1) critical action, (2) ordinary action, (3) noncritical action. Conversely, deformation-controlled actions are intended to have an inelastic response during a seismic event as to create sacrificial components whose potential failure would be relatively inconsequential to the structure. Good practice is to identify components as force- or action-controlled and provide table similar to Table 5-1 in the design.

Table 5-1: Force-controlled vs. deformation-controlled actions for gravity framing

<table>
<thead>
<tr>
<th>Gravity Framing Components</th>
<th>Force-Controlled Action</th>
<th>Deformation-Controlled Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial in columns and beams</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Shear in beams, columns, beam-column connections, slab-column</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>connections, slab-column connections, slab-wall connections</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flexure in beams, beam-column joints slab-column connections,</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>slab-wall connections</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

As shown in Table 5-1, axial and shear of gravity framing elements are defined as force-controlled actions, whereas flexure in these elements is typically defined as a deformation-controlled action. It is
especially important to consider where the inelastic action is occurring for gravity columns and slab-column connections. For the purposes of reinforced concrete design for seismic resistance, inelastic action is typically the consequence of lateral drifts inducing larger moments and shears. The expectation of inelastic action indicates where additional detailing provisions may be required.

5.3. GRAVITY COLUMN DESIGN

Detailing requirements for gravity framing columns can be found in ACI 318 §18.14.2. Gravity columns must be designed for axial, shear, and flexural forces due to gravity loads. Although lateral force resistance is not the function of gravity framing, these elements must still be able to withstand earthquake displacements during a seismic event. Adequate detailing can be achieved without determining design displacements as per ACI 318 §18.14.3.3. Alternatively, the reduced reinforcement detailing of ACI §18.14.3.2 requires that displacement induced forces do not exceed the capacities of the element. This requires the determination of design displacement ($\delta_u$).

The studied failures of Section 2.1 occurred largely due to misjudgement of whether the gravity columns will undergo inelastic response. Because main focus of seismic design is lateral system, the imposed displacements on gravity columns might be overlooked. As stated, ACI 318 permits this omission if gravity framing elements are then designed per ACI §18.14.3.3. However, when a gravity column is not detailed to exhibit proper confinement and shear strength during a seismic event, it can lose capacity resulting in its failure and the successive failure of surrounding elements. This constitutes a major reason why determining design displacements is a critical part of the gravity framing design process per ACI §18.14.3.2.

*Figure 5-1* and *Figure 5-2* provide a summary of ACI code provisions for RC gravity columns with respect to seismic detailing per ACI 318-19 §18.7.5.2. *Figure 5-1* provides guidance to cross-section requirements as they relate to axial load and concrete strength, whereas *Figure 5-2* details requirements along the length of the column. Where induced moments and shears exceed the corresponding design
strengths or if induced moments and shears are not calculated, column design requirements given in (b) apply.

Where induced moments and shears do not exceed the design moment and shears, the design requirements of (a) suffice. In both cases, shear strength must satisfy $\Phi V_n \geq V_e$, where shear force $V_e$ is determined using the maximum probable moment strengths at column ends (but can be limited by maximum probably moment of in-framing elements) and $V_n$ is the nominal shear strength of the gravity column. Several additional design requirements are emphasized in Box 1.
- Longitudinal reinforcement satisfies $0.01 \leq A_{st}/A_g \leq 0.06$ [18.7.4.1]
- Transverse reinforcement is spirals, circular hoops, or rectilinear hoops and crossties, designed to resist shear corresponding to $M_p$ and considers $P_u$ [18.7.6]
- Transverse reinforcement to be designed such that shear strength of concrete is considered zero ($V_c = 0$) when [18.7.6.2.1]:
  a. Earthquake-induced shear force: $V_u \geq \frac{1}{2} Vn$ within distance, $l_o$
  b. Factored axial compressive force including earthquake effects: $P_u \leq \frac{A_g f'_c}{20}$
- Rectilinear hoops and crossties [18.7.5.2]:
  a. Must engage at least corner and alternate longitudinal bars.
  b. Maximum longitudinal spacing between column transverse reinforcement: $s_{max} = \min(6d_b, 6 \text{ in.})$.
  c. If $P_u \geq 0.3A_g f'_c$ or $f'_c > 10,000 \text{ psi}$:
     - $h_x \leq 8 \text{ in.}$ on center, every longitudinal bar along perimeter is supported, and both tie-ends bent at 135 degrees (b)
  d. If $P_u < 0.3A_g f'_c$ and $f'_c \leq 10,000 \text{ psi}$:
     - $h_x \leq 14 \text{ in.}$ on center and alternating tie-ends of 90 degrees and 135 degrees is permissible (a)
  e. Diameter of crossties [25.7.2.2]:
     - No. 3 if enclosing longitudinal bars of $\leq$ No.10.
     - No. 4 if enclosing longitudinal bars of $\geq$ No.11

**Box 1: Gravity column detailing requirements**

**Figure 5-1**: Cross-section requirements for gravity columns
Figure 5-2: Requirements for gravity columns based on displacement induced moment and shear demands (Moehle, 2015)
5.4. GRAVITY BEAMS

Similar to gravity columns, the detailing of gravity beams is related to whether moments and shears induced by $\delta_n$ in combination with factored gravity loads exceed design capacities. Additionally, gravity beams are generally less critical to global stability because hinge formation within a beam element does not have the same potential to cause a story mechanism as with columns. Therefore, there are few differences in the detailing of gravity beams when demands do not exceed design strengths versus when demands do exceed design strengths. These differences are shown in Figure 5-3(a) and Figure 5-3(b), respectively.

If induced demands are checked and found to exceed the capacity of the beam element, the gravity beam shear strength must satisfy $\Phi V_n \geq V_e$ where $V_e = 0$ along length $l_0$ when same conditions as for columns occur (see Box 1). $V_e$ is calculated using the probable moment strengths at the ends of the element. Additionally, beams in which seismically induced demand exceeds design strengths shall use a Type 2 mechanical splice when splicing occurs within twice the beam depth from the column face.

![Figure 5-3: (a) Requirements for gravity beams based on displacement induced moment and shear demands (Moehle, 2015)](image-url)
5.5. GRAVITY FRAMING CONNECTIONS

A typical lateral-force-resisting system features strong-column weak-beam design. This configuration encourages beam mechanisms as opposed to a single-story or intermediate mechanism in which plastic hinges form in columns. Gravity framing is not intended to transfer lateral loads down to foundations; it simply must be capable of withstanding lateral forces during ground shaking. As a result, hinging may occur in either the horizontal or vertical elements of gravity framing. Figure 5-4 shows rotations in the slab while the column remains rigid.

Figure 5-4: Rigid Column and Rotated Slab Response to Lateral Drifts

With this in mind, the structural integrity of gravity framing connections can benefit from minor changes with consideration to the two primary failure modes previously discussed: axial capacity loss in columns and punching failure in slab-column connections. For conventionally reinforced concrete,
these recommendations are summarized in Figure 5-5.

Figure 5-5: Typical details for structural integrity of conventionally reinforced concrete slab-column joints (Moehle, 2015)

For slab-column connections, providing continuous bottom reinforcement through the column core prevents against progressive collapse in the event of shear failure at the joint. As recommended by ACI 352.1, two top bars should also be continuous through the column core. For connections of exterior members, these recommendations are addressed by providing fully developed reinforcement as shown in Figure 5-5(b).

Prestressed connections meet structural integrity requirements as per ACI 318 §8.7.5.6 by satisfying one of two options. Figure 5-6 outlines the detailing for both.

1. A minimum of two tendons must pass through or be anchored within the column. These tendons are to pass under any orthogonal tendons in adjacent spans. Where the two tendons are anchored within the column, ensure the anchorage is located beyond the column centroid. This arrangement is to prevent progressive collapse in the event of a punching shear failure.

2. Provide continuous bottom reinforcement through the column core or anchored in exterior supports where bottom reinforcement satisfies both \( A_v \geq 4 \frac{4.5 \sqrt{f'_c}}{f_y} bd, \text{psi} \) and \( A_v \geq 300 \frac{bd}{f_y}, \text{psi} \) (ACI §8.7.5.6.3.1).
5.5.1. BEAM-COLUMN JOINTS

ACI 318 does not have unique requirements for beam-column joints of gravity framing members. Instead beam-column joint analysis and design adheres to requirements of non-seismic conditions or specific provisions of special moment frames depending on whether earthquake-induced demands exceed design strengths of the gravity framing members (i.e. 18.14.3.3(d)).

5.5.1.1. DEMANDS DO NOT EXCEED DESIGN STRENGTHS

Where demands in beams and columns do not exceed design strengths, beam-column joints must satisfy the non-seismic requirements of ACI 318, Chapter 15. Joint shear force shall consider the maximum moment transferred between the beam and column from factored loads or the beam nominal moment strength $M_n$. The design shear strength must satisfy $\Phi V_u \geq V_u$. Design strengths are determined in accordance with ACI 318 Table 15.4.2.3 in which the effective area of the joint $A_j$, continuity of the column, and confinement of the joint by inframing beams impact the calculation for nominal joint shear strength.

Commentary in ACI 318 §R15.3.1 states that transverse reinforcement is not required for interior joints with beams framing into each face of the column. Additionally, the joint is considered confined if two beams frame into opposite sides of the column and satisfy (a) each beam is at least $\frac{3}{4}$ the width of the column, transverse beams extend $\geq$ one beam depth beyond the joint face, and
transverse beams contain at least two continuous top and bottom bars with No. 3 or larger stirrups,
(b) the joint is not part of the lateral-force-resisting system, and (c) the joint is not part of a structure
assigned to seismic design category D, E, or F (ACI 318 §15.3.1.1). Connections not satisfying
these provisions shall have transverse reinforcement in each principal direction satisfying:

\[
A_v \geq 0.75 \sqrt{f'c} \frac{b_w s}{f_{yt}} \geq 50 \frac{b_w s}{f_{yt}}, \text{psi}
\]

The center-to-center tie spacing should not exceed 12 in. and transverse spacing shall not exceed 8
in. within the depth of the deepest beam framing into the joint. In the shallowest beam, at least two
layers of horizontal transverse reinforcement shall be provided. Confinement requirements for
joints can also be satisfied by continuing the column transverse reinforcement through the joint at
potentially reduced spacing.

5.5.1.2. DEMANDS DO EXCEED DESIGN STRENGTHS

Where large drifts create demands in either beams or columns that exceed design strengths, beam-
column joints adhere to ACI 318 detailing provisions for intermediate moment frames (ACI 318
§18.14.3.3(d), ACI 318 §18.4). Joint transverse reinforcement prevents strength loss during load
reversals experienced during ground shaking. Joint shear forces are determined with consideration
to the maximum probable moment, \(M_{pr}\).

Joints shall be reinforced with transverse reinforcement equivalent to confinement along \(l_o\) of
column. For Grade 60 steel, spacing shall be the least of \(8d_b\), 8 in., and \(\frac{1}{2}\) the smallest cross-sectional
column dimension. For Grade 80 steel, spacing shall be the least of \(6d_b\), 6 in, and \(\frac{1}{2}\) the smallest
cross-sectional column dimension. Where beams frame into all four sides of the joint and each beam
is at least three-fourths the column width, the transverse reinforcement can be halved. If a beam
framing into column has depth exceeding twice the column depth, design is recommended to follow
the strut-and-tie method of ACI 318 Chapter 23 where \(\Phi V_n\) does not exceed its value per ACI 318
§15.4.2.
5.5.2. SLAB-COLUMN FRAMING

As described in Section 3.2, due to the potential for progressive collapse, punching failure at the slab-column joint is a primary concern for gravity framing design. Therefore, provisions for slab-column joints are predominantly concerned with addressing shear strength capacity. As shown in Figure 5-7, ACI 318 approximates that the relationship between story drift ratio and gravity shear stress can be used to determine the need for shear reinforcement for slabs. The larger drift between adjacent stories above and below the slab-column joint shall be used.

![Figure 5-7: Criterion to determine whether slab shear reinforcement is required (ACI 318 R18.14.5.1)](image)

Multiplying the stress terms shown along the x-axis of Figure 5-7 by the length of the critical section around column times the distance from the extreme fiber $b_o d$ results in the shear demand without moment transfer and shear strength of slab concrete. Critical sections are the locations surrounding the slab-column joint at risk for punching failure. ACI 318 §8.7.7 describes two shear critical sections as shown in Figure 5-8. The first shear critical section exists around the column, capital, or drop panel at a distance of $d/2$. Where shear reinforcement for the slab-column connection is required, a second shear critical section exists at a distance of $d/2$ away from the termination of addition shear reinforcement.
Where shear reinforcement is required, stud rails are the most common form of reinforcement. Shear strength capacity shall provide $v_s \geq 2\sqrt{f'c}$ at critical sections per ACI 318 §22.6.8.3. The reinforcement should extend a length no less than $4h_s$ from the column face. As recommended by ACI 318 §8.7.7, the arrangement of stud rails is shown in Figure 5-8. Per ACI 318 Table 8.7.7.2.1, center-to-center spacing between adjacent stud rails should not exceed $2d$ where $d$ is defined per Figure 5-8 as bottom slab face to top of stud. The maximum spacing between studs is typically $d/2$. Prestressed slabs conforming to ACI 318 §22.6.5.4 have a maximum spacing of $3d/4$.

5.5.3. SLAB-WALL FRAMING

ACI 318 provides minimal commentary on slab-wall connections for gravity framing. Nonetheless, reinforcement details should anticipate local deformations during earthquake shaking. Research by Schwaighofer and Collins (1972) show seismically induced forces can result in punching failure for slab-wall coupling. As shown in Figure 5-9(b), this typically appears when two walls are separated by a narrow entryway and connected by a slab. As with slab-column rotations shown in Figure 5-4,
rotations can occur in the slab at slab-wall connections when subjected to seismic loading. These rotations cause increased moment and shear demands at connection locations. To ensure moment transfer for large lateral drift ratios, Schwaighofer and Collins (1972) recommend reinforcement be placed at a width of \( l_h + b \) across the coupling walls and a \( 3l_h \) centered at entryway. Per these boundaries, shear strength at the critical section shown in Figure 5-9(a) can be estimated as \( V_n = 4\sqrt{f'_c \text{ (psi)}} [3(b + d)d] \).

\[ \text{(a) Slab critical section} \]

\[ \text{(b) Crack pattern formation due to lateral loading} \]

\[ \text{(c) Effective width for reinforcement} \]

**Figure 5-9: Slab-wall coupling effective sections for design (Schwaighofer and Collins, 1972)**

Two termination details for tendons at a slab-wall connection are shown in Figure 5-10. The second case, studied by Klemencic et al. (2006), was shown to perform adequately when the tendon termination was placed one slab height away from the wall face such that shear and moment transfer was conducted via the use of dowels. This second connection detail may increase constructability where wall elements are cast ahead of floor slabs, typical in high-rise construction.
Figure 5-10: Typical details for slab-wall connections (Moehle, 2015)
6. Detailing, Constructability and Material Selection

The performance of gravity framing elements during earthquake shaking is reliant on effective detailing and proper placement. Placement recommendations for gravity framing are similar to those for special moment frames, which are well discussed by Moehle (2015). Additionally, the reinforcement specifications for RC gravity framing should reflect ACI 318-19 Chapter 20. The following discussion highlights several important aspects of detailing, constructability, and material selection.

6.1. DETAILING AND CONSTRUCTABILITY ISSUES

6.1.1. LONGITUDINAL BAR COMPATIBILITY

Where architectural requirements push the reduction of column cross-sectional areas, congestion can occur. Additionally, these size-reduced columns might then support axial loads above their balance point. The proper detailing and placement of reinforcement is critical to maintaining axial capacities in members. The performance of the members during ground shaking depends on confinement detailing to support column cores and protect against shear failure.

Element connections present the greatest challenge to reinforcement constructability in terms of placement and spacing for concrete pour. Connections also represent locations where failures tend to occur. Therefore, careful placement is critical both for construction sequencing and building performance. If beams and columns are the same width, reinforcement conflicts and requires bending and offsetting. Typically, the beam reinforcement will be bent to accommodate column reinforcement as to minimize bar eccentricities in vertical members. When bending beam reinforcement, one option is to guide the reinforcement between the longitudinal bars of the column. Beam longitudinal reinforcement can be supported by smaller discontinuous reinforcement, which is not considered in moment strength calculations.
If the widths of the beam and column differ by at least 2 inches on each side, bending and offsetting can usually be avoided. However, this can increase forming cost and go against architectural intent. The cost of architectural enclosures to address the appearance can be more costly than required by the previously commented upon construction methods for bar bending.

Multiple layers of longitudinal reinforcement present a challenge in terms of constructability. The interior layers are difficult to support requiring additional intermediary ties and supports. This can drive up material cost and construction time. If possible, sizing up the element’s cross-section might provide a more economical alternative.

6.1.2. BEAM AND COLUMN CONFINEMENT

As discussed throughout this Guide, confinement is crucial to the performance of beams, columns, and joints during ground shaking. Confinement can assist ductility in beams when needed. Confinement for beams can be achieved by either continuously wound reinforcement closed by seismic hooks or by stirrups with seismic hooks closed by tie caps. Confinement for columns requires continuous hoop reinforcement at its perimeter. For lower axial loads, the use of crossties with alternating 90 degree and 135 degree hooks are permitted. Where axial loads are high, both ends of the cross tie must be a seismic hook. In comparison to 135 degrees, a 90 degree bend is more prone to opening during ground shaking, potentially causing a significant loss in capacity.

ACI 318 has provisions allowing horizontal spacing of confinement reinforcement to reach up to 14 inches. However, the effectiveness of confinement reinforcement is closely related to both vertical and horizontal spacing. Performance is improved when longitudinal reinforcement spacing be limited to no more than 6 or 8 inches. In terms of constructability, tighter horizontal spacing allows an increase in vertical spacing thereby creating more space to maneuver between hoop sets during construction. The diameter of hoop set reinforcement should not exceed No. 5 bars. Bar diameters greater than No. 5 are difficult to bend and place.
6.1.3. BAR SPLICES

Bar splicing causes stress concentrations and effectively doubles the steel ratio in an element. Additionally, splicing requires that overlap be fully developed creating areas along the element length with additional congestion. This congestion can be reduced through the use of mechanical splices. As discussed in Section 5.3, gravity framing elements have no limits to splicing locations when induced shears and moment remain below the element’s capacity. However, when induced forces exceed design capacity for columns, splicing is limited to column midspans. This is to better ensure the effectiveness of confinement at hinging locations during ground shaking.

6.2. REINFORCEMENT SELECTION FOR GRAVITY FRAMING

ACI 318-19 Chapter 20 provides specifications for permitted properties of steel reinforcement. Elements such as inserts, anchor bolts, or plain bars for dowels at joints are not discussed. Per ACI 318-19 §20.2.1.3, non-prestressed deformed steel reinforcement is specified as ASTM A615 or ASTM A706. Prestressed strands, wires, and bars are specified per ACI §20.3 and are not discussed further by this Guide.

ASTM A615 steel is a carbon steel and can be specified as Grade 40, 60, 80, and 100 depending on its application. ASTM A706 is a low-alloy steel that is typically specified for seismic design, where controlled tensile properties are required, or for reinforcement with welded connection designs. As of the 2019 version of ACI 318, ASTM A706 can also be specified as Grade 40, 60, 80, and 100 with adherence to application limits per Table 20.2.2.4(a).

ACI 318-19 Table 20.2.2.4(a) outlines the allowable reinforcement types by usage and application for nonprestressed deformed reinforcement. For gravity framing, the designer typically choses between ASTM A615 and A706 for steel reinforcement. The decision largely depends on whether induced moments and shears are calculated and whether the reduced provisions of ACI 318-19 §18.14.3.2 are satisfied. If induced moment and shears do not exceed the design moment and shears of the member,
ASTM A615 can be specified. However, if induced moment and shears are not calculated or if they exceed the design moment and shears, then ACI 318 clearly shows that these elements are required to meet seismic specifications—particularly the seismic specifications of special moment frames found in ACI 318-19 §18.7.

ASTM A706 is typically specified with regard to seismic design because it has a lower and upper limit on the actual yield strength of the steel and requires a minimum tensile-to-yield strength ratio (ACI 318, 2019). This is important because steel that exhibits substantially higher strength than assumed in design will lead to higher shear and bond stresses, which may lead to brittle failures. Therefore, when induced moment and shears are not calculated or when they exceed the design moment and shears, gravity framing elements are then designed largely with respect to seismic provisions. This means that ASTM A706 should be specified as per ACI 318-19 Table 20.2.2.4(a). ASTM A615 Grade 60 provides an alternative only if the designer specifies that the provisions of ACI 318-19 §20.2.2.5(b) be satisfied. These requirements for ASTM A615 deal largely with ensuring ductile response in the occurrence of lateral loads.

This decision-making process for material selection is outlined in *Flow Chart 6-1*. 
Flow Chart 6-1: Specifications of deformed nonprestressed steel reinforcement for gravity framing elements with regards to seismic design
7. References

ACI (2013). *Design specification for unbonded post-tensioned precast concrete special moment frames satisfying ACI 374.1 (ACI 550.3-13) and commentary*, American Concrete Institute, Farmington Hills, MI.

ACI (2019). *Building code requirements for structural concrete (ACI 318-19) and Commentary on building code requirements for structural concrete (ACI 318R-19)*, American Concrete Institute, Farmington Hills, MI.


ASCE (2016). *Minimum design loads and associated criteria for buildings and other structures*, ASCE/SEI 7-16, American Society of Civil Engineers, Reston, VA.

ASCE (2017). *Seismic evaluation and retrofit of existing buildings*, ASCE/SEI 41-17, American Society of Civil Engineers, Reston, VA.


DESIGN EXAMPLE:
SEISMIC DETAILING OF
CAST-IN-PLACE CONCRETE
GRAVITY FRAMING
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1. Gravity Framing Design Example 1: Podium Structure

This design example utilizes an eight-story podium structure—three stories of reinforced concrete and five stories of light wood framing—to illustrate the design and detailing of secondary frame members, also referred to as gravity framing. Gravity framing consists of beams and columns that are not part of the lateral-force-resisting system, diaphragms, and collectors. The example structure is classified as a bearing-wall system with special reinforced concrete shear walls.

Seismic forces are distributed to the elements considered as part of the lateral-force-resisting system. As discussed in the accompanying Guide, standard code requirements apply modification factors and design procedures to control the yielding mechanism of the lateral-force-resisting system. These elements are designed to maintain their structural integrity while undergoing cyclic action resulting in energy dissipation through inelastic deformation. It is common to assign the lateral loads for design to a limited part of the structure. This typically yields several benefits both economical and performance based. The remaining beams and columns are designated as gravity framing.

In this design example, the structure resembles a typical podium structure with a concrete lower structure and a light wood frame attached on top. The lateral-force-resisting system consists of four sets of shear walls configured such that each surrounds an elevator shaft or stairwell. The asymmetrical floor plan is not uncommon for podium structures. Furthermore, the floor plan is intended to induce a more significant response under earthquake loading for perimeter columns such that they transition to the more detailed reinforcement requirements as outlined in Section 5.3 of the Guide.

Podium structures are commonly used in both residential and commercial projects. This design example seeks to address concerns over the analysis and design of reinforced concrete gravity framing as it relates to the global stability of a podium structure. The design process can be generally applied to non-podium structures.

The design example is intended to further illustrate the previous discussion on gravity framing design.
Figure 1-1: 3D model of podium structure (upper stories wood frame conceptual only)
2. Building Geometry and Loads

2.1. GIVEN INFORMATION

The seismic design coefficients are determined using ASCE 7-26 §11.4.2 and a hazard tool such as ATC Hazards by Location tool at hazards.atcouncil.org.

Hazard tool output (randomized location in the San Francisco: Bay Area)

\[ S_s = 2.016g; \quad S_{MS} = 2.016g; \quad S_{DS} = 1.344g \]
\[ S_I = 0.774g; \quad S_{MI} = 1.316g; \quad S_{DI} = 0.877g \]

The example structure utilizes the two-stage analysis procedure as outlined by ASCE 7-16 §12.2.3.2. The lower structure is classified as a bearing-wall system with special reinforced shear walls. The upper structure is classified as light-frame (wood) walls sheathed with wood structural panels rated for shear resistance. Seismic design parameters are computed as follows:

**Special reinforced shear walls**

- Seismic importance factor, \( I = 1.0 \) §11.5.1
- Seismic Design Category D §11.6
- Response modification factor, \( R = 5 \) §12.2-1
- System overstrength factor, \( \Omega = 2.5 \) §12.2-1
- Deflection amplification factor, \( C_d = 5 \) §12.2-1
- Redundancy, \( \rho = 1.0 \) §12.3.4.2

**Light-frame walls**

- Seismic importance factor, \( I = 1.0 \) §11.5.1
- Seismic Design Category D §11.6
- Response modification factor, \( R = 6.5 \) §12.2-1
- System overstrength factor, \( \Omega = 3 \) §12.2-1
- Deflection amplification factor, \( C_d = 4 \) §12.2-1
- Redundancy, \( \rho = 1.0 \) §12.3.4.2
Material Properties

Concrete strength (slabs), $f'_c = 4000$ psi

Concrete strength (columns and walls), $f'_c = 6000$ psi

Mild reinforcement yield stress, $f_y = 60$ ksi

P/T tendon ultimate stress, $f_{u,PT} = 270$ ksi

P/T tendon effective stress = 90 ksi

The example structure is square in plan and measures 125 feet by 125 feet. The lower three stories are reinforced concrete with shear walls and the upper five stories are light-frame (wood) walls sheathed with wooden structural panels rated for shear resistance. The building is 96 feet tall with a typical story height of 12 feet. The typical floor plan for Levels 1 through 3 is shown in Figure 2-1. A courtyard rests on top of Level 3 between gridlines A-C and 6-7. As shown in Figure 2-2, the cut out to accommodate the courtyard extends to all light-frame levels.

For the reinforced concrete structure analyzed in this design example, the lateral-force-resisting system is shear walls. There are four sets of shear walls surround either elevator shafts or stairwells. As shown in Figure 2-1, two sets of core shear walls exist at the structure’s center, one at the northeast most corner, and one at the southeast most corner. Each measures 25 feet by 12 feet with an opening of 9 feet on one side. All shear walls are 12 inches thick. Shear wall design omitted from this design example.

The gravity framing is columns supporting unbonded post-tensioned flat plates. The PT slabs on Level 1 and 2 are 8 inches thick and the PT slab on Level 3 is 12 inches thick to accommodate the transfer of forces from the upper wooden-frame structure. Gravity columns have 25 in. x 25 in. cross-sections. Spacing between columns is 25 feet in both directions.
Figure 2-1: Floor plan for level 1 through level 3
2.2. MATERIAL WEIGHTS

Light-frame wood structure (approx.): 30 psf

Concrete: 150 lb/ft³

Exterior Cladding: 17 psf (wall vertical surface)

2.3. LIVE LOADS

ASCE 7 Table 4.3-1 lists the minimum uniformly distributed live loads per occupancy type. This example structure is designated as residential apartments (40 psf) for the upper structure and commercial office (50 psf) use for the lower structure.

In accordance with ASCE 7 §4.3.2, partitions contribute to live loads with a minimum magnitude of 15 psf. Per ASCE 7 §12.7.2 partition load shall be included when calculating the effective seismic weight at a minimum magnitude of 10 psf.
The example structure qualifies for live load reduction. However, to accommodate future changes in use, live load reduction was not performed. In load combinations, live loads are factored by 0.5 where applicable.

2.4. SEISMIC MASS

The seismic mass was determined as 10,595 kips details shown in Table 2-1.

Table 2-1: Seismic Weight Details

<table>
<thead>
<tr>
<th>Level 1 - Level 2</th>
<th>Slab(+SIDL): [(14,425ft²)(2/3ft)]150lb/ft³+(10psf)(14,425ft²)</th>
<th>1586.75 kips</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear Walls: [(260ft)(1ft)(12ft)]150lb/ft³</td>
<td>468 kips</td>
</tr>
<tr>
<td></td>
<td>Columns: 28[(25&quot;) (25&quot;) (12ft)] 150lb/ft³</td>
<td>218.75 kips</td>
</tr>
<tr>
<td></td>
<td>Partition+Cladding: (10psf)(14,425ft²)+(500ft)(12ft)(17psf)</td>
<td>246.25 kips</td>
</tr>
</tbody>
</table>

| Level 3            | Slab(+SIDL): [(14,425ft²)(1ft)]150lb/ft³+(10psf)(14,425ft²) | 2308 kips     |
|                    | Partition+Cladding: (10psf)(13,175ft²)+(500ft)(12ft)(17psf)| 254.15 kips   |

| Level 4 - Level 7  | Partition+Cladding: (10psf)(13,175ft²)+(600ft)(12ft)(17psf)| 254.15 kips   |
|                    | Light-frame: [13,175ft²]30psf                               | 395.25 kips   |

| Level 8 (roof)     | Light-frame: [13,175ft²]30psf                               | 395.25 kips   |
|                    | Total                                                       | 10,595 kips   |
3. Load Combinations for Design

Load combinations for strength design and seismic load combinations are listed in ASCE 7-16 §2.3.1 and ASCE 7 §2.3.6, respectively. The following load combinations are used in this example (including live load factor of 0.5 for relevant combinations):

\[
\begin{align*}
1.4D & \quad \text{§2.3.1, Combination 1} \\
1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) & \quad \text{§2.3.1, Combination 2} \\
1.2D + E_v + E_h + 0.5L + 0.2S & \quad \text{§2.3.6, Combination 6a} \\
0.9D - E_v + E_h & \quad \text{§2.3.6, Combination 7a} \\
1.2D + E_v + E_{mh} + 0.5L + 0.2S & \quad \text{§2.3.6, Combination 6b} \\
0.9D - E_v + E_{mh} & \quad \text{§2.3.6, Combination 7b}
\end{align*}
\]

The horizontal seismic load effect \(E_h\) without overstrength applied is defined as:

\[
E_h = \rho Q_E \quad \text{ASCE 7 Eq. 12.4-3}
\]

The horizontal seismic load effect \(E_{mh}\) including overstrength is defined as:

\[
E_{mh} = \Omega_o Q_E \quad \text{ASCE 7 Eq. 12.4-7}
\]

The vertical seismic load effect \(E_v\) for all relevant combinations is defined as:

\[
E_v = 0.2S_DSD \quad \text{ASCE 7 Eq. 12.4-4a}
\]
4. Lateral Analysis

4.1. STRUCTURAL SYSTEM

The example structure is a podium structure with three levels of reinforced concrete shear walls supporting five levels of light-frame wood construction. In a complete lateral analysis, the effects of horizontal and vertical irregularities would be evaluated. These procedures are outlined in ASCE 7 Chapter 12. To focus on gravity framing design, procedures with respect to irregularity, torsion, and redundancy are omitted. As stated in the introduction, the example structure does feature an asymmetrical floor plan which might induce a significant torsional response as defined in ASCE Table 12.3-1. This is intended such that induced moment and shears of perimeter columns will exceed the design capacity such that the more stringent reinforcement detailing per ACI §18.14.3.1 is invoked.

4.2. ANALYSIS PROCEDURE

This structure is analyzed using the equivalent lateral force procedure (ELF). This is the most common method of analysis for simple podium structures and is permissible per ASCE 7 Table 12.6-1. The example structure qualifies as a structure less than 160 ft. height. Notably, its structural irregularities remain undefined per the comment in Section 4.1. In the full design of a structure, the engineer should check for irregularities and torsion to confirm an adequate analysis procedure is being used (in addition to other items of importance related to these building characteristics).

4.3. STRUCTURAL MODELING

The example structure does not require a 3D model be built. However, a 3D ETABS model has been used to verify hand calculations and facilitate the determination of design forces and drifts.

The built model does not include the light-frame upper structure. Instead, the resulting forces of the light-frame are summed and applied as a distributed load on Level 3 of the lower RC structure. To capture these loads for lateral analysis, they are assigned as a seismic mass in the ETABS model.

Element stiffnesses are as follows:

*Table 4-1: ETABS section property modifiers*

<table>
<thead>
<tr>
<th>Element</th>
<th>Properties</th>
<th>Modification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravity Columns</td>
<td>M2, M3</td>
<td>0.70 (0.01 for drift analysis)</td>
</tr>
<tr>
<td>Shear Walls</td>
<td>f11, f22</td>
<td></td>
</tr>
<tr>
<td>PT Slabs</td>
<td>f11, f22</td>
<td>0.50</td>
</tr>
</tbody>
</table>
4.4. PERIOD AND BASE SHEAR FOR TWO-STAGE ANALYSIS

4.4.1. PERIOD DETERMINATION

For two-stage analysis, the period of the upper and lower structure is to be found separately.

**Upper structure**

\[ T_a = C_t h_n^x \]  
\[ C_t = 0.02, x = 0.75, h = 60\text{ ft} \]  
\[ \therefore T_a = 0.02 \times 60^{0.75} = 0.431\text{ s} \]

**Lower structure**

\[ T_a = C_t h_n^x \]  
\[ C_t = 0.02, x = 0.75, h = 36\text{ ft} \]  
\[ \therefore T_a = 0.02 \times 36^{0.75} = 0.294\text{ s} \]

4.4.2. SESIMIC BASE SHEAR DETERMINATION

**Upper structure**

\[ V = C_s W_{upper} \]  
\[ C_s = \frac{S_{DS}}{R} = 0.207g \]  

Where

\[ C_s \leq \frac{S_{DL}}{T(R)^2} \rightarrow 0.313g \]  
\[ C_s \geq 0.044S_{DS}l \rightarrow 0.059g \]  
\[ C_s \geq \frac{0.5S_1}{R} \rightarrow 0.002g \]

\[ \therefore V = 0.207 \times 2993 = 620\text{ kips} \]

Contributing to lower structure per ASCE 7 §12.2.3.2(d): \[ V \times \frac{R/R_{upper}}{R/R_{lower}} = 805\text{ kips} \]

**Lower structure**

\[ V = C_s W_{lower} \]  
\[ C_s = \frac{S_{DS}}{R} = 0.269 \]  

Where
\[
C_s \leq \frac{5\rho_1}{T(R)} \rightarrow 0.597g \quad \text{ASCE 7 Eq. 12.8-3}
\]
\[
C_s \geq 0.044S_D S \rightarrow 0.059g \quad \text{ASCE 7 Eq. 12.8-5}
\]
\[
C_s \geq \frac{0.5S_1}{R} \rightarrow 0.002g \quad \text{ASCE 7 Eq. 12.8-6}
\]

\[\therefore V = 0.269 \times 7602 = 2045 \text{ kips}\]
\[\therefore \text{Total base shear: } V = 2043 + 805 = 2850 \text{ kips}\]

Equivalently, the base shear can also be determined by multiplying the total effective seismic weight by the seismic response coefficient of the lower structure. This is possible because both the upper structure and lower structure have the same redundancy factor, \( \rho \).

Total base shear: \( V = C_{s, \text{Lower Structure}} W_{total} = 0.269 \times (7602 + 2993) = 2850 \text{ kips} \)

### 4.5. VERTICAL DISTRIBUTION OF SESIMIC FORCES

Story shears at each level of the structure are determined per ASCE §12.8. The table below shows only Level 1 – Level 3 per two-stage analysis guidelines. The calculations for lateral forces \( F_x \) of the lower concrete structure are entirely independent of the upper structure. When determining story shear \( V_x \), the shear of the upper story (calculated in Section 4.4.2) is added onto the shear per the lower structure alone. In this way, the total shear at Level 1 equates the total shear previously calculated.

\[
F_x = C_{vx} V
\]

where

\[
C_{vx} = \frac{w_i h_i^k}{\sum w_i h_i^k} \quad \text{ASCE 7 Eq. 12.8-12}
\]

**Table 4-2: Seismic Force Distribution**

<table>
<thead>
<tr>
<th>Wx_lower</th>
<th>hx</th>
<th>( w_i h_i^k )</th>
<th>Cvx</th>
<th>Fx_lower</th>
<th>( V_x =V_i + V_{\text{light frame}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 3</td>
<td>2560 kips</td>
<td>36 ft</td>
<td>92230 kip*ft</td>
<td>0.517</td>
<td>1060</td>
</tr>
<tr>
<td>Level 2</td>
<td>2390 kips</td>
<td>24 ft</td>
<td>57340 kip*ft</td>
<td>0.322</td>
<td>660</td>
</tr>
<tr>
<td>Level 1</td>
<td>2390 kips</td>
<td>12 ft</td>
<td>28670 kip*ft</td>
<td>0.161</td>
<td>330</td>
</tr>
<tr>
<td>( \Sigma )</td>
<td>7340 kips</td>
<td>96 ft</td>
<td>178240 kip*ft</td>
<td>1.000</td>
<td>2050.0</td>
</tr>
</tbody>
</table>

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5. Column Design for Gravity Framing

For this design example, column demands were organized by three primary locations: interior, exterior, and corner. When applicable, columns were further organized by floor due to slab thickness change at podium level. The procedure is simplified such that only one cross-section is designed. In practice, this procedure could result in a range of cross-section sizes and reinforcement schedules. Columns were first designed following ACI 318-19 §18.14.4.2 where earthquake induced forces are considered to be less than the column’s capacity. After this initial design, the column section was checked against induced forces to see whether design requirements shift to ACI 318-19 §18.14.4.3.

5.1. SHEAR DESIGN

Columns for this example structure are all 25 in. by 25 in. The cross-section area was determined iteratively based on the axial, flexure, and shear requirements detailed below. Notably, the cross-section was originally conceived to be 14 in. x 18 in. based on axial demands, but this proved to be inadequate due to punching shear.

Columns must be designed for axial, flexural, and shear design loads determined as per gravity and seismic combinations in Section 3. Gravity columns are not considered as part of the lateral-load-resisting system but must be capable of withstanding building deformations under seismic loads. Table 5-1 displays the design forces for each column using data from the ETABS model. Note that these design forces were determined when gravity column stiffness was “on” using a 0.7 modification factor as discussed in Section 4.3.
Design displacements were also determined using the ETABS model. Data extracted from ETABS was modified by $C_d/I_e$ as outlined in Section 4.2.1 where $C_d = 5$ and $I_e = 1$ per ASCE 7-16 §11.5.1 and Table 12.2-1. The resulting design displacements and story drifts at the center of mass are shown in Table 5-2.

**Table 5-2: Design Displacements**

<table>
<thead>
<tr>
<th>Level</th>
<th>Height (ft)</th>
<th>X-Design Displacement (in)</th>
<th>Y-Design Displacement (in)</th>
<th>X-Drifts (%)</th>
<th>Y-Drifts (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CM</td>
<td>Ground</td>
<td>0</td>
<td>0</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td>12 ft</td>
<td>12</td>
<td>0.8</td>
<td>1.5</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>24 ft</td>
<td>24</td>
<td>2.2</td>
<td>3.9</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>36 ft</td>
<td>36</td>
<td>3.8</td>
<td>6.6</td>
<td>1.1</td>
</tr>
</tbody>
</table>

This displacement data was extracted when gravity column stiffness was turned “off” (i.e. the stiffness modification factor was set to 0.01). This structure is quite stiff due to its lateral-force-resisting system being shear walls. Therefore, only a small difference between drift values occurs when stiffness of gravity framing is “on” vs “off” as shown in Figure 5-1. While the difference is marginal for this
design example, the stiffness contribution of gravity framing might be more pronounced for flexible lateral-force-resisting systems (e.g. moment frames).

Figure 5-1: Earthquake induced displacements at center-of-mass with and without gravity column stiffness contribution (East-West vs North-South)

5.2. AXIAL AND FLEXURE DESIGN
5.2.1. LONGITUDINAL REINFORCEMENT

From Table 5-1, the governing axial demands on the columns were as follows:

<table>
<thead>
<tr>
<th>Column Location</th>
<th>Axial Demand, ( P_u ) (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
<td>223</td>
</tr>
<tr>
<td>Exterior</td>
<td>450</td>
</tr>
<tr>
<td>Corner</td>
<td>950</td>
</tr>
</tbody>
</table>

The final axial column capacity was determined to be 1860 kips. This strength was determined by satisfying strength requirements for axial force \( \Phi P_n \geq P_u \).

\[
\Phi P_{n,\text{max}} = \Phi 0.8 P_o
\]

ACI 318 Table 22.4.2.1

where:

\[
\Phi = 0.65 \quad \text{(ACI 318-19 Table 21.2.1)}
\]
This relationship was reversed to determine the longitudinal steel requirements. The result was to use 12 No. 7 bars in each column. This longitudinal steel recommendation has a steel reinforcement ration of 1.15%, which satisfies ACI §18.7.4.3: 0.01 \leq \frac{A_s}{A_g} \leq 0.06.

5.2.2. TRANSVERSE REINFORCEMENT

As discussed in Section 5.3 of the Guide, the transverse reinforcement detailing is dependent on whether the gravity column is loaded beyond 30% of the concrete compressive strength per ACI 318-19 §18.7.5.2(f). Check 30% maximum compressive strength:

\[0.3 f' c \, A_g = 1125 \text{ kip}\]

where:

\[A_g = c_1 c_2 = 625 \text{ in}^2\]

where \( P_u < 0.3A_g f'_c \) and \( f'_c \leq 10,000 \text{ psi} \)

where \( P_u \geq 0.3A_g f'_c \) or \( f'_c > 10,000 \text{ psi} \)

It is concluded that for interior, exterior, and corner columns, detailing requirements of (a) are permissible.

Next, transverse reinforcement was determined iteratively following design requirements in ACI 318-19 Chapter 18. It was determined that 4 No. 4 bars would be required spanning in each primary direction across the column face. Per ACI 318-19 Table 18.7.5.4, the following expressions must be satisfied:

\[\frac{A_{sh}}{sb_c} = 0.3 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}\]  
ACI 318 Eq. 18.7.5.4(a)

\[\frac{A_{sh}}{sb_c} = 0.09 \frac{f'_c}{f_{yt}}\]  
ACI 318 Eq. 18.7.5.4(b)
where:

\[ A_{sh} = \text{area of transverse steel} \]

\[ s = \text{longitudinal spacing of transverse reinforcement} \]

\[ b_c = c_1 - 2 \times \text{cover} = 22 \text{ in} \]

\[ A_{ch} = (c_1 - 2 \times \text{cover})(c_2 - 2 \times \text{cover}) = 484 \text{in}^2 \]

\[ a_s = 20 \text{ (corner)}, 30 \text{ (exterior)}, 40 \text{ (interior)} \]

The longitudinal spacing of transverse reinforcement was determined per ACI 318-19 §18.14.3.2 to be 5 inches: \( s = 6d_b = 5.25 \text{ in} \) where \( d_b = 0.875 \text{ in} \) for No. 4 bars.

The conclusion for the column cross-section at this stage:

Using this cross-section, the PM interaction diagram was created to further evaluate the adequacy of the column design. The maximum force and moment combinations experienced by the structure during ground shaking are shown as black dots on the PM diagram. This confirms adequate flexural strength for the column.
5.3. SHEAR DESIGN

Columns were checked against punching failure at the column-slab connection. The following load combinations were used to determine demand:

1. \(1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)\)  
   ASCE 7 §2.3.1, Combination 2

2. \(1.2D + E_v + 0.5L\)  
   ASCE 7 §2.3.6, Combination 6a

ASCE 7 §2.3.1, Combination 2 governs with the following results:

<table>
<thead>
<tr>
<th>Column Location</th>
<th>Shear Demand, (V_u) (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
<td>45</td>
</tr>
<tr>
<td>Exterior</td>
<td>90</td>
</tr>
<tr>
<td>Corner</td>
<td>180</td>
</tr>
</tbody>
</table>

Using the 25in x 25in cross-section \((c_1xc_2)\), all columns were confirmed to satisfy \(\Phi V_n \geq V_u\) for punching shear strength using the minimum of the following:

\[
\Phi V_n = \Phi (3.5\lambda \sqrt{f_c'} + 0.3 f_{pc} + \frac{V_p}{b_o d}) \quad \text{ACI 318 Eq. 22.6.5.5a}
\]

\[
\Phi V_n = \Phi ((1.5 + \frac{c_d}{b_o})\lambda \sqrt{f_c'} + 0.3 f_{pc} + \frac{V_p}{b_o d}) \quad \text{ACI 318 Eq. 22.6.5.5b}
\]

where:

\(\Phi = 0.75\)
\[ \lambda = 1 \]
\[ a_s = 20 \text{ (corner)}, 30 \text{ (exterior)}, 40 \text{ (interior)} \]
\[ d = 12 \text{ in} - 1.5 \text{ in} = 10.5 \text{ in} \]
\[ b_o = 4L_o \]
\[ L_o = c_1 + \frac{2}{d} \]

Next, transverse reinforcement adequacy must be verified. Using the previously built PM diagram, a conservative estimate of the probable moment can be assigned as \( M_{pr} = 13,000 \text{ kip} \times \text{ in} \). This value can be used to determine the shear demand \( V_e \) per ACI 318-19 §18.6.5.1.

\[ V_e = \frac{2M_{pr}}{l_n} = 200 \text{kips} \]

where:
\[ l_n = l_u - h = 12 \text{ft} - 1 \text{ ft} = 11 \text{ ft}. \]

It must be shown that the shear capacity of concrete and steel exceeds the shear demand related to probable moment: \( \Phi V_n = \Phi (V_c + V_s) \geq V_e \). The shear strength of steel was found to be:

\[ V_s = \frac{A_v f_y d_s}{s} = 225 \text{ kip}. \]

Per ACI 318-19 §18.6.5.2, the shear strength of concrete \( V_c \) is calculated as non-zero if both \( V_e \) is 1.2 the maximum required strength and if the axial demand satisfied:

\[ P_u \leq \frac{A_g f_c'}{20}. \]

For this design example, both requirements are satisfied and \( V_c \) is calculated according to ACI 318-19 Table 22.5.5.1 where \( A_v \geq A_{v,min} \).

\[ V_c = [2\lambda \sqrt{f_c'} + \frac{P_u}{6A_g}] \]  
ACI 318 Table 22.5.5.1(a)

where:
\[ A_v = \text{area of tranverse reinforcement}=0.8 \text{in}^2 \]
\[ A_{v,min} = \max \left( 0.75 \sqrt{f_c} \frac{c_1 d}{f_y}, 0.05 \frac{c_1 s}{f_y} \right) = 0.60 \text{in}^2 \]  
ACI 318 Table 9.6.3.4

\[ \lambda = 1 \]

The column design was determined to satisfy the required shear demands. The table below summarizes the calculated shear capacities of the three column types considered.
Finally, the need for slab shear reinforcement in the PT slab was evaluated. The requirement for shear reinforcement in the slab is determined based on ACI 318 Fig. R18.14.5.1, which was discussed in Section 5.5.2 of the Guide. Per the ACI figure, no slab shear reinforcement is required for this design example for any column location. This was determined using drift data from Table 5-2 and shear strength and demand as previously discussed.

**Table 5-3: Shear reinforcement based on shear demand ratio and drift demand ratio**

<table>
<thead>
<tr>
<th>Location</th>
<th>$V_s$ (kips)</th>
<th>$V_c$ (kips)</th>
<th>Total Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
<td>225</td>
<td>125</td>
<td>350</td>
</tr>
<tr>
<td>Exterior</td>
<td>225</td>
<td>160</td>
<td>385</td>
</tr>
<tr>
<td>Corner</td>
<td>225</td>
<td>240</td>
<td>465</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>$v_u/\Phi v_n$</th>
<th>$\delta_{uxi}/h_i$</th>
<th>$\delta_{uyi}/h_i$</th>
<th>Shear Reinforcement Required (Y/N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corner, Level 1</td>
<td>0.20</td>
<td>0.002</td>
<td>0.004</td>
<td>N</td>
</tr>
<tr>
<td>Exterior, Level 1</td>
<td>0.35</td>
<td>0.002</td>
<td>0.004</td>
<td>N</td>
</tr>
<tr>
<td>Interior, Level 1</td>
<td>0.71</td>
<td>0.002</td>
<td>0.004</td>
<td>N</td>
</tr>
<tr>
<td>Corner, Level 2</td>
<td>0.20</td>
<td>0.002</td>
<td>0.003</td>
<td>N</td>
</tr>
<tr>
<td>Exterior, Level 2</td>
<td>0.35</td>
<td>0.002</td>
<td>0.003</td>
<td>N</td>
</tr>
<tr>
<td>Interior, Level 2</td>
<td>0.71</td>
<td>0.002</td>
<td>0.003</td>
<td>N</td>
</tr>
<tr>
<td>Corner, Level 3</td>
<td>0.22</td>
<td>0.001</td>
<td>0.002</td>
<td>N</td>
</tr>
<tr>
<td>Exterior, Level 3</td>
<td>0.45</td>
<td>0.001</td>
<td>0.002</td>
<td>N</td>
</tr>
<tr>
<td>Interior, Level 3</td>
<td>0.89</td>
<td>0.001</td>
<td>0.002</td>
<td>N</td>
</tr>
</tbody>
</table>
5.4. EVALUATE COLUMN STRENGTH BASED ON EARTHQUAKE INDUCED DEMANDS

As emphasized in the Guide, gravity elements must be able to maintain their structural integrity during a seismic event. To perform this check, lateral seismic forces in the ETABS model were magnified by the deflection amplification factor, $C_d = 5$. The results are shown graphically on the same PM interaction diagram:

The induced forces can be seen to exceed the capacity of the column. Therefore, ACI 318-19 §18.14.4.2 design guidance is no longer adequate such that the columns must now satisfy ACI 318-19 §18.14.4.3.

5.4.1. TRANSVERSE STEEL SPACING

A primary design shift in ACI 318-19 §18.14.4.3 is spacing requirements. Following the diagram on the right, updated spacing can be determined as follows:

$$l_o = \max\{c_1, c_2, \frac{1}{6} l_u, 18\text{ in}\} = 25\text{ in} \quad \text{ACI 318 §18.7.5.1}$$

$$s_1 = \min\{c_1, c_2, \frac{1}{4}, 6d_b, s_o, \text{as required by shear}\} = 5\text{ in} \quad \text{ACI 318 §18.7.5.3}$$

$$s_2 = \min\{6d_b, \text{as required by shear}\} = 5\text{ in} \quad \text{ACI 318 §18.7.5.5}$$

$$s_o = 4 + \left(\frac{14-h}{2}\right) \text{ where } 4\text{ in} \leq s_o \leq 6 \text{ in} = 6.4\text{ in} \quad \text{ACI 318 Eq. 18.7.5.3}$$

Here we see no change in the detailing of the column.
There is an additional check with regards to confinement that must be evaluated. This step was not part of the previous design because axial demand is less than 30% of the compressive capacity. However, per ACI 318-19 §18.14.4.3, confinement must be satisfied regardless of axial demand.

Confinement is satisfied when the transverse steel ratio is greater than the following:

\[
\rho_t \geq \max \left\{ 0.3 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_t}{f_y}, 0.09 \frac{f'_t}{f_y} \right\}
\]

ACI 318 Table 18.7.5.4

where:

\[
\rho_t = 0.006
\]

\[
\rho_{\text{required}} = 0.009
\]

This check is **not** satisfied. Reduce transverse spacing to 4 in and check again.

now:

\[
\rho_t = 0.009
\]

\[
\rho_{\text{required}} = 0.009
\]

This check is now satisfied with a reduced spacing. All other checks for axial and shear are satisfied. The updated column cross-section is shown below.
A new PM interaction diagram was created with the updated transverse spacing. Notice that the induced forces during a seismic event continue to exceed the design capacity of the column. This is acceptable. Because gravity columns are not considered part of the lateral-force-resisting system, forces are permitted to exceed design capacities so long as the reinforcement requirements of ACI 318-19 §18.14.4.3 are followed.