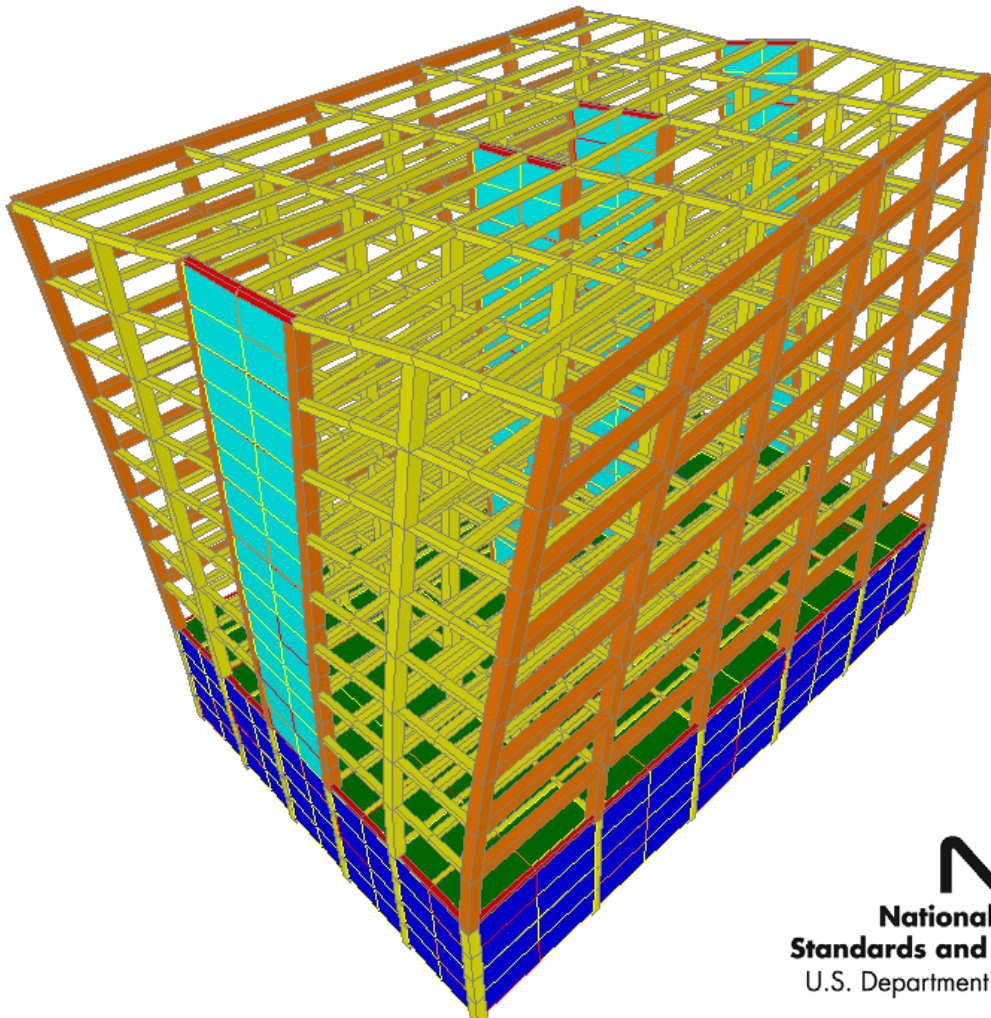


NISTIR 9988

# Evaluation of Contemporary Design of Reinforced Concrete Lateral Resisting Systems using Current Performance Objective Assessment Criteria

Travis Welt



NISTIR 7766

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August, 2010



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## **ABSTRACT**

Seismic design and assessment methods for structures vary in approach. This pilot investigation thoroughly examines the correlation between ASCE/SEI 7-10, *Minimum Design Loads on Buildings and Other Structures*, design methods and ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings*, assessment methods. The project focuses on reinforced concrete buildings composed of 'Special Reinforced Concrete Moment Resisting Frames' and 'Special Reinforced Concrete Shear Walls'. A primary goal of this pilot project is to identify typical inconsistencies or modeling issues for a single building, to provide a guide for the larger and more comprehensive parent investigation. A full seismic design of an archetype structure, using ASCE/SEI 7-10 is completed using both the Equivalent Lateral Force method (for drift) and the Modal Response Spectrum Analysis method (for strength).

Assessment methods in ASCE/SEI 41-06 include linear and nonlinear procedures. Linear procedures model components as perfectly elastic, while approximating nonlinear stress-strain behavior through the use of global stiffness reduction factors. Typically, these methods provide conservative but simple approximations of component behavior. Nonlinear methods incorporate the use of stress-strain relationships beyond the elastic limit, capturing more accurately the nonlinear behavior of a material. Although not exact, this approximation typically provides a more accurate representation of overall system behavior.

Using these linear and nonlinear methods prescribed in ASCE/SEI 41-06, the archetype structure was found to exhibit a large number of component failures, both in flexure and shear. Linear methods, static and dynamic, yielded the highest percentage of failures, indicating notable inconsistencies between design and assessment methods. Additionally, analysis using nonlinear dynamic analysis, based on ASCE/SEI 41-06 parameters, results in a 50% collapse rate, with respect to time history records, at the collapse prevention seismic hazard level. In a practical design situation, these results would require the structure to be either redesigned or retrofitted, and reevaluated. The findings of this pilot investigation indicate the need for further, larger scale evaluation of design and assessment methods for various structure layouts and types.

## EXECUTIVE SUMMARY

Modern seismic design is strictly governed by building code requirements, such as those found in ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures*. Similarly, modern seismic assessment of existing buildings (and seismic rehabilitation) is governed by requirements found in ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings*. However, few studies have examined the agreement between code requirements for design and assessment. The following report is a pilot investigation into the correlation between design and assessment methods for reinforced concrete lateral systems. The report focuses on ‘Special Reinforced Concrete Moment Resisting Frames’ and ‘Special Reinforced Concrete Shear Walls’. The parent project aims to relate design and assessment for a broad spectrum of building layouts and heights, for both reinforced concrete and structural steel lateral resisting systems.

The first phase of the project involves familiarization with all pertinent design and assessment codes and methods, as well as a review of related research. During this process, design and assessment spreadsheets are developed to maintain efficiency throughout the project.

The second phase of the project includes a full seismic design of an archetype structure using ASCE/SEI 7-10 provisions. The structure is composed of ‘Special Reinforced Concrete Moment Resisting Frames’ in one orthogonal direction, with ‘Special Reinforced Concrete Shear Walls’ in the other. The building is analyzed for drift using the Equivalent Lateral Force method, and analyzed for strength using the Modal Response Spectrum Analysis method, both as prescribed in ASCE/SEI 7-10. As the lateral systems in use are primarily reserved for regions with higher seismicity, Seismic Category D is assigned to the structure. Additionally, the pilot project focuses on the upper-bound, or  $D_{MAX}$ , seismic hazard; the parent project will encompass both  $D_{MAX}$  and  $D_{MIN}$  seismic hazards. Wind analysis is also performed to ensure that seismic actions govern the behavior of the structure.

Lastly, the archetype structure is assessed under ASCE/SEI 41-06 provisions. Within ASCE/SEI 41-06, various earthquake hazards, based on probability of exceedance, are used to assess the structure for varying levels of damage. These hazards, called performance objectives, include Immediate Occupancy, Life Safety, and Collapse Prevention. Immediate Occupancy relates to minor and repairable structural damage, which does not prevent immediate re-occupancy of the building. Life Safety involves repairable structural damage to the building, with the possibility for typically non-life-threatening injuries. Lastly, Collapse Prevention refers to major structural damage, and typically imminent collapse. Often times, structural damage at the collapse prevention level cannot be practically repaired.

ASCE/SEI 41-06 provides four different methods for assessment of existing buildings. These methods include the linear static procedure, the linear dynamic procedure, the nonlinear static procedure, and the nonlinear dynamic procedure. Linear methods are based on elastic theory of materials. In general, stiffness reduction factors are used to approximate nonlinearities in a system. These procedures are typically more conservative, but also provide a simple and functional model of the system. Nonlinear

methods are based on the full stress-strain relationships of specific materials. Nonlinear modeling includes stress-strain relationships beyond the elastic limit, and therefore can capture strain-hardening and strain-softening effects. Static methods employ the use of pseudo-lateral forces, similar to the Equivalent Lateral Force method in ASCE/SEI 7-10, as well as pushover analysis for the nonlinear static procedure. These methods are typically a conservative approximation of the behavior of the structure during a seismic event. Conversely, dynamic methods use response spectrum analysis and time history analysis, both of which more accurately represent the seismic demands on a structure through inertial forces. As this investigation aims to correlate design and assessment as a whole, all four methods are evaluated.

The results of this investigation indicate inconsistencies between seismic design and assessment. Specifically, the use of linear procedures results in flexural and shear failures in both the moment frames and the shear walls. These failures are likely due to the variance in modeling and analysis between design and assessment. For example, stiffness reduction factors are much lower using ASCE/SEI 41-06. This leads to larger deformations, and as a result, larger demands. Additionally, the actions on the structure are much higher using ASCE/SEI 41-06. ASCE/SEI 41-06 adjusts component capacities to account for ductility, where ASCE/SEI 7-10 reduces actions based on ductility of the system. Although this poses difficulties for comparison, the results of this investigation do indicate differences in the end result.

Nonlinear methods yielded fewer failures, although under these procedures the archetype structure does not satisfy all ASCE/SEI 41-06 requirements. Additionally, nonlinear dynamic analysis yielded a 50% collapse rate (in terms of time history records) at the collapse prevention performance objective. As a result, in a practical situation, the archetype structure would require redesign.

It is important to pursue further evaluation of design and assessment methods. This pilot investigation was successful in locating notable discrepancies between the two methods, which should be examined on a large scale, as is intended in the parent project. Specifically, the linear procedures appear to be overly conservative, yielding high rates of component failures. Nonlinear procedures, however, are also worthy of further review for comparison, as a number of catastrophic failures occurred as a result of these methods.

## **ACKNOWLEDGEMENTS**

This project was financially supported through an ongoing parent project, *Assessment of First Generation Performance-Based Seismic Design Methods for New Concrete Buildings*, at the National Institute of Standards and Technology (NIST), Building and Fire Research Laboratory. Throughout the investigation, immediate guidance and support was provided by Dr. Jeff Dragovich and Dr. Jack Hayes, both members of the National Earthquake Hazards Reduction Program (NEHRP) at NIST. Additionally, academic advising and oversight was provided by Dr. Larry Fahnstock of the Civil and Environmental Engineering Department at the University of Illinois at Urbana-Champaign. I would like to thank each of them for their advice and suggestions throughout this duration of this project. This project was only made possible through their invaluable contributions.

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## 1. INTRODUCTION

Seismic analysis of buildings plays an integral role in modern structural engineering practice. Such analyses can be applied to design, rehabilitation, and assessment, amongst others uses. However, the method with which seismic analysis is used in these applications varies. Specifically, methods with which new buildings are designed to resist seismic motions differ from those used to assess existing buildings. The following report outlines an investigation in which an archetype structure, designed using contemporary methods governed by ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010), is assessed for compliance with assessment criteria prescribed by ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings*, (ASCE, 2006).

This project is a pilot investigation for a larger ongoing project. The parent investigation aims to evaluate archetypal structures of various heights and layouts, including both reinforced concrete and structural steel buildings. This pilot investigation helps to provide a guide through the entire process for the large scale project, surfacing areas of concern and possibilities for increased efficiency.

Specifically, this investigation examines reinforced concrete structures, composed of commonly used lateral resisting systems; reinforced concrete moment frames and reinforced concrete shear walls will be evaluated. In order to practically evaluate the archetype structure, an upper-bound seismic hazard is used for design and assessment. In regions of such high seismicity, codes require that reinforced concrete structures adhere to strict detailing requirements. These requirements are implemented in “Special Reinforced Concrete Moment Frames” and “Special Reinforced Concrete Shear Wall”, as defined in ASCE/SEI 7-10. This report focuses solely on these lateral resisting systems in its evaluation.

Prior to the design phase, a detailed review and familiarization with current design practices was completed. For this purpose, the design example in Chapter 7 of FEMA P752, *NEHRP Recommended Provisions: Design Examples*, (FEMA, In-Print), was thoroughly completed. This process ensures that every detail of design, prescribed in either ASCE/SEI 7-10 or ACI 318-08, *Building Code Requirements for Structural Concrete and Commentary*, (ACI, 2008), is understood and implemented correctly. During this phase of the project, a design spreadsheet is developed for use in design of the archetype structure, using Microsoft Excel. Similarly, prior to the assessment phase of the project, a comprehensive review and familiarization of assessment methods in ASCE/SEI 41-06 was completed. An assessment spreadsheet is also developed using Microsoft Excel to maintain efficiency during the actual assessment phase.

## 2. ARCHETYPE STRUCTURE ANALYSIS AND DESIGN

### 2.1. Seismic Hazard Definition and Design Criteria

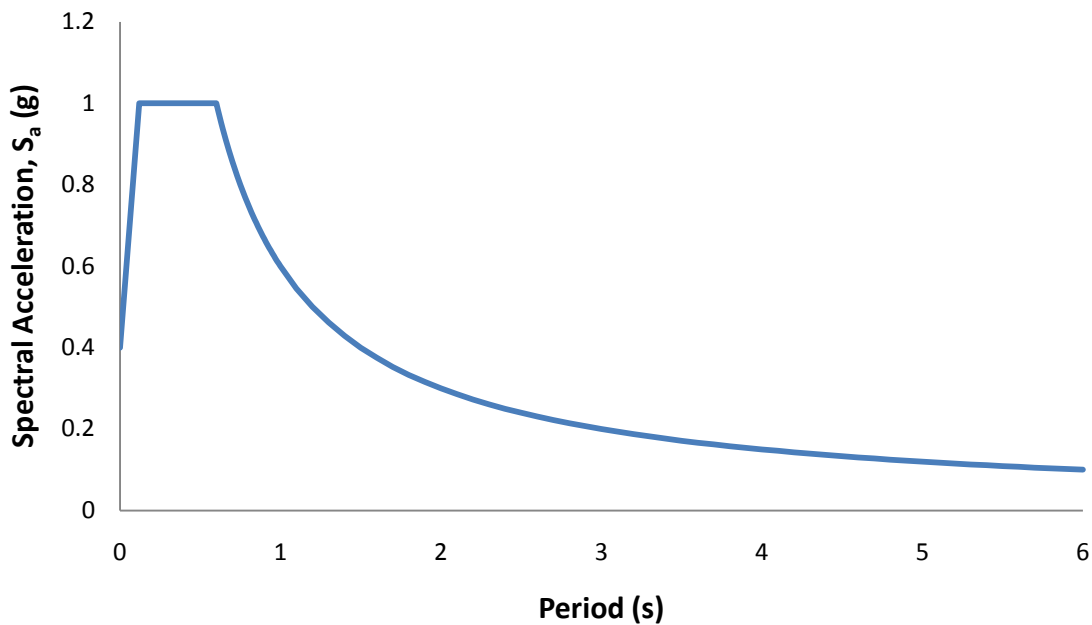
Development of Seismic Hazard for the archetype structure considers documentation within FEMA P695, *NEHRP Quantification of Building Seismic Performance Factors*, (FEMA, 2009), which uses probabilistic regions of Maximum Considered Earthquake (MCE) ground motions, as defined in ASCE/SEI

7-10 Section 21.2. FEMA P695 also provides lower and upper bounds on the short-period and 1-second spectral response accelerations,  $S_s$  and  $S_1$ . The investigation is limited to the upper bound ground motions (denoted  $D_{max}$ ). The archetype structure is not located in a specific region, and therefore is assigned to Site Class D as required in ASCE/SEI 7-10 Section 11.4.2. Design parameters are calculated using ASCE/SEI 7-10 Chapter 11 methods, calculations for which are located in Appendix A. Table 1 shows the Seismic Hazard for the archetype structure.

**Table 1.**  $D_{MAX}$  Seismic Hazard Parameters

Design Category	$S_s$ (g)	$S_1$ (g)	$S_{DS}$ (g)	$S_{D1}$ (g)
$D_{MAX}$	1.50	.60	1.00	.60

As permitted in ASCE/SEI 7-10 Section 12.6, both Equivalent Lateral Force (ELF) and Modal Response Spectrum Analysis (MRSA) are used in the design of the archetype structure. Development of the Response Spectrum conforms to ASCE/SEI 7-10 Section 11.4.5. Figure 1 shows the calculated response spectrum corresponding to  $D_{MAX}$  design parameters.



**Figure 1.** ASCE/SEI 7-10 Design Earthquake Response Spectrum for Seismic Design Category D

The archetype structure, although simplified, is intended to emulate a multi-story mixed use (retail and office) building, with basement parking levels. Corresponding live loads conform to ASCE/SEI 7-10 Table 4-1. A typical cladding load of 15 PSF (vertical) is applied to at the perimeter of the structure. Table 2 shows the uses and super-imposed loads for each level in the archetype structure. Wind loads are also considered in the design, using a 3-second gust speed of 115 mph at an Exposure Category C. These parameters are representative of typical upper-bound, non-hurricane zone wind speeds. ASCE/SEI 7-10 Figure 26.5-1A provides guidance for obtaining such 3-second gust wind speeds.

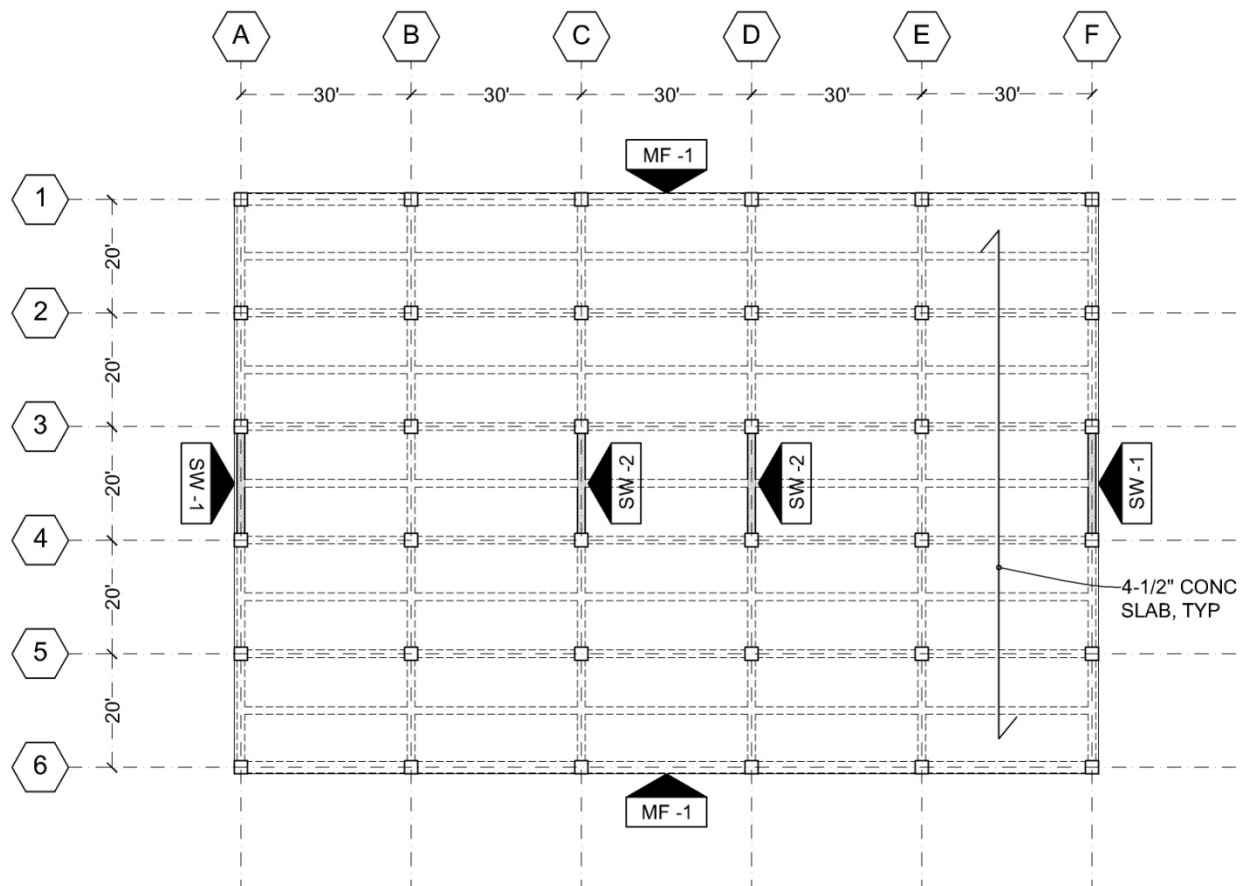
**Table 2.** Building Uses and Live Loads

Level	Use	Live Load (PSF)	Partition/Roofing Load (PSF)	M/E/P/Ceiling Load (PSF)
Roof	-	20	10	10
Level 08	Office	50	15	10
Level 07	Office	50	15	10
Level 06	Office	50	15	10
Level 05	Office	50	15	10
Level 04	Office	50	15	10
Level 03	Office	50	15	10
Level 02	Office	50	15	10
Level 01	Retail	100	15	10
Level A	Parking	40	15	10
Level B	Parking	40	15	10

## 2.2. Structural Layout and Configuration

The layout of the archetype structure is the result of maintaining simplicity while ensuring modern design practices. In order to provide simplicity, a rectangular floor plan with orthogonal lateral resisting systems is used. In addition, the lateral resisting systems do not share common components, eliminating the need for concurrent lateral loading requirements. During the initial design phase, multiple floor plan options are reviewed by licensed professional engineers to ensure a practical design. Additionally, specific layout details such as bay widths, story heights, and preliminary sizes are chosen to closely resemble previously peer reviewed structures. Both FEMA P752 and Haselton (2006), *Assessing Seismic Collapse Safety of Modern Reinforced Concrete Moment-Frame Buildings*, are used for this purpose. Figure 2 shows the final floor plan layout for the archetype structure.

The finalized layout is composed of two perimeter lines of Special Reinforced Concrete Moment Resisting Frames in the East-West (E-W) direction, and 4 full height Special Reinforced Concrete Shear Walls in the North-South (N-S) direction. Interior and exterior columns not acting as part of the lateral systems are designed to resist gravity loads only. Interior beams are designed to resist gravity loads only, and are configured to provide a one-way slab system with a 4-1/2 inch thick slab spanning 10 feet. Throughout the structure, beams acting as part of the moment-resisting frame are 24 inches wide by 32 inches deep, while the interior beams are 16 inches wide by 20 inches deep. Columns throughout the structure are 28 inches square. The shear walls are 16 inches thick, with columns acting as boundary elements on each side of each wall. All structural reinforced concrete members use normal weight concrete with  $f'_c$  of 5000 PSI and ASTM A615 reinforcing steel.



**Figure 2.** Archetype Structure Floor Framing Plan

### 2.3. Analysis Methodology

In modern building design, there are three analysis methods used to determine the seismic behavior of a structure. As noted in ASCE/SEI 7-10, these methods include the Equivalent Lateral Force (ELF) method, the Modal Response Spectrum Analysis (MRSA) method, and the Time History Analysis (THA) method. Additionally, ASCE/SEI 7-10 permits the use of auxiliary rational analysis methods, however such methods are out of the scope of this project. ASCE/SEI 7-10 Table 12.6-1 provides guidance on analysis procedure selection. In general, this selection is based on seismic design category, structural system, dynamic properties, and structural regularity.

Table 3 lists the horizontal and vertical irregularities in the archetype structure in accordance with ASCE/SEI 7-10 Table 12.3-1 and Table 12.3-2. In order to exclusively use the ELF method, the structure must not exhibit specific structural irregularities, and also must meet size limitations. The archetype structure exceeds the 2 story (above grade) requirement, but is within the 160 foot height restriction. As shown in Table 3, no irregularities exist in the structure, and therefore it is permissible by code to design the structural using only the ELF method. However, as the ELF method typically results in conservative designs, this research will use both the ELF method and the MRSA method in order to produce a more

efficient design. The THA method also provides a more representative analysis model of ground motions, however due to the simplicity of the structure, this method would be unlikely to produce a largely more efficient design when compared with the MRSA method.

**Table 3.** Horizontal and Vertical Irregularities

Horizontal Irregularities	Existence	Vertical Irregularities	Existence
1a. Torsional	No <sup>1</sup>	1a. Stiffness-Soft Story	No <sup>1</sup>
1b. Extreme Torsional	No <sup>1</sup>	1b. Stiffness-Extreme Soft Story	No <sup>1</sup>
2. Reentrant Corner	No	2. Weight (Mass)	No <sup>1</sup>
3. Diaphragm Discontinuity	No	3. Vertical Geometric	No
4. Out-of-Plane Offset	No	4. In-Plane Discontinuity	No
5. Nonparallel System	No	5a. Weak Story	No <sup>1</sup>
-		5b. Extreme Weak Story	No <sup>1</sup>

1. Reference Appendix A for supporting calculations

ASCE/SEI 7-10 also provides load combination requirements for seismic design. Pertinent combinations include:

- $(1.2 + 0.2S_{DS})D + \rho Q_E + 0.5L$
- $(0.9 - 0.2 S_{DS})D + \rho Q_E$

Where,

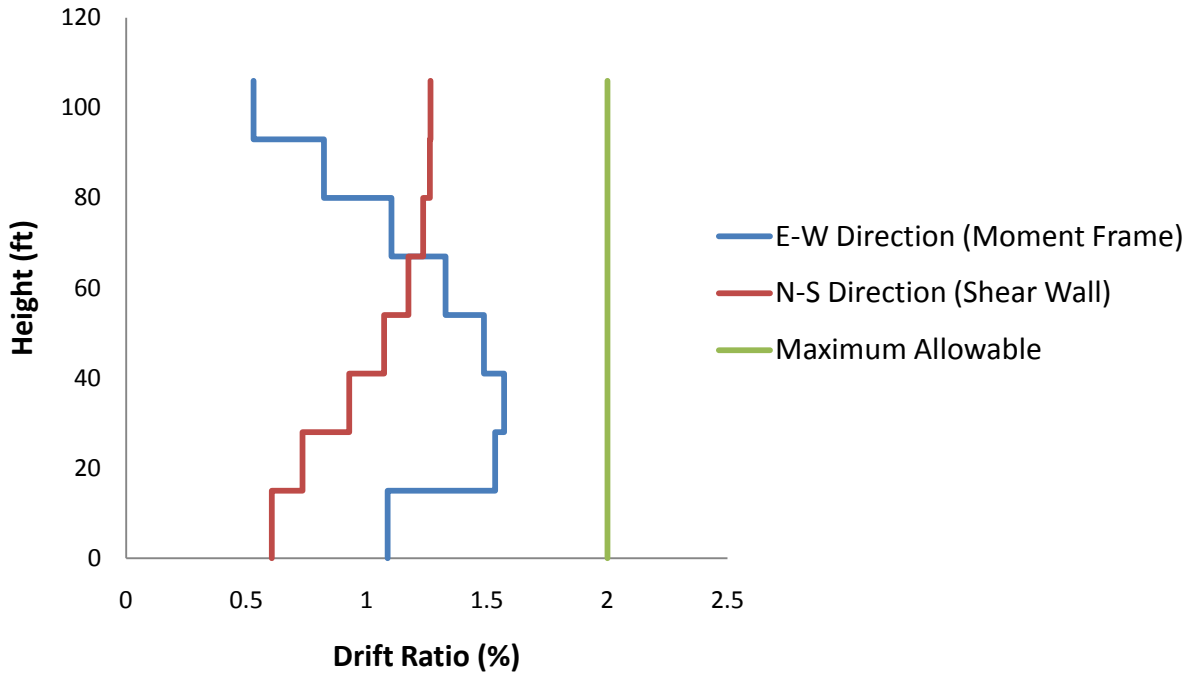
- $S_{DS}$  = Short-period design spectral acceleration
- $S_{D1}$  = 1-second design spectral acceleration
- D = Dead Load
- L = Live Load
- $Q_E$  = Effect of horizontal seismic forces
- $\rho$  = Redundancy Factor

### 2.3.1. Equivalent Lateral Force Method

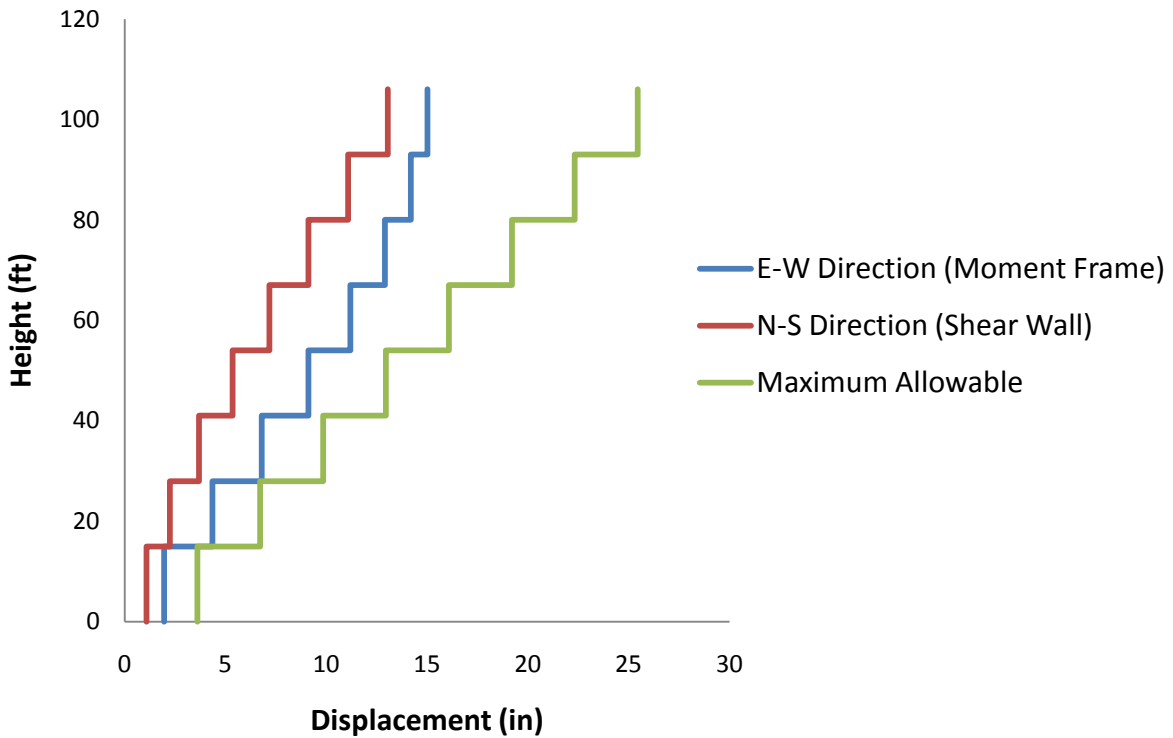
The general process of the ELF method includes calculating equivalent lateral static forces that represent the inertial forces that a structure would experience during a seismic event. These forces are derived using a seismic response coefficient, based on seismic design parameters, that is applied to the seismic weight of the structure. This overall base shear is then distributed vertically based on the distribution of seismic weight at each level. The structure must be able to resist seismic forces in any direction, and therefore typically the ELF distribution is applied in both orthogonal directions. If orthogonal lateral resisting systems share components, concurrent loading is required. For this investigation, the structure was not analyzed for strength using the ELF method, as a more efficient design can typically be obtained by using the MRSA method. However, as required by ASCE/SEI 7-10, the ELF method is used in calculating maximum drifts. Calculations concerning the equivalent lateral static loads and distributions can be found in Appendix A.

As noted in ASCE/SEI 7-10 Section 12.12.1, story drifts and displacements may be calculated using either the ELF or MRSA methods. However, as drifts and displacements are of high importance to the integrity of a structure, the ELF method is used to provide a conservative approximation. In order to calculate these deformations, both inherent and accidental torsion must be accounted for in the model. Inherent torsion is that torsion which comes from forces acting through the center of mass of the structure being resisted by forces located at the center of rigidity. As the archetype structure is symmetric in both directions, there is no inherent torsion. Accidental torsion typically is caused by a number of factors, most commonly the variance in propagation of seismic waves, and the unlikelihood of a perfectly constructed symmetric structure. Accidental torsion is accounted for by applying a torsion equal to a 5% eccentricity of shear loading, to the center of mass of the structure. Calculations concerning torsional effects can be found in Appendix A.

The results of the model, including the effects of torsion, are used to calculate the existence of torsional irregularities, as well as any required accidental torsion amplification. If amplification is required, the model must be analyzed again for finalized deformations. This process theoretically can be iterated a large number of times until the amplification factor converges to unity, however typical design practice implements only one iteration. Additionally, if design uses the MRSA method, drifts are required to be scaled by 85% of the ratio of the ELF base shear to the MRSA base shear. This is required only if the MRSA base shear is less than 85% of the ELF base shear. Finally, drifts must be multiplied by the deflection amplification factor, which is based on the lateral system type. For the archetype structure, drift ratios are limited to 2%, as required by ASCE/SEI 7-10 Table 12.12-1. Figure 3 shows the calculated story drifts for the archetype structure in each orthogonal direction. Figure 4 shows the calculated story deflections for the archetype structure in each orthogonal direction. Calculations for story drift and MRSA scaling factors can be found in Appendix A.



**Figure 3.** Archetype Structure Story Drift Ratios



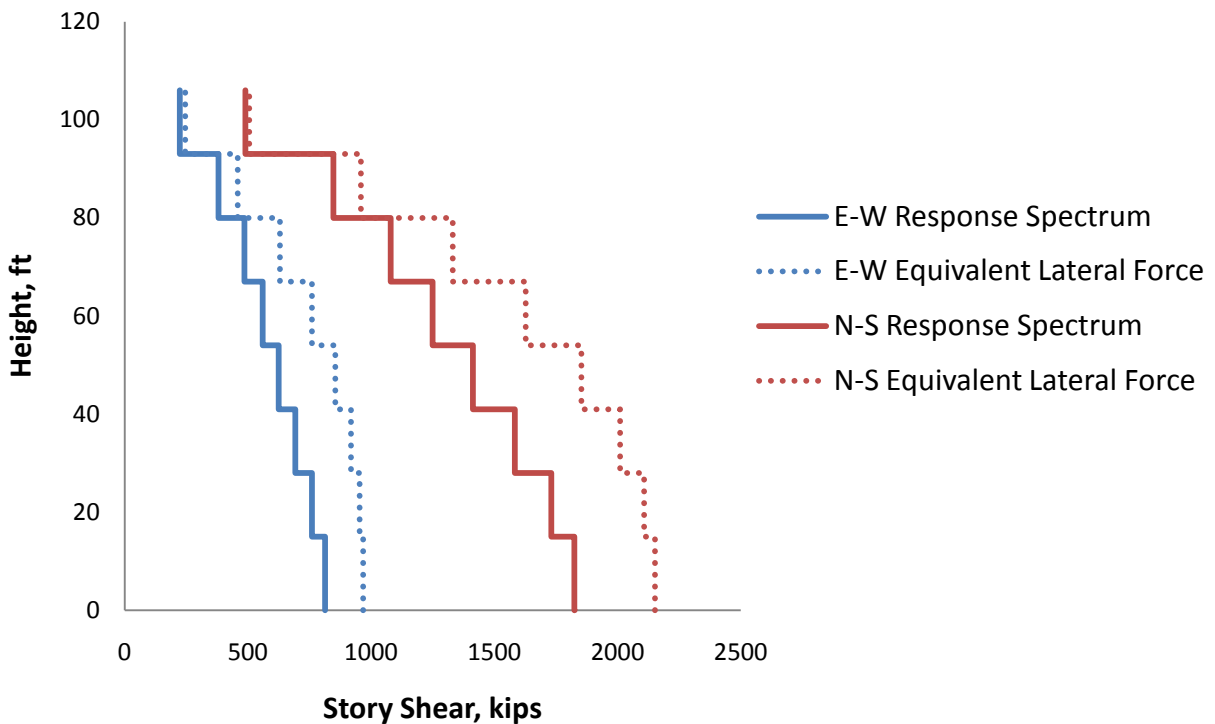
**Figure 4.** Archetype Structure Story Deflections

**2.3.2. Modal Response Spectrum Analysis Method**



The MRSA method models the actions on a structure from seismic events more realistically than the ELF method through the use of ground motions; this typically results in a more accurate depiction of the response of the structure to seismic loading. Demands from the MRSA method are the result of inertial forces in the structure that develop due to the ground motion. The general theory behind this method can be related to basic structural dynamics of multi-degree of freedom systems. The structure typically has as many mode shapes as levels, however only a relatively small percentage of the mass participates in any mode other than the fundamental mode. ASCE 7-10 Section 12.9.1 requires that a structure model includes a sufficient number of modes in order to reach a mass participation factor of at least 90%. This ensures that the majority of the actions from seismic activity are adequately incorporated. Calculations indicating compliance with this requirement can be found in Appendix A.

As mentioned, in this investigation the MRSA method is used to determine the strength demands on the structure, as it can typically provide more efficient designs. Moment, shear, and axial diagrams for a typical moment frame, and a typical exterior and interior shear wall can be found in Appendix B. In support of the decision to use the MRSA method for strength demands, a story shear comparison, found in Figure 5, clearly indicates the lower demands on the structure using the MRSA method.



**Figure 5.** Story Shear Comparison between ELF and MRSA Method

### 2.3.3. Redundancy Factor

ASCE/SEI 7-10 provides additional requirements which take into account the redundancy in the lateral system of a structure. These requirements are used to ensure that in the event of a lateral system component failure, the structure can continue to support lateral loads for the remainder of the seismic

(or wind) event and until repairs can be made. For this reason, in Seismic Design Categories D and above, ASCE/SEI 7-10 Section 12.3.4 requires that a redundancy factor be applied to the lateral resisting systems of a structure. This redundancy factor,  $\rho$ , is permitted to be taken as unity for the specific conditions noted in ASCE/SEI 7-10 Section 12.3.4.1. Otherwise, ASCE/SEI 7-10 Section 12.3.4.2 requires that  $\rho$  equal to 1.3 be applied to all lateral resisting systems. As an exception, the code provides guidance on calculating the inherent redundancy in a structure; if the structure is adequately redundant,  $\rho$  is permitted to be taken as unity. This analysis ensures that the loss of a single moment frame bay or shear wall segment does not result in the loss of more than 33% of the story strength, nor does the resulting structure have an extreme torsional irregularity. Calculations regarding structure redundancy in each direction can be found in Appendix A, which indicate that  $\rho$  may only be taken as unity for the moment frames.

#### **2.3.4. P-Delta Effects**

P-Delta effects are the additional stresses on a system component due to gravity loads acting on the deformed configuration of a system. These effects can be captured either through a nonlinear analysis, or a simplified linear analysis which implements geometric adjustments that represent these deformations. A full analysis must include P-Delta effects as they can greatly increase the stresses in a system component. However, ASCE/SEI 7-10 Section 12.8.7 permits P-Delta effects to be neglected in the case where the stability coefficient,  $\theta$ , is less than or equal to 0.10. Design calculations, found in Appendix A, indicate that P-Delta effects need not be considered for the archetype structure.

### **2.4. Archetype Structure Design Modeling**

ASCE/SEI 7-10 Section 12.7.3 provides guidance on structural modeling. In general, a model must account for the stiffness and strength of structural components, including the reduced stiffness of reinforced concrete components resulting from cracking. Additionally, force distributions, deformations, and masses (for the MRSA method) must be included in the model. The code permits 2-Dimensional models of lateral systems if certain irregularities do not exist, however in order to capture a more accurate representation, the archetype structure is modeled as a 3-Dimensional structure. SAP2000 (CSI, 2010) structural analysis software is used in this investigation.

#### **2.4.1. Structural Component Modeling**

Stiffness and strength of components is important in providing an accurate model of the archetype structure. Material properties, section properties, flanged construction, and anticipated cracking are all important factors in modeling of component stiffness. Theoretically, the cracked section of a concrete member varies along the span with the moment demand. However, modeling such a variance is difficult for large scale models, and so various stiffness reduction factors are used for different structure components. Reduction factors used in the modeling of the archetype structure were adopted from FEMA P752. Table 4 shows the various factors corresponding to different components. Flanged construction, or T-Beams, for the archetype structure are modeled using major axis moment of inertia scaling factors in SAP2000. Beams are modeled as rectangular sections and therefore simple calculations, found in Appendix A, are required to obtain a scale factor that provides a rectangular section with the equivalent T-Beam moment of inertia.

**Table 4.** Stiffness Reduction Factors for Structural Modeling

	<b>Beams</b>	<b>Columns</b>	<b>Walls</b>
<b>Stiffness Reduction</b>	$0.5 * E I_g$	$0.7 * E I_g$	$0.8 * E I_g$

Strength is not explicitly modeled in the SAP2000 model. The model is, however, used to determine strength requirements for various components. As this investigation does not make use of the design capabilities of SAP2000, such strength requirements are used in the design spreadsheet during the design process. More information on the design of structural components can be found in Section 2.5.

#### **2.4.2. Mass Modeling**

Mass modeling of structures can be accomplished through many different methods. Each method has strengths and weaknesses, such as efficiency, accuracy, and ease of use. Two methods are explored to examine the possibility that a sufficiently accurate, yet simple method is employed. The archetype structure is modeled using both a “smeared mass” approach, and a “lumped mass” approach. These methods are explained in more detail below, however the final model uses the smeared mass approach to achieve the most accurate results.

##### **2.4.2.1. Smeared Mass Approach**

In this more complex method, the mass is distributed over the entire floor system at each level. Similarly sized floor (shell) elements are used to discretize the floor system. The option to refine the mesh is at the discretion of the designer, however it is prudent to place nodes at all of the columns within the structure so that the area masses are lumped directly to each column. Smeared area masses are then lumped at the columns associated with each floor element. Additionally, the seismic contribution of non-structural elements, such as curtain walls, is linearly distributed (using vertical tributary area) at the perimeter frame or wall elements of the structure. From this point, it is simply a matter of running a multi degree of freedom dynamic model of the structure (in this case, SAP2000 was used for this analysis) to obtain the structure modal response.

In this method, all lateral resisting members, including columns, beams, and structural shear walls, are assigned a self mass. This is done to ensure the center of mass of these elements is in agreement with the actual structure. Floor elements and secondary framing is assigned a zero mass, as additional area masses will be manually added to accommodate for both the self mass and superimposed seismic masses (such as roofing, utilities, etc). Lastly, the ground floor and below are assigned a zero mass, as these elements are assumed to experience relatively small displacements, and therefore do not significantly contribute to the modal response of the building. A more in depth look into the effects of the below ground levels (and site conditions) to the modal response of the structure is out of the scope of this project.

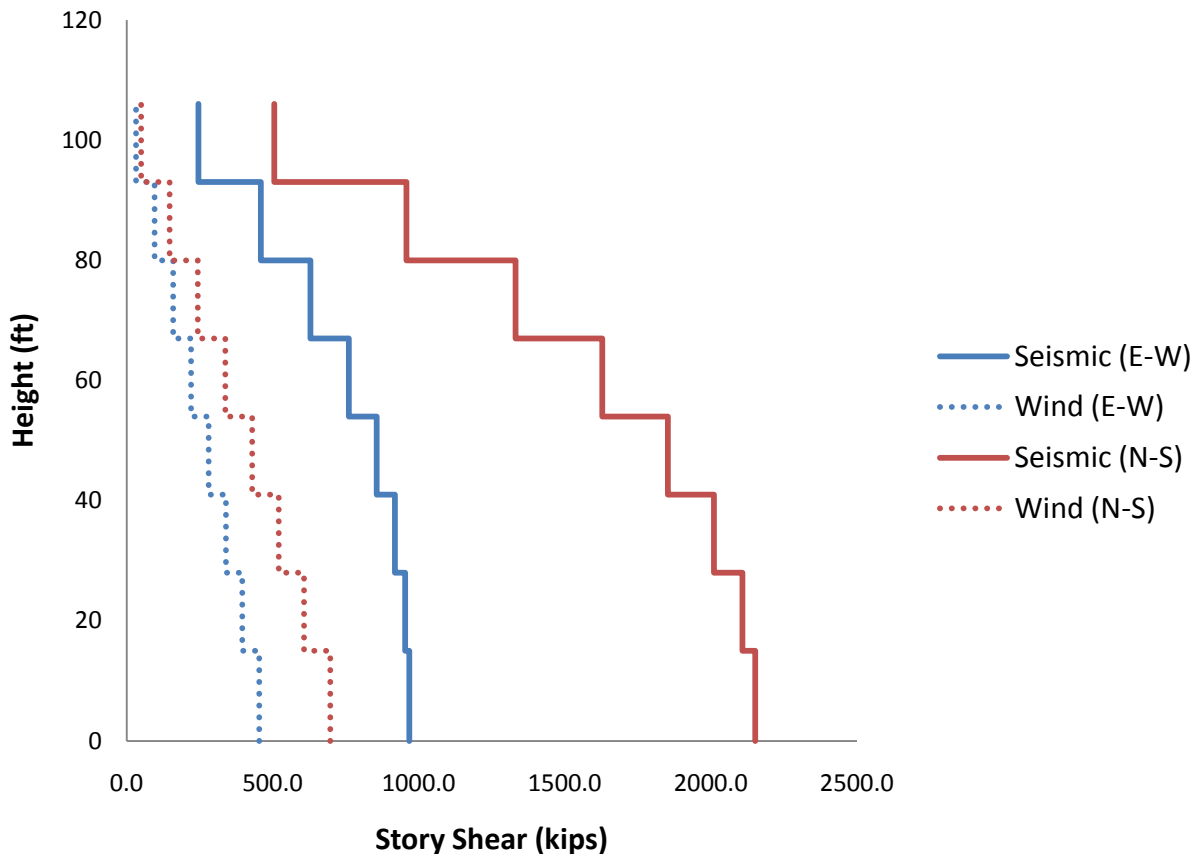
##### **2.4.2.2. Lumped Mass Approach**

The lumped mass approach takes a more simplified approach than in the smeared mass approach. In this method, the full seismic mass of each floor is taken as a single lumped mass located at the center of mass of each floor. To retain torsional effects, this lumped mass is distributed amongst the corner

columns. In this method, all elements, including structural shear walls, beams, columns, and curtain walls are assigned a zero mass. In this process, special attention must be paid in calculating the correct total mass at each level to ensure an accurate model. Note that this does not reduce the model to a simple stick model, as the structure geometry as well as the various element stiffnesses are still included in the model. Keeping with consistency, the model is analyzed using SAP2000 modal analysis.

## 2.5. Lateral System Design

The lateral design of a structure is based typically on wind and seismic forces. Although this investigation is intended to be a review of seismic design and assessment methods, it is pertinent to examine the behavior of the structure under design wind loads to ensure that the seismic demands control the lateral system design. ASCE/SEI 7-10 Chapter 26 and 27 provide requirements for design wind forces and directional combinations. Calculations using the Directional Method can be found in Appendix A. Figure 6 shows a comparison between story shears resulting from MRSA and wind analyses. Clearly, wind demands do not control the design of the archetype structure.



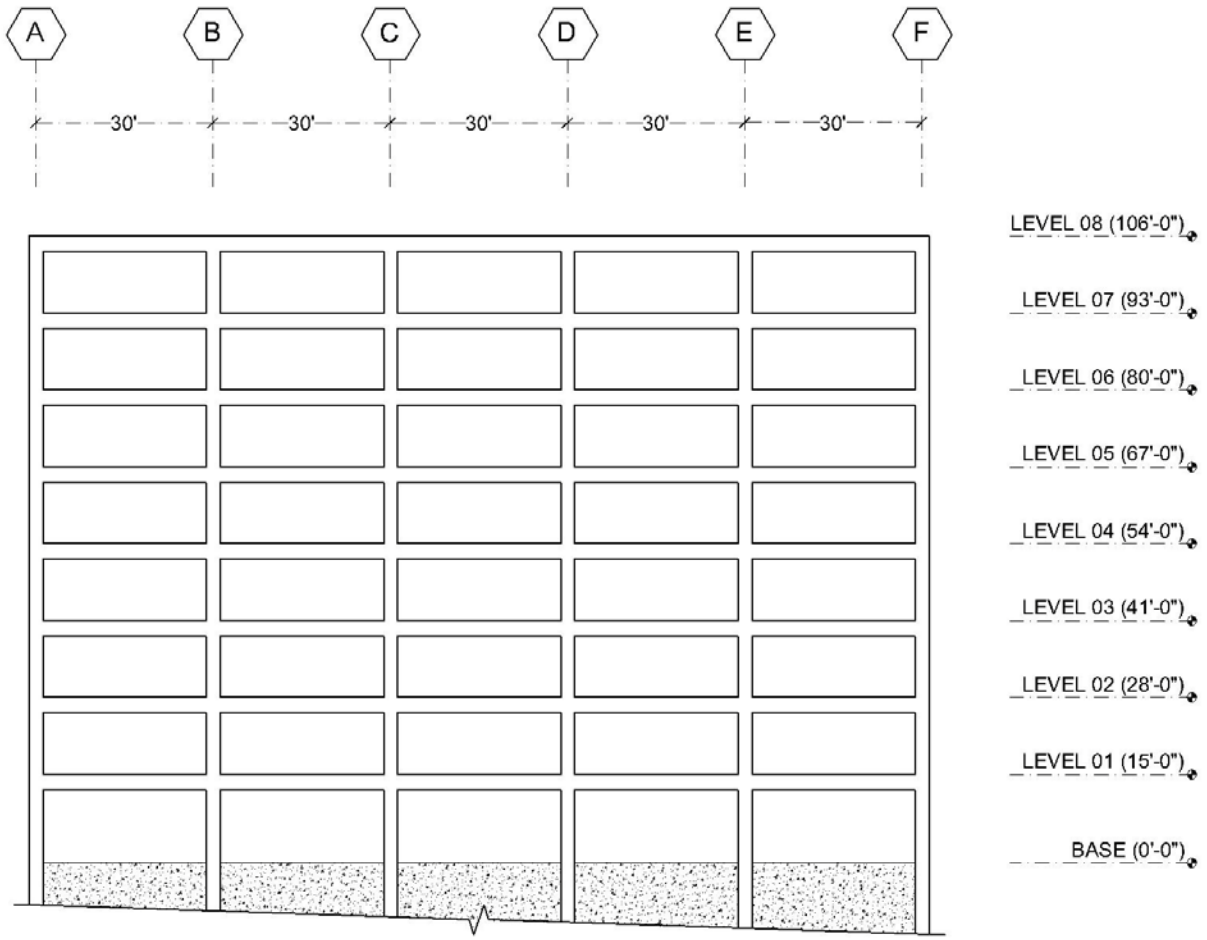
**Figure 6.** Story Shear Comparison between Seismic and Wind Demands

Additionally, in an effort to represent a viable gravity system, nominal calculations were made to ensure secondary framing members, including the slab, are capable of supporting the gravity loads of each floor.

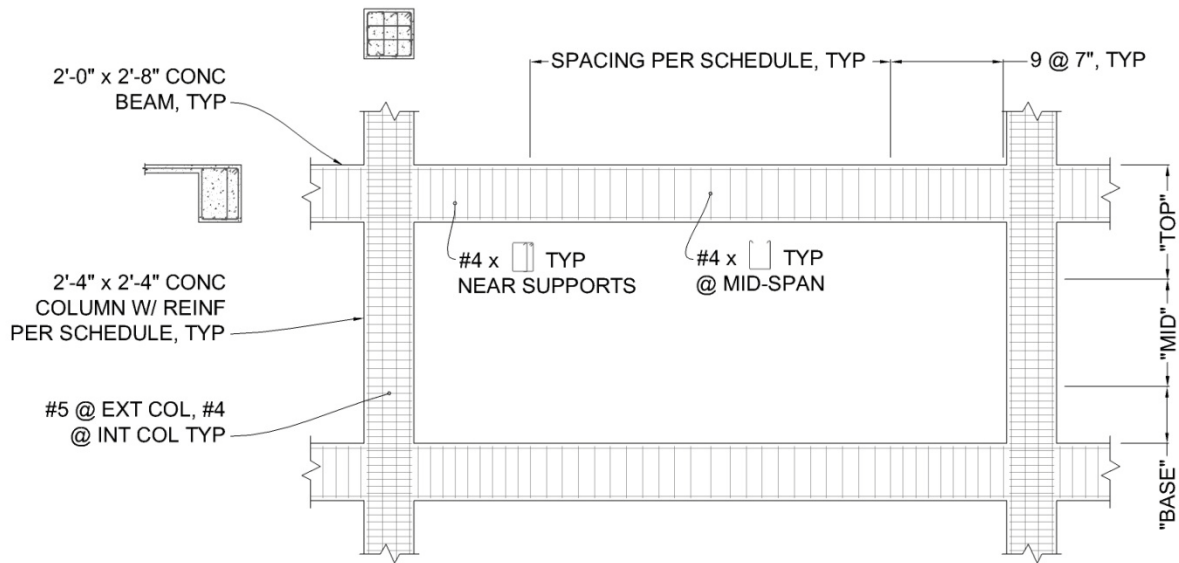
### **2.5.1. Special Reinforced Concrete Moment Resisting Frame Design**

The general lateral design procedure for the moment frame adheres to the capacity design philosophy. Unless otherwise specified, materials are manufactured with strengths at or above the specified minimums. This increased strength (referred to in design as “probable strengths”) can cause a component to yield in an unanticipated mode, such as shear. Shear is not a preferred failure mode as it is a brittle failure (non-ductile) and typically occurs without warning.

Design of the moment frame is also based on the strong column weak beam (SCWB) premise. It is important to instigate this principle as it provides a level of protection against catastrophic failure in the event of a simple component failure. To prevent this type of failure, columns are designed to resist the full flexural capacity of the beams which frame into the joint. This ensures that the column will not fail prior to, or concurrently with the beam. The SCWB method is prescribed for design through the requirements in ACI 318-08 Chapter 21. Moment, shear, and axial diagrams used in design from MRSA analysis can be found in Appendix B. Figure 7 shows a typical layout elevation for moment frames, and Figure 8 shows typical moment frame reinforcing layout with member cross sections. Additionally, Figure 9 is a moment frame elevation which indicates various flexural, shear, and axial demand to capacity ratios. As the building is symmetric, this elevation represents a typical structural design for the two moment frames.



**Figure 7.** Typical Moment Frame Elevation



**Figure 8.** Typical Moment Frame Bay Reinforcing Layout

Moment frame beams in the archetype structure are designed with flexural reinforcing capable of resisting moment demands from MRSA. Specified material strengths are used to ensure that the minimum capacity of the beam is adequate to resist such demands. The probable flexural strength of the beam is then used to calculate the probable shear demand at the onset of flexural yielding. This is done to satisfy capacity design principles, which ensure flexural yielding prior to shear failure. Figure 10 shows the reinforcing schedule for the moment frame beams. Supporting calculations for moment frame beams can be found in Appendix A.

To provide SCWB action, the probable flexural strengths of adjacent beams are used to determine the flexural demands on the columns. As columns experience both flexural and axial demands, interaction analysis is used to determine the adequacy of column reinforcement. Interaction diagrams for all columns are calculated using PCA Column, and can be found in Appendix C. These interaction diagrams represent the various axial and flexural demands on all columns of a certain reinforcing pattern. Similar to the beams, the shear demand on the columns is based on the probable onset of flexural yielding in the columns. Figure 11 provides transverse reinforcing information for the moment frame columns. Corner columns along the moment frame have (12) #8 longitudinal bars while interior edge columns have (10) #10 longitudinal bars. Supporting calculations for the moment frame columns can be found in Appendix A.

	Grid A		Grid B		Grid C		Grid D		Grid E		Grid F
Level 08	Joint $\Phi V_{pr}/V_n = 0.755$	0.276	0.750	0.171	0.597	0.201	0.301	0.378	0.288	0.378	0.755
Story 8	$(M_u/\Phi M_n)_{PMAX} = 0.442$							0.578		0.578	0.442
	$(M_u/\Phi M_n)_{PMIN} = 0.482$							0.611		0.611	0.482
	$V_{pr}/\Phi V_n = 0.870$							0.899		0.899	0.870
	$P_{MAX}/A_g f_c' = 0.017$							0.025		0.025	0.017
Level 07	Joint $V_{pr}/\Phi V_n = 0.634$	0.355	0.878	0.395	0.710	0.457	0.391	0.445	0.385	0.445	0.634
Story 7	$(\Phi M_u/M_n)_{PMAX} = 0.512$							0.691		0.691	0.512
	$(\Phi M_u/M_n)_{PMIN} = 0.610$							0.760		0.760	0.610
	$V_{pr}/\Phi V_n = 0.870$							0.899		0.899	0.870
	$P_{MAX}/A_g f_c' = 0.038$							0.052		0.052	0.038
Level 06	Joint $V_{pr}/\Phi V_n = 0.738$	0.404	0.813	0.650	0.693	0.685	0.435	0.510	0.429	0.510	0.738
Story 6	$(\Phi M_u/M_n)_{PMAX} = 0.460$							0.645		0.645	0.460
	$(\Phi M_u/M_n)_{PMIN} = 0.597$							0.726		0.726	0.597
	$V_{pr}/\Phi V_n = 0.870$							0.899		0.899	0.870
	$P_{MAX}/A_g f_c' = 0.060$							0.073		0.073	0.060
Level 05	Joint $V_{pr}/\Phi V_n = 0.738$	0.441	0.921	0.835	0.805	0.858	0.469	0.510	0.461	0.510	0.738
Story 5	$(\Phi M_u/M_n)_{PMAX} = 0.420$							0.606		0.606	0.420
	$(\Phi M_u/M_n)_{PMIN} = 0.587$							0.698		0.698	0.587
	$V_{pr}/\Phi V_n = 0.870$							0.899		0.899	0.870
	$P_{MAX}/A_g f_c' = 0.083$							0.106		0.106	0.083
Level 04	Joint $V_{pr}/\Phi V_n = 0.743$	0.472	0.994	0.988	0.899	0.995	0.495	0.510	0.486	0.510	0.743
Story 4	$(\Phi M_u/M_n)_{PMAX} = 0.488$							0.716		0.716	0.488
	$(\Phi M_u/M_n)_{PMIN} = 0.725$							0.844		0.844	0.725
	$V_{pr}/\Phi V_n = 0.870$							0.741		0.741	0.870
	$P_{MAX}/A_g f_c' = 0.106$							0.123		0.123	0.106
Level 03	Joint $V_{pr}/\Phi V_n = 0.936$	0.502	0.863	0.902	0.791	0.892	0.520	0.648	0.510	0.648	0.936
Story 3	$(\Phi M_u/M_n)_{PMAX} = 0.456$							0.684		0.684	0.456
	$(\Phi M_u/M_n)_{PMIN} = 0.718$							0.818		0.818	0.718
	$V_{pr}/\Phi V_n = 0.870$							0.741		0.741	0.870
	$P_{MAX}/A_g f_c' = 0.129$							0.148		0.148	0.129
Level 02	Joint $V_{pr}/\Phi V_n = 0.936$	0.526	0.912	0.967	0.853	0.976	0.539	0.648	0.529	0.648	0.936
Story 2	$(\Phi M_u/M_n)_{PMAX} = 0.430$							0.723		0.723	0.430
	$(\Phi M_u/M_n)_{PMIN} = 0.714$							0.794		0.794	0.714
	$V_{pr}/\Phi V_n = 0.870$							0.741		0.741	0.870
	$P_{MAX}/A_g f_c' = 0.153$							0.173		0.173	0.153
Level 01	Joint $V_{pr}/\Phi V_n = 0.966$	0.537	0.895	0.967	0.866	0.954	0.534	0.663	0.534	0.663	0.966
Story 1	$(\Phi M_u/M_n)_{PMAX} = 0.409$							0.784		0.784	0.409
	$(\Phi M_u/M_n)_{PMIN} = 0.710$							0.771		0.771	0.710
	$V_{pr}/\Phi V_n = 0.729$							0.621		0.621	0.729
	$P_{MAX}/A_g f_c' = 0.178$							0.199		0.199	0.178

Figure 9. Typical Moment Frame Demand Capacity Ratios





The joints at which the beams and columns intersect are designed for a shear demand representing the onset of flexural yielding in adjacent framing members. As joints are integral in providing support of gravity loads, this component is typically designed not to fail until the ultimate collapse of the structure. Supporting calculations for the moment frame joints can be found in Appendix A.

### **2.5.2. Special Reinforced Concrete Shear Wall Design**

Design of the shear walls and corresponding boundary columns is based on ACI 318-08 Chapter 21 provisions. Shear reinforcement is calculated using the shear demand results from the MRSA. Shear reinforcing is based on developing adequate shear capacity, calculated using ACI 318-08 Equation 21-7.

Design for flexural and axial loads is similar to that of the moment frame columns, in that interaction diagrams are required to ensure adequate capacity. Again, PCA Column is used for this analysis, and representative design interaction diagrams for the base level of each shear wall can be found in Appendix C.

Boundary elements are used to ensure adequate ductility in the shear wall. Two methods are prescribed in ACI 318-08 Section 21.9.6.2 and Section 21.9.6.3 to calculate where they are required. The second method, a stress-based procedure, is used in this investigation. By analysis, neither the interior or exterior shear walls require additional boundary element reinforcing.

Design drawings for exterior and interior shear walls can be found in Figure 12 and Figure 13, respectively. Exterior and interior shear wall elevations indicating demand to capacity ratios are shown in Figure 14. Support calculations for shear reinforcing in the wall panel, flexural and shear reinforcing in the boundary columns, and adequacy of boundary elements can be found in Appendix A.

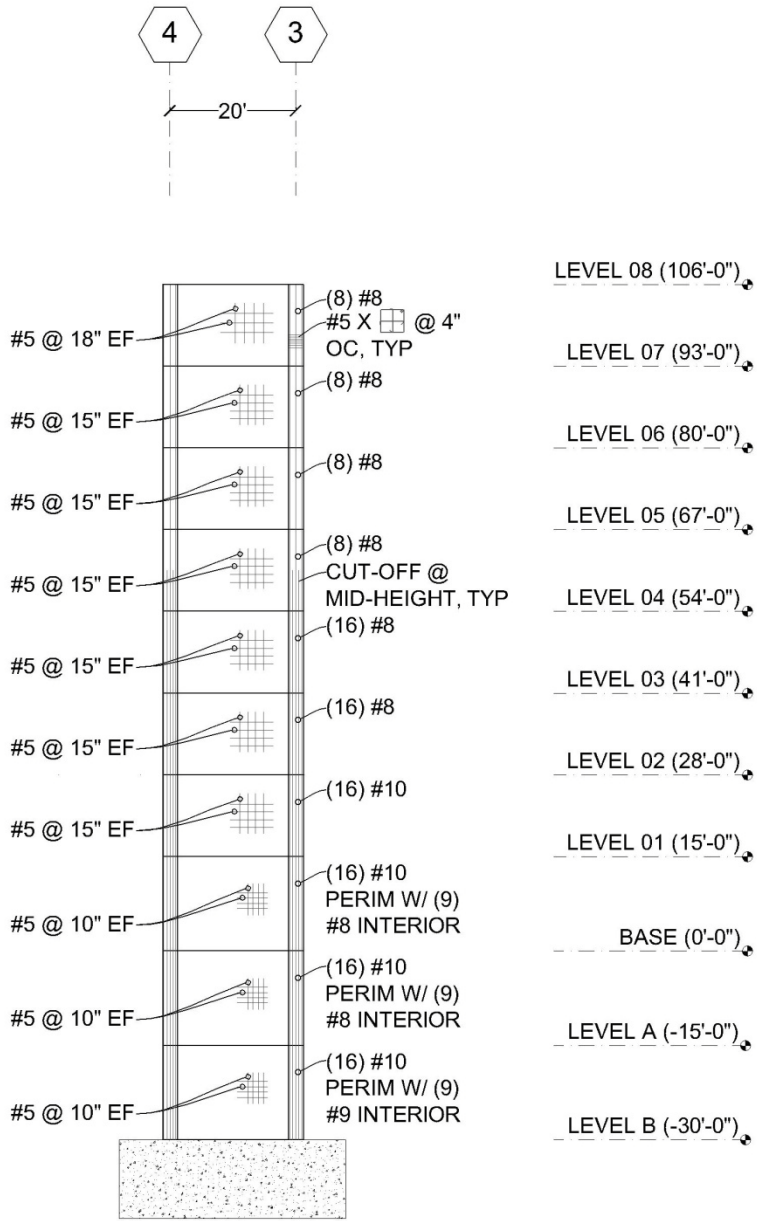
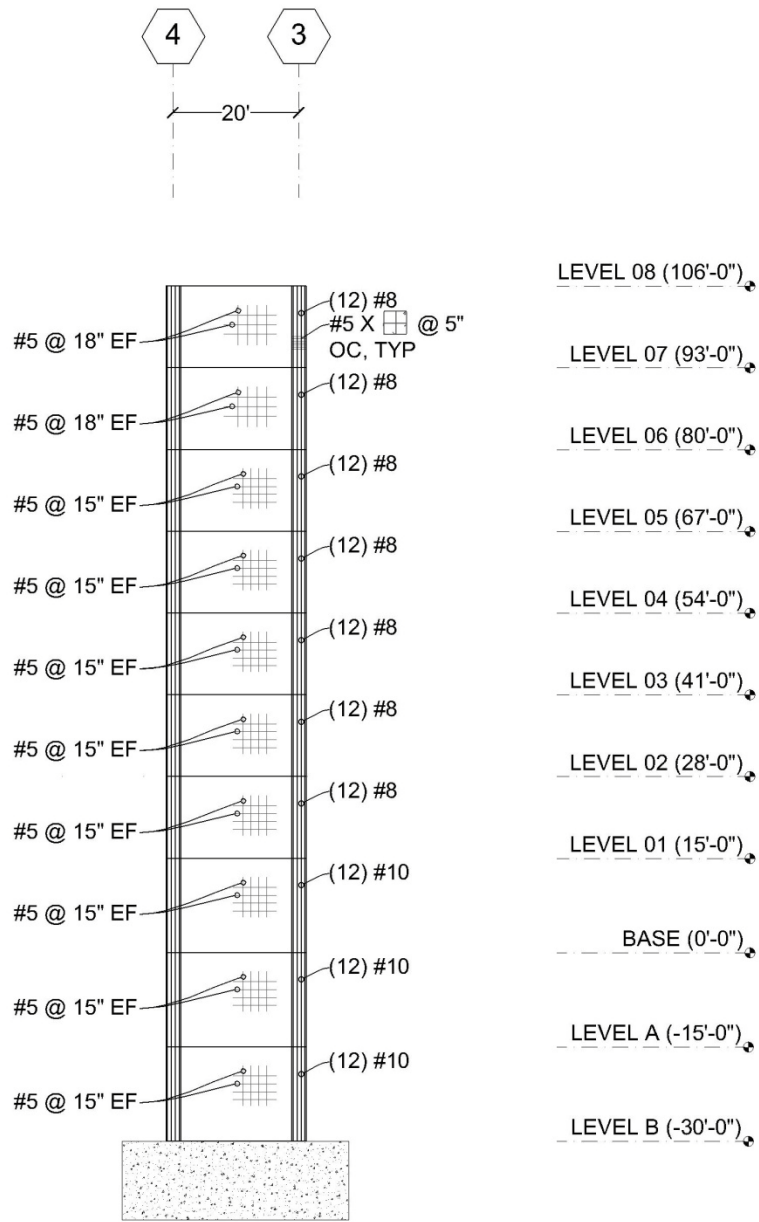


Figure 12. Exterior Shear Wall Design Elevation



**Figure 13.** Interior Shear Wall Design Elevation

	Grid 4	Grid 5		Grid 4	Grid 5
Level 08	$(M_u/\Phi M_n)_{PMAX} = 0.106$ $(M_u/\Phi M_n)_{PMIN} = 0.112$ $V_u/\Phi V_n = 0.178$ $P_{max}/A_g f'_c = 0.008$		Level 08	$(M_u/\Phi M_n)_{PMAX} = 0.076$ $(M_u/\Phi M_n)_{PMIN} = 0.081$ $V_u/\Phi V_n = 0.176$ $P_{max}/A_g f'_c = 0.012$	
Level 07	$(M_u/\Phi M_n)_{PMAX} = 0.267$ $(M_u/\Phi M_n)_{PMIN} = 0.296$ $V_u/\Phi V_n = 0.319$ $P_{max}/A_g f'_c = 0.017$		Level 07	$(M_u/\Phi M_n)_{PMAX} = 0.190$ $(M_u/\Phi M_n)_{PMIN} = 0.213$ $V_u/\Phi V_n = 0.313$ $P_{max}/A_g f'_c = 0.025$	
Level 06	$(M_u/\Phi M_n)_{PMAX} = 0.454$ $(M_u/\Phi M_n)_{PMIN} = 0.525$ $V_u/\Phi V_n = 0.418$ $P_{max}/A_g f'_c = 0.026$		Level 06	$(M_u/\Phi M_n)_{PMAX} = 0.299$ $(M_u/\Phi M_n)_{PMIN} = 0.350$ $V_u/\Phi V_n = 0.366$ $P_{max}/A_g f'_c = 0.038$	
Level 05	$(M_u/\Phi M_n)_{PMAX} = 0.643$ $(M_u/\Phi M_n)_{PMIN} = 0.771$ $V_u/\Phi V_n = 0.497$ $P_{max}/A_g f'_c = 0.035$		Level 05	$(M_u/\Phi M_n)_{PMAX} = 0.414$ $(M_u/\Phi M_n)_{PMIN} = 0.503$ $V_u/\Phi V_n = 0.424$ $P_{max}/A_g f'_c = 0.051$	
Level 04	$(M_u/\Phi M_n)_{PMAX} = 0.606$ $(M_u/\Phi M_n)_{PMIN} = 0.701$ $V_u/\Phi V_n = 0.572$ $P_{max}/A_g f'_c = 0.044$		Level 04	$(M_u/\Phi M_n)_{PMAX} = 0.523$ $(M_u/\Phi M_n)_{PMIN} = 0.655$ $V_u/\Phi V_n = 0.475$ $P_{max}/A_g f'_c = 0.063$	
Level 03	$(M_u/\Phi M_n)_{PMAX} = 0.760$ $(M_u/\Phi M_n)_{PMIN} = 0.897$ $V_u/\Phi V_n = 0.657$ $P_{max}/A_g f'_c = 0.053$		Level 03	$(M_u/\Phi M_n)_{PMAX} = 0.625$ $(M_u/\Phi M_n)_{PMIN} = 0.804$ $V_u/\Phi V_n = 0.517$ $P_{max}/A_g f'_c = 0.076$	
Level 02	$(M_u/\Phi M_n)_{PMAX} = 0.779$ $(M_u/\Phi M_n)_{PMIN} = 0.907$ $V_u/\Phi V_n = 0.759$ $P_{max}/A_g f'_c = 0.062$		Level 02	$(M_u/\Phi M_n)_{PMAX} = 0.714$ $(M_u/\Phi M_n)_{PMIN} = 0.940$ $V_u/\Phi V_n = 0.524$ $P_{max}/A_g f'_c = 0.089$	
Level 01	$(M_u/\Phi M_n)_{PMAX} = 0.844$ $(M_u/\Phi M_n)_{PMIN} = 0.908$ $V_u/\Phi V_n = 0.761$ $P_{max}/A_g f'_c = 0.071$		Level 01	$(M_u/\Phi M_n)_{PMAX} = 0.716$ $(M_u/\Phi M_n)_{PMIN} = 0.938$ $V_u/\Phi V_n = 0.396$ $P_{max}/A_g f'_c = 0.103$	

(A)

(B)

**Figure 14.** Exterior (A) and Interior (B) Shear Wall Demand Capacity Ratios

### 3. ARCHETYPE STRUCTURE ASSESSMENT

Seismic assessment of existing structures is important to maintain a confident level of safety. ASCE/SEI 41-06 provides guidance on analysis methods and acceptance criteria for both assessment and rehabilitation. The archetype structure is completely known, and therefore analysis throughout this section uses a knowledge factor of 1.0, and all material testing requirements are not applicable. As this investigation is intended to be a general evaluation of the assessment of new construction methods using the existing building criteria in ASCE/SEI 41-06, all prescribed analysis methods are examined.

#### 3.1. Performance Objectives and Seismic Hazard Definitions

ASCE/SEI 41-06 is based on the principle of performance objectives, specifically for rehabilitation projects. These performance objectives include: Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). ASCE/SEI 41-06 Section C1.5.1.1 describes the IO objective as retention of lateral and gravity system stiffnesses and strengths, and relatively minor structural damage for which repairs are not required prior to re-occupancy. ASCE/SEI 41-06 Section C.1.5.1.3 describes the LS objective as severe structural damage, however partial or full collapse is not imminent. Additionally, the LS objective does include the possibility of earthquake related injuries, however these injuries are not expected to be life-threatening. Lastly, ASCE/SEI 41-06 Section C.1.5.1.5 describes the CP objective as the state at which partial or full collapse of the structure is imminent. Structural components have significant reductions in stiffness and strength, and the risk of injury due to falling components is high. The structure may also be impractical to repair.

The performance objectives in ASCE/SEI 41-06 are defined seismically by probabilistic earthquake levels. Probabilistic earthquake hazards are defined using the desired probability of exceedance and minimum expected structure life. Refer to ASCE/SEI 41-06 Section 1.6 for more information regarding probabilistic earthquake hazards. As mentioned earlier, the spectral accelerations,  $S_s$  and  $S_{1,}$  are defined at the  $D_{MAX}$  level for the archetype structure. ASCE/SEI 41-06 Section 1.6 provides guidance on calculating the various seismic hazards for corresponding performance objectives. Contrary to ASCE/SEI 7-10, ASCE/SEI 41-06 denotes the design spectral response accelerations as  $S_{XS}$  and  $S_{X1}$ . Table 5 shows each performance objective, and the corresponding probability of exceedance and seismic hazard. Support calculations can be found in Appendix D.

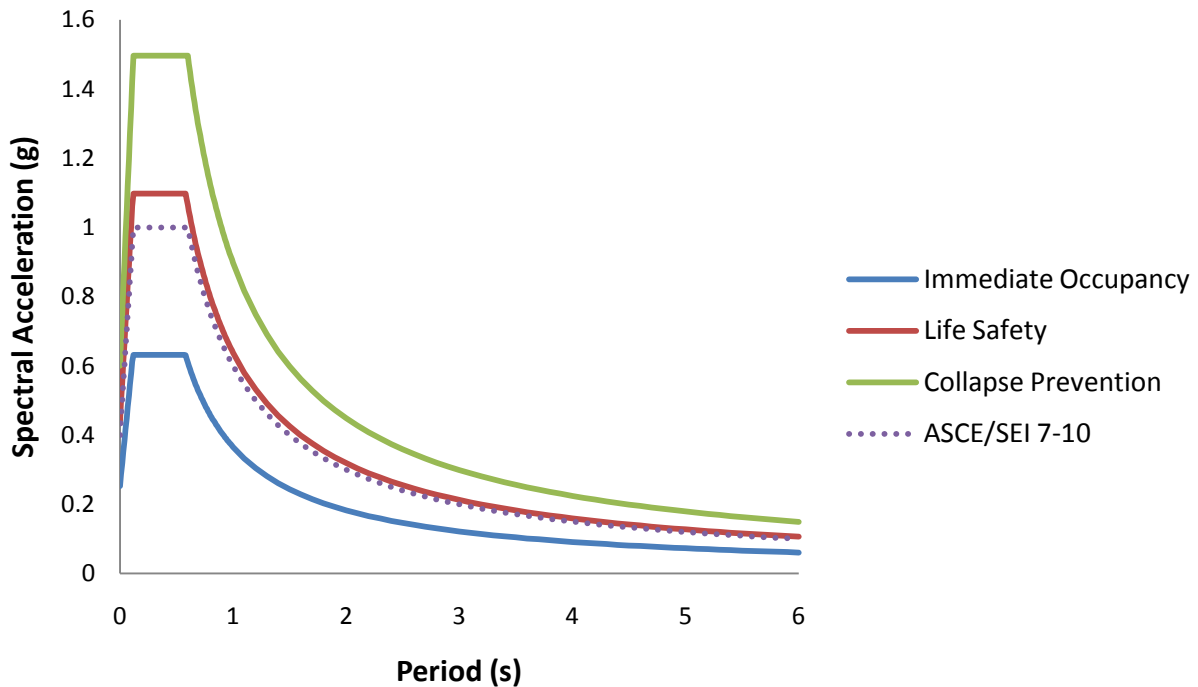
**Table 5.** Performance Objective Seismic Hazards

	Probability of Exceedance	$S_{XS}$ (g)	$S_{X1}$ (g)
<b>Immediate Occupancy</b>	50% / 50 Year	0.633	0.367
<b>Life Safety</b>	10% / 50 Year	1.100	0.640
<b>Collapse Prevention</b>	2% / 50 Year	1.500	0.900

##### 3.1.1. Response Spectrum

Development of the response spectrum is based on ASCE/SEI 41-06 Section 1.6.1.5. ASCE/SEI 41-06 uses a slightly different method for calculating a response spectrum as that found in ASCE/SEI 7-10. Figure 15

shows the calculated response spectrums for each performance objective. The response spectrum calculated using ASCE/SEI 7-10, found in Figure 1, is also plotted for comparison. Discussion of response spectrum methods and use is included later in this report.



**Figure 15.** ASCE/SEI 41-06 Performance Objective Response Spectrums

### 3.1.2. Time Histories

In addition to response spectrum analysis, complete ASCE/SEI 41-06 assessment requires the use of time history analysis. To maintain consistency, a set of 44 time history records defined in the FEMA P695 are used in analysis. These records are applied in analysis in each direction, using both maximum and minimum gravity load combinations. Each record is normalized and accompanied by a previously calculated response spectrum, with which a scale factor is determined for the specific record. Additionally, performance objective scale factors are calculated using these time history response spectrums and ASCE/SEI 41-06 calculated response spectrums shown in Figure 15. These scale factors are calculated by matching the average (Square Root of the Sum of Squares method) 1-second spectral accelerations of the time histories with the corresponding 1-second spectral acceleration of the target response spectrum. This scaled average, with the corresponding target response spectrum, is shown in Figure 16. Support calculations for time history record scaling can be found in Appendix F.

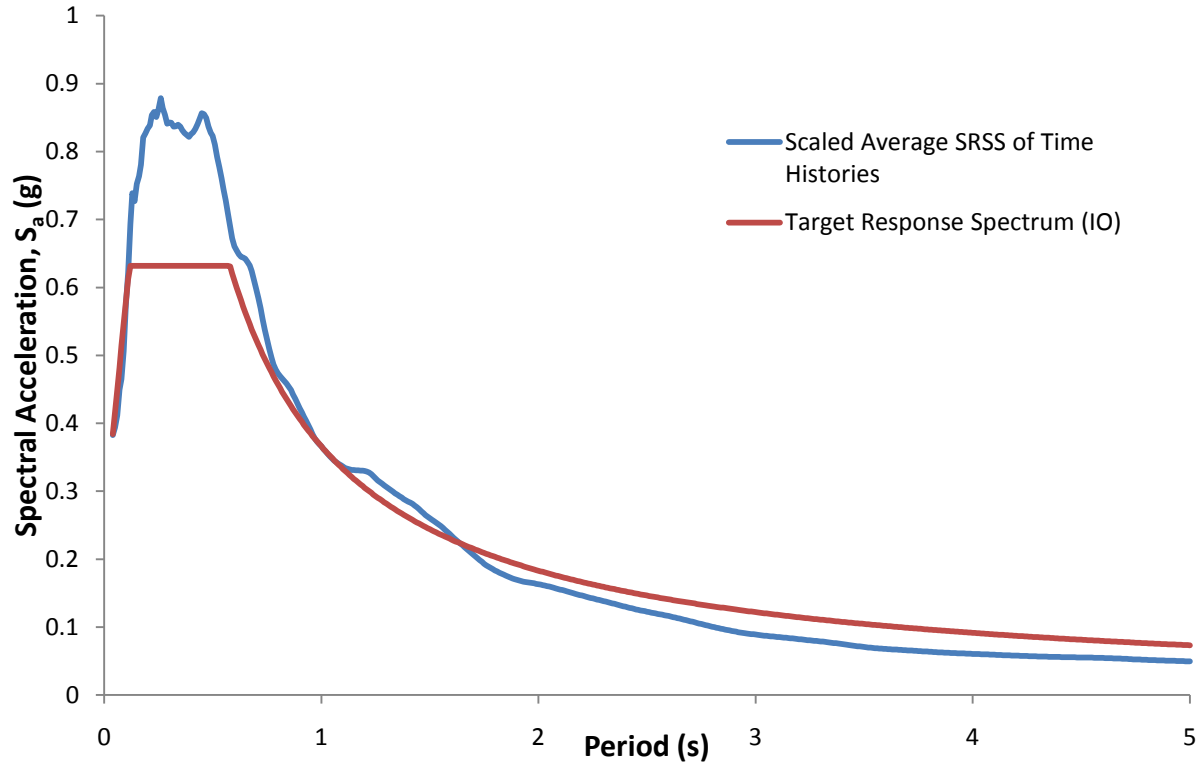


Figure 16. Time History Record Scaling

### 3.2. Linear Procedures

Linear procedures incorporate traditional linear stress-strain relationships for component strengths. For this reason, solely linear analysis can only accurately capture elastic behavior. As elastic behavior is an unrealistic design assumption for seismic design, adjustments are made to account for probable nonlinearities in the system. ASCE/SEI 41-06 linear procedures employ adjustment factors,  $m$  and  $J$ , for deformation-controlled and force-controlled actions, respectively, to account for these nonlinearities. Although this process is a simplification of complex behavior, it provides a good estimate for simple and regular structures. For this reason, ASCE/SEI 41-06 Section 2.4.1.1 provides various restrictions on use of linear procedures. As mentioned previously however, the archetype structure is assessed using all methods prescribed in ASCE/SEI 41-06.

The archetype structure uses the same model for all linear procedures. The model accounts for both flexural, shear, and axial stiffness through the use of stiffness reduction factors prescribed in ASCE/SEI 41-06 Table 6-5. Table 6 summarizes ASCE/SEI 41-06 Table 6-5 with pertinent structural components. Due to the relatively low gravity loads, all moment frame are assigned flexural stiffness reduction factors equal to, or near,  $0.3 \cdot E I_g$ . Note that flexural stiffness reduction factors prescribed in ASCE/SEI 41-06 Table 6-5 are much less than those used in design.

Table 6. ASCE/SEI 41-06 Stiffness Reduction Factors

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
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<b>Beams</b>	$0.3*EI_g$	$0.4*EA_w$	(-)
<b>Columns (<math>P_g \geq 0.5*A_g*f_c'</math>)</b>	$0.7*EI_g$	$0.4*EA_w$	$1.0*EA_g$
<b>Columns (<math>P_g \leq 0.1*A_g*f_c'</math>)</b>	$0.3*EI_g$	$0.4*EA_w$	$1.0*EA_g$
<b>Walls</b>	$0.5*EI_g$	$0.4*EA_w$	$1.0*EA_g$

ASCE/SEI 41-06 accounts for capacity design principles through the use of deformation-controlled and force-controlled actions. Typically ductile failure modes, such as flexural yielding in beams, are defined as deformation-controlled actions while brittle failure modes, such as shear in beams, are considered force-controlled. Component strengths are calculated using similar design principles as in design, however lower-bound and expected strengths are used for deformation-controlled and force-controlled actions, respectively. ASCE/SEI 41-06 Table 6-4 provides factors for use in calculating expected strengths. Component strengths, similar to design, are not explicitly modeled in SAP2000; demands calculated using SAP2000 are checked against calculated strengths. More information regarding deformation-controlled and force-controlled actions can be found in ACSE/SEI 41-06 Section 2.4.4.3.

ASCE/SEI 41-06 Section 3.2.5 requires that P-Delta effects be included in linear analysis. P-Delta effects are accounted for through the use of a “leaning” column and nonlinear static gravity loads in SAP2000. The “leaning” column does not contribute to the stiffness of the structure, nor does it develop any moment. Remaining gravity loads on the structure not resisted by the lateral systems are assigned to the “leaning” column.

### 3.2.1. Linear Static Procedure

#### 3.2.1.1. Analysis Methodology

The general premise behind the linear static procedure (LSP) is similar to the equivalent lateral force method defined in ASCE/SEI 7-10 (reference section 2.3.1). ASCE/SEI 41-06 Section 3.3.1.3 defines a pseudo-lateral force distributed horizontally and vertically on the structure. Such factors as structure period, mass, and design spectral acceleration affect the magnitude of this pseudo-lateral force. Calculated base shears for corresponding performance objectives, compared with the design base shear (from MRSA), are shown in Table 7. The sizeable differences in base shear arise from the method with which the seismic coefficients are calculated. ASCE/SEI 41-06 does not account for lateral system ductility in calculations of the pseudo-lateral force while ASCE/SEI 7-10 does, through the use of R factors. Support calculations for Table 7 can be found in Appendix D.

**Table 7.** Linear Static Procedure and Design Base Shears

<b>Direction</b>	<b>IO (kips)</b>	<b>LS (kips)</b>	<b>CP (kips)</b>	<b>Design (kips)</b>
<b>Moment Frame</b>	1,520	2,652	3,729	412
<b>External Shear Wall</b>	1,755	3,063	4,307	571
<b>Internal Shear Wall</b>	1,622	2,831	3,980	571

The LSP also accounts for both inherent and accidental torsion, as defined in ASCE/SEI 41-06 Section 3.2.2.2. For 2-Dimensional models, torsion is accounted for by increasing the lateral forces by a  $\eta$  factor.

This  $\eta$  factor requires 3-Dimensional analysis that accounts for torsional effects, and represents the ratio of maximum displacement to average displacement of a floor. ASCE/SEI 41-06 is however not clear on torsion requirements when a low  $\eta$  factor is calculated. ASCE/SEI 41-06 Section 3.2.2.2.3 indicates that increased forces and displacements due to torsion are not required when the  $\eta$  factor at each level is less than 1.1, however ASCE/SEI 41-06 Section 3.2.2.2.6 states that torsional effects for 2-Dimensional models should be calculated and incorporated. As a conservative approach, this investigation accounts for torsional effects in the 2-Dimensional models, though the  $\eta$  factor for the moment frames and interior shear walls is less than 1.1 at each level. Support calculations for horizontal torsion effects can be found in Appendix D.

Acceptance criteria for the LSP are based on demand and capacity principles. ASCE/SEI 41-06 Section 3.4.2.2 provides demand-capacity requirements for both deformation-controlled and force-controlled actions. In general, capacities of deformation-controlled components are increased by the  $m$  factor, while seismic demands of force-controlled components are decreased by the  $J$  factor.

#### 3.2.1.2. Results

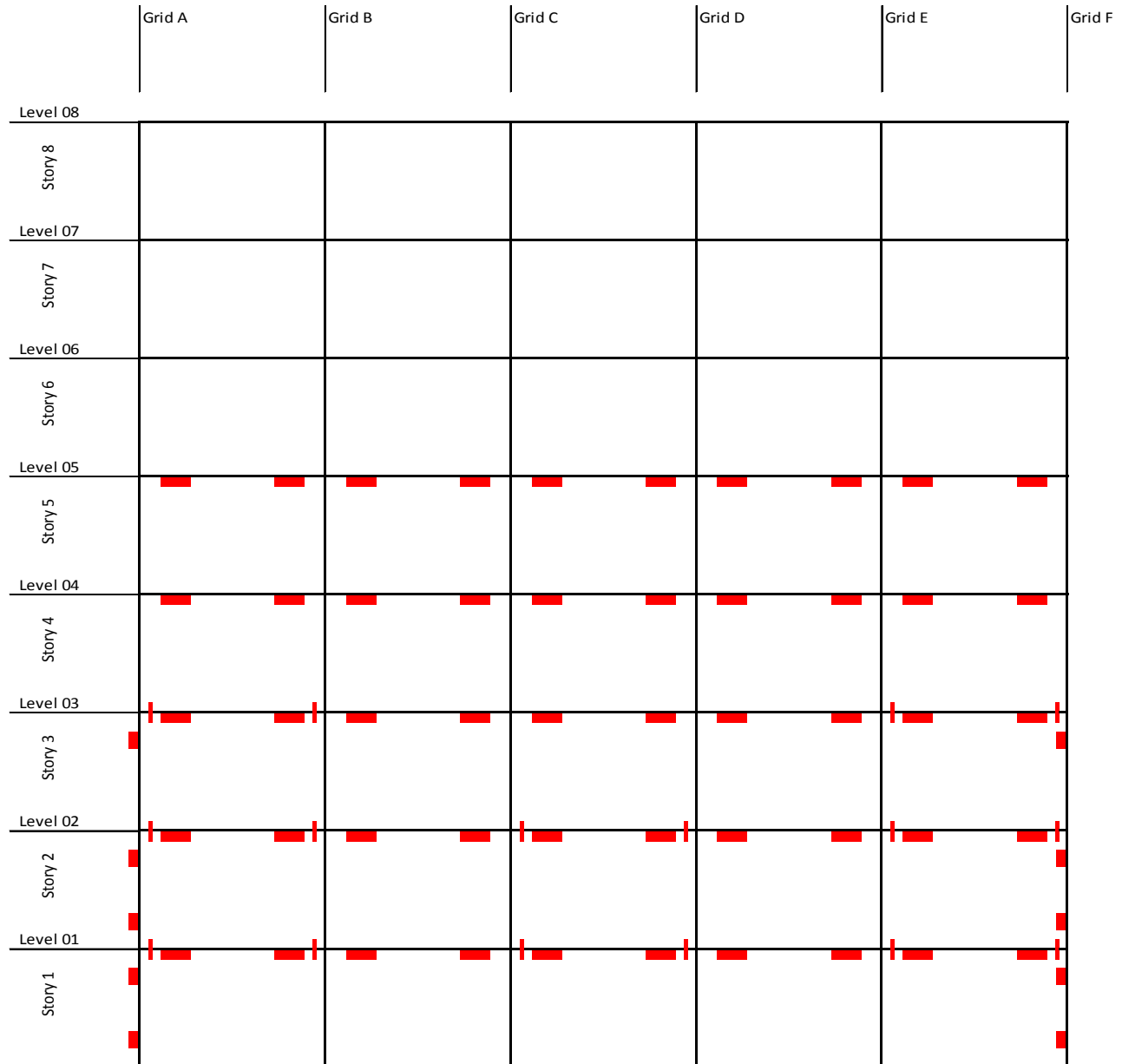
Results from the LSP clearly indicate inconsistencies between design and assessment. In general, component failures occur in the lower levels of the structure for all performance objectives. Typically, the magnitude and rate of occurrence of failure increases with more stringent performance objectives.

In moment frame beams, positive moment capacity is the most common failure. In general, these failures occur with demand to capacity ratios (DCR) ranging from around 1.25 at the IO performance objective, to 2.0 at the CP performance objective. This could be due to the significantly higher lateral loads calculated using ASCE/SEI 41-06. In design, positive moment demand at beam ends is typically lower than negative moment demand because the gravity loads produce relatively significant increases in negative moment demand. However, when lateral loads represent such a large percentage of the total demands, the positive and negative moment demands have similar magnitudes. This results in failure in positive flexure, as the positive moment capacity of the beams is lower than that of the negative moment capacity. Additionally, due to the large increase in lateral load, shear failures occur for all performance objectives, typically in end bays on lower levels. Typically, DCR's for shear failure ranges from approximately 1.0 at the IO performance objective to 2.0 at the CP performance objective.

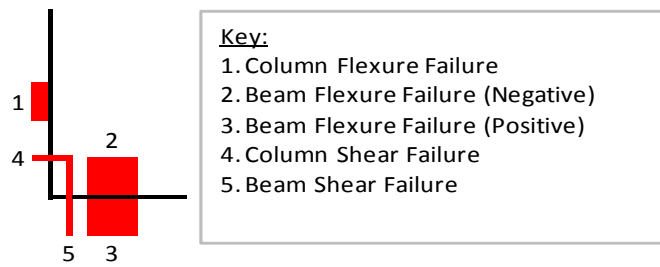
In moment frame columns, the occurrence of failure in flexure is low at all performance objectives. However, it is important to note that edge columns experience net tension under the comparatively large lateral loads, and therefore interaction failure in these columns is common. Column shear and joint shear demands are adequately resisted throughout all performance objectives.

Under ASCE/SEI 41-06 loading, both interior and exterior shear walls fail in flexural-axial interaction at the lower levels for all performance objectives. The cause of these failures is likely the reduced gravity load combinations prescribed by ASCE/SEI 41-06 in combination with much higher lateral demands. Typically, shear walls that exhibit flexural-axial interaction failures have DCR values ranging between 1.0 and 1.5 for all performance objectives. Shear walls are adequate in shear capacity for all performance objectives.

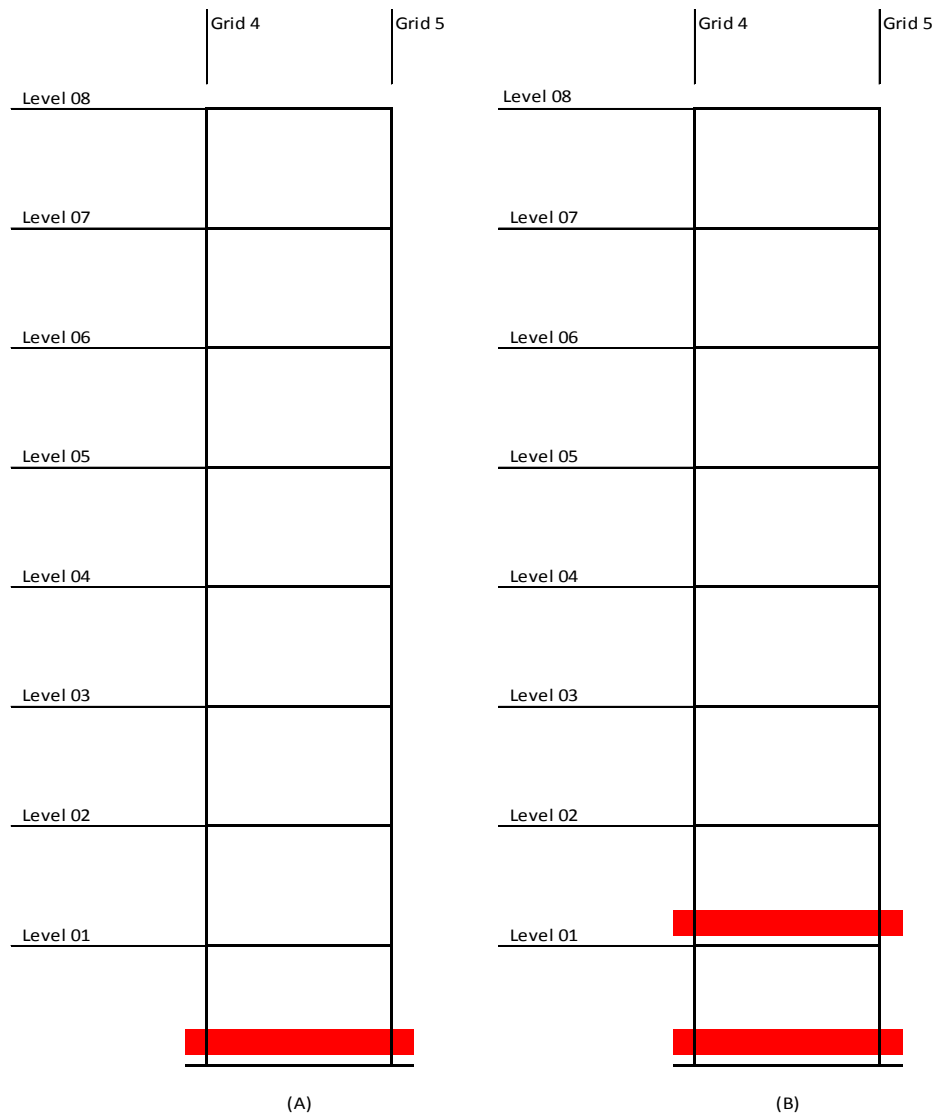
A representative moment frame elevation is shown in Figure 17, indicating IO performance objective failures. Similarly, Figure 19 represents failures in interior and exterior shear walls under the IO performance objective. A key for moment frame elevations, and similar subsequent elevations, is shown in Figure 18. A key for shear wall elevations, and similar subsequent elevations, is shown in Figure 20. Similar elevations for LS and CP objectives can be found in Appendix E. Support calculations for all lateral systems under LS and CP performance objectives can be found in Appendix D.



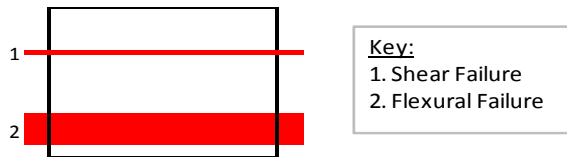
**Figure 17.** LSP Failure Elevation for Moment Frames at the IO Performance Objective



**Figure 18.** Moment Frame Elevation Key for LSP and LDP Failures



**Figure 19.** LSP Failure Elevation for Exterior (A) and Interior (B) Shear Walls at the IO Performance Objective



**Figure 20.** Shear Wall Elevation Key for LSP and LDP Failures

### 3.2.2. Linear Dynamic Procedure

#### 3.2.2.1. Analysis Methodology

The linear dynamic procedure (LDP) incorporates the use of both modal response spectrum analysis and time history analysis. Similar to the LSP, the LDP imposes linear elastic demands on the structure, and accounts for inelasticity through adjustment factors. This method provides a more accurate model of the motions that are experienced during a seismic event, however maintains simplifications to account for nonlinear response behavior.

Use of modal response spectrum analysis requires models that can sufficiently capture at least 90% of the participating mass of the structure. Maximum forces and displacements are combined using the complete quadratic combination, as permitted per ASCE/SEI 41-06 Section 3.3.2.2.3. The archetype structure is assigned a typical structure damping ratio of 5%, per ASCE/SEI 41-06 Section 1.6.1.5.3. Time history analysis employs the described set of time history records in Section 3.1.2. As permitted by ASCE/SEI 41-06 Section 3.3.2.2.4, the maximum response parameters, including forces and displacements, are the average of those produced by each record. Records are implemented in SAP2000 using modal analysis of linear, transient time histories. For linearly defined loading, the total response of a member or structure to multiple load types (i.e. gravity and seismic) is the sum of the responses of each load type. For this reason, order of action does not affect the total response of the structure, and therefore is not used in the LDP.

Table 8 and Table 9 show the base shears for the various performance objectives using modal response spectrum analysis and time history analysis, respectively. As shown, the base shear values used for various performance objectives are much higher than those used in design. This is a result of the differing methods used to define seismic hazard and response in ASCE/SEI 7-10 and ASCE/SEI 41-06. ASCE/SEI 7-10 reduces the base shear based on the type of lateral resisting system used; this reduction is a function of the ductility of the lateral system. On the contrary, ASCE/SEI 41-06 uses system ductility to increase the capacity of various components. This discrepancy presents a difficult comparison for base shear between the two methods, although also provides a good explanation for the varying results obtained using ASCE/SEI 7-10 design and ASCE/SEI 41-06 assessment.

**Table 8.** Linear Dynamic Procedure (Response Spectrum Analysis) and Design Base Shears

Direction	IO (kips)	LS (kips)	CP (kips)	Design (kips)
Moment Frame	1,748	3,049	4,284	412
External Shear Wall	1,679	2,927	4,087	571
Internal Shear Wall	1,562	2,722	3,799	571

**Table 9.** Linear Dynamic Procedure (Time History Analysis) and Design Base Shears

<b>Direction</b>	<b>IO (kips)</b>	<b>LS (kips)</b>	<b>CP (kips)</b>	<b>Design (kips)</b>
<b>Moment Frame</b>	1,499	2,617	3,680	412
<b>External Shear Wall</b>	1,288	2,249	3,158	571
<b>Internal Shear Wall</b>	1,188	2,074	2,913	571

Additionally, as required by ASCE/SEI 41-06, torsional effects are included in analysis by amplifying maximum forces and displacements by the maximum  $\eta$  factor, similar to the LSP. Acceptance criteria for the LDP are similar to those of the LSP.

### 3.2.2.2. Results

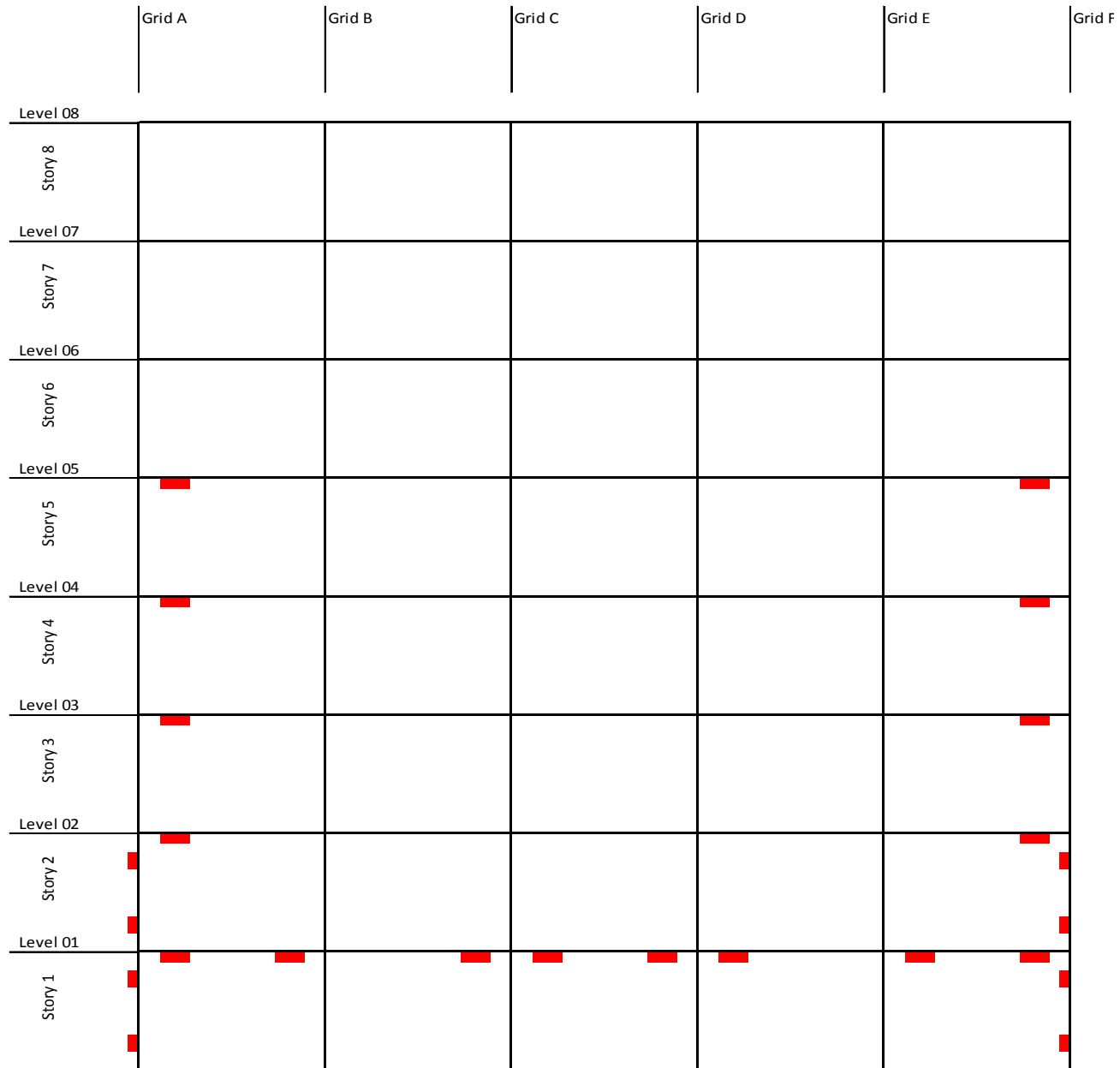
Moment frame assessment is considerably better using the modal response spectrum analysis method when compared with the LSP. At the IO performance objective, beams, columns, and joints are adequate in resisting demands on the structure, with the exception of first level edge columns. These specific columns are inadequate in flexural-axial interaction, likely due to the large overturning forces that result in net tension in the columns. Similar to the LSP, beams fail in flexure at the LS and CP performance objectives, however at a considerably lower rate. Columns also experience flexural-axial interaction failures at the LS and CP performance objectives, a likely result of lower axial forces caused by high lateral demands. Again, joint shear is adequately resisting in the moment frames for all performance objectives. Comparatively, time history analysis results in an increase in failure occurrence in beams and columns. The failures themselves, however, are similar to those found in modal response spectrum analysis.

Interior shear walls exhibit flexural-axial interaction failures within the first three floors of the archetype structure under response spectrum analysis demands for various performance objectives. However, such failures do not occur in exterior shear walls, or when analyzed using time history analysis demands. Shear failures do not occur in any shear walls under either analysis. It is important to note the correlation between performance of shear walls and moment frames under response spectrum analysis and time history analysis. Both lateral systems met a higher percentage of ASCE/SEI 41-06 acceptance criteria using time history analysis as compared with response spectrum analysis. This can also be seen through base shear comparisons, found in Table 8 and Table 9.

Representative moment frame elevations indicating IO performance objective failures for response spectrum analysis and time history analysis are shown in Figure 21 and Figure 22, respectively. Figure 23 is a representative shear wall elevation at the IO performance objective using response spectrum analysis. As no failures, at the IO performance objective level, occur during time history analysis, an elevation is unnecessary. Similar elevations for LS and CP objectives can be found in Appendix E. Support calculations for all lateral systems under LS and CP performance objectives can be found in Appendix D.

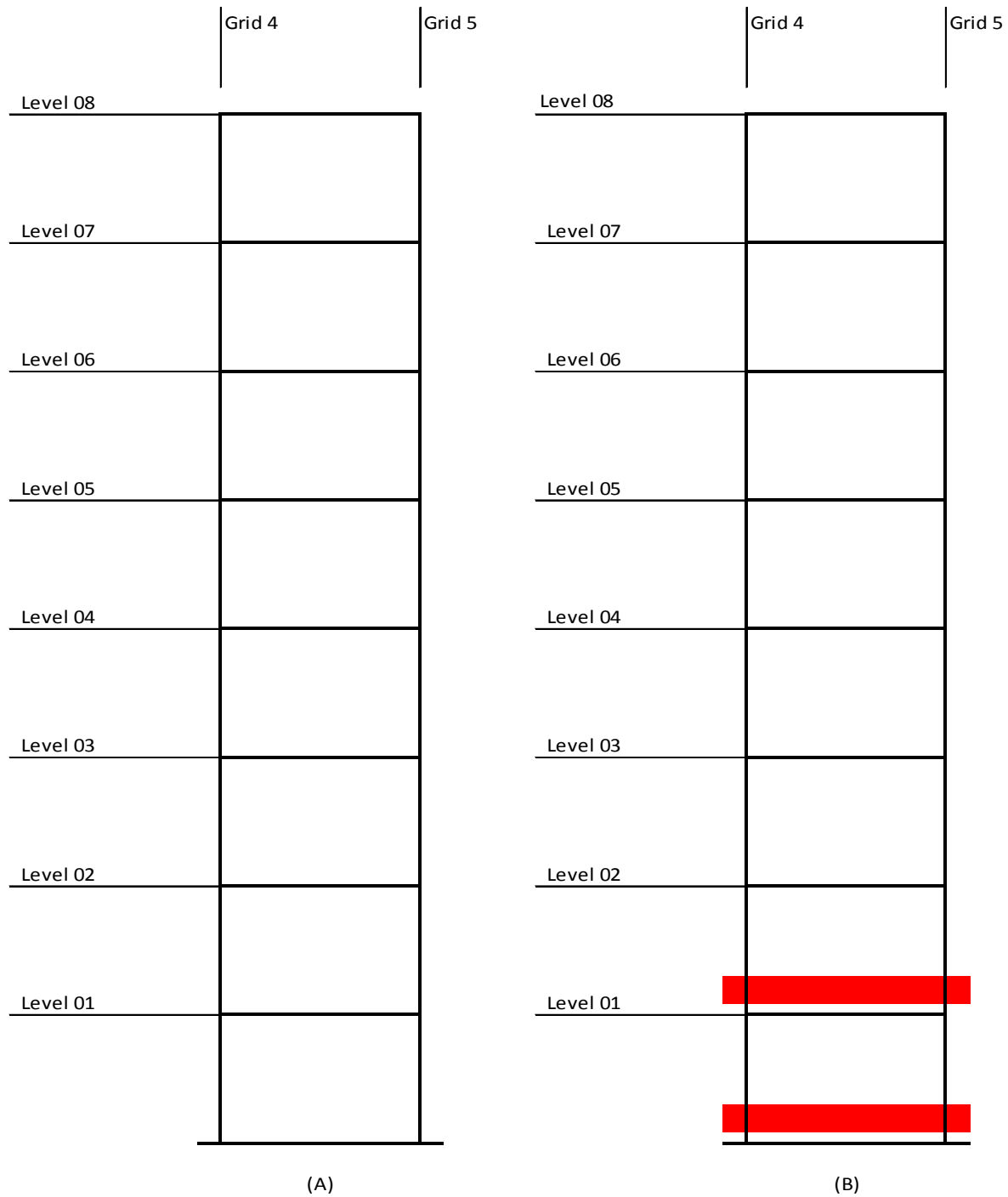
	Grid A	Grid B	Grid C	Grid D	Grid E	Grid F
Level 08						
Story 8						
Level 07						
Story 7						
Level 06						
Story 6						
Level 05						
Story 5						
Level 04						
Story 4						
Level 03						
Story 3						
Level 02						
Story 2						
Level 01						
Story 1						

**Figure 21.** LDP (Response Spectrum) Failure Elevation for Moment Frames at the IO Performance Objective



**Figure 22.** LDP (Time History Analysis) Failure Elevation for Moment Frames at the IO Performance Objective



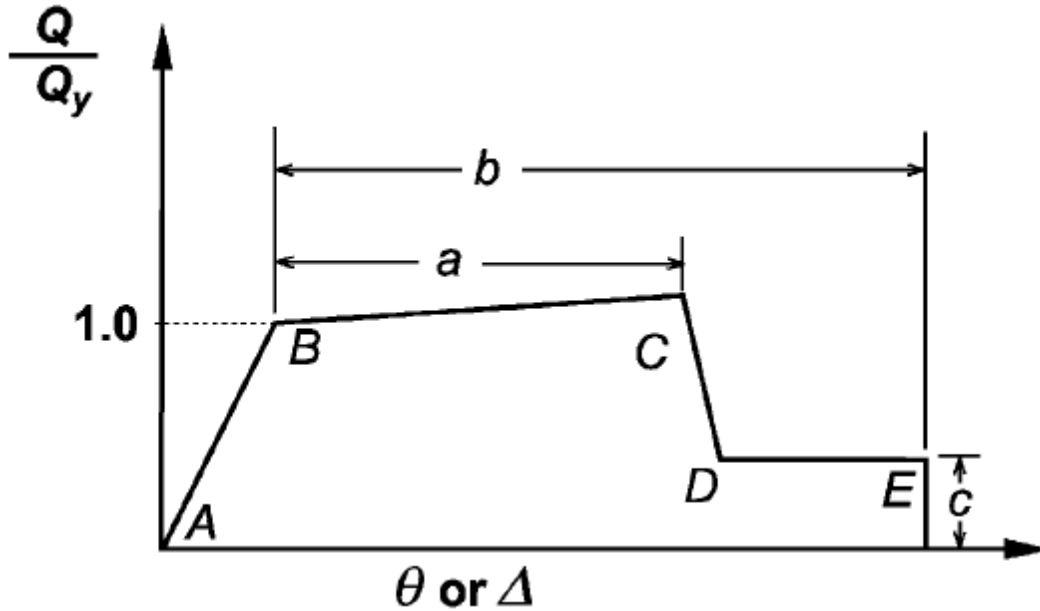


**Figure 23.** LDP (Response Spectrum) Failure Elevation for Exterior (A) and Interior (B) Shear Walls at the IO Performance Objective

### 3.3. Nonlinear Procedures

Contrary to linear procedures, nonlinear procedures provide a much more accurate depiction of structure behavior for inelastic response. As opposed to approximating nonlinearity through the use of stiffness modifiers and capacity adjustment factors, nonlinear procedures model the inelastic stress-

resultant (i.e. load-displacement or moment-curvature) relationships for each component anticipated to experience yielding. Such relationships include strain-hardening and strain-softening properties for the specific material. Although materials such as concrete and steel typically exhibit nonlinear strain-hardening and strain-softening properties, linear approximations of these regions can be made to significantly reduce calculation intensity while maintaining accuracy. As noted in ASCE/SEI 41-06 Section 6.3.1.2.2, reinforced concrete behavior can be modeled using a force-deformation curve defined in ASCE/SEI 41-06 Figure 6-1, as shown in Figure 24. This curve is normalized by the yield force, although this is not necessary. Additionally, the parameters  $a$ ,  $b$ , and  $c$ , are limits defined in ASCE/SEI 41-06.



**Figure 24.** Typical Force-Displacement Curve for Reinforced Concrete

For both moment frames and shear walls, major axis flexural hinges are used to model the nonlinear behavior of the archetype structure. To provide a more accurate model of nonlinear behavior, columns and shear walls should employ interaction hinges, which are capable of adjusting inelastic stress-strain relationships based on levels of axial load. ASCE/SEI 41-06 does indicate that flexural-axial interaction should be accounted for, including varying axial demand, although no clear guidance is provided. For this reason, average gravity loads are used to define inelastic stress-strain relationships for a constant axial load, which is then used to define the corresponding flexural hinges. This assumption is based on methods used in FEMA P695. Although this does not account for the axial load produced by lateral loads, it provides, for the most part, a conservative approximation of the flexural behavior.

Nominal shear wall and column interaction diagrams are calculated using PCA Column, although the traditional moment-curvature method is used in calculating yielding interaction diagrams. Concrete strength used in these calculations accounts for confined and unconfined strengths through the use of the Hognestad parabola. An isotropic hardening hysteresis model is used for simplicity, although a more detailed model accounting for stiffness degradation, such as the Takeda model, could be used to gain more behavioral accuracy. Additionally, shear wall models for nonlinear procedures are simplified as

multi-degree of freedom stick models. Each stick element represents a wall segment for a given level, and incorporates stiffness and mass properties that resemble the actual shear wall. In order to develop a more accurate model of the shear walls, a fiber model is required, which is out of the scope of this investigation.

Although the flexural hinges replace the use of approximate stiffness reduction factors, reductions in stiffness are required where hinges are not anticipated to occur as these areas still experience cracking. Guidance for defining this stiffness reduction is not clearly prescribed in ASCE/SEI 41-06, although practical estimates are permitted. For modeling purposes, similar stiffness reductions as those used in design are assigned to beams, columns, and shear walls in the archetype structure in order to account for component cracking.

For nonlinear procedures, connection behavior is modeled implicitly using provisions in ASCE/SEI 41-06 Section C6.4.2.2.1. Inherently, actual rotations cannot be directly calculated using these implicit models, as they use either 100%, 50%, or 0% rigidity. In order to capture joint rotations directly, explicit models incorporating rotational springs, or a similar method, are required. Results from linear procedures indicate joints are well within the prescribed limits and therefore this investigation does not examine the adequacy of joints using nonlinear methods.

Similar to linear procedures, P-Delta effects are required to be included in nonlinear procedures. Again, “leaning” columns and nonlinear gravity loads are used to account for P-Delta effects in the archetype structure. However, during analysis of the archetype structure under the nonlinear static and nonlinear dynamic procedures, convergence problems were encountered in the model that could only be immediately resolved by neglecting P-Delta effects. It is important to note, however, that P-Delta effects increase the stresses on a system, and therefore would produce larger deformations. As this investigation aims to review the assessment methods in ASCE/SEI 41-06, failures that neglect P-Delta effects represent a minimum collection of failures.

### **3.3.1. Nonlinear Static Procedure**

#### **3.3.1.1. Analysis Methodology**

The nonlinear static procedure (NSP), also known as “pushover analysis”, combines simplistic loading mechanisms with complex modeling parameters to approximate the response of a structure. The premise behind the NSP uses incremental displacement controlled loading up to a specific target displacement. Preliminary structure pushover analysis is required to calculate these specific target displacements. From this analysis, an idealized force-displacement curve is derived based on provisions in ASCE/SEI 41-06 Section 3.3.3.2.5. This idealized force-displacement curve is used in determining the effective fundamental period of the structure in each direction. As the response of the structure throughout the elastic range is linear, the effective fundamental period may be taken as equal to the elastic period from modal analysis. Figure 25 and Figure 26 show pushover analyses for the moment frame and shear walls, respectively, which clearly indicate the linear elastic range of the lateral systems.

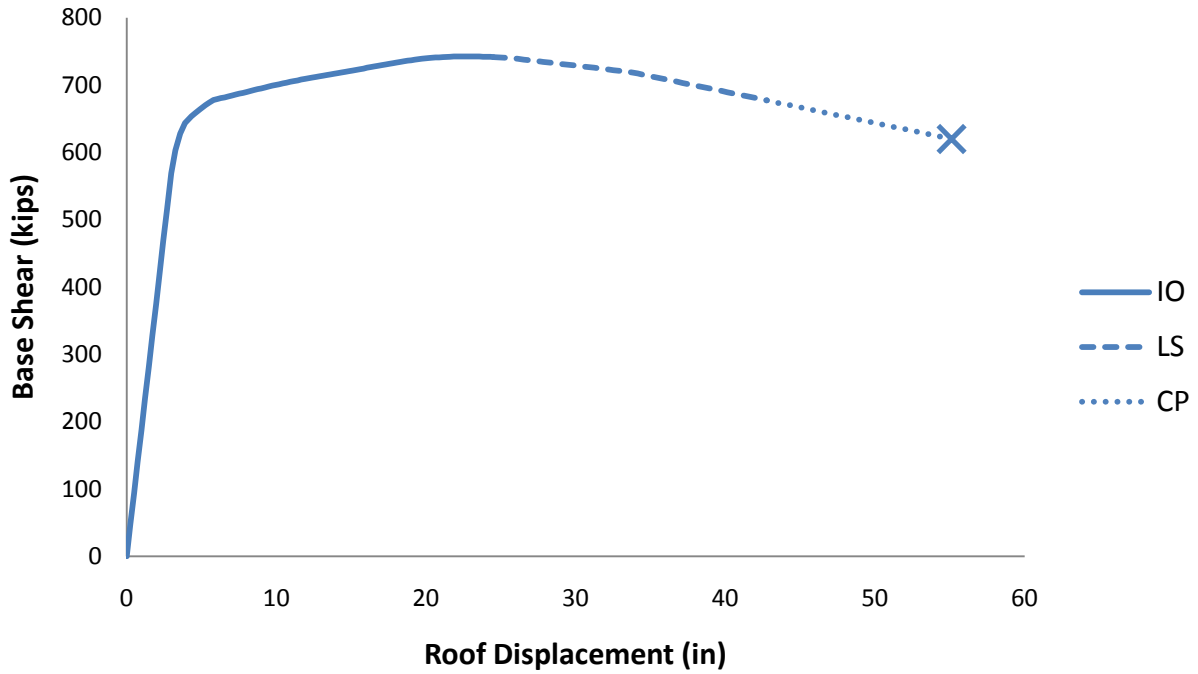


Figure 25. Pushover Curve for Moment Frames

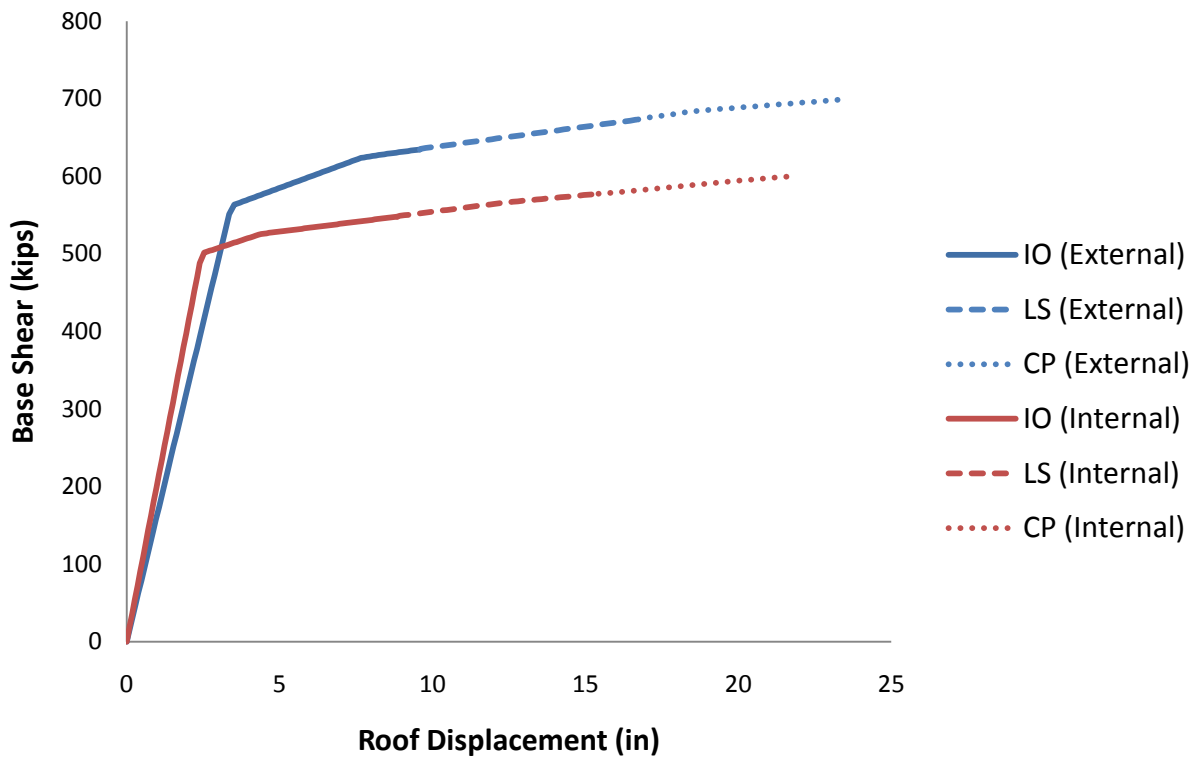


Figure 26. Pushover Curve for Exterior and Interior Shear Walls

Target displacements for various performance objectives are based on spectral accelerations and effective fundamental period. Additionally, a modification factor,  $C_o$ , is used to relate the spectral displacement of an equivalent single-degree of freedom system to roof displacement of the structure multi-degree of freedom system. Target displacements are applied to a control node, which is located at the center of mass of the roof of the structure. Additionally, ASCE/SEI 41-06 Section 3.3.3.2 requires the structure to be loaded to 150% of the target displacement to investigate probable behavior under extreme seismic conditions. This is generally applied to rehabilitation design to verify that a structure will not collapse under extreme loading, and therefore is not included in the archetype structure assessment. It is important to note, however, that the moment frame exhibits a collapse mechanism at approximately 55 inches, which is less than the 150% target displacement of 60 inches. This is an example of why structures are required to be analyzed beyond the calculated target displacements. Table 10 shows the calculated target displacements for each lateral resisting system at various performance objectives. During analysis, these target displacements are also increased by the maximum  $\eta$  factor in order to account for torsional effects. Support calculations for target displacements can be found in Appendix D.

**Table 10.** Target Displacements for Lateral Systems

<b>System</b>	<b>IO (in)</b>	<b>LS (in)</b>	<b>CP (in)</b>
<b>Moment Frame</b>	16.3	28.5	40.1
<b>External Shear Wall</b>	6.4	11.1	15.7
<b>Internal Shear Wall</b>	5.9	10.3	14.5

Under the target displacements shown above, Table 11 shows the calculated base shears for the various performance objectives, and those calculated for design. Note that the performance objective base shears are similar because they correspond to the calculated target displacement for each objective.

**Table 11.** Nonlinear Static Procedure and Design Base Shears

<b>Direction</b>	<b>IO (kips)</b>	<b>LS (kips)</b>	<b>CP (kips)</b>	<b>Design (kips)</b>
<b>Moment Frame</b>	726	742	742	412
<b>External Shear Wall</b>	605	645	670	571
<b>Internal Shear Wall</b>	533	555	574	571

Acceptance criteria for the NSP are based on hinge rotation limits. ASCE/SEI 41-06 Chapter 6 includes various tables which provide guidance for calculating maximum rotations for various components, based on performance objective. Maximum allowable hinge rotations prescribed by ASCE/SEI 41-06, unless stated otherwise, refer to plastic hinge rotations, which are defined as any hinge rotation occurring after yielding.

### 3.3.1.2. Results

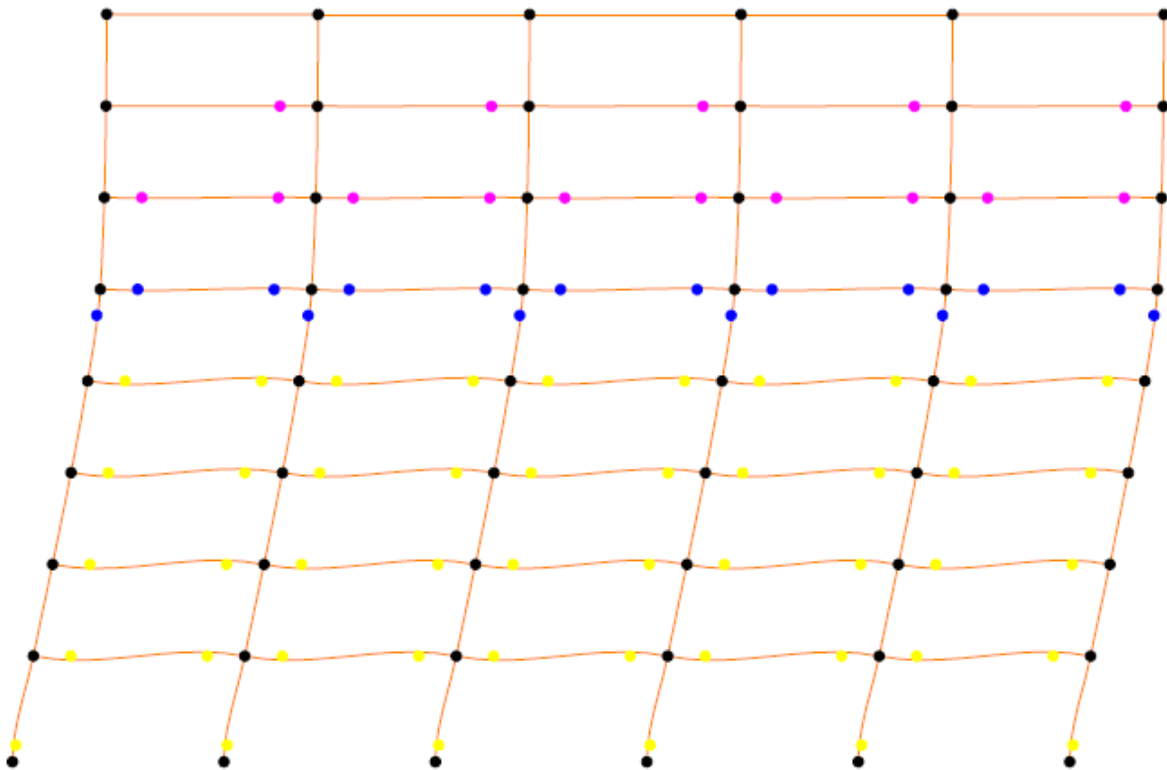
At target displacements, moment frame beams on the first and second levels of the archetype structure exceed the maximum allowable ASCE/SEI 41-06 hinge rotations for all performance objectives.

Additionally, beam hinge rotations on the third level exceed the maximum allowable rotations for the IO performance objective. These failures are likely caused by the relatively high variance in lateral loading used in ASCE/SEI 7-10 design, and ASCE/SEI 41-06 assessment. By analysis, beams at levels 6 and above remained elastic.

Plastic rotation in columns exceeds the maximum ASCE/SEI 41-06 hinge rotations in the moment frame columns located in the lower levels of the structure. Column deformation failures are limited to the top of level 3 columns and the bottom of level 1 columns. However, nearly all column hinges that did not exceed allowable rotation limits remained elastic for all performance objectives.

Analysis of both interior and exterior shear walls using the NSP indicate that all shear walls remain within the prescribed hinge rotation limits prescribed in ASCE/SEI 41-06 at all performance levels.

Figure 27 shows a representative moment frame elevation for performance level CP. The elevation indicates the various rotation range that each hinge reaches under the given target displacement. Additionally, Figure 28 and Figure 29 show the exterior and interior shear wall elevation, respectively, representing the CP performance objective. Specifically, yellow indicates that the hinge has rotated beyond the allowable ASCE/SEI 41-06 prescribed limit for the CP performance objective. Support calculations for the NSP can be found in Appendix D.



**Figure 27.** NSP Moment Frame Hinge Behavior (CP Objective)

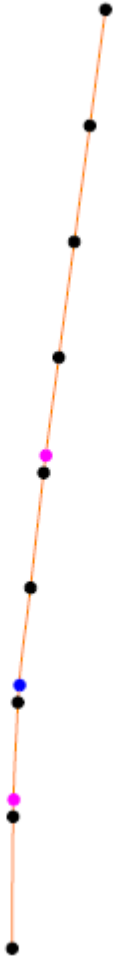


Figure 28. NSP Exterior Shear Wall Hinge Behavior (CP Objective)



**Figure 29.** NSP Interior Shear Wall Hinge Behavior (CP Objective)

### 3.3.2. Nonlinear Dynamic Procedure

#### 3.3.2.1. Analysis Procedure

The analysis procedure for the NDP is somewhat similar to the LDP, although only time history analysis is used. Response spectrum analysis uses modal coordinates to determine dynamic response in a system. However, in nonlinear behavior, the stiffness of a component varies with time and load intensity, often never regaining original elastic stiffness. Therefore by nature, the response spectrum method cannot be used to capture any nonlinear effects, which is why it is not used in the NDP. Time history analysis, however, imposes varying accelerations into the system over time, which allows nonlinear behavior in system components to propagate. Also, the method for implementing the time history records is different than that of the LDP. As nonlinear behavior is dependent on time and load intensity, it is sensitive to the order of loading. For this reason, gravity loads are applied to the model prior to running the time history records. As mentioned above, P-Delta effects are not included in the NDP due to convergence issues (reference section 3.3 for more information). Table 12 indicates the average base shears from time history analysis for various performance objectives, as well as those used in design.



**Table 12.** Nonlinear Dynamic Procedure and Design Base Shears

Direction	IO (kips)	LS (kips)	CP (kips)	Design (kips)
<b>Moment Frame</b>	802	943	*	412
<b>External Shear Wall</b>	847	1,122	1,465	571
<b>Internal Shear Wall</b>	745	1,017	1,282	571

\*Note: Base shear results for the CP objective include collapse mechanisms, and therefore an average value is not representative of the analyzed results.

### 3.3.2.2. Results

Generally, moment frame beam and column plastic hinge rotations remain within prescribed ASCE/SEI 41-06 limits. However, certain time history records result in collapse mechanisms under the LS and CP performance objectives. Table 13 lists the various time history records which result in collapse of the structure.

**Table 13.** Time History Records causing Collapse Mechanisms

	LS Objective	CP Objective
<b>Time History Record</b>	9, 10, 17, 13, 15, 38, 41	3, 9, 10, 11, 13, 15, 17, 18, 20, 21, 24, 28, 29, 30, 31, 33, 34, 37, 38, 39, 41, 42

As shown, approximately 16% of LS and 50% of CP performance objective time history records result in collapse of the structure. When more than 7 records are used, ASCE/SEI 41-06 permits the design actions on a structure to be taken as the average of those actions caused by each time history record. By eliminating the specific time history records that result in a collapse mechanism, sufficient time history records remain that indicate the archetype structure moment frame meets the assessment criteria for the NDP. However, such records cannot be simply ignored, as they indicate the possibility of structure collapse. In an actual assessment or rehabilitation, greater consideration would be needed to ensure that the structure would not collapse under the specific suite of time histories used in analysis.

Plastic rotations occur in moment frame columns typically at the first and fourth levels. This indicates that the archetype structure deformation stems from multiple modes. As column reinforcing is typical throughout the height of the archetype structure, the column hinge locations are not a function of differing reinforcement patterns. Figure 30 and Figure 31 show bottom and top of column plastic hinge rotations at various performance levels, respectively. Column hinge numbers are ordered in ascending floor level, for grids A through F. Additionally, these figures indicate the maximum and minimum hinge rotations. The first and fourth floor mechanisms can be clearly seen in Figure 30. Both figures represent time history records which do not result in collapse mechanisms. Acceptance criteria for each performance objective are shown as dashed lines.

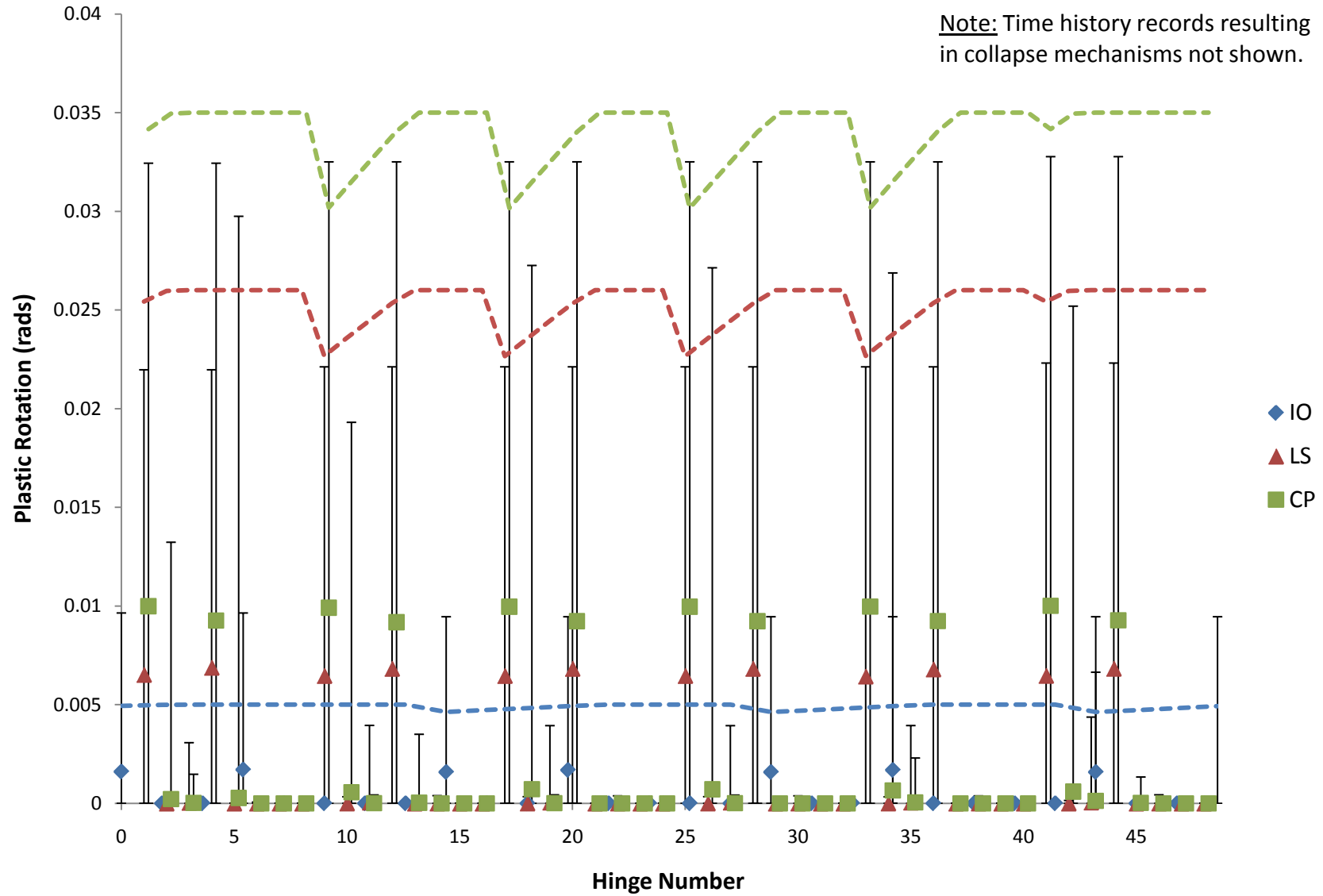


Figure 30. Bottom of Column Plastic Hinge Rotations

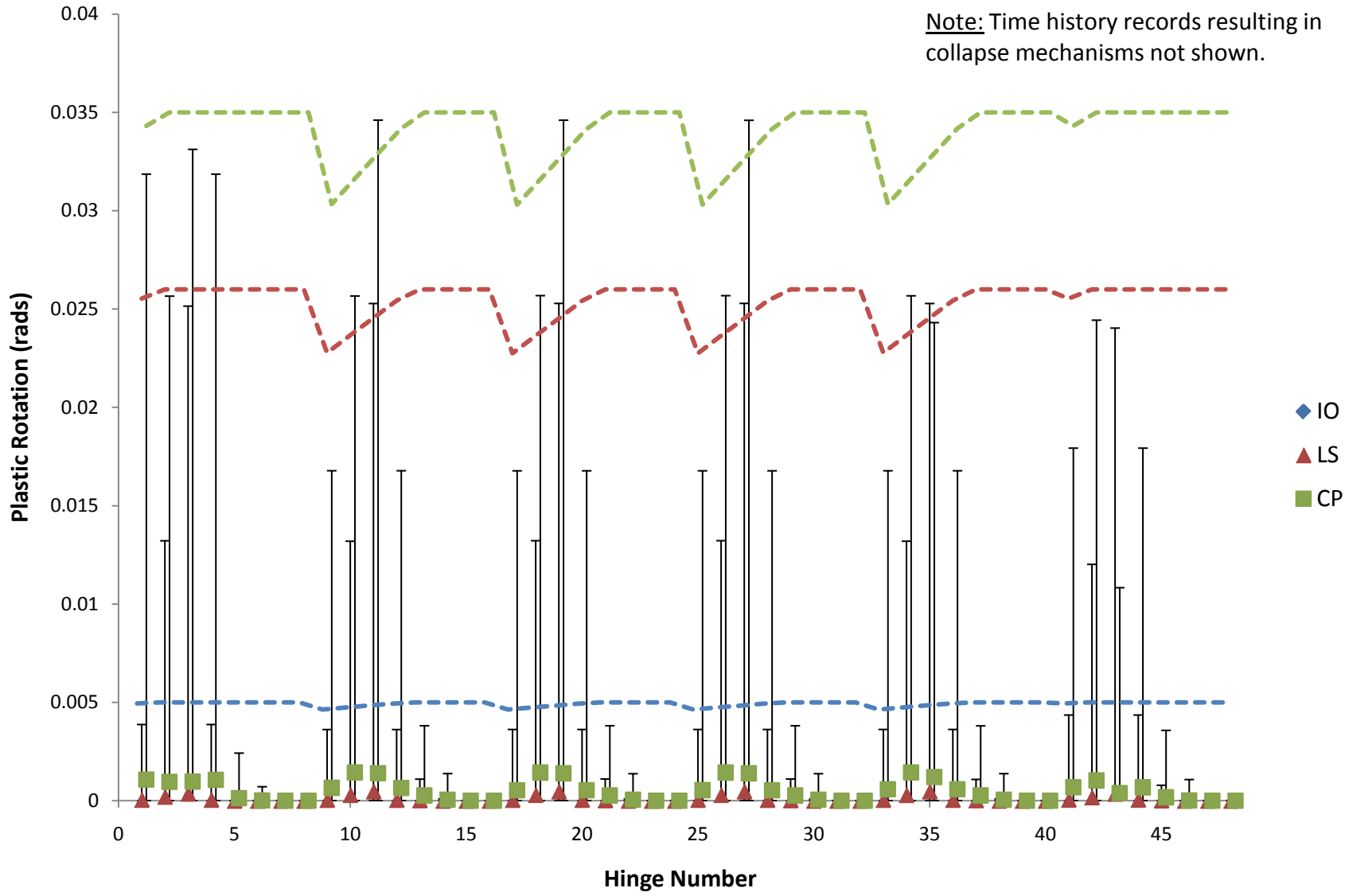


Figure 31. Top of Column Plastic Hinge Rotations

Plastic hinge rotations in the beams are generally within the prescribed ASCE/SEI 41-06 limits for all performance objectives. However, analysis indicates two IO level failures, occurring on the first and second levels. Similar to columns, however, these results are based on those time history records that do not cause a collapse mechanism. Figure 32 and Figure 33 indicate the various beam plastic hinge rotations at each performance objective for positive and negative flexure, respectively. The hinge numbers are arranged by level, all beams on one level in a row. Hinge numbers 1 through 40 represent hinges on the left side of the beam, while hinge numbers 41 through 80 represent hinges on the right side of the beam. Additionally, these figures indicate the maximum and minimum hinge rotations. As shown, the rotations at higher performance objectives, such as LS and CP, are relatively similar to those of the IO performance objective. This indicates that as the beams begin to yield, load is immediately transferred into the columns. This could be the cause of the collapse mechanisms found during analysis.

The overall behavior of the moment frame is indicative of the intended strong-column weak-beam design philosophy. As shown in the figures above, flexural rotations and yielding occur generally in the beams, and not in the columns. The locations of column yielding indicate a story shear type failure which results in collapse, and therefore these hinges are likely occurring just prior to collapse. Although this level of yielding is not desired at any performance objective, it does indicate behavior representative of design objectives. Figure 34 is a representative moment frame elevation showing the typical collapse mechanism for the CP performance objective. Again, yellow hinges indicate rotations beyond the CP objective limit. Support calculations for the moment frame using the NDP can be found in Appendix D.

Both exterior and interior shear walls experience plastic hinge rotations within prescribed ASCE/SEI 41-06 limits, at all performance objectives. Figure 35 and Figure 36 show plastic hinge rotations at each level of the interior and exterior shear walls, respectively. Hinge numbers are ordered by ascending floor level, and also indicate maximum and minimum rotation values. As shown in Figure 35, the largest plastic rotations occur on the second floor. This could be due to the relatively large change in reinforcing at this level. Conversely, as shown in Figure 36, rotations in the interior shear wall are at a maximum at the base and decrease relatively linearly with the height of the structure. This is likely the result of the generally typical reinforcing layout found in the interior shear walls. Rotations corresponding to the IO and LS objectives do, however, indicate clearly the relatively small change in reinforcing found at the first level of the interior shear walls. Support calculations for the exterior and interior shear walls using the NDP can be found in Appendix D.

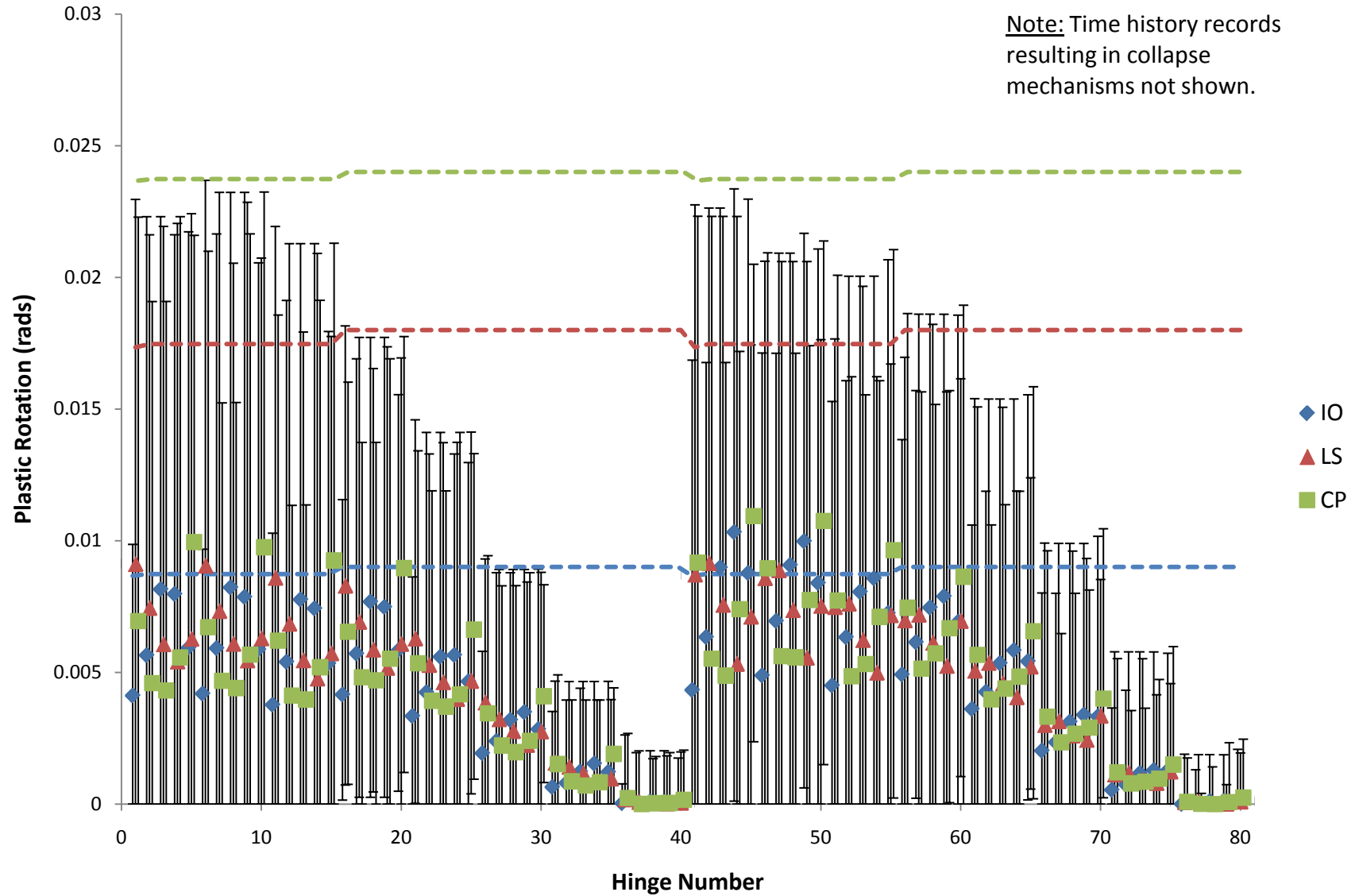


Figure 32. Positive Flexure Plastic Hinge Rotations in Beams

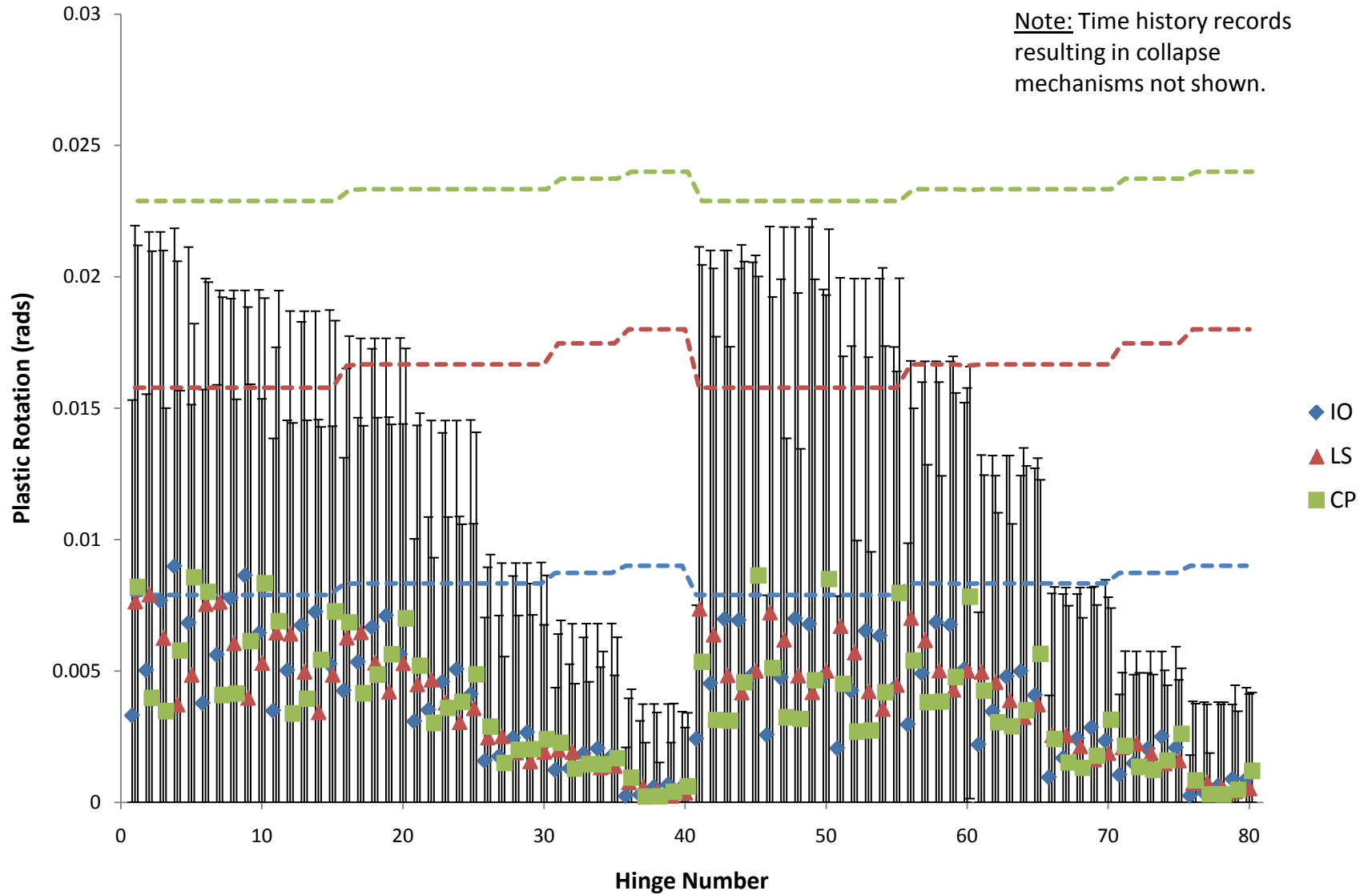
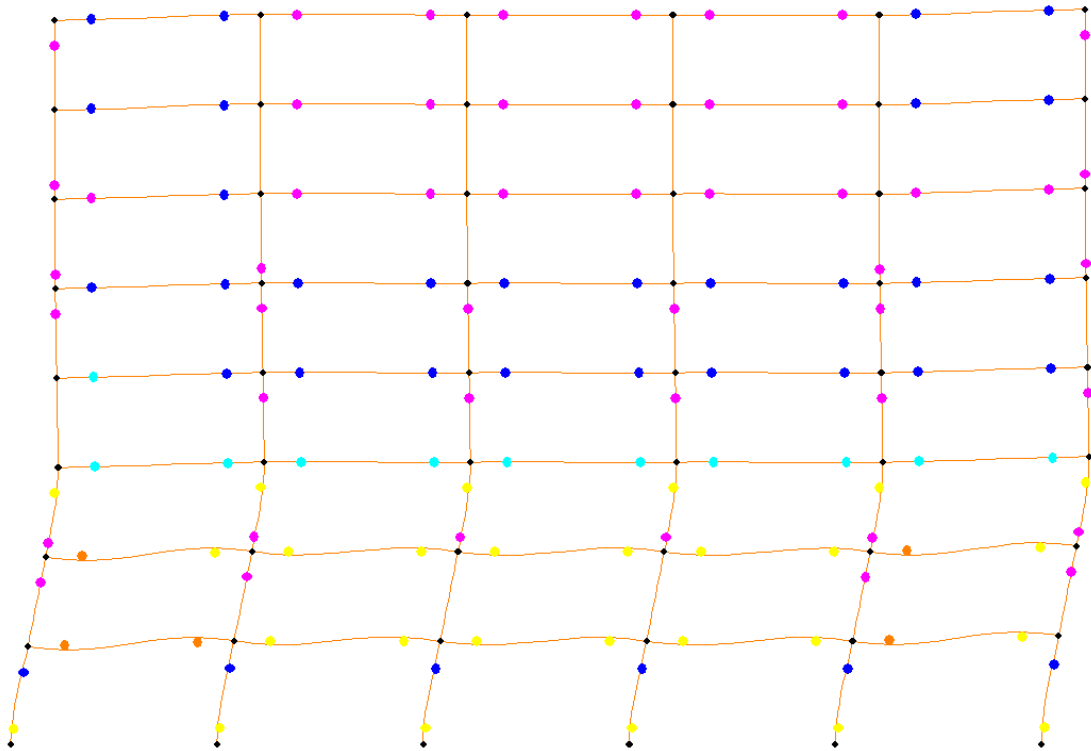


Figure 33. Negative Flexure Plastic Hinge Rotations in Beams



**Figure 34.** Collapse Mechanism for Moment Frame (Record CP 11)

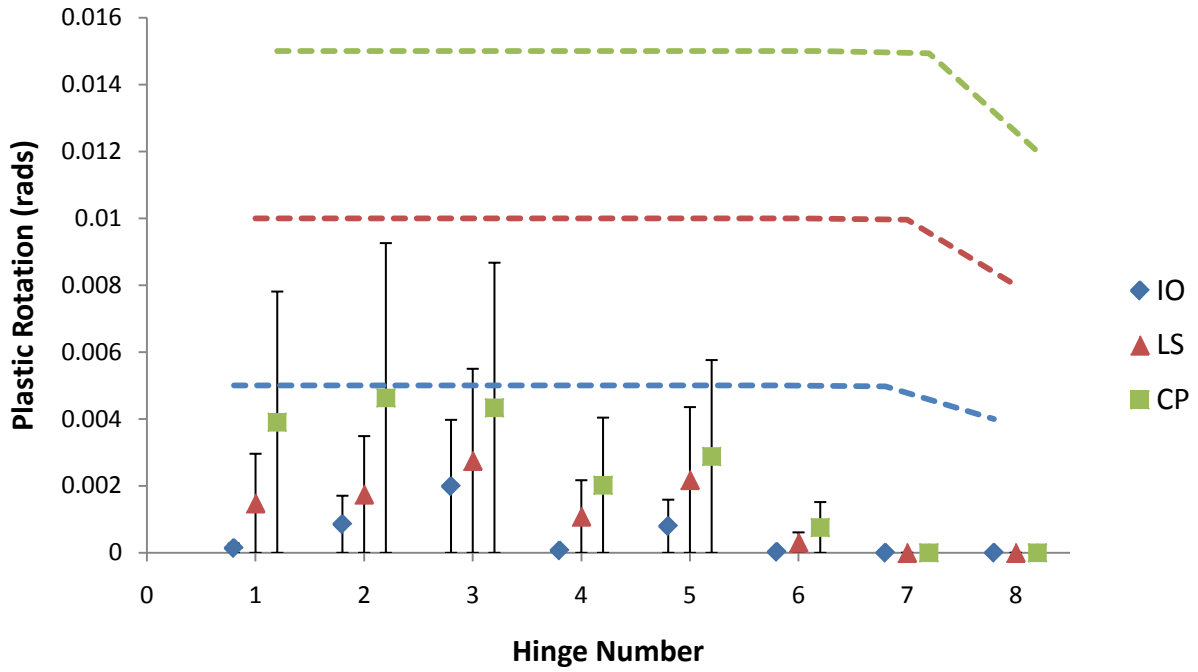


Figure 35. Exterior Shear Wall Plastic Hinge Rotations

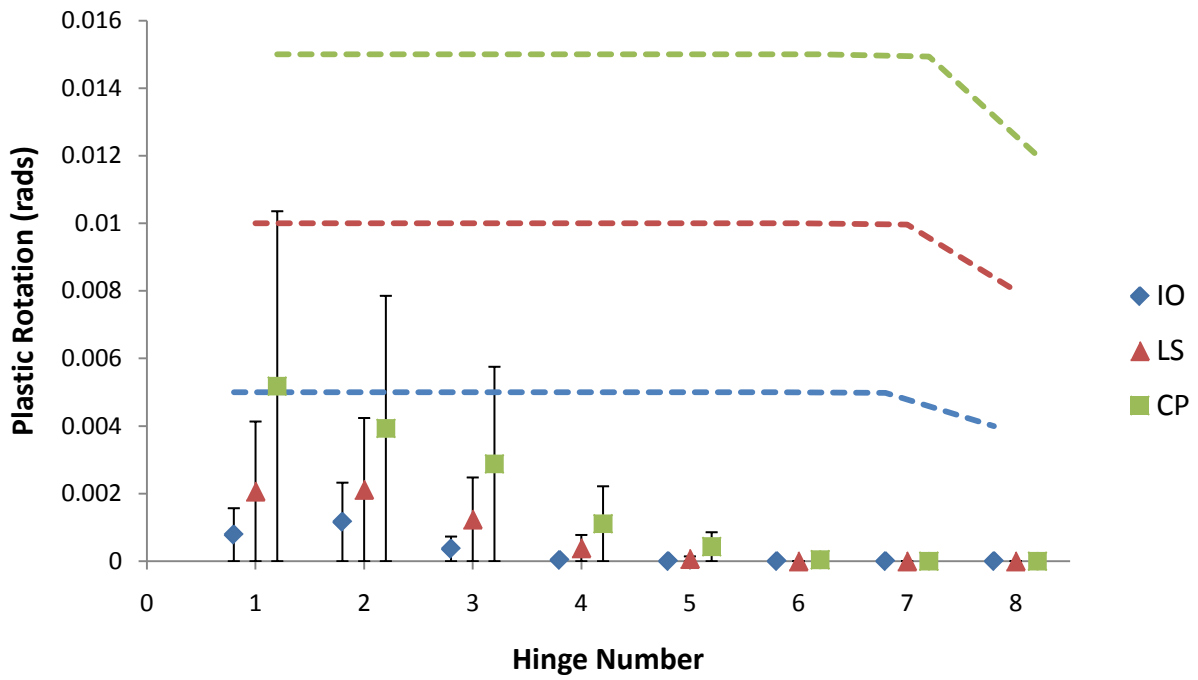


Figure 36. Interior Shear Wall Plastic Hinge Rotations



## **4. CONCLUSIONS AND FUTURE RESEARCH NEEDS**

### **4.1. Conclusions**

A primary goal of ASCE/SEI 41-06 is to provide guidance on assessing the seismic performance of existing structures. Four different methods are prescribed, which include simpler, yet more conservative options, as well as complex options which more accurately represent the behavior of the structure. Assessment of the archetype structure exemplifies these varying approaches, as acceptance criteria are met at a much higher percentage for the more complex methods. However, the main objective of this investigation is to evaluate the procedures outlined in ASCE/SEI 41-06 for use in assessing new structures designed under ASCE/SEI 7-10 requirements.

Overall, the moment frames do not meet the acceptance criteria prescribed in ASCE/SEI 41-06. Throughout all linear and nonlinear procedures, beams and columns repeatedly do not meet assessment criteria. Additionally, use of the nonlinear dynamic procedure indicates a high collapse rate for the structure among various time history records at the collapse prevention performance objective. Although by code, the acceptance criteria can still be met, these collapse mechanisms would not be ignored in a design situation.

Shear walls, similar to moment frames, do not meet all of the acceptance criteria prescribed in ASCE/SEI 41-06. However, shear walls in general do meet the acceptance criteria at a much higher rate than the moment frames. In fact, use of nonlinear procedures results in both exterior and interior shear walls meeting all ASCE/SEI 41-06 requirements.

A likely cause for these discrepancies is the varying methods with which actions are imposed and resisted. ASCE/SEI 7-10 provides reduced design values, based on the ductility of the system, while ASCE/SEI 41-06 provides adjustment factors for component capacities. As these are different approaches, it is difficult to draw direct comparisons between the adjustment methods. However, the findings of this investigation do indicate that the two methods do not provide similar end results.

### **4.2. Further Research**

Further research is needed to provide a more global view into the discrepancies found in this investigation. Varying parameters such as building layout, size, and lateral system could provide insight into the repeatability of the findings of this investigation.

Additionally, more detailed modeling should be investigated. As mentioned, an isotropic hardening hysteresis model is used in this investigation for the nonlinear dynamic procedure. To gain more accuracy, a Takeda model, which accounts for stiffness degradation, could be implemented. In addition, P-Delta effects were neglected for nonlinear procedures in order to resolve convergence issues. Further study should incorporate P-Delta effects to gain a more accurate view of how the structure behaves under seismic loading. Lastly, for nonlinear procedures, stick models are used for all shear walls to maintain simplicity. Flexural behavior in shear walls, however, is not fully captured using this method.

Use of a detailed fiber model of the shear walls should be used for the archetype structure in this investigation to compare the corresponding levels of accuracy.

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## 6. APPENDICES

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## **Appendix A – Design Calculations**



## Section 1. Seismic Design Requirements

### A. Site Information

**D** Site Class (ASCE 7-10 Section 11.4.2)

### B. Response Parameters

$S_S$	<b>1.500</b>	Short Period Spectral Response Acceleration, g (USGS Hazards Map)
$S_1$	<b>0.600</b>	1 Second Spectral Response Acceleration, g (USGS Hazards Map)
$F_a$	<b>1.000</b>	Short Period Site Coefficient (ASCE 7-10 Table 11.4-1)
$F_v$	<b>1.500</b>	Velocity Based Site Coefficient (ASCE 7-10 Table 11.4-2)
$S_{MS}$	<b>1.500</b>	$(F_a * S_S)$ Site Adjusted Short Period Spectral Response Acceleration, g (ASCE 7-10 Equation 11.4-1)
$S_{M1}$	<b>0.900</b>	$(F_v * S_1)$ Site Adjusted 1 Second Spectral Response Acceleration, g (ASCE 7-10 Equation 11.4-2)
$S_{DS}$	<b>1.000</b>	$(2/3 * S_{MS})$ Short Period Design Spectral Acceleration, g (ASCE 7-10 Equation 11.4-3)
$S_{D1}$	<b>0.600</b>	$(2/3 * S_{D1})$ 1 Second Design Spectral Acceleration, g (ASCE 7-10 Equation 11.4-4)
$T_S$	<b>0.600</b>	$(S_{D1}/S_{DS})$ Transition Period, sec
$T_0$	<b>0.120</b>	$(0.2S_{D1}/S_{DS})$ Short Period, sec

### C. Use & Seismic Design Category

**II** Occupancy Category (ASCE 7-10 Table 1.5-1)

**I** Importance Factor (ASCE 7-10 Table 1.5-2)

**D** Seismic Design Category (ASCE 7-10 Table 11.6-1 & 11.6-2)

### D. Structural Systems

Select Structural System Types (ASCE 7-10 Table 12.2-1)

a. N-S Direction

#### A.1 "Special Reinforced Concrete Shear Walls"

R	<b>5</b>	Response Modification Factor (ASCE 7-10 Table 12.2-1)
$\Omega_0$	<b>2.5</b>	System Overstrength Factor (ASCE 7-10 Table 12.2-1)
$C_d$	<b>5</b>	Deflection Amplification Factor (ASCE 7-10 Table 12.2-1)

a. E-W Direction

#### C.5 "Special Reinforced Concrete Moment Frames"

R	<b>8</b>	Response Modification Factor (ASCE 7-10 Table 12.2-1)
$\Omega_0$	<b>3</b>	System Overstrength Factor (ASCE 7-10 Table 12.2-1)
$C_d$	<b>5.5</b>	Deflection Amplification Factor (ASCE 7-10 Table 12.2-1)

### E. Structural Configuration



Reference ASCE 7-10 Table 12.3-1,2 and apply required adjustments accordingly

\*Note: Many Structural Irregularities (Horizontal, Vertical, and Torsional) require assumptions that must be verified after analysis is completed

## F. Load Combinations

Basic Load Combinations (ASCE 7-10 Section 12.4.2.3 & 2.3.2)

1.4D

1.2D + 1.6L + 0.5L<sub>r</sub>

1.2D + 1.6L<sub>r</sub> + L

1.2D + 1.6L<sub>r</sub> + 0.5W

1.2D + 1.0W + L + 0.5L<sub>r</sub>

(1.2 + 0.2S<sub>DS</sub>)D + 0.5L<sup>1</sup> +/- E

0.9D + 1.0W

(0.9 - 0.2S<sub>DS</sub>)D +/- E

1. Load factor for L permitted to be 0.5 for Live Loads less than or equal to 100 PSF

## Section 2. Material Properties & Loading

### A. Material Properties

#### Beams

	Normal-Weight	Concrete Type
f <sub>c</sub> '	5000	Concrete Compressive Strength, psi
<b>**Permitted per ACI 318 Section 21.2.4**</b>		
w <sub>c</sub>	150	Unit Weight of Concrete, pcf
f <sub>y</sub>	60	Reinforcing Steel Yield Strength, ksi
	ASTM A615*	Reinforcing Steel Designation
<i>*Note: Ensure compliance with ACI 318-08 Section 21.1.5.2.a,b</i>		
SC <sub>E</sub>	0.5	Cracked moment of inertia scaling factor (applied to E in SAP2000)

\*Note: Cracked scaling factor may be calculated or reasonably assumed per ACI 318-08 Section 10.10.4.1

#### Columns

	Normal-Weight	Concrete Type
f <sub>c</sub> '	5000	Concrete Compressive Strength, psi
<b>**Permitted per ACI 318 Section 21.2.4**</b>		
w <sub>c</sub>	150	Unit Weight of Concrete, pcf
f <sub>y</sub>	60	Reinforcing Steel Yield Strength, ksi
	ASTM A615*	Reinforcing Steel Designation
<i>*Note: Ensure compliance with ACI 318-08 Section 21.1.5.2.a,b</i>		
SC <sub>E</sub>	0.7	Cracked moment of inertia scaling factor (applied to E in SAP2000)

\*Note: Cracked scaling factor may be calculated or reasonably assumed per ACI 318-08 Section 10.10.4.1





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*Walls*

	<b>Normal-Weight</b>	Concrete Type
$f'_c$	<b>5000</b>	Concrete Compressive Strength, psi
<b>**Permitted per ACI 318 Section 21.2.4**</b>		
$w_c$	<b>150</b>	Unit Weight of Concrete, pcf
$f_y$	<b>60</b>	Reinforcing Steel Yield Strength, ksi
	<b>ASTM A615*</b>	Reinforcing Steel Designation
<i>*Note: Ensure compliance with ACI 318-08 Section 21.1.5.2.a,b</i>		
$SC_E$	<b>0.8</b>	Cracked moment of inertia scaling factor (applied to E in SAP2000)
<i>*Note: Cracked scaling factor may be calculated or reasonably assumed per ACI 318-08 Section 10.10.4.1</i>		

*Slabs*

	<b>Normal-Weight</b>	Concrete Type
$f'_c$	<b>5000</b>	Concrete Compressive Strength, psi
<b>**Permitted per ACI 318 Section 21.2.4**</b>		
$w_c$	<b>150</b>	Unit Weight of Concrete, pcf
$f_y$	<b>60</b>	Reinforcing Steel Yield Strength, ksi
	<b>ASTM A615*</b>	Reinforcing Steel Designation
<i>*Note: Ensure compliance with ACI 318-08 Section 21.1.5.2.a,b</i>		
$SC_E$	<b>1.0</b>	Cracked moment of inertia scaling factor (applied to E in SAP2000)
<i>*Note: Cracked scaling factor may be calculated or reasonably assumed per ACI 318-08 Section 10.10.4.1</i>		

**B. Structural Design Loads**

h	<b>4.5</b>	Typical slab thickness, in
L	<b>102.3</b>	<b>N-S</b> Building length parallel to secondary framing, ft
W	<b>152.3</b>	<b>E-W</b> Building length perpendicular to secondary framing, ft

	SW (PSF)	SDL <sub>seis</sub> (PSF)	SDL <sub>nonseis</sub> (PSF)	CL (PSF)	LL (PSF)	DL <sub>seis</sub> (PSF)	DL <sub>nonseis</sub> (PSF)
Level 08	<b>56.25</b>	<b>20</b>	<b>20</b>	<b>15</b>	<b>20</b>	<b>76</b>	<b>76</b>
Level 07	<b>56.25</b>	<b>20</b>	<b>25</b>	<b>15</b>	<b>50</b>	<b>76</b>	<b>81</b>
Level 06	<b>56.25</b>	<b>20</b>	<b>25</b>	<b>15</b>	<b>50</b>	<b>76</b>	<b>81</b>
Level 05	<b>56.25</b>	<b>20</b>	<b>25</b>	<b>15</b>	<b>50</b>	<b>76</b>	<b>81</b>
Level 04	<b>56.25</b>	<b>20</b>	<b>25</b>	<b>15</b>	<b>50</b>	<b>76</b>	<b>81</b>
Level 03	<b>56.25</b>	<b>20</b>	<b>25</b>	<b>15</b>	<b>50</b>	<b>76</b>	<b>81</b>
Level 02	<b>56.25</b>	<b>20</b>	<b>25</b>	<b>15</b>	<b>50</b>	<b>76</b>	<b>81</b>
Level 01	<b>56.25</b>	<b>20</b>	<b>25</b>	<b>15</b>	<b>50</b>	<b>76</b>	<b>81</b>
Level 00	<b>56.25</b>	<b>20</b>	<b>25</b>	<b>15</b>	<b>100</b>	<b>76</b>	<b>81</b>
Level A	<b>56.25</b>	<b>20</b>	<b>25</b>	<b>15</b>	<b>40</b>	<b>76</b>	<b>81</b>
Level B	<b>56.25</b>	<b>20</b>	<b>25</b>	<b>15</b>	<b>40</b>	<b>76</b>	<b>81</b>

Legend:



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- h Slab Thickness
- SW Self weight of slab
- SDL Superimposed dead load, including roofing, partitions, utilities, etc.
- LL Live Load (per ASCE 7-05 Table 4-1)

\*Note: Applied cladding load uses user defined vertical tributary loads found in **Section 3**

### Section 3. Section Properties, Stiffness Modifiers & Member Loads

\*Note: Program is set up for typical floor framing configurations, table below may be duplicated and edited if levels have unique framing

#### A. Frame Beams

\*Note: Beams are Modeled in ETABS as rectangular shapes below the slab as opposed to T-Beams

	$W_{STEM}$	$H_{STEM}$	$b_{eff}$	$A_{STEM}$	$\bar{y}$	$I_{ETABS}$	$I_{ACTUAL}$	$SC_{I33}$
	in	in	in	in <sup>2</sup>	in	in <sup>4</sup>	in <sup>4</sup>	
<b>FB1</b>	<b>24</b>	<b>27.5</b>	<b>27</b>	<b>660</b>	<b>17.88</b>	<b>41595.6</b>	<b>97131.7</b>	<b>2.335</b>

Legend:

- $W_{STEM}$  Stem width to be used in SAP2000
- $H_{STEM}$  Height of stem below slab to be used in SAP2000
- $b_{eff}$  Calculated effective flange extension, per ACI 318-08 Section 8.12
- $SC_{I33}$  Major axis moment of inertia scaling factor used in SAP2000
- $A_{STEM}$  Cross sectional area of stem

*Unfactored Loads by Beam*

Beam	Level	Horiz Trib (ft)	Vert Trib (ft)	LL (PLF)	CL (PLF)	DL <sub>TOT</sub> (PLF)	
Level 08	<b>FB1</b>	<b>Level 08</b>	<b>6.2</b>	<b>6.5</b>	<b>123</b>	97.5	<b>568</b>
Level 07	<b>FB1</b>	<b>Level 07</b>	<b>6.2</b>	<b>13</b>	<b>308</b>	195	<b>696</b>
Level 06	<b>FB1</b>	<b>Level 06</b>	<b>6.2</b>	<b>13</b>	<b>308</b>	195	<b>696</b>
Level 05	<b>FB1</b>	<b>Level 05</b>	<b>6.2</b>	<b>13</b>	<b>308</b>	195	<b>696</b>
Level 04	<b>FB1</b>	<b>Level 04</b>	<b>6.2</b>	<b>13</b>	<b>308</b>	195	<b>696</b>
Level 03	<b>FB1</b>	<b>Level 03</b>	<b>6.2</b>	<b>13</b>	<b>308</b>	195	<b>696</b>
Level 02	<b>FB1</b>	<b>Level 02</b>	<b>6.2</b>	<b>13</b>	<b>308</b>	195	<b>696</b>
Level 01	<b>FB1</b>	<b>Level 01</b>	<b>6.2</b>	<b>14</b>	<b>308</b>	210	<b>711</b>

#### B. Frame Columns and Shear Walls

- 28** Typical column dimension, in
- 16** Typical shearwall width, in

#### C. Gravity Beam Unfactored Loads

*Distributed Load Beams*

Beam	Level	Grid Loc'n	Horiz Trib (ft)	LL (PLF)	DL (PLF)
------	-------	------------	--------------------	-------------	-------------



<b>GB1</b>	<b>08</b>	<b>1.5 - 5.5</b>	<b>10</b>	<b>200</b>	<b>763</b>
<b>GB1</b>	<b>07 - 01</b>	<b>1.5 - 5.5</b>	<b>10</b>	<b>500</b>	<b>813</b>
<b>GB1</b>	<b>00</b>	<b>1.5 - 5.5</b>	<b>10</b>	<b>1000</b>	<b>813</b>
<b>GB1</b>	<b>A</b>	<b>1.5 - 5.5</b>	<b>10</b>	<b>400</b>	<b>813</b>

*Cladding Load Beams*

<b>Beam</b>	<b>Level</b>	<b>Loc'n</b>	<b>Vert Trib</b>	<b>CL (DL) (PLF)</b>
<b>GB2</b>	<b>08</b>	<b>Perim</b>	<b>6.5</b>	<b>97.5</b>
<b>GB2</b>	<b>07 - 02</b>	<b>Perim</b>	<b>13</b>	<b>195</b>
<b>GB2</b>	<b>01</b>	<b>Perim</b>	<b>14</b>	<b>210</b>



## Section 4. Seismic Modeling

### A. Modeling Criteria

1. Foundation modeling is out of the scope of this program
2. Effective Seismic Weight (ASCE 7-10 Section 12.7.2)

\*Note 1: In storage areas (not including public garages and open parking structures), a minimum of 25% of the floor live load should be included in the seismic weight of the structure

\*Note 2: When partition loading is required, the greater of the partition weight or 10 psf should be included in the seismic weight of the structure

\*Note 3: Total weight of operating machinery should be included in the seismic weight of the structure

\*Note 4: Where the flat roof snow load exceeds 30 psf, a minimum of 20% of the uniform design snow load (regardless of roof slope) should be included in the seismic weight of the structure

### B. Analysis Procedure Notes

\*Note 1: Reference ASCE 7-10 Table 12.6-1 for permitted design procedures

\*Note 2: Ensure after analysis that procedure is still permitted

\*Note 3: Equivalent Lateral Method is required for base shear calculations, regardless of permitted analysis procedures, for scaling purposes

### C. Equivalent Lateral Force Procedure, ELF (ASCE 7-10 Section 12.8)

#### 1. Seismic Weight per Floor (From ETABS Model)

Level	Total Floor Weight (kips)	Total Floor Mass (k-s <sup>2</sup> /ft)	Relative Mass Ratio (%)	Check <sup>1</sup>
Level 08	2277.2	70.72	94.34	PASS
Level 07	2413.7	74.96	100.00	PASS
Level 06	2413.7	74.96	100.00	PASS
Level 05	2413.7	74.96	100.00	PASS
Level 04	2413.7	74.96	100.00	PASS
Level 03	2413.7	74.96	100.00	PASS
Level 02	2413.7	74.96	100.00	PASS
Level 01	2460.1	76.40	101.92	PASS
Total	19220	597		

\*Note: If check fails, a Mass Irregularity exists per ASCE 7-10 Table 12.3-2

#### 2.a. Determine Period (N-S)

$h_n$  **106** Height above base to highest level of structure, ft

$C_w$  **0.0387** Coefficient for shear walls (if necessary)

\*Note: Calculation of  $C_w$  based on ASCE 7-10 Equation 12.8-10

SW  $T_a$  **1.02** ( $C_t \cdot h_n^x$ ) Approximate Period, sec (ASCE 7-10 Equation 12.8-7)

\*Note: Values of  $C_t$  and  $x$  from ASCE 7-10 Table 12.8-2

$T_a \cdot C_u$  **1.43** Upper Limit Approx. Period, sec (ASCE 7-10 Section 12.8.2)

\*Note: Value of  $C_u$  from ASCE 7-10 Table 12.8-1

$T_{calc}$  **1.072** Calculated (ETABS, etc) Period, sec



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T	<b>1.07</b>	Period to be used for ELF, sec
<i>*Note: Use only the values of T corresponding to the structural system in use</i>		
T <sub>drift</sub>	<b>1.07</b>	Period for use in calculating drifts, sec

Seismic Base Shear Calculation

T <sub>L</sub>	<b>8</b>	Long Period Transition Period, sec (ASCE 7-10 Figure 22-15)
C <sub>s,max</sub>	<b>0.2000</b>	(S <sub>DS</sub> /(R/I)) Max Seismic Response Coefficient (ASCE 7-10 Equation 12.8-2)
C <sub>s,T</sub>	<b>0.1119</b>	Adjusted Seismic Response Coefficient (ASCE 7-10 Equation 12.8-3,4)
C <sub>s,T</sub>	<b>0.1119</b>	Adjusted Seismic Response Coefficient for Drift Calculations (No TaCu Limit per ASCE 7-10 Section 12.8.6.2)
C <sub>s,min</sub>	<b>0.0440</b>	Min Seismic Response Coefficient (ASCE 7-10 Equation 12.8-5)
C <sub>s,S1</sub>	<b>0.0600</b>	Min Seismic Response Coefficient for S1 ≥ .6g (ASCE 7-10 Equation 12.8-6)
C <sub>s</sub>	<b>0.1119</b>	Seismic Response Coefficient to be used
C <sub>s,drift</sub>	<b>0.1119</b>	Seismic Response Coefficient for Drift
V	<b>2151</b>	(C <sub>s</sub> *W) Seismic Base Shear, kips (ASCE 7-10 Equation 12.8-1)
V <sub>drift</sub>	<b>2151</b>	Seismic Base Shear for Drift Calculations, kip
S <sub>drift,NS</sub>	<b>1.00</b>	V <sub>drift</sub> / V

2.b. Determine Period (E-W)

h <sub>n</sub>	<b>106</b>	Height above base to highest level of structure, ft
C <sub>w</sub>	<b>NA</b>	Coefficient for shear walls (if necessary)

*\*Note: Calculation of C<sub>w</sub> based on ASCE 7-10 Equation 12.8-10*

MRF	T <sub>a</sub>	<b>1.06</b>	(C <sub>t</sub> *h <sub>n</sub> <sup>x</sup> ) Approximate Period, sec (ASCE 7-10 Equation 12.8-7)
<i>*Note: Values of C<sub>t</sub> and x from ASCE 7-10 Table 12.8-2</i>			
	T <sub>a</sub> *C <sub>u</sub>	<b>1.49</b>	Upper Limit Approx. Period, sec (ASCE 7-10 Section 12.8.2)
<i>*Note: Value of C<sub>u</sub> from ASCE 7-10 Table 12.8-1</i>			
	T <sub>calc</sub>	<b>1.868</b>	Calculated (ETABS, etc) Period, sec
	T	<b>1.49</b>	Period to be used for ELF, sec
<i>*Note: Use only the values of T corresponding to the structural system in use</i>			
	T <sub>drift</sub>	<b>1.87</b>	Period for use in calculating drifts, sec

Seismic Base Shear Calculation

T <sub>L</sub>	<b>8</b>	Long Period Transition Period, sec (ASCE 7-10 Figure 22-15)
C <sub>s,max</sub>	<b>0.1250</b>	(S <sub>DS</sub> /(R/I)) Max Seismic Response Coefficient (ASCE 7-10 Equation 12.8-2)



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$C_{s,T}$	<b>0.0504</b>	Adjusted Seismic Response Coefficient (ASCE 7-10 Equation 12.8-3,4)
$C_{s,T}$	<b>0.0401</b>	Adjusted Seismic Response Coefficient for Drift Calculations (No $T_z C_u$ Limit per ASCE 7-10 Section 12.8.6.2)
$C_{s,min}$	<b>0.0440</b>	Min Seismic Response Coefficient (ASCE 7-10 Equation 12.8-5)
$C_{s,S1}$	<b>0.0375</b>	Min Seismic Response Coefficient for $S1 \geq .6g$ (ASCE 7-10 Equation 12.8-6)
$C_s$	<b>0.0504</b>	Seismic Response Coefficient to be used
$C_{s,drift}$	<b>0.0401</b>	Seismic Response Coefficient for Drift
$V$	<b>968</b>	$(C_s * W)$ Seismic Base Shear, kips (ASCE 7-10 Equation 12.8-1)
$V_{drift}$	<b>772</b>	Seismic Base Shear for Drift Calculations, kip
$S_{drift,EW}$	<b>0.797</b>	$V_{drift} / V$

3.a. Vertical Force Distribution (N-S)

$k$	<b>1.29</b>	Structure Period Exponent (ASCE 7-10 Section 12.8.3)					
Level	Height	$w_x * h_x^k$	$C_{vx}^a$	$F_x^b$	$F_{x,drift}$	$V_x^c$	$M_x^d$
	(ft)			(kips)	(k)	(kips)	(k-ft)
Level 08	106	916097	0.235	505.3	505.3		
Level 07	93	820643.5	0.210	452.6	452.6	505.3	6,569
Level 06	80	676175.1	0.173	372.9	372.9	957.9	19,021
Level 05	67	538291.7	0.138	296.9	296.9	1330.9	36,323
Level 04	54	407890.7	0.105	225.0	225.0	1627.8	57,483
Level 03	41	286236.8	0.073	157.9	157.9	1852.7	81,569
Level 02	28	175279.3	0.045	96.7	96.7	2010.6	107,707
Level 01	15	80057.6	0.021	44.2	44.2	2107.3	135,102
<b>Total</b>			<b>1.000</b>	<b>2151</b>	<b>2151</b>	<b>2151.4</b>	<b>167,373</b>

a. Per ASCE 7-10 Equation 12.8-12

b. Per ASCE 7-10 Equation 12.8-11

c. Story shear

d. Overturning Moment

3.b. Vertical Force Distribution (E-W)

$k$	<b>1.49</b>	Structure Period Exponent (ASCE 7-10 Section 12.8.3)					
Level	Height	$w_x * h_x^k$	$C_{vx}^a$	$F_x^b$	$F_{x,drift}$	$V_x^c$	$M_x^d$
	(ft)			(kips)	(k)	(kips)	(k-ft)
Level 08	106	2424735	0.253	245.2	195.5		
Level 07	93	2113573	0.221	213.7	170.4	245.2	3,187
Level 06	80	1687614	0.176	170.6	136.0	458.8	9,152
Level 05	67	1294663	0.135	130.9	104.4	629.5	17,335
Level 04	54	937842.5	0.098	94.8	75.6	760.4	27,220
Level 03	41	621364.6	0.065	62.8	50.1	855.2	38,338



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Level 02	28	351384.2	0.037	35.5	28.3	918.0	50,272
Level 01	15	140888.9	0.015	14.2	11.4	953.5	62,668
<b>Total</b>			<b>1.000</b>	<b>968</b>	<b>772</b>	<b>967.8</b>	<b>77,185</b>

- a. Per ASCE 7-10 Equation 12.8-12
- b. Per ASCE 7-10 Equation 12.8-11
- c. Story shear
- d. Overturning Moment

For Modelling Purposes, Ratio of Drift Base Shear to Strength Base Shear

**0.797** E-W Ratio  
**1.000** N-S Ratio

4. Horizontal Force Distribution & Accidental Torsion

\*Note: Accidental torsion calculated by multiplying the story force,  $F_x$ , by an eccentricity equal to 5% of the respective building width (ASCE 7-10 Section 12.8.4.2)

Level	Force, $F_{NS}$ (kips)	Force, $F_{EW}$ (kips)	NS Width (ft)	EW Width (ft)	NS Tors (k-ft)	EW Tors (k-ft)
Level 08	<b>505.3</b>	<b>245.2</b>	<b>152.3333</b>	<b>102.3333</b>	<b>3849</b>	<b>1254</b>
Level 07	<b>452.6</b>	<b>213.7</b>	<b>152.3333</b>	<b>102.3333</b>	<b>3448</b>	<b>1093</b>
Level 06	<b>372.9</b>	<b>170.6</b>	<b>152.3333</b>	<b>102.3333</b>	<b>2841</b>	<b>873</b>
Level 05	<b>296.9</b>	<b>130.9</b>	<b>152.3333</b>	<b>102.3333</b>	<b>2261</b>	<b>670</b>
Level 04	<b>225.0</b>	<b>94.8</b>	<b>152.3333</b>	<b>102.3333</b>	<b>1714</b>	<b>485</b>
Level 03	<b>157.9</b>	<b>62.8</b>	<b>152.3333</b>	<b>102.3333</b>	<b>1202</b>	<b>321</b>
Level 02	<b>96.7</b>	<b>35.5</b>	<b>152.3333</b>	<b>102.3333</b>	<b>736</b>	<b>182</b>
Level 01	<b>44.2</b>	<b>14.2</b>	<b>152.3333</b>	<b>102.3333</b>	<b>336</b>	<b>73</b>

\*Note: Forces are derived from ELF Method 16387      4952

D. Modal Response Spectrum Analysis (ASCE 7-10 Section 12.9)

1. Elastic Response Spectrum Development (ASCE 7-10 Section 11.4.5)

Reference sheet "Response Spectrum Data" and copy spectrum to a .txt file, the user may now add this file in SAP as a Response Function to be used in analysis

2. Modal Mass Participation Factors

\*Note: Prior to continuing through this section, the SAP model should be run to determine modes, etc.

Mode	Period (s)	% Effective Mass		
		N-S	E-W	Torsion
1	<b>2.100</b>	<b>82.848</b>	<b>0.000</b>	<b>18.128</b>
2	<b>1.072</b>	<b>0.000</b>	<b>70.729</b>	<b>34.864</b>
3	<b>0.954</b>	<b>0.000</b>	<b>0.000</b>	<b>20.245</b>
4	<b>0.671</b>	<b>9.596</b>	<b>0.000</b>	<b>2.100</b>
5	<b>0.373</b>	<b>3.398</b>	<b>0.000</b>	<b>0.744</b>
6	<b>0.243</b>	<b>1.624</b>	<b>0.000</b>	<b>0.355</b>
7	<b>0.195</b>	<b>0.000</b>	<b>21.422</b>	<b>10.559</b>



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8	<b>0.189</b>	<b>0.000</b>	<b>0.000</b>	<b>5.810</b>
9	<b>0.173</b>	<b>0.827</b>	<b>0.000</b>	<b>0.181</b>
10	<b>0.146</b>	<b>0.000</b>	<b>0.000</b>	<b>0.000</b>
11	<b>0.146</b>	<b>0.000</b>	<b>0.000</b>	<b>0.000</b>
12	<b>0.139</b>	<b>0.000</b>	<b>0.000</b>	<b>0.000</b>
Total		<b>98.29</b>	<b>92.15</b>	<b>92.99</b>
Check		<b>OK</b>	<b>OK</b>	<b>OK</b>

\*Note: Per ASCE 7-10 Section 12.9.1, the number of modes investigated must represent at least 90% of the total mass

2. Scaling Requirements (ASCE 7-10 Section 12.9.4)

\*Note: The elastic response spectrum requires scaling to achieve an approximate inelastic response spectrum as well as to ensure compliance with ASCE 7-10 Section 12.9.4. The final scaling factor should be applied as a load combination scale factor in SAP

\*Note: The R factors for the two orthogonal building directions have been embedded into the calculated response spectrums in this workbook

\*Note: A unit conversion of  $1g = 32.2 \text{ ft/s}^2$  has been embedded into the response spectrums - ensure that the units in SAP comply, and scale the response spectrum accordingly if not

$\xi$  **0.05** Damping Ratio

\*Note: Typically, a damping ratio of 0.05 is used

Direction	$V_{ELF}$ (kips)	$0.85 \cdot V_{ELF}$ (kips)	$V_{MRSA}$ (kips)	Force Scale Factor
N-S	<b>2151</b>	<b>1829</b>	<b>2282.9</b>	<b>1.00</b>
E-W	<b>968</b>	<b>823</b>	<b>625</b>	<b>1.32</b>

\*Note: These scale factors should be applied in SAP as load combination scale factors (in addition to any other combination scaling)

E. Torsional Irregularities and Accidental Torsion Amplification

\*Note: Input Displacements Based on Drift Load Combinations

Level	E-W Displacements		N-S Displacements	
	N Side (in)	S Side (in)	E Side (in)	W Side (in)
Level 08	<b>3.607</b>	<b>3.754</b>	<b>3.770</b>	<b>2.851</b>
Level 07	<b>3.431</b>	<b>3.555</b>	<b>3.191</b>	<b>2.410</b>
Level 06	<b>3.137</b>	<b>3.239</b>	<b>2.612</b>	<b>1.971</b>
Level 05	<b>2.735</b>	<b>2.815</b>	<b>2.046</b>	<b>1.543</b>
Level 04	<b>2.248</b>	<b>2.307</b>	<b>1.507</b>	<b>1.137</b>
Level 03	<b>1.699</b>	<b>1.738</b>	<b>1.016</b>	<b>0.769</b>
Level 02	<b>1.112</b>	<b>1.134</b>	<b>0.594</b>	<b>0.453</b>
Level 01	<b>0.523</b>	<b>0.532</b>	<b>0.265</b>	<b>0.207</b>
Base	<b>0.005</b>	<b>0.005</b>	<b>0.035</b>	<b>0.033</b>

\*Note: The user should import or transpose the story drifts from the SAP model  
Check E-W direction for Torsional Irregularity





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Level	Drift <sub>N</sub> (in)	Drift <sub>S</sub> (in)	Drift <sub>AVE</sub> (in)	Max/Ave	Torsional Irregularity Exisits?
Level 08	<b>0.141</b>	<b>0.158</b>	<b>0.150</b>	<b>1.059</b>	NO
Level 07	<b>0.234</b>	<b>0.252</b>	<b>0.243</b>	<b>1.037</b>	NO
Level 06	<b>0.320</b>	<b>0.338</b>	<b>0.329</b>	<b>1.027</b>	NO
Level 05	<b>0.388</b>	<b>0.405</b>	<b>0.397</b>	<b>1.021</b>	NO
Level 04	<b>0.438</b>	<b>0.453</b>	<b>0.446</b>	<b>1.017</b>	NO
Level 03	<b>0.468</b>	<b>0.481</b>	<b>0.475</b>	<b>1.014</b>	NO
Level 02	<b>0.470</b>	<b>0.481</b>	<b>0.475</b>	<b>1.011</b>	NO
Level 01	<b>0.413</b>	<b>0.420</b>	<b>0.416</b>	<b>1.008</b>	NO
Amplification Needed?					<b>NO</b>

Check N-S direction for Torsional Irregularity

Level	Drift <sub>E</sub> (in)	Drift <sub>W</sub> (in)	Drift <sub>AVE</sub> (in)	Max/Ave	Torsional Irregularity Exisits?
Level 08	<b>0.579</b>	<b>0.441</b>	<b>0.510</b>	<b>1.135</b>	NO
Level 07	<b>0.579</b>	<b>0.439</b>	<b>0.509</b>	<b>1.137</b>	NO
Level 06	<b>0.566</b>	<b>0.428</b>	<b>0.497</b>	<b>1.139</b>	NO
Level 05	<b>0.538</b>	<b>0.405</b>	<b>0.472</b>	<b>1.141</b>	NO
Level 04	<b>0.491</b>	<b>0.369</b>	<b>0.430</b>	<b>1.143</b>	NO
Level 03	<b>0.422</b>	<b>0.316</b>	<b>0.369</b>	<b>1.144</b>	NO
Level 02	<b>0.329</b>	<b>0.246</b>	<b>0.288</b>	<b>1.144</b>	NO
Level 01	<b>0.230</b>	<b>0.174</b>	<b>0.202</b>	<b>1.139</b>	NO
Amplification Needed?					<b>NO</b>

Torsional Irregularity Requirements

- 1** *Increase in seismic forces calculated from ASCE 7-10 Section 12.8.1 for connections of diaphragms to vertical elements. Collectors and their connections shall be increased unless they are designed for the load combinations with overstrength factor of ASCE 7-10 Section 12.4.3.2 (ASCE 7-10 Section 12.3.3.4)*
- NO** *3D Modeling requirement per ASCE 7-10 Section 12.7.3?*
- NO** *Story drift shall be calculated as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration (ASCE 7-10 Section 12.12.1)*

Redundancy Factor

$\rho_{EW}$	<b>1.0</b>	Note Requirements per ASCE 7-10 Section 12.3.4
$\rho_{NS}$	<b>1.3</b>	Note Requirements per ASCE 7-10 Section 12.3.4

\*Note: For  $\rho$  to be taken as 1.0, ensure all levels shown below that resist more than 35% of the total Base Shear comply with ASCE 7-10 Section 12.3.4.2.a or b

\*Note:  $\rho$  applied to load combinations in SAP model where applicable



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Level	EW		NS	
	Story Force (kips)	% Base Shear	Story Force (kips)	% Base Shear
Level 08	<b>245.2</b>	<b>25.3</b>	<b>505.3</b>	<b>23.5</b>
Level 07	<b>458.8</b>	<b>47.4</b>	<b>957.9</b>	<b>44.5</b>
Level 06	<b>629.5</b>	<b>65.0</b>	<b>1330.9</b>	<b>61.9</b>
Level 05	<b>760.4</b>	<b>78.6</b>	<b>1627.8</b>	<b>75.7</b>
Level 04	<b>855.2</b>	<b>88.4</b>	<b>1852.7</b>	<b>86.1</b>
Level 03	<b>918.0</b>	<b>94.9</b>	<b>2010.6</b>	<b>93.5</b>
Level 02	<b>953.5</b>	<b>98.5</b>	<b>2107.3</b>	<b>97.9</b>
Level 01	<b>967.8</b>	<b>100.0</b>	<b>2151.4</b>	<b>100.0</b>

Check redundancy = 1.0 (Moment Frames) - Single bay removed on main floor

Level	Disp <sub>N</sub> (in)	Disp <sub>S</sub> (in)	Drift <sub>N</sub> (in)	Drift <sub>S</sub> (in)	Drift <sub>AVE</sub> (in)	Max/Ave	Extreme Torsional Irregularity?
Level 08	3.722	3.856	0.141	0.158	0.150	1.057	NO
Level 07	3.545	3.658	0.234	0.252	0.243	1.036	NO
Level 06	3.251	3.342	0.321	0.338	0.329	1.026	NO
Level 05	2.849	2.919	0.389	0.405	0.397	1.021	NO
Level 04	2.362	2.411	0.439	0.454	0.447	1.017	NO
Level 03	1.811	1.841	0.473	0.486	0.479	1.013	NO
Level 02	1.218	1.232	0.495	0.504	0.500	1.009	NO
Level 01	0.597	0.599	0.476	0.478	0.477	1.002	NO

Check redundancy = 1.0 (Shear Walls) - Single wall pier removed on main floor

Level	Disp <sub>E</sub> (in)	Disp <sub>W</sub> (in)	Drift <sub>E</sub> (in)	Drift <sub>W</sub> (in)	Drift <sub>AVE</sub> (in)	Max/Ave	Extreme Torsional Irregularity?
Level 08	3.528	2.100	0.511	0.334	0.422	1.208	NO
Level 07	3.018	1.766	0.511	0.332	0.421	1.212	NO
Level 06	2.507	1.434	0.502	0.323	0.412	1.217	NO
Level 05	2.005	1.111	0.481	0.304	0.393	1.225	NO
Level 04	1.524	0.807	0.447	0.274	0.360	1.239	NO
Level 03	1.077	0.532	0.395	0.232	0.314	1.260	NO
Level 02	0.682	0.300	0.327	0.176	0.251	1.301	NO
Level 01	0.355	0.125	0.355	0.125	0.240	1.480	YES

Accidental Torsion Amplification

Level	EW				NS			
	Max Disp (in)	Ave Disp (in)	A <sub>x</sub>	A <sub>x</sub> M <sub>ta</sub> (k-ft)	Max Disp (in)	Ave Disp (in)	A <sub>x</sub>	A <sub>x</sub> M <sub>ta</sub> (k-ft)
Level 08	<b>3.754</b>	<b>3.681</b>	<b>1.00</b>	<b>1254</b>	<b>3.770</b>	<b>3.311</b>	<b>1.00</b>	<b>3849</b>
Level 07	<b>3.555</b>	<b>3.493</b>	<b>1.00</b>	<b>1093</b>	<b>3.191</b>	<b>2.800</b>	<b>1.00</b>	<b>3448</b>
Level 06	<b>3.239</b>	<b>3.188</b>	<b>1.00</b>	<b>873</b>	<b>2.612</b>	<b>2.292</b>	<b>1.00</b>	<b>2841</b>



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Level 05	<b>2.815</b>	<b>2.775</b>	<b>1.00</b>	<b>670</b>	<b>2.046</b>	<b>1.794</b>	<b>1.00</b>	<b>2261</b>
Level 04	<b>2.307</b>	<b>2.277</b>	<b>1.00</b>	<b>485</b>	<b>1.507</b>	<b>1.322</b>	<b>1.00</b>	<b>1714</b>
Level 03	<b>1.738</b>	<b>1.719</b>	<b>1.00</b>	<b>321</b>	<b>1.016</b>	<b>0.892</b>	<b>1.00</b>	<b>1202</b>
Level 02	<b>1.134</b>	<b>1.123</b>	<b>1.00</b>	<b>182</b>	<b>0.594</b>	<b>0.524</b>	<b>1.00</b>	<b>736</b>
Level 01	<b>0.532</b>	<b>0.527</b>	<b>1.00</b>	<b>73</b>	<b>0.265</b>	<b>0.236</b>	<b>1.00</b>	<b>336</b>

\*Note: Rerun the SAP model at this point with the amplified accidental torsion values

### F. Drift and P-Delta Effects

\*NOTE: The structure model should be analyzed for drift with Force + Torsion AND Force-Torsion to obtain the max drift values on each side of the building (this should be done in both orthogonal directions as well)

#### Story Displacements Including Amplified Torsions, Based on Drift Load Combinations

Level	E-W Displacements		N-S Displacements	
	N Side (in)	S Side (in)	E Side (in)	W Side (in)
Level 08	<b>3.607</b>	<b>3.754</b>	<b>3.770</b>	<b>2.851</b>
Level 07	<b>3.431</b>	<b>3.555</b>	<b>3.191</b>	<b>2.410</b>
Level 06	<b>3.137</b>	<b>3.239</b>	<b>2.612</b>	<b>1.971</b>
Level 05	<b>2.735</b>	<b>2.815</b>	<b>2.046</b>	<b>1.543</b>
Level 04	<b>2.248</b>	<b>2.307</b>	<b>1.507</b>	<b>1.137</b>
Level 03	<b>1.699</b>	<b>1.738</b>	<b>1.016</b>	<b>0.769</b>
Level 02	<b>1.112</b>	<b>1.134</b>	<b>0.594</b>	<b>0.453</b>
Level 01	<b>0.523</b>	<b>0.532</b>	<b>0.265</b>	<b>0.207</b>
Base	<b>0.005</b>	<b>0.005</b>	<b>0.035</b>	<b>0.033</b>

#### E-W Story Drift Calculations

**2.0** Maximum drift ratio per ASCE 7-10 Table 12.12-1, %

\*Note: Reference table notes for additional requirements on drift limit

Level	Story Drift (North) (in)	Story Drift (South) (in)	C <sub>d</sub> * Drift <sub>MAX</sub> (in)	Story Height (in)	Max Drift Ratio (%)	Drift Check	Story Stiffness <sup>1</sup> (k/in)	Relative Stiffness <sup>2</sup> (%)	Relative Stiffness <sup>3</sup> (%)
Level 08	<b>0.141</b>	<b>0.158</b>	<b>0.871</b>	<b>156</b>	<b>0.559</b>	<b>OK</b>	<b>1638.8</b>	<b>NA</b>	<b>NA</b>
Level 07	<b>0.234</b>	<b>0.252</b>	<b>1.387</b>	<b>156</b>	<b>0.889</b>	<b>OK</b>	<b>1886.9</b>	<b>115.1</b>	<b>NA</b>
Level 06	<b>0.320</b>	<b>0.338</b>	<b>1.859</b>	<b>156</b>	<b>1.191</b>	<b>OK</b>	<b>1912.8</b>	<b>101.4</b>	<b>NA</b>
Level 05	<b>0.388</b>	<b>0.405</b>	<b>2.229</b>	<b>156</b>	<b>1.429</b>	<b>OK</b>	<b>1916.0</b>	<b>100.2</b>	<b>105.7</b>
Level 04	<b>0.438</b>	<b>0.453</b>	<b>2.494</b>	<b>156</b>	<b>1.599</b>	<b>OK</b>	<b>1919.0</b>	<b>100.2</b>	<b>100.7</b>
Level 03	<b>0.468</b>	<b>0.481</b>	<b>2.648</b>	<b>156</b>	<b>1.697</b>	<b>OK</b>	<b>1934.1</b>	<b>100.8</b>	<b>100.9</b>
Level 02	<b>0.470</b>	<b>0.481</b>	<b>2.643</b>	<b>156</b>	<b>1.694</b>	<b>OK</b>	<b>2006.2</b>	<b>103.7</b>	<b>104.3</b>
Level 01	<b>0.413</b>	<b>0.420</b>	<b>2.309</b>	<b>180</b>	<b>1.283</b>	<b>OK</b>	<b>2324.7</b>	<b>115.9</b>	<b>119.0</b>

1. Story Stiffness is taken as the Story Shear divided by the Story Drift

2. Relative stiffness to floor above

3. Relative stiffness to average of 3 floors above



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\*Note: If relative stiffness cell turns yellow, a stiffness-soft story exists per ASCE 7-10 Table 12.3-2 (1a)

\*Note: If relative stiffness cell turns red, a stiffness-extreme soft story exists per ASCE 7-10 Table 12.3-2 (1b)

N-S Story Drift Calculations

**2.0** Maximum drift ratio per ASCE 7-10 Table 12.12-1, %

\*Note: Reference table notes for additional requirements on drift limit

Level	Story Drift (East) (in)	Story Drift (West) (in)	$C_d$ * Drift <sub>MAX</sub> (in)	Story Height (in)	Max Drift Ratio (%)	Drift Check	Story Stiffness <sup>1</sup> (k/in)	Relative Stiffness <sup>2</sup> (%)	Relative Stiffness <sup>3</sup> (%)
Level 08	0.579	0.441	2.897	156	1.857	OK	989.9	NA	NA
Level 07	0.579	0.439	2.894	156	1.855	OK	1882.4	190.2	NA
Level 06	0.566	0.428	2.832	156	1.815	OK	2676.2	142.2	NA
Level 05	0.538	0.405	2.692	156	1.726	OK	3449.2	128.9	186.5
Level 04	0.491	0.369	2.456	156	1.574	OK	4310.0	125.0	161.5
Level 03	0.422	0.316	2.110	156	1.352	OK	5449.8	126.4	156.7
Level 02	0.329	0.246	1.644	156	1.054	OK	7328.2	134.5	166.4
Level 01	0.230	0.174	1.149	180	0.638	OK	10663.8	145.5	187.2

1. Story Stiffness is taken as the Story Shear divided by the Story Drift

2. Relative stiffness to floor above

3. Relative stiffness to average of 3 floors above

\*Note: If relative stiffness cell turns yellow, a stiffness-soft story exists per ASCE 7-10 Table 12.3-2 (1a)

\*Note: If relative stiffness cell turns red, a stiffness-extreme soft story exists per ASCE 7-10 Table 12.3-2 (1b)

E-W P-Delta Effects

Level	$\beta$	$\theta_{max}$	Story DL (kips)	Story LL (kips)	Total (kips)	Accum (kips)	$\theta$	Check
Level 08	1.0	0.091	2277	312	2589	2589	0.01073	OK
Level 07	1.0	0.091	2492	779	3271	5860	0.01152	OK
Level 06	1.0	0.091	2492	779	3271	9131	0.01126	OK
Level 05	1.0	0.091	2492	779	3271	12402	0.01118	OK
Level 04	1.0	0.091	2492	779	3271	15673	0.01112	OK
Level 03	1.0	0.091	2492	779	3271	18944	0.01100	OK
Level 02	1.0	0.091	2492	779	3271	22216	0.01057	OK
Level 01	1.0	0.091	2538	779	3317	25533	0.00800	OK

\*Note: If all stories are within stability limits, P-Delta effects need not be considered

Legend:

- $\beta$  Ratio of shear demand to shear capacity between the story and the story below, taken as 1.0 conservatively if desired (ASCE 7-10 Section 12.8.7)
- $\theta$  Story Stability Coefficient
- $\theta_{max}$  Max stability coefficient (ASCE 7-10 Equation 12.8-17)

N-S P-Delta Effects



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Level	$\beta$	$\theta_{max}$	Story DL (kips)	Story LL (kips)	Total (kips)	Accum (kips)	$\theta$	Check
Level 08	1.0	0.100	2277	312	2589	2589	0.0190	OK
Level 07	1.0	0.100	2492	779	3271	5860	0.0227	OK
Level 06	1.0	0.100	2492	779	3271	9131	0.0249	OK
Level 05	1.0	0.100	2492	779	3271	12402	0.0263	OK
Level 04	1.0	0.100	2492	779	3271	15673	0.0266	OK
Level 03	1.0	0.100	2492	779	3271	18944	0.0255	OK
Level 02	1.0	0.100	2492	779	3271	22216	0.0222	OK
Level 01	1.0	0.100	2538	779	3317	25533	0.0152	OK

\*Note: If all stories are within stability limits, P-Delts effects need not be considered

Legend:

- $\beta$  Ratio of shear demand to shear capacity between the story and the story below, taken as 1.0 conservatively if desired (ASCE 7-10 Section 12.8.7)
- $\theta$  Story Stability Coefficient
- $\theta_{max}$  Max stability coefficient (ASCE 7-10 Equation 12.8-17)

5. Additional Notes

- a. Verify if a reentrant corner exists as defined in ASCE 7-10 Table 12.3-1(2)
- b. Verify if the diaphragm has a discontinuity irregularity as define in ASCE 7-10 Table 12.3-1(3)
- c. Verify if out-of-plane offsets exist as defined in ASCE 7-10 Table 12.3-1(4)
- d. Verify that the vertical resisting system components are parallel as defined in ASCE 7-10 Table 12.3-1(5)
- e. Verify if a vertical geometric irregularity exists as defined in ASCE 7-10 Table 12.3-2(3)
- f. Verify if an in-plane vertical irregularity exists as defined in ASCE 7-10 Table 12.3-2(4)
- g. Moment frame design should be checked for weak story requirements prescribed in ASCE 7-10 Table 12.3-2(5a, 5b)



## Concrete Lateral Systems - Wind Forces

### Section 5. Wind Design (ASCE 7-10 Chapter 26 & 27)

#### A. General Requirements

W	<b>102.3</b>	N-S Building Dimension, ft
L	<b>152.3</b>	E-W Building Dimension, ft
V	<b>110</b>	Basic Wind Speed (3 second gust), mph
Exp	<b>C</b>	Exposure Category (ASCE 7-10 Section 26.7.3)
AK	<b>No</b>	Structure located in the State of Alaska?
Bldg	<b>Enclosed</b>	Enclosure Category (ASCE 7-10 Section 26.2)
h	<b>106</b>	Mean Roof Height, ft
K <sub>d</sub>	<b>0.85</b>	Wind Directionality Factor (ASCE 7-10 Table 26.6-1)

#### B. Directional Procedure

	<b>N-S</b>	Direction of Wind Loading
L	<b>102.3</b>	Horizontal Dimension of Building Parallel to Wind Pressure, ft
B	<b>152.3</b>	Horizontal Dimension of Building Normal to Wind Pressure, ft
n	<b>0.93</b>	(1/T) Natural Frequency of Structure in Direction of Wind Loading, Hz
	<b>Flexible</b>	Structure Rigidity (ASCE 7-10 Section 26.9.2)
K <sub>zt</sub>	<b>1.0</b>	Topographic Factor (ASCE 7-10 Section 26.8.2)

#### A. Rigid Structures (ASCE 7-10 Section 26.9.4)

c	<b>0.2</b>	(Table 26.9-1)
z <sub>min</sub>	<b>15</b>	ft (Table 26.9-1)
z <sub>bar</sub>	<b>63.6</b>	ft (Section 26.9.4)
l <sub>z</sub>	<b>0.18</b>	(US Units)
l	<b>500</b>	(Table 26.9-1)
ε <sub>bar</sub>	<b>0.20</b>	(Table 26.9-1)
L <sub>zbar</sub>	<b>570.1</b>	(US Units)
Q	<b>0.85</b>	(Equation 26.9-8)
G	<b>0.85</b>	(Equation 26.9-6)

#### B. Flexible Structures (ASCE 7-10 Section 26.9.5)

n <sub>1</sub>	<b>0.93</b>	Fundamental Frequency, Hz
V <sub>zbar</sub>	<b>116.00</b>	Mean Hourly Wind Speed, ft/s
N <sub>1</sub>	<b>4.58</b>	(Equation 26.9-14)
R <sub>h</sub>	<b>0.22</b>	(Equation 26.9-15a)
R <sub>B</sub>	<b>0.16</b>	(Equation 26.9-15a)
R <sub>L</sub>	<b>0.08</b>	(Equation 26.9-15a)
R <sub>n</sub>	<b>0.05</b>	(Equation 26.9-13)
β	<b>0.05</b>	Damping Ratio
R	<b>0.15</b>	(Equation 26.9-12)
g <sub>R</sub>	<b>4.17</b>	(Equation 26.9-11)
G <sub>f</sub>	<b>0.86</b>	(Equation 26.9-10)

Level	Height (ft)	Lower TBL (ft)	Row #	K <sub>z</sub> (B)	K <sub>z</sub> (C)	K <sub>z</sub> (D)	<b>K<sub>z</sub></b>	q <sub>z</sub> (PSF)
Level 08	106	100	11	0.996	1.275	1.445	<b>1.275</b>	33.6
Level 07	93	90	10	0.969	1.246	1.409	<b>1.246</b>	32.8
Level 06	80	80	9	0.930	1.210	1.380	<b>1.210</b>	31.9
Level 05	67	60	7	0.878	1.158	1.331	<b>1.158</b>	30.5
Level 04	54	50	6	0.826	1.106	1.286	<b>1.106</b>	29.1



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Level 03	41	40	5	0.765	1.045	1.225	<b>1.045</b>	27.5
Level 02	28	25	3	0.684	0.964	1.144	<b>0.964</b>	25.4
Level 01	15	15	1	0.570	0.850	1.030	<b>0.850</b>	22.4
Mean Ht	106	100	11	0.996	1.275	1.445	<b>1.275</b>	33.6 <- q <sub>h</sub>

\*Note: K<sub>z</sub> and K<sub>h</sub> calculated per ASCE 7-10 Table 27.3-1

C <sub>p,windward</sub>	<b>0.8</b>	Windward Pressure Coefficient (ASCE 7-10 Table 27.4-1)
C <sub>p,leeward</sub>	<b>-0.5</b>	Leeward Pressure Coefficient (ASCE 7-10 Table 27.4-1)
C <sub>p,side</sub>	<b>-0.7</b>	Side Pressure Coefficient (ASCE 7-10 Table 27.4-1)
G	<b>0.86</b>	Gust Factor (Calculated Above)

Level	P <sub>windward</sub> (PSF)	P <sub>leeward</sub> (PSF)	P <sub>side</sub> (PSF)
Level 08	29.2	-20.5	-26.3
Level 07	28.7	-20.5	-26.3
Level 06	28.1	-20.5	-26.3
Level 05	27.1	-20.5	-26.3
Level 04	26.2	-20.5	-26.3
Level 03	25.1	-20.5	-26.3
Level 02	23.6	-20.5	-26.3
Level 01	21.5	-20.5	-26.3

	<b>E-W</b>	Direction of Wind Loading
L	<b>152.3</b>	Horizontal Dimension of Building Parallel to Wind Pressure, ft
B	<b>102.3</b>	Horizontal Dimension of Building Normal to Wind Pressure, ft
n	<b>0.54</b>	(1/T) Natural Frequency of Structure in Direction of Wind Loading, Hz
	<b>Flexible</b>	Structure Rigidity (ASCE 7-10 Section 26.9.2)
K <sub>zt</sub>	<b>1.0</b>	Topographic Factor (ASCE 7-10 Section 26.8.2)

A. Rigid Structures (ASCE 7-10 Section 26.9.4)

c	<b>0.2</b>	(Table 26.9-1)
z <sub>min</sub>	<b>15</b>	ft (Table 26.9-1)
z <sub>bar</sub>	<b>63.6</b>	ft (Section 26.9.4)
l <sub>z</sub>	<b>0.18</b>	(US Units)
l	<b>500</b>	(Table 26.9-1)
ε <sub>bar</sub>	<b>0.20</b>	(Table 26.9-1)
L <sub>zbar</sub>	<b>570.1</b>	(US Units)
Q	<b>0.87</b>	(Equation 26.9-8)
G	<b>0.86</b>	(Equation 26.9-6)

B. Flexible Structures (ASCE 7-10 Section 26.9.5)

n <sub>1</sub>	<b>0.54</b>	Fundamental Frequency, Hz
V <sub>zbar</sub>	<b>116.00</b>	Mean Hourly Wind Speed, ft/s
N <sub>1</sub>	<b>2.63</b>	(Equation 26.9-14)
R <sub>h</sub>	<b>0.35</b>	(Equation 26.9-15a)
R <sub>B</sub>	<b>0.36</b>	(Equation 26.9-15a)
R <sub>L</sub>	<b>0.13</b>	(Equation 26.9-15a)
R <sub>n</sub>	<b>0.08</b>	(Equation 26.9-13)
β	<b>0.05</b>	Damping Ratio
R	<b>0.33</b>	(Equation 26.9-12)
g <sub>R</sub>	<b>4.04</b>	(Equation 26.9-11)
G <sub>f</sub>	<b>0.90</b>	(Equation 26.9-10)



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Level	Height (ft)	Lower TBL (ft)	Row #	$K_z$ (B)	$K_z$ (C)	$K_z$ (D)	<b><math>K_z</math></b>	$q_z$ (PSF)
Level 08	106	100	11	0.996	1.275	1.445	<b>1.275</b>	33.6
Level 07	93	90	10	0.969	1.246	1.409	<b>1.246</b>	32.8
Level 06	80	80	9	0.930	1.210	1.380	<b>1.210</b>	31.9
Level 05	67	60	7	0.878	1.158	1.331	<b>1.158</b>	30.5
Level 04	54	50	6	0.826	1.106	1.286	<b>1.106</b>	29.1
Level 03	41	40	5	0.765	1.045	1.225	<b>1.045</b>	27.5
Level 02	28	25	3	0.684	0.964	1.144	<b>0.964</b>	25.4
Level 01	15	15	1	0.570	0.850	1.030	<b>0.850</b>	22.4
Mean Ht	106	100	11	0.996	1.275	1.445	<b>1.275</b>	33.6 <- $q_h$

\*Note:  $K_z$  and  $K_h$  calculated per ASCE 7-10 Table 27.3-1

$C_{p,windward}$	<b>0.8</b>	Windward Pressure Coefficient (ASCE 7-10 Table 27.4-1)
$C_{p,leeward}$	<b>-0.40</b>	Leeward Pressure Coefficient (ASCE 7-10 Table 27.4-1)
$C_{p,side}$	<b>-0.7</b>	Side Pressure Coefficient (ASCE 7-10 Table 27.4-1)
G	<b>0.90</b>	Gust Factor (Calculated Above)

Level	$P_{windward}$ (PSF)	$P_{leeward}$ (PSF)	$P_{side}$ (PSF)
Level 08	30.3	-18.2	-27.2
Level 07	29.7	-18.2	-27.2
Level 06	29.0	-18.2	-27.2
Level 05	28.0	-18.2	-27.2
Level 04	27.1	-18.2	-27.2
Level 03	25.9	-18.2	-27.2
Level 02	24.4	-18.2	-27.2
Level 01	22.2	-18.2	-27.2

C. Loading (ASCE 7-10 Section 27.4.6)

Level	Vert Trib (ft)	E-W Force <sup>1</sup> (k)	N-S Force <sup>2</sup> (k)	$M_{T,E-W}$ (k-ft)	$M_{T,N-S}$ (k-ft)
Level 08	6.5	32.3	49.3	371.4	844.6
Level 07	13	63.8	97.5	734.3	1671.3
Level 06	13	62.9	96.2	723.8	1649.1
Level 05	13	61.6	94.4	708.7	1617.0
Level 04	13	60.2	92.5	693.5	1584.9
Level 03	13	58.7	90.3	675.8	1547.2
Level 02	13	56.7	87.4	652.2	1497.2
Level 01	14	57.9	89.7	666.6	1536.6

1. Force due to wind applied in the E-W Direction
2. Force due to wind applied in the N-S Direction





### D. Comparison to Seismic Loads

Level	East-West			North-South		
	V <sub>seis</sub> (k)	V <sub>wind</sub> (k)	Control	V <sub>seis</sub> (k)	V <sub>wind</sub> (k)	Control
Level 08	245.2	32.3	<b>Seismic</b>	505.3	49.3	<b>Seismic</b>
Level 07	458.8	96.0	<b>Seismic</b>	957.9	146.8	<b>Seismic</b>
Level 06	629.5	158.9	<b>Seismic</b>	1330.9	243.0	<b>Seismic</b>
Level 05	760.4	220.5	<b>Seismic</b>	1627.8	337.4	<b>Seismic</b>
Level 04	855.2	280.7	<b>Seismic</b>	1852.7	429.9	<b>Seismic</b>
Level 03	918.0	339.4	<b>Seismic</b>	2010.6	520.2	<b>Seismic</b>
Level 02	953.5	396.1	<b>Seismic</b>	2107.3	607.5	<b>Seismic</b>
Level 01	967.8	454.0	<b>Seismic</b>	2151.4	697.2	<b>Seismic</b>
<b>Total</b>	<b>967.8</b>	<b>454.0</b>		<b>2151.4</b>	<b>697.2</b>	











Column Shear Stress Level ( $\sqrt{f'_c} \cdot b \cdot d$ )	Weak Story Check			Relative Str. (%)
	$\Phi V_n$ (kips)	Story	Strength (kips)	
4.42	306.0	Level 08	2320.2	(-)
5.28	306.0	Level 07	2320.2	100%
5.28	306.0	Level 06	2320.2	100%
5.28	306.0	Level 05	2320.2	100%
5.28	306.0	Level 04	2020.4	87%
5.28	306.0	Level 03	2020.4	100%
5.28	306.0	Level 02	2020.4	100%
5.28	306.0	Level 01	2020.4	100%
5.31	427.1			
6.33	427.1			
6.33	427.1			
6.33	427.1			
6.33	352.1			
6.33	352.1			
6.33	352.1			
6.33	352.1			
5.31	427.1			
6.33	427.1			
6.33	427.1			
6.33	427.1			
6.33	352.1			
6.33	352.1			
6.33	352.1			
6.33	352.1			
6.33	352.1			
5.31	427.1			
6.33	427.1			
6.33	427.1			
6.33	427.1			
6.33	352.1			
6.33	352.1			
6.33	352.1			
6.33	352.1			
6.33	352.1			
4.42	306.0			
5.28	306.0			
5.28	306.0			
5.28	306.0			
5.28	306.0			
5.28	306.0			
5.28	306.0			



Section 8. Frame Design - Joints

V\_col = ((M\_pr,L + M\_pr,R) + (V\_e,L + V\_e,R) \* h) / l\_c

Typical Beam Properties

- 5000 Concrete Compressive Strength, psi
60 Reinforcing Steel Yield Strength, ksi
28 Column width, in
1.5 Clear cover, in

Table with columns: JOINT ID, l\_c, BEAM ID, b\_w, x, I\_n,L, M\_br+, M\_br-, BEAM ID, b\_w, x, I\_n,R, M\_br+, M\_br-, V\_e,L, V\_e,R, V\_col, A\_s,pos, A\_s,neg, T = C\_pos, T = C\_neg, V\_j, A\_j, Confine, Phi\_vn, Joint Shear Stress Ratio (sqrt(f\_c)\*A\_j)





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## Section 9. Concrete Shear Wall Design (Ext Wall)

### I. Shear Design

$h_w$	<b>106</b>	Height of shear wall, ft								
$\Phi$	<b>0.60</b>	Strength reduction factor for shear in walls								
$f'_c$	<b>5000</b>	Compressive strength of concrete, psi								
$f_y$	<b>60</b>	Yield stress of reinforcing steel, ksi								
Level	$f'_c$ (psi)	Shear, $V_u$ (kips)	Wall Thickness (in)	$l_w^1$ (ft)	$\alpha_c$	Horiz (tr) bar size	Vert (Ing) bar size	Curtains <sup>2</sup>	$s_h$ (in)	$s_v$ (in)
Level 8	5000	154.6	16	22.33	2	#5	#5	2	18.0	18.0
Level 7	5000	303.8	16	22.33	2	#5	#5	2	15.0	15.0
Level 6	5000	398.7	16	22.33	2	#5	#5	2	15.0	15.0
Level 5	5000	473.6	16	22.33	2	#5	#5	2	15.0	15.0
Level 4	5000	545.7	16	22.33	2	#5	#5	2	15.0	15.0
Level 3	5000	625.9	16	22.33	2	#5	#5	2	15.0	15.0
Level 2	5000	723.2	16	22.33	2	#5	#5	2	15.0	15.0
Level 1	5000	915.4	16	22.33	2	#5	#5	2	10.0	10.0
Level B	5000	2603.7	16	102.33	3	#5	#5	2	10.0	10.0

Level	$A_{cv}$ (in <sup>2</sup> )	$A_{cv} * f'_c^{1.0.5}$	$\rho_{t,min}$	$\rho_{l,min}$	$\rho_{t,req}$	$\rho_t$	$\rho_l$	$\Phi V_n$ (kips)
Level 8	4287	303.2	0.002	<b>0.0012</b>	<b>0.002</b>	<b>0.00215</b>	<b>0.00215</b>	870.1
Level 7	4287	303.2	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00258</b>	<b>0.00258</b>	953.1
Level 6	4287	303.2	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00258</b>	<b>0.00258</b>	953.1
Level 5	4287	303.2	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00258</b>	<b>0.00258</b>	953.1
Level 4	4287	303.2	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00258</b>	<b>0.00258</b>	953.1
Level 3	4287	303.2	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00258</b>	<b>0.00258</b>	953.1
Level 2	4287	303.2	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00258</b>	<b>0.00258</b>	953.1
Level 1	4287	303.2	0.0025	<b>0.0025</b>	<b>0.0036</b>	<b>0.00388</b>	<b>0.00388</b>	1202.4
Level B	19647	1389.3	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00388</b>	<b>0.00388</b>	8735.8

1.  $l_w$  should be the length of the wall measured to the outer edge of the boundary element

2. Minimum number of curtains per ACI 318-08 Section 21.9.2.2

\*Note: Design table is specific to a given wall and loading demands

\*Note: "Level" indicates the supported level

\*Note: If concrete compressive strength is defined at a wall other than defined above, ensure compliance with ACI318-08 Section 21.2.4

### II. Flexural/Axial Design (PCA Column)

\*Note: Each uniquely reinforced wall should be analyzed in PCA Column, and the corresponding wall levels should be checked against the corresponding interaction diagram

**28** Boundary column dimension, in

**1.5** Clear cover to extreme boundary element steel, in

Level	$P_{u,max}$ (k)	$P_{u,min}$ (k)	$M_{u,x}$ (k-in)	$M_{u,y}$ (k-in)	Col Reinf	Col Reinf	shear rein	shear rein	
Level 8	204	99	1985	0	8	#8	4	#5	<b>3b</b>



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Level 7	424	202	5919	0	8	#8	4	#5	3b
Level 6	645	304	11037	0	8	#8	4	#5	3b
Level 5	865	407	16986	0	8	#8	4	#5	3b
Level 4	1085	510	23643	0	16	#8	4	#5	5a
Level 3	1305	612	31126	0	16	#8	4	#5	5a
Level 2	1525	715	39804	0	16	#10	4	#5	5a
Level 1	1762	825	52897	0	16,9	#10,#9	4	#5	5a
Level B	4308	1881	72957	0	16,9	#10,#9	4	#5	5a

\*Note: "Level" indicates the supported level

\*Note:  $P_{u,max}$  and  $P_{u,min}$  are calculated from governing load combinations

\*Note: "x" is taken as the major bending axis (in-plane) for the shear wall

\*Note: "c" is depth to neutral axis for given loading from PCA Column

### III. Boundary Elements

Level	$I_x^1$ (in <sup>4</sup> )	$S_x$ (in <sup>3</sup> )	$A_z^2$ (in <sup>2</sup> )	Maximum Stress (ksi)	$(*f_c')$	Boundary Element Required <sup>3</sup> ?	$C_{pmax}$ (in)	$C_{pmin}$	$L_B^{5,6}$
Level 8	35385800	264113	4959.36	0.049	0.01	NO	92.76	41.68	NA
Level 7	35385800	264113	4959.36	0.108	0.02	NO	26.24	22.1	NA
Level 6	35385800	264113	4959.36	0.172	0.03	NO	28.19	23.03	NA
Level 5	35385800	264113	4959.36	0.239	0.05	NO	30.21	23.92	NA
Level 4	35385800	264113	4959.36	0.308	0.06	NO	32.47	24.82	NA
Level 3	35385800	264113	4959.36	0.381	0.08	NO	34.8	25.81	NA
Level 2	35385800	264113	4959.36	0.458	0.09	NO	75.93	35.94	NA
Level 1	35385800	264113	4959.36	0.556	0.11	NO	23.92	22.98	NA
Level B	35385800	57633.5	20319.36	1.478	0.30	YES	25.87	23.85	12.94

1. Moment of inertia, taken from PCA Column output

2. Area of wall perpendicular to axial loading

3. Boundary elements may be discontinued above the highest level indicated "Cutoff"

4. Controlling depth to neutral axis from PCA Column data

5. Minimum length of boundary element (ACI 318-08 Section 21.9.6.4a)

6. Adjustments will need to be made to ensure the boundary elements can fit within the column

\*Note: "Level" indicates the supported level

#### Boundary Element Trans. Reinforcement Requirements (ACI 318-08 Section 21.9.6.4)

$f_c'$	<b>5000</b>	Compressive strength of column, psi
	<b>#10</b>	Size of column reinf used in PCA Column
	<b>16</b>	Number of longitudinal bars used
	<b>5a</b>	Hoop Style (Ref attached sheet for diagrams)
	<b>#5</b>	Size of transverse reinforcing
	<b>4.4</b>	Clear spacing between longitudinal bars
	<b>16</b>	Total available longitudinal bars (must equal value above)
	<b>13.1</b>	Maximum crosstie/hoop spacing, in ( $\leq 14$ " per ACI 318-08 Section 21.6.4.2)
	<b>OK</b>	Compliance with ACI 318-08 Section 7.10.5.3



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$s_{\text{spc-req}}$	<b>4.29</b>	Maximum spacing of transverse reinforcement within hinge zone (ACI 318-08 Section 21.6.4.3), in
$b_c$	<b>25.0</b>	Maximum core dimension measured to outside of transverse reinf, in
$A_{\text{ch}}$	<b>625.0</b>	Cross section area measured to outside of transverse reinf, in <sup>2</sup>
$A_{\text{tr}}$	<b>0.93</b>	Actual area of steel of transverse reinf for given layout, in <sup>2</sup>
$s_{\text{area-req}}$	<b>4.96</b>	Maximum spacing of transverse reinforcement within hinge zone (ACI 318-08 Section 21.6.4.4)
$s_{\text{hinge}}$	<b>4.0</b>	Hinge Zone Spacing (smaller of $s_{\text{spc-req}}$ and $s_{\text{area-req}}$ , rounded down to nearest .5"), in

*\*Note: Ensure that any columns designed using the Frame Column tab are checked against these tighter spacing requirements*



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## Section 9a. Concrete Shear Wall Design (Int Wall)

### I. Shear Design

$h_w$	<b>106</b>	Height of shear wall, ft
$\Phi$	<b>0.60</b>	Strength reduction factor for shear in walls
$f'_c$	<b>5000</b>	Compressive strength of concrete, psi
$f_y$	<b>60</b>	Yield stress of reinforcing steel, ksi

Level	$f'_c$ (psi)	Shear, $V_u$ (kips)	Wall Thickness (in)	$l_w^1$ (ft)	$\alpha_c$	Horiz (tr) bar size	(Ing) bar	$s^2$	$s_h$ (in)	$s_v$ (in)
Level 8	5000	153	16	22.33	2	#5	#5	2	18.0	18.0
Level 7	5000	272	16	22.33	2	#5	#5	2	18.0	18.0
Level 6	5000	349	16	22.33	2	#5	#5	2	15.0	15.0
Level 5	5000	404	16	22.33	2	#5	#5	2	15.0	15.0
Level 4	5000	453	16	22.33	2	#5	#5	2	15.0	15.0
Level 3	5000	493	16	22.33	2	#5	#5	2	15.0	15.0
Level 2	5000	499	16	22.33	2	#5	#5	2	15.0	15.0
Level 1	5000	377	16	22.33	2	#5	#5	2	15.0	15.0
Level B	5000	1503	16	102.33	3	#5	#5	2	15.0	15.0

Level	$A_{cv}$ (in <sup>2</sup> )	$A_{cv} * f'_c^{0.5}$	$\rho_{t,min}$	$\rho_{l,min}$	$\rho_{t,req}$	$\rho_t$	$\rho_l$	$\Phi V_n$ (kips)
Level 8	4287	303.2	0.002	<b>0.0012</b>	<b>0.002</b>	<b>0.00215</b>	<b>0.00215</b>	870.1
Level 7	4287	303.2	0.002	<b>0.0012</b>	<b>0.002</b>	<b>0.00215</b>	<b>0.00215</b>	870.1
Level 6	4287	303.2	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00258</b>	<b>0.00258</b>	953.1
Level 5	4287	303.2	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00258</b>	<b>0.00258</b>	953.1
Level 4	4287	303.2	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00258</b>	<b>0.00258</b>	953.1
Level 3	4287	303.2	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00258</b>	<b>0.00258</b>	953.1
Level 2	4287	303.2	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00258</b>	<b>0.00258</b>	953.1
Level 1	4287	303.2	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00258</b>	<b>0.00258</b>	953.1
Level B	19647	1389.3	0.0025	<b>0.0025</b>	<b>0.0025</b>	<b>0.00258</b>	<b>0.00258</b>	7213.2

1.  $l_w$  should be the length of the wall measured to the outer edge of the boundary element

2. Minimum number of curtains per ACI 318-08 Section 21.9.2.2

\*Note: Design table is specific to a given wall and loading demands

\*Note: "Level" indicates the supported level

\*Note: If concrete compressive strength is defined at a wall other than defined above, ensure compliance with ACI318-08 Section 21.2.4

### II. Flexural/Axial Design (PCA Column)

\*Note: Each uniquely reinforced wall should be analyzed in PCA Column, and the corresponding wall levels should be checked against the corresponding interaction diagram

**28** Boundary column dimension, in

**1.5** Clear cover to extreme boundary element steel, in

Level	$P_{u,max}$ (k)	$P_{u,min}$ (k)	$M_{u,x}$ (k-in)	$M_{u,y}$ (k-in)	Col Reinf	Col Reinf	shear rein spc	shear rein size	shear rein type
Level 8	293	141	1964	0	12	#8	4	#4	<b>4c</b>



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Level 7	613	286	5483	0	12	#8	4	#4	4c
Level 6	933	430	9945	0	12	#8	4	#4	4c
Level 5	1253	575	14987	0	12	#8	4	#4	4c
Level 4	1572	720	20439	0	12	#8	4	#4	4c
Level 3	1892	865	26194	0	12	#8	4	#4	4c
Level 2	2212	1010	31893	0	12	#8	4	#4	4c
Level 1	2547	1163	36452	0	12	#10	4	#4	4c
Level B	2839	1280	13910	0	12	#10	4	#4	4c

\*Note: "Level" indicates the supported level

\*Note:  $P_{u,max}$  and  $P_{u,min}$  are calculated from governing load combinations

\*Note: "x" is taken as the major bending axis (in-plane) for the shear wall

\*Note: "c" is depth to neutral axis for given loading from PCA Column

### III. Boundary Elements

Level	$I_x^1$ (in <sup>4</sup> )	$S_x$ (in <sup>3</sup> )	$A_z^2$ (in <sup>2</sup> )	Maximum Stress (ksi)	Maximum Stress (* $f_c'$ )	Boundary Element Required <sup>3</sup> ?	$C_{pmax}$ (in)	$C_{pmin}$ (in)	$L_B^{5,6}$
Level 8	35385800	264113	4959.4	0.067	0.01	NO	92.76	41.68	NA
Level 7	35385800	264113	4959.4	0.144	0.03	NO	26.24	22.1	NA
Level 6	35385800	264113	4959.4	0.226	0.05	NO	28.19	23.03	NA
Level 5	35385800	264113	4959.4	0.309	0.06	NO	30.21	23.92	NA
Level 4	35385800	264113	4959.4	0.394	0.08	NO	32.47	24.82	NA
Level 3	35385800	264113	4959.4	0.481	0.10	NO	34.8	25.81	NA
Level 2	35385800	264113	4959.4	0.567	0.11	NO	75.93	35.94	NA
Level 1	35385800	264113	4959.4	0.652	0.13	NO	23.92	22.98	NA
Level B	35385800	57633.5	20319	0.381	0.08	NO	25.87	23.85	NA

1. Moment of inertia, taken from PCA Column output

2. Area of wall perpendicular to axial loading

3. Boundary elements may be discontinued above the highest level indicated "Cutoff"

4. Controlling depth to neutral axis from PCA Column data

5. Minimum length of boundary element (ACI 318-08 Section 21.9.6.4a)

6. Adjustments will need to be made to ensure the boundary elements can fit within the column

\*Note: "Level" indicates the supported level

#### Boundary Element Trans. Reinforcement Requirements (ACI 318-08 Section 21.9.6.4)

$f_c'$	<b>5000</b>	Compressive strength of column, psi
	<b>#10</b>	Size of column reinf used in PCA Column
	<b>12</b>	Number of longitudinal bars used
	<b>4c</b>	Hoop Style (Ref attached sheet for diagrams)
	<b>#5</b>	Size of transverse reinforcing
	<b>6.2</b>	Clear spacing between longitudinal bars
	<b>12</b>	Total available longitudinal bars (must equal value above)
	<b>9.4</b>	Maximum crosstie/hoop spacing, in ( $\leq 14$ " per ACI 318-08 Section 21.6.4.2)
	<b>OK</b>	Compliance with ACI 318-08 Section 7.10.5.3



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$s_{spc-req}$	<b>5.54</b>	Maximum spacing of transverse reinforcement within hinge zone (ACI 318-08 Section 21.6.4.3), in
$b_c$	<b>25.0</b>	Maximum core dimension measured to outside of transverse reinf, in
$A_{ch}$	<b>625.0</b>	Cross section area measured to outside of transverse reinf, in <sup>2</sup>
$A_{tr}$	<b>1.2</b>	Actual area of steel of transverse reinf for given layout, in <sup>2</sup>
$s_{area-req}$	<b>6.61</b>	Maximum spacing of transverse reinforcement within hinge zone (ACI 318-08 Section 21.6.4.4)
$s_{hinge}$	<b>5.0</b>	Hinge Zone Spacing (smaller of $s_{spc-req}$ and $s_{area-req}$ , rounded down to nearest .5"), in

*\*Note: Ensure that any columns designed using the Frame Column tab are checked against these tighter spacing requirements*



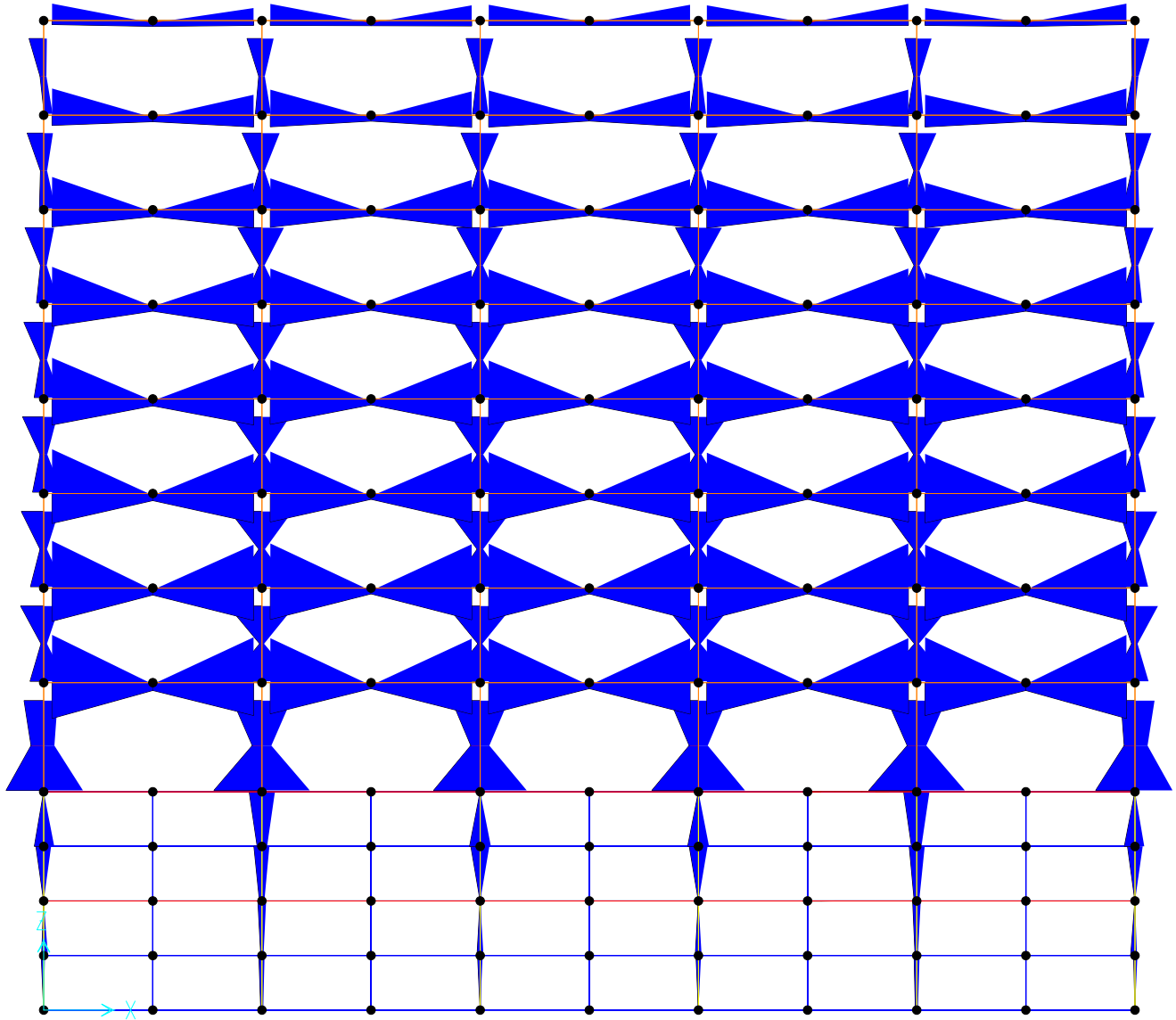
National Earthquake Hazards Reduction Program

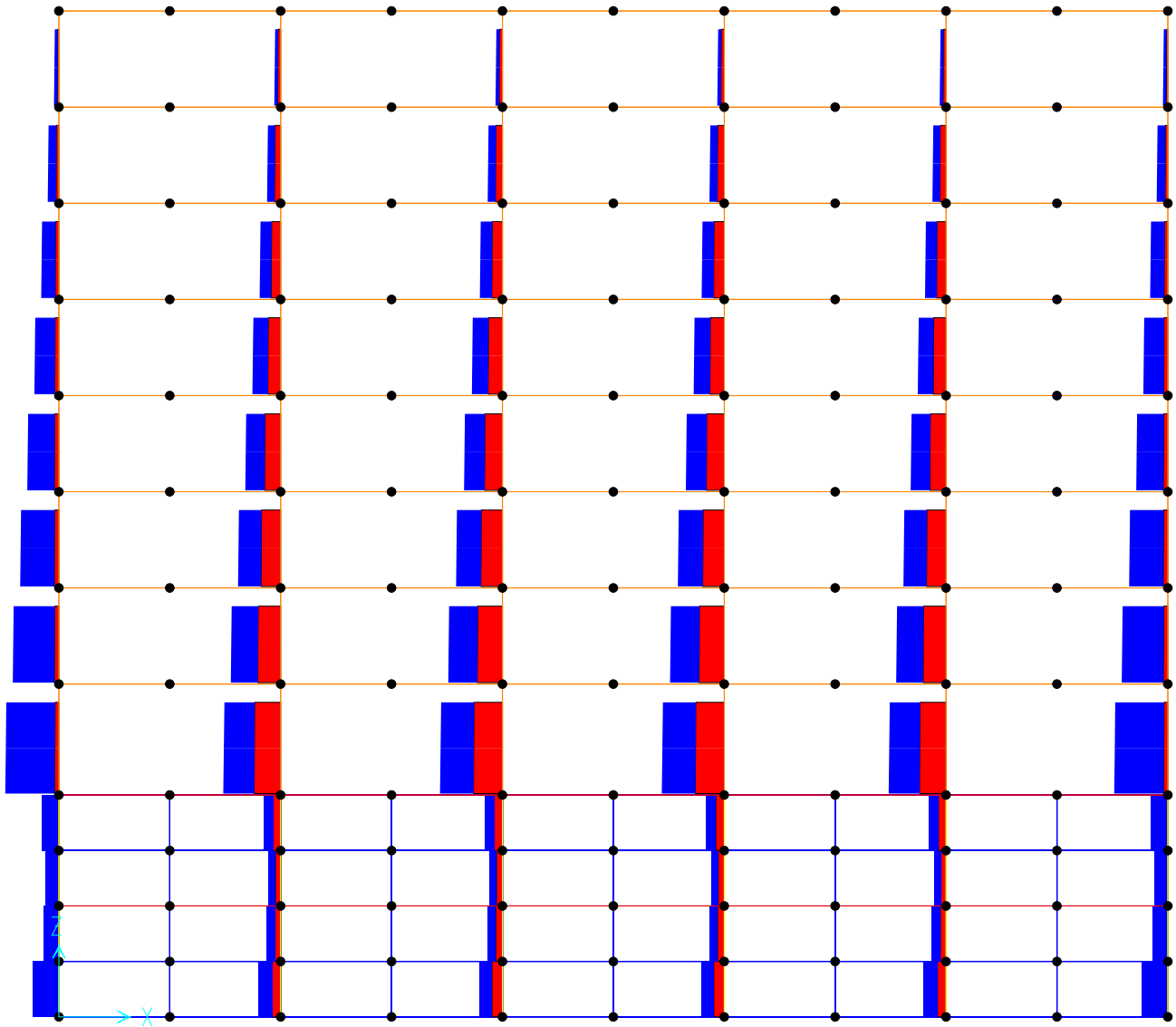
Weak Story Check

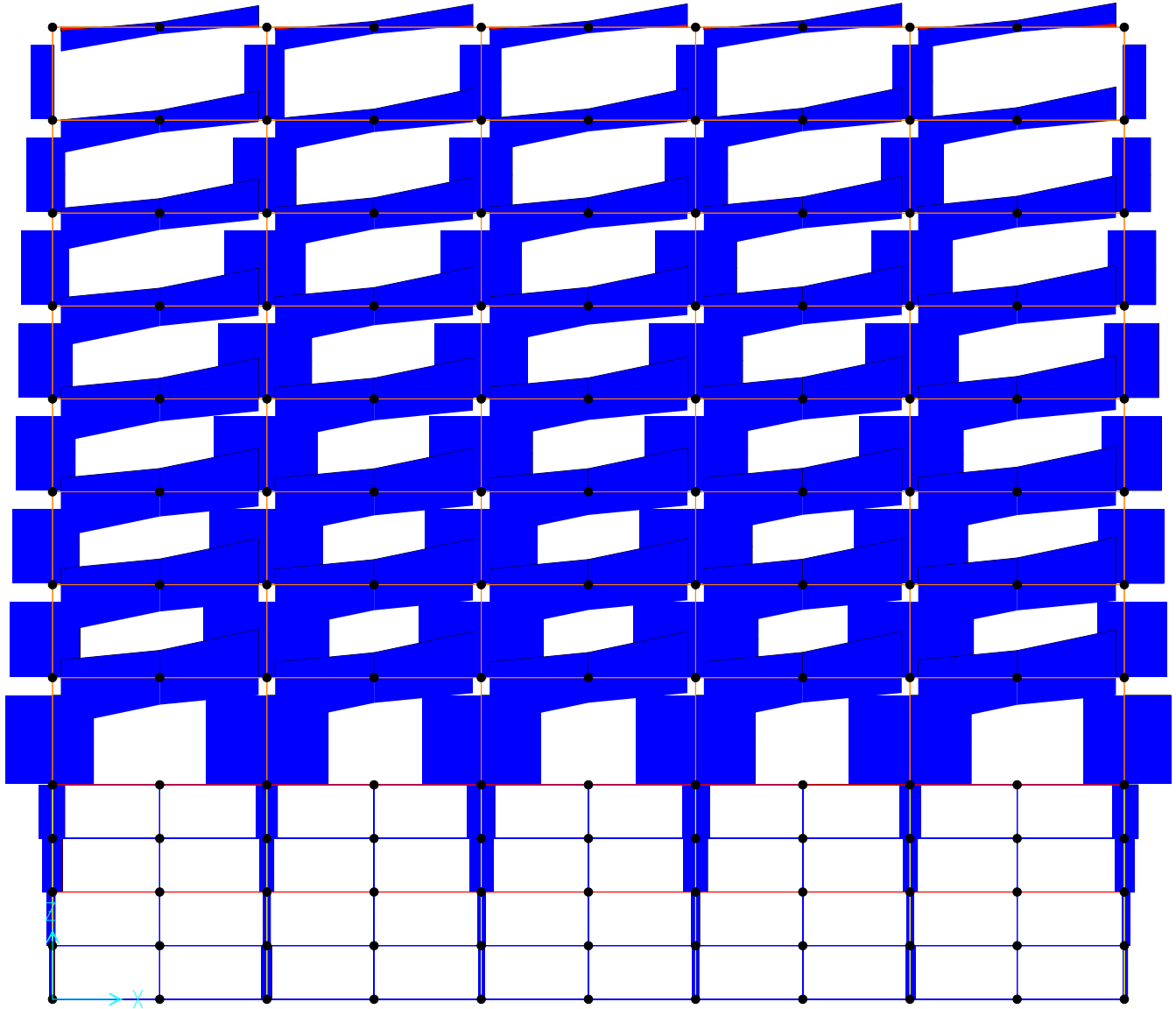
Level	$\Phi V_n$ (kips)	Relative Str. (%)
Level 08	3480.3	(-)
Level 07	3646.5	105%
Level 06	3812.6	105%
Level 05	3812.6	100%
Level 04	3812.6	100%
Level 03	3812.6	100%
Level 02	3812.6	100%
Level 01	4311.0	113%

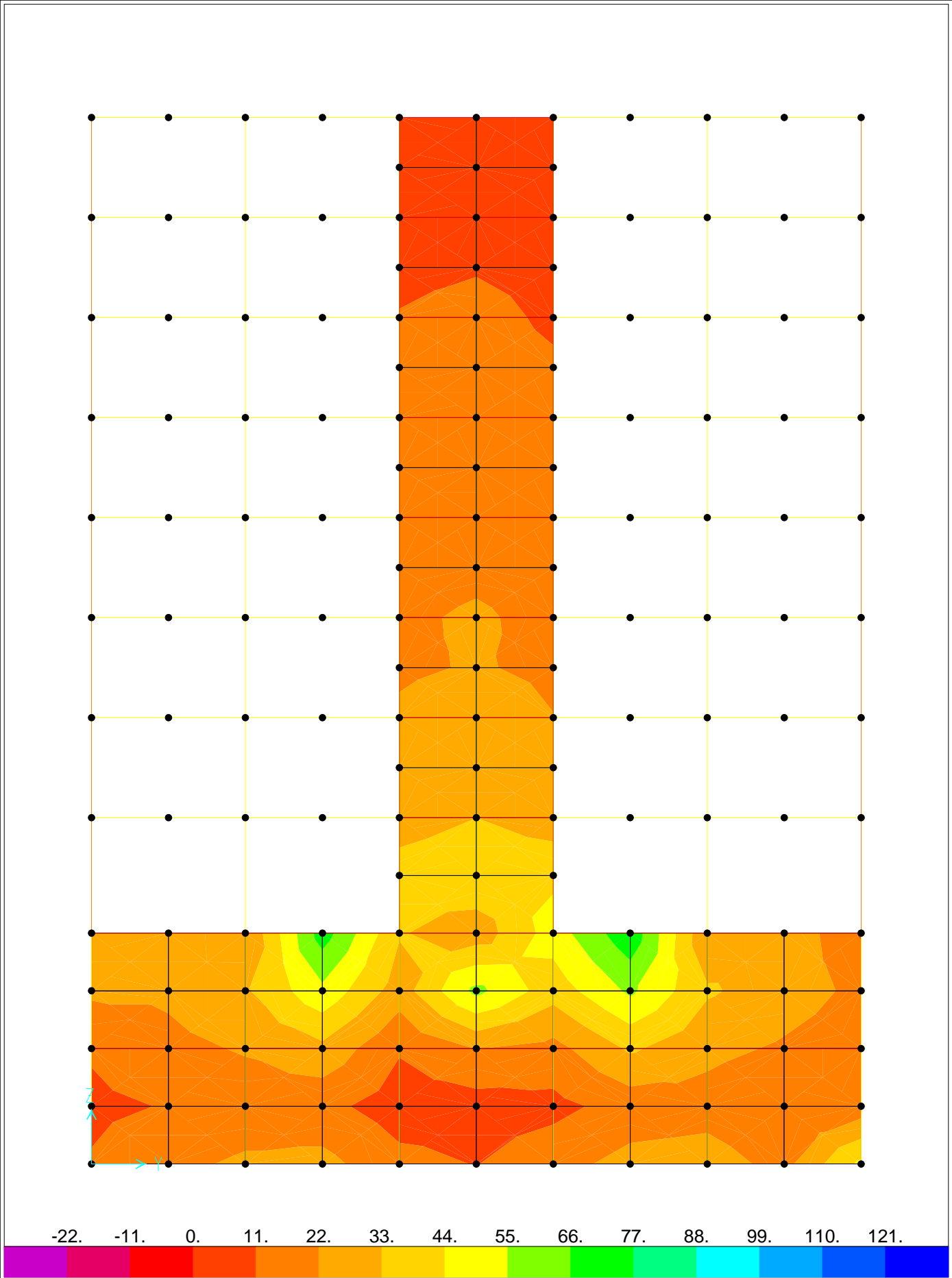
## **Appendix B – Design Moment, Shear, and Axial Load Diagrams**



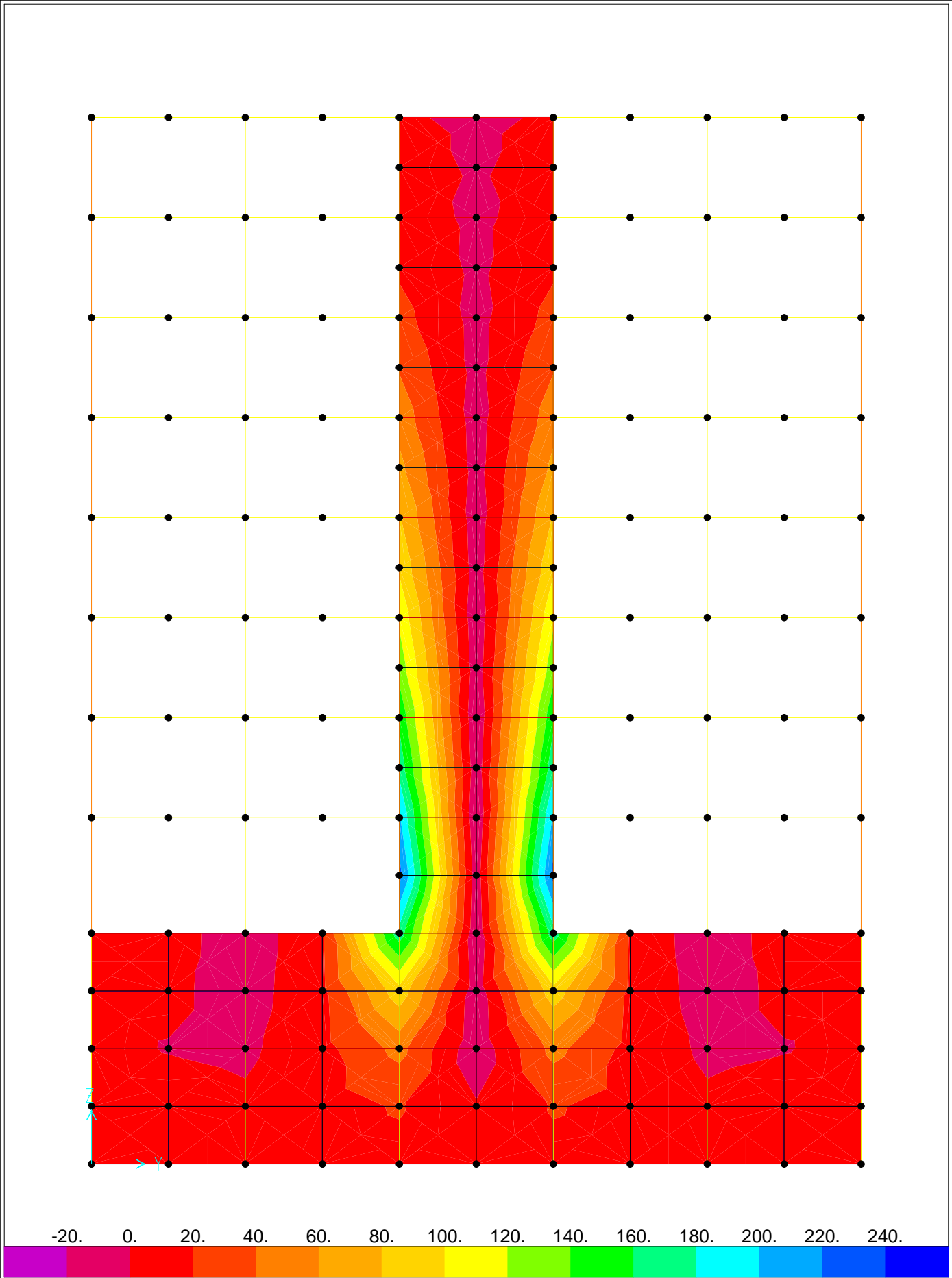






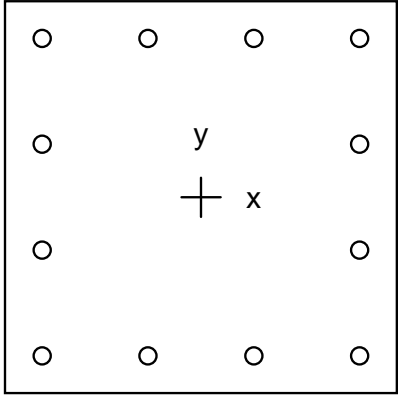


SAP2000 v14.2.0 - File:StructModel03 - Stress S12 Diagram - Visible Face (NS RESPONSE - Max) - Kip, ft, F Units



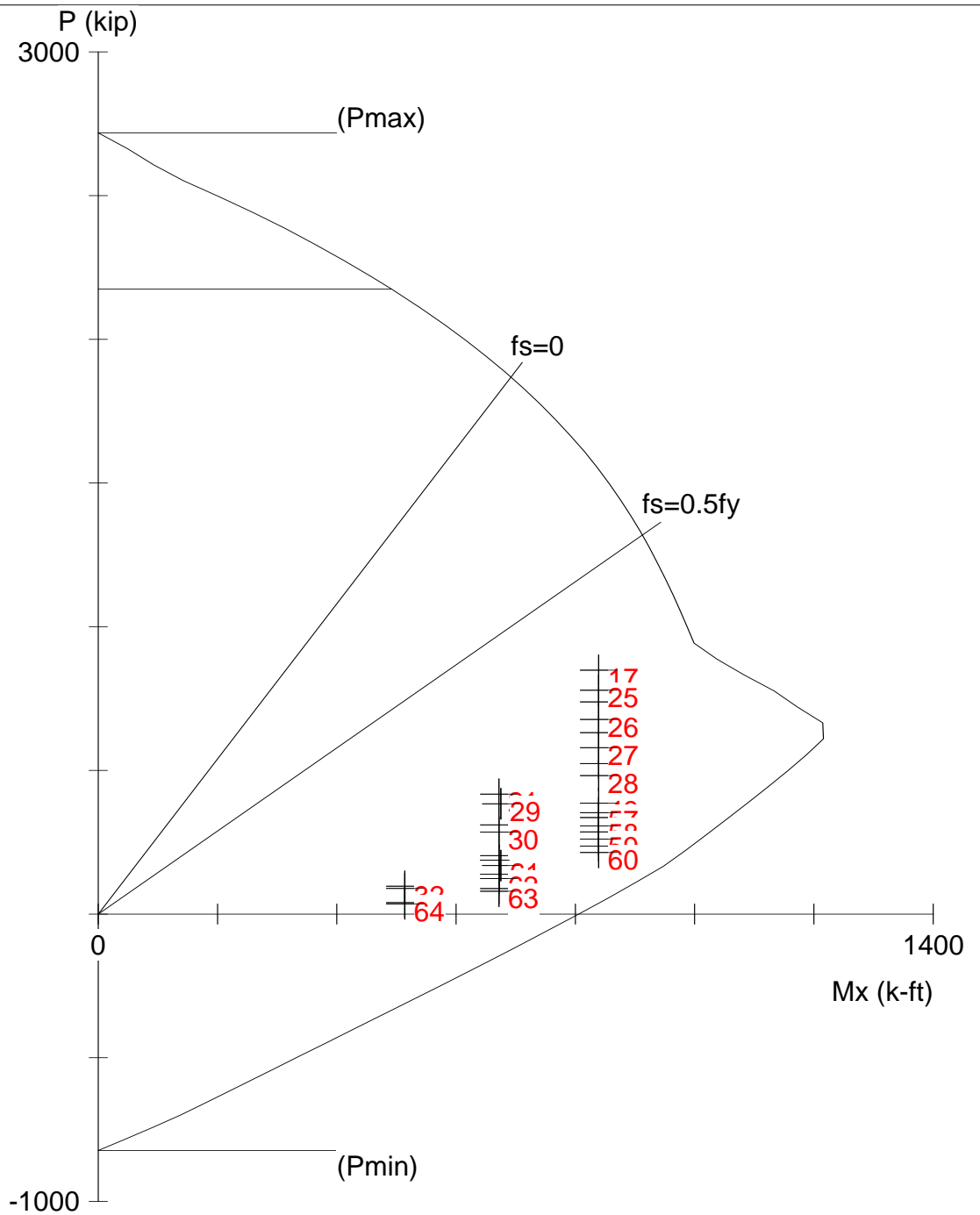
SAP2000 v14.2.0 - File:StructModel03 - Stress S22 Diagram - Visible Face (NS RESPONSE - Max) - Kip, ft, F Units

## **Appendix C – Design Interaction Diagrams**



28 x 28 in

Code: ACI 318-08  
 Units: English  
 Run axis: About X-axis  
 Run option: Investigation  
 Slenderness: Not considered  
 Column type: Structural  
 Bars: ASTM A615  
 Date: 08/09/10  
 Time: 13:33:24



spColumn v4.50. Licensed to: National Institute of Standards and Technology . License ID: 56132-1019166-4-2473A-20EE4

File: Z:\Documents\Report Files 6-17-10\PCA Columns\12number10col.col

Project:

Column: (12) #10

Engineer: TSW

$f'_c = 5$  ksi

$f_y = 60$  ksi

$A_g = 784$  in<sup>2</sup>

12 #10 bars

$E_c = 4031$  ksi

$E_s = 29000$  ksi

$A_s = 15.24$  in<sup>2</sup>

$\rho = 1.94\%$

$f_c = 4.25$  ksi

$X_o = 0.00$  in

$I_x = 51221.3$  in<sup>4</sup>

$e_u = 0.003$  in/in

$Y_o = 0.00$  in

$I_y = 51221.3$  in<sup>4</sup>

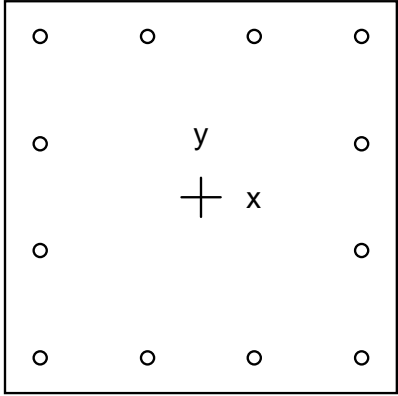
Beta1 = 0.8

Min clear spacing = 6.31 in

Clear cover = 2.00 in

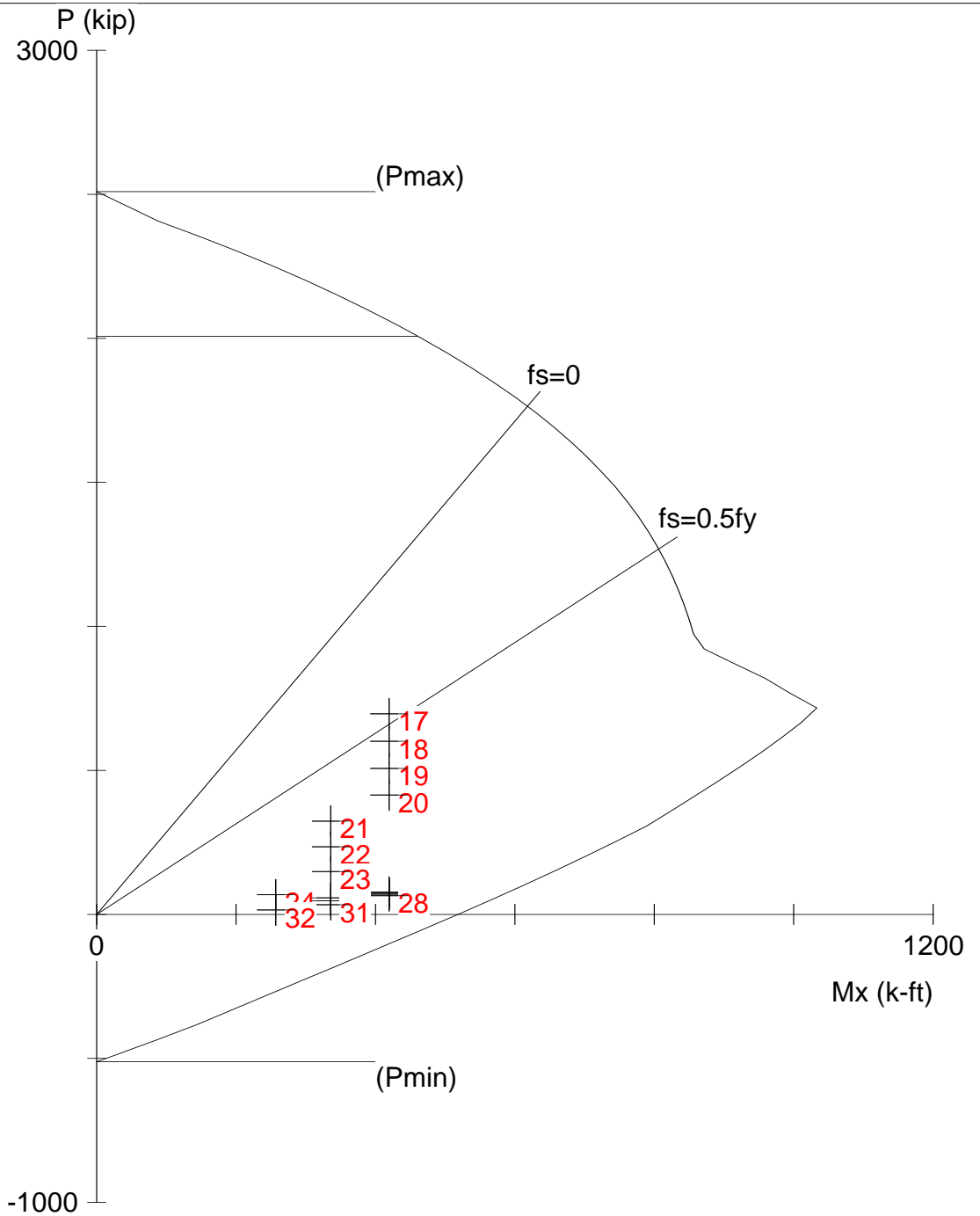
Confinement: Tied

$\phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65$



28 x 28 in

Code: ACI 318-08  
 Units: English  
 Run axis: About X-axis  
 Run option: Investigation  
 Slenderness: Not considered  
 Column type: Structural  
 Bars: ASTM A615  
 Date: 08/09/10  
 Time: 13:32:24



spColumn v4.50. Licensed to: National Institute of Standards and Technology . License ID: 56132-1019166-4-2473A-20EE4

File: Z:\Documents\Report Files 6-17-10\PCA Columns\12number8col.col

Project:

Column: (12) #8

Engineer: TSW

$f'_c = 5$  ksi

$f_y = 60$  ksi

$A_g = 784$  in<sup>2</sup>

12 #8 bars

$E_c = 4031$  ksi

$E_s = 29000$  ksi

$A_s = 9.48$  in<sup>2</sup>

$\rho = 1.21\%$

$f_c = 4.25$  ksi

$X_o = 0.00$  in

$I_x = 51221.3$  in<sup>4</sup>

$e_u = 0.003$  in/in

$Y_o = 0.00$  in

$I_y = 51221.3$  in<sup>4</sup>

Beta1 = 0.8

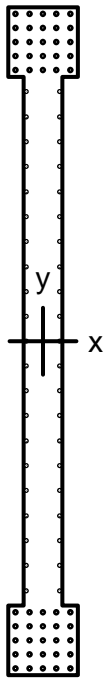
Min clear spacing = 6.67 in

Clear cover = 2.00 in

Confinement: Tied

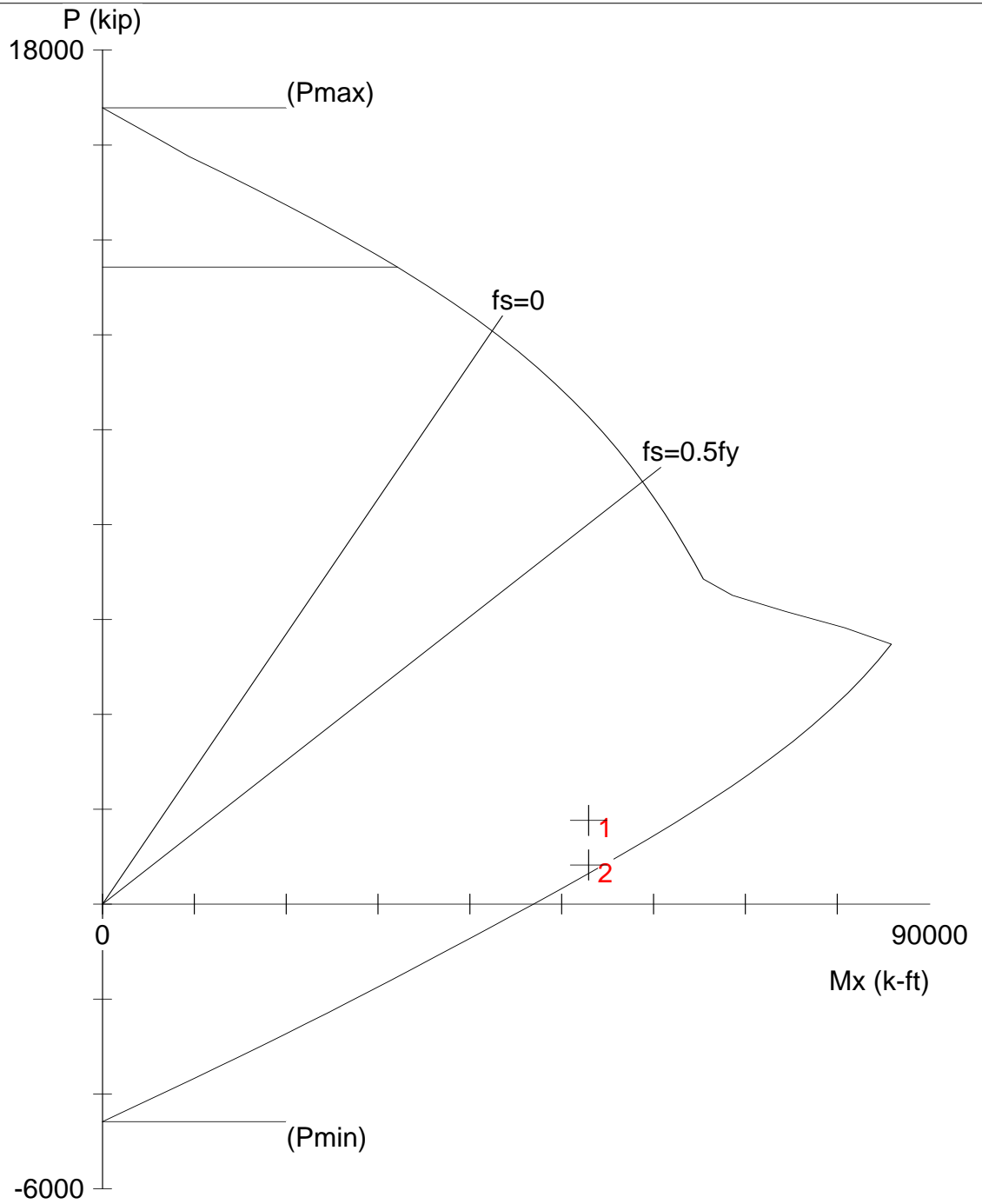
$\phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65$





28 x 268 in

Code: ACI 318-08  
 Units: English  
 Run axis: About X-axis  
 Run option: Investigation  
 Slenderness: Not considered  
 Column type: Structural  
 Bars: ASTM A615  
 Date: 08/09/10  
 Time: 13:38:05



spColumn v4.50. Licensed to: National Institute of Standards and Technology . License ID: 56132-1019166-4-2473A-20EE4

File: Z:\Documents\PCA Column Files\Shear Walls\5at10\5at10sw with16number10and9number9.col

Project:

Column: EXT SW L1

Engineer:

$f'_c = 5$  ksi

$f_y = 60$  ksi

$A_g = 4960$  in<sup>2</sup>

92 bars

$E_c = 4031$  ksi

$E_s = 29000$  ksi

$A_s = 84.89$  in<sup>2</sup>

$\rho = 1.71\%$

$f_c = 4.25$  ksi

$X_o = 0.00$  in

$I_x = 3.53858e+007$  in<sup>4</sup>

$e_u = 0.003$  in/in

$Y_o = 0.00$  in

$I_y = 174805$  in<sup>4</sup>

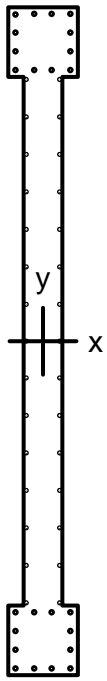
Beta1 = 0.8

Min clear spacing = 4.35 in

Clear cover = N/A

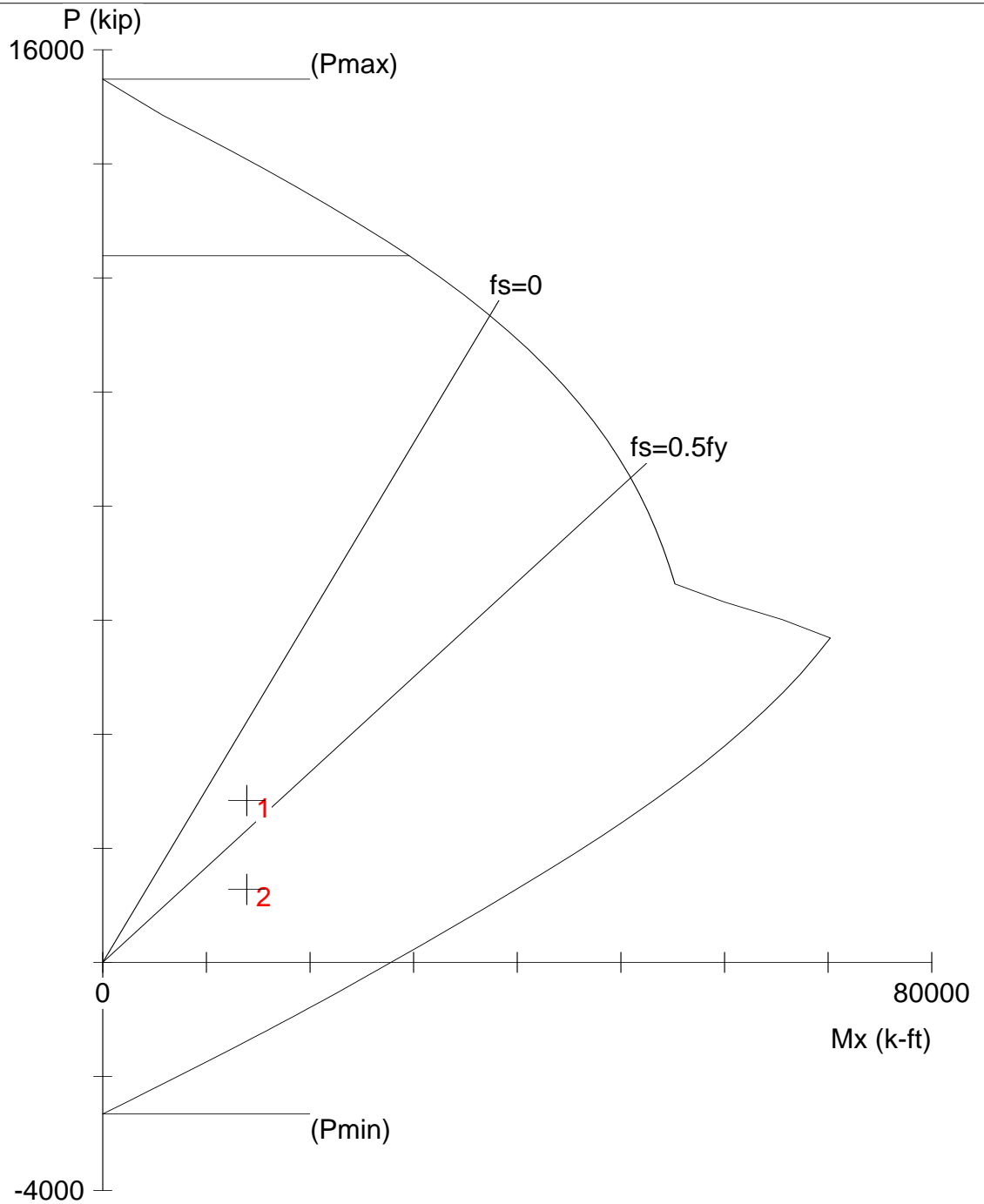
Confinement: Tied

$\phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65$



28 x 268 in

Code: ACI 318-08  
 Units: English  
 Run axis: About X-axis  
 Run option: Investigation  
 Slenderness: Not considered  
 Column type: Structural  
 Bars: ASTM A615  
 Date: 08/09/10  
 Time: 13:37:26



spColumn v4.50. Licensed to: National Institute of Standards and Technology . License ID: 56132-1019166-4-2473A-20EE4

File: Z:\Documents\PCA Column Files\Shear Walls\5at15\5at15sw with12number10.col

Project:

Column: INT SW L1

Engineer:

$f'_c = 5$  ksi

$f_y = 60$  ksi

$A_g = 4960$  in<sup>2</sup>

54 bars

$E_c = 4031$  ksi

$E_s = 29000$  ksi

$A_s = 49.23$  in<sup>2</sup>

$\rho = 0.99\%$

$f_c = 4.25$  ksi

$X_o = 0.00$  in

$I_x = 3.53858e+007$  in<sup>4</sup>

$e_u = 0.003$  in/in

$Y_o = 0.00$  in

$I_y = 174805$  in<sup>4</sup>

Beta1 = 0.8

Min clear spacing = 3.84 in

Clear cover = N/A

Confinement: Tied

$\phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65$

## **Appendix D – LSP, LDP, NSP, and NDP Calculations**



## Section 1: Modeling & Analysis Procedure

### 1. Analytical Determination of Period (ASCE 41-06 3.3.1.2.1)

Stiffness Reduction Factors for Cracked Concrete - For Use in SAP2000 Analysis

	Moment			Shear			Axial				
Beams	0.3	$E_c I_g$		0.4	$E_c A_w$		(-)				
Col (>.5)	0.7	$E_c I_g$		0.4	$E_c A_w$		(-)				
Col (<.1)	0.3	$E_c I_g$		0.4	$E_c A_w$		1.0	$E_c A_g$			
Walls	0.7	$E_c I_g$		0.4	$E_c A_w$		1.0	$E_c A_g$			
	E-W Direction						N-S Direction				
LFRS	Concrete Moment Resisting Frame						LFRS	Concrete Shear Wall			
T	<b>2.314</b> s, Calculated Period (old)						T	<b>1.107</b> s, Calculated Period (old)			

### 2. Hazard Definitions (ASCE 41-06 Table C1-1, Section 1.6)

IO - Immediate Occupancy

LS - Life Safety

CP - Collapse Prevention

Site Class **D**

	EQ	$S_s$	$S_1$	$S_{X5}$	$S_{X1}$	$T_s$	$T_0$	$B_1$
		(g)	(g)	(g)	(g)	(s)	(s)	
IO	50%/50yr	0.436	0.175	0.633	0.367	0.579	0.116	1.002365
LS	10%/50yr (BSE-1)	1.000	0.400	1.100	0.640	0.582	0.116	1.002365
CP	2%/50yr (BSE-2)	1.500	0.600	1.500	0.900	0.600	0.120	1.002365

### 3. Force Distribution Calculations (ASCE 41-06 Section 3.3)

E-W Direction					N-S Direction				
$C_1$	1				$C_1$	1			
$C_2$	1				$C_2$	1			
$C_m$	1				$C_m$	1			
	CP	LS	IO			CP	LS	IO	
$S_a$	0.388	0.276	0.158	g	$S_a$	0.811	0.577	0.331	
W	19220	19220	19220	kips	W	19220	19220	19220	
V	7458	5303	3040	kips	V	15589	11086	6354	
k	1.907				k	1.304			

### 3. Force Distribution (ASCE 41-06 3.3.1.3.2)

W **152.33** ft, Building Width in EW Direction

L **102.33** ft, Building Width in NS Direction

Level	w	h	$w_i h_i^k$ EW	$w_i h_i^k$ NS	$C_{vx,EW}$	$C_{vx,NS}$
	(kip)	(ft)				



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Level 08	2277	106	16582744	993995	0.289	0.236
Level 07	2414	93	13695643	888389	0.238	0.211
Level 06	2414	80	10277277	730068	0.179	0.174
Level 05	2414	67	7328416	579394	0.128	0.138
Level 04	2414	54	4856913	437382	0.085	0.104
Level 03	2414	41	2872528	305456	0.050	0.073
Level 02	2414	28	1388084	185804	0.024	0.044
Level 01	2460	15	430284	83943	0.007	0.020
Total	19220		57431890	4204431	1.000	1.000

Base Forces

Level	F <sub>xEW</sub> (kips)	F <sub>xNS</sub> (kips)	M <sub>EW</sub> (k-in)	M <sub>NS</sub> (k-in)	Limit State Factors <sup>1</sup>			
					IO	LS	CP	
Level 08	5550	4544	507217	278987				
Level 07	4583	4061	418909	249347	EW	0.1582	0.2759	0.3880
Level 06	3439	3337	314351	204910	NS	0.3306	0.5768	0.8111
Level 05	2453	2649	224154	162620				
Level 04	1625	1999	148558	122761				
Level 03	961	1396	87862	85733				
Level 02	465	849	42457	52150				
Level 01	144	384	13161	23561				

1. For use in adjusting SAP2000 Load Combinations

4. Torsion Amplification Check (ASCE 41-06 3.2.2.2.2)

Immediate Occupancy

Level	$\delta_{EW}$ (N) (in)	$\delta_{EW}$ (S) (in)	$\delta_{NS}$ (E,Ext) (in)	$\delta_{NS}$ (W,Ext) (in)	$\eta_{EW}$	$\eta_{NS,EXT}$	A <sub>x,EW</sub>	A <sub>x,NS,EXT</sub>
Level 08	17.427	18.040	8.096	6.567	1.051	1.104	0.00	1.00
Level 07	16.544	17.063	6.869	5.572	1.031	1.105	0.00	1.00
Level 06	15.049	15.472	5.642	4.577	1.023	1.105	0.00	1.00
Level 05	13.037	13.367	4.439	3.603	1.018	1.106	0.00	1.00
Level 04	10.651	10.893	3.291	2.676	1.015	1.106	0.00	1.00
Level 03	8.018	8.180	2.240	1.827	1.012	1.107	0.00	1.00
Level 02	5.248	5.340	1.330	1.092	1.010	1.106	0.00	1.00
Level 01	2.472	2.511	0.610	0.510	1.008	1.101	0.00	1.00
Base	0.013	0.014	0.086	0.082				
					1.051	1.107		<-- MAX

Level	$\delta_{NS}$ (E,Int) (in)	$\delta_{NS}$ (W,Int) (in)	$\eta_{NS,INT}$
Level 08	7.485	7.179	1.021
Level 07	6.350	6.091	1.021
Level 06	5.216	5.003	1.021



National Earthquake Hazards Reduction Program

Level 05	4.104	3.937	1.021
Level 04	3.045	2.922	1.021
Level 03	2.075	1.992	1.021
Level 02	1.235	1.187	1.021
Level 01	0.570	0.550	1.020
Base	0.084	0.084	1.005
			1.021 <-- MAX

5. Base Forces for 2D Model

Level	F <sub>x,MF</sub> (kips)	F <sub>x,EXTSW</sub> (kips)	F <sub>x,INTSW</sub> (kips)
Level 08	2914.9	1257.0	1160.2
Level 07	2407.4	1123.4	1036.9
Level 06	1806.5	923.2	852.1
Level 05	1288.2	732.7	676.3
Level 04	853.7	553.1	510.5
Level 03	504.9	386.3	356.5
Level 02	244.0	235.0	216.9
Level 01	75.6	106.2	98.0

3917.2

5. P-Delta Effects (ASCE 41-06 3.2.5)

Note: P-Delta effects are included in the SAP2000 Analysis through use of Leaner Columns. 3D Loads should represent total floor load, and 2D Loads should be for a single frame. Input Loads should be cumulative (from Section Cuts in SAP2000)

Level	Floor DL <sub>3D</sub> (kips)	Floor LL <sub>3D</sub> (kips)	Floor DL <sub>2D</sub> (kips)	Floor LL <sub>2D</sub> (kips)	Cumlat DL (kips)	Cumal LL (kips)	L Col DL (kips)	L Col LL (kips)
Level 08	2495.6	307.0	346.1	33.5	901.7	120.0	<b>901.7</b>	<b>120.0</b>
Level 07	5112.7	767.4	717.0	83.7	1839.3	300.0	<b>937.6</b>	<b>300.0</b>
Level 06	7729.7	1534.8	1088.0	167.4	2776.9	600.0	<b>937.6</b>	<b>300.0</b>
Level 05	10346.8	2302.2	1459.0	251.1	3714.5	900.0	<b>937.6</b>	<b>300.0</b>
Level 04	12963.9	3069.6	1829.9	334.8	4652.0	1200.0	<b>937.6</b>	<b>300.0</b>
Level 03	15581.0	3837.0	2200.9	418.5	5589.6	1500.0	<b>937.6</b>	<b>300.0</b>
Level 02	18198.1	4604.4	2571.9	502.2	6527.2	1800.0	<b>937.6</b>	<b>300.0</b>
Level 01	20912.9	5371.8	2955.2	585.9	7501.3	2100.0	<b>974.1</b>	<b>300.0</b>

6. Actions of Gravity Loads Acting Simultaneously with Seismic Loads

Q <sub>G</sub>	1.1	*(Q <sub>D</sub> + .25Q <sub>L</sub> )
Q <sub>G</sub>	0.9	*Q <sub>D</sub>

















Section 3a: LSP & NSP Moment Frame Column Assessment (IO)

Typical Column Properties
w = 28 in, Width of Column
c = 1.5 in, Cover
beta\_1 = 0.8
f'\_c = 5000 psi, Lower Bound Compressive Strength (ASCE 41-06 Table 6-3)
f\_y = 60 ksi, Lower Bound Yield Strength of Reinforcing Steel (ASCE 41-06 Table 6-2)
f\_u = 90 ksi, Lower Bound Ultimate Strength of Reinforcing Steel (ASCE 41-06 Table 6-2)
f'\_e = 7500 psi, Expected Compressive Strength (ASCE 41-06 Table 6-4)
f\_y = 75 ksi, Expected Yield Strength of Reinforcing Steel (ASCE 41-06 Table 6-4)
f'\_e = 112.5 ksi, Expected Ultimate Strength of Reinforcing Steel (ASCE 41-06 Table 6-4)
k = 1 Knowledge Factor (ASCE 41-06 2.2.6.4)
C\_1 = 1
C\_2 = 1
eta = 5.875

Parameters Below Assume P/A\_g f'\_c is less than 0.6, s/d is less than or equal to 0.5, mu is greater than or equal to 0.006, and V\_u/(A\_g f'\_c) is less than 0.6. Check these values prior to using this portion of the table

PCA Column Models should use expected behaviors

Main data table with columns for COL ID, I\_n, Longit Reinf, Yield Zone Reinforcing, Non-Yield Zone Reinforcing, Top/Bottom V\_DESIGN, rho, Top/Bottom P\_DESIGN, Top/Bottom Q\_u/A\_g f'\_c, Top/Bottom Q\_u/A\_g f'\_c \* E\_s/E\_c, Flexure-Axial J^P\_DESIGN, and Flexure-Axial M\_u/m. Includes a final column for Failure Mode (F\_i) with sub-columns for various assessment criteria (DCR\_max, DCR\_min, etc.)

None

Linear Static Procedure											Nonlinear Static Procedure									
exure-Axial Acceptance											Hinge Rotation		Column Hinge			Shear Acceptance				
Bottom	Top	Bottom	$KQ_{L,yield}$	$KQ_{L,non-yield}$	J	$Q_G$	Shear	Yield Zone Shear	Non-Yield Zone		Top	Bottom	$\theta_{allow, TOP}$	$\theta_{allow, BOT}$	Rotation	Acceptance	$V_{OES}$	$V_E$		
$P_{max}$	$P_{min}$	$P_{min}$	(k)	(k)		(k)	$Q_E$	Acceptance	Shear Acceptance		(radians)	(radians)	(radians)	(radians)			(kips)	(kips)		
PASS	FAIL	FAIL	408.0	408.0	1.0	6.00	201.4	PASS	PASS		0	0.014185	0.0049	0.0049	PASS	FAIL	99.0	92.99	PASS	
PASS	FAIL	FAIL	408.0	408.0	1.0	11.39	165.6	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	61.2	49.84	PASS	
PASS	FAIL	PASS	408.0	408.0	1.0	10.18	164.1	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	51.5	41.34	PASS	
PASS	PASS	PASS	408.0	408.0	1.0	10.79	161.8	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	38.0	27.21	PASS	
PASS	PASS	PASS	408.0	408.0	1.0	10.78	143.4	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	22.8	11.99	PASS	
PASS	PASS	PASS	408.0	408.0	1.0	11.18	119.9	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	9.1	-2.05	PASS	
PASS	PASS	PASS	408.0	408.0	1.0	10.35	89.0	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	6.0	-4.31	PASS	
PASS	PASS	PASS	408.0	408.0	1.0	13.81	41.9	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	10.8	-2.98	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.16	282.5	PASS	PASS		0	0.012068	0.0046	0.0046	PASS	FAIL	126.8	126.62	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.02	297.7	PASS	PASS		0	0	0.0047	0.0047	PASS	PASS	131.3	131.30	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.32	281.2	PASS	PASS		0	0	0.0048	0.0048	PASS	PASS	116.6	116.32	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.45	273.3	PASS	PASS		0	0	0.0049	0.0049	PASS	PASS	97.5	97.02	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.63	247.8	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	79.6	78.99	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.71	209.7	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	59.4	58.65	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.85	155.7	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	36.8	35.95	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.94	87.1	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	18.0	17.02	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.02	283.9	PASS	PASS		0	0.012051	0.0046	0.0046	PASS	FAIL	127.1	127.04	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.00	297.9	PASS	PASS		0	0	0.0047	0.0047	PASS	PASS	131.2	131.24	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.01	295.7	PASS	PASS		0	0	0.0048	0.0048	PASS	PASS	136.4	136.41	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.01	266.4	PASS	PASS		0	0	0.0049	0.0049	PASS	PASS	97.3	97.30	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.01	244.6	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	79.5	79.44	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.01	208.0	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	61.1	61.12	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.02	156.0	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	39.7	39.64	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.02	90.3	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	22.2	22.21	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.01	283.7	PASS	PASS		0	0.012052	0.0046	0.0046	PASS	FAIL	127.1	127.09	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.00	297.8	PASS	PASS		0	0	0.0047	0.0047	PASS	PASS	131.3	131.29	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.01	295.7	PASS	PASS		0	0	0.0048	0.0048	PASS	PASS	116.4	116.42	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.01	266.4	PASS	PASS		0	0	0.0049	0.0049	PASS	PASS	97.4	97.37	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.02	244.6	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	79.5	79.47	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.02	208.0	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	61.1	61.04	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.02	156.1	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	39.7	39.64	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.04	90.3	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	22.1	22.10	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.24	283.8	PASS	PASS		0	0.012067	0.0046	0.0046	PASS	FAIL	127.2	126.93	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.03	298.6	PASS	PASS		0	0	0.0047	0.0047	PASS	PASS	131.6	131.59	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.19	281.9	PASS	PASS		0	0	0.0048	0.0048	PASS	PASS	116.7	116.48	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.27	273.9	PASS	PASS		0	0	0.0049	0.0049	PASS	PASS	97.8	97.50	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.39	248.0	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	79.0	78.57	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.43	209.8	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	61.6	61.17	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.56	155.7	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	40.7	40.12	PASS	
PASS	PASS	PASS	469.5	375.6	1.0	0.53	87.1	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	22.9	22.36	PASS	
PASS	FAIL	FAIL	408.0	408.0	1.0	4.02	201.4	PASS	PASS		0	0.011996	0.0049	0.0049	PASS	FAIL	97.4	93.39	PASS	
PASS	FAIL	FAIL	408.0	408.0	1.0	7.61	165.6	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	86.7	79.10	PASS	
PASS	PASS	PASS	408.0	408.0	1.0	6.79	164.1	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	72.6	65.85	PASS	
PASS	PASS	PASS	408.0	408.0	1.0	7.20	161.8	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	62.2	54.98	PASS	
PASS	PASS	PASS	408.0	408.0	1.0	7.17	143.4	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	45.7	38.53	PASS	
PASS	PASS	PASS	408.0	408.0	1.0	7.50	119.9	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	34.9	27.37	PASS	
PASS	PASS	PASS	408.0	408.0	1.0	6.70	89.0	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	29.9	23.20	PASS	
PASS	PASS	PASS	408.0	408.0	1.0	9.89	41.9	PASS	PASS		0	0	0.0050	0.0050	PASS	PASS	22.8	12.93	PASS	



Linear Static Procedure										Nonlinear Static Procedure									
Top		Bottom		$\kappa_{CL,yield}$ (k)	$\kappa_{CL,non-yield}$ (k)	J	Shear		Yield Zone Shear Acceptance	Non-Yield Zone Shear Acceptance	Hinge Rotation		Column Hinge		Shear Acceptance				
$P_{min}$	$P_{max}$	$Q_G$	$Q_E$				Top (radians)	Bottom (radians)			$\theta_{allow, TOP}$ (radians)	$\theta_{allow, BOT}$ (radians)	Rotation Acceptance	$V_{DES}$ (kips)	$V_E$ (kips)				
FAIL	FAIL	408.0	408.0	2.0	6.00	363.7	PASS	PASS	0	0.035784	0.0255	0.0254	PASS	FAIL	106.6	100.56	PASS		
FAIL	FAIL	408.0	408.0	2.0	11.39	299.8	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	64.7	53.33	PASS		
FAIL	FAIL	408.0	408.0	2.0	10.18	294.8	PASS	PASS	0.013895	0	0.0260	0.0260	PASS	PASS	56.4	46.22	PASS		
FAIL	PASS	408.0	408.0	2.0	10.79	288.8	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	10.7	-0.05	PASS		
PASS	PASS	408.0	408.0	2.0	10.78	254.3	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	8.1	-2.71	PASS		
PASS	PASS	408.0	408.0	2.0	11.18	211.3	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	9.4	-1.76	PASS		
PASS	PASS	408.0	408.0	2.0	10.35	156.2	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	4.6	-5.77	PASS		
PASS	PASS	408.0	408.0	2.0	13.81	73.3	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	14.7	0.92	PASS		
PASS	FAIL	469.5	375.6	2.0	0.16	492.7	PASS	PASS	0	0.030559	0.0228	0.0227	PASS	FAIL	123.1	122.94	PASS		
PASS	PASS	469.5	375.6	2.0	0.02	519.4	PASS	PASS	0	0	0.0237	0.0236	PASS	PASS	120.5	120.52	PASS		
PASS	PASS	469.5	375.6	2.0	0.32	490.8	PASS	PASS	0.014347	0	0.0246	0.0245	PASS	PASS	92.2	91.83	PASS		
PASS	PASS	469.5	375.6	2.0	0.45	477.1	PASS	PASS	0	0	0.0254	0.0253	PASS	PASS	63.7	63.29	PASS		
PASS	PASS	469.5	469.5	2.0	0.63	432.7	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	42.4	41.74	PASS		
PASS	PASS	469.5	469.5	2.0	0.71	366.3	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	31.9	31.20	PASS		
PASS	PASS	469.5	469.5	2.0	0.85	272.0	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	17.1	16.24	PASS		
PASS	PASS	469.5	469.5	2.0	0.94	152.1	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	7.4	6.42	PASS		
PASS	PASS	469.5	375.6	2.0	0.02	496.0	PASS	PASS	0	0.030486	0.0228	0.0227	PASS	FAIL	122.7	122.67	PASS		
PASS	PASS	469.5	375.6	2.0	0.00	520.5	PASS	PASS	0	0	0.0236	0.0236	PASS	PASS	120.3	120.29	PASS		
PASS	PASS	469.5	375.6	2.0	0.01	516.5	PASS	PASS	0.014306	0	0.0245	0.0244	PASS	PASS	91.5	91.51	PASS		
PASS	PASS	469.5	375.6	2.0	0.01	465.3	PASS	PASS	0	0	0.0254	0.0253	PASS	PASS	64.4	64.41	PASS		
PASS	PASS	469.5	469.5	2.0	0.01	427.2	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	41.9	41.85	PASS		
PASS	PASS	469.5	469.5	2.0	0.01	363.2	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	33.1	33.11	PASS		
PASS	PASS	469.5	469.5	2.0	0.02	272.6	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	19.9	19.90	PASS		
PASS	PASS	469.5	469.5	2.0	0.02	157.7	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	11.2	11.20	PASS		
PASS	PASS	469.5	375.6	2.0	0.01	495.5	PASS	PASS	0	0.030482	0.0228	0.0227	PASS	FAIL	122.4	122.38	PASS		
PASS	PASS	469.5	375.6	2.0	0.00	520.2	PASS	PASS	0	0	0.0236	0.0236	PASS	PASS	119.9	119.94	PASS		
PASS	PASS	469.5	375.6	2.0	0.01	516.5	PASS	PASS	0.014311	0	0.0245	0.0244	PASS	PASS	91.7	91.68	PASS		
PASS	PASS	469.5	375.6	2.0	0.01	465.5	PASS	PASS	0	0	0.0254	0.0253	PASS	PASS	64.4	64.43	PASS		
PASS	PASS	469.5	469.5	2.0	0.02	427.3	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	42.0	41.96	PASS		
PASS	PASS	469.5	469.5	2.0	0.02	363.4	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	33.1	33.09	PASS		
PASS	PASS	469.5	469.5	2.0	0.02	272.6	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	19.9	19.89	PASS		
PASS	PASS	469.5	469.5	2.0	0.04	157.7	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	11.1	11.10	PASS		
PASS	FAIL	469.5	375.6	2.0	0.24	496.6	PASS	PASS	0	0.030518	0.0228	0.0227	PASS	FAIL	122.6	122.41	PASS		
PASS	PASS	469.5	375.6	2.0	0.03	522.1	PASS	PASS	0	0	0.0237	0.0236	PASS	PASS	120.2	120.13	PASS		
PASS	PASS	469.5	375.6	2.0	0.19	492.9	PASS	PASS	0.014319	0	0.0246	0.0245	PASS	PASS	92.2	92.03	PASS		
PASS	PASS	469.5	375.6	2.0	0.27	478.7	PASS	PASS	0	0	0.0254	0.0253	PASS	PASS	64.5	64.25	PASS		
PASS	PASS	469.5	469.5	2.0	0.39	433.4	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	41.8	41.40	PASS		
PASS	PASS	469.5	469.5	2.0	0.43	366.5	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	34.0	33.54	PASS		
PASS	PASS	469.5	469.5	2.0	0.56	272.0	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	21.0	20.49	PASS		
PASS	PASS	469.5	469.5	2.0	0.53	152.1	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	12.3	11.74	PASS		
FAIL	FAIL	408.0	408.0	2.0	4.02	363.7	PASS	PASS	0	0.030939	0.0255	0.0254	PASS	FAIL	96.7	92.63	PASS		
FAIL	FAIL	408.0	408.0	2.0	7.61	299.8	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	82.3	74.68	PASS		
FAIL	FAIL	408.0	408.0	2.0	6.79	294.8	PASS	PASS	0.014174	0	0.0260	0.0260	PASS	PASS	59.0	52.26	PASS		
PASS	FAIL	408.0	408.0	2.0	7.20	288.8	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	35.3	28.08	PASS		
PASS	PASS	408.0	408.0	2.0	7.17	254.3	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	25.8	18.63	PASS		
PASS	PASS	408.0	408.0	2.0	7.50	211.3	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	18.2	10.68	PASS		
PASS	PASS	408.0	408.0	2.0	6.70	156.2	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	18.9	12.15	PASS		
PASS	PASS	408.0	408.0	2.0	9.89	73.3	PASS	PASS	0	0	0.0260	0.0260	PASS	PASS	19.0	9.12	PASS		





Section 3c: LSP & NSP Moment Frame Column Assessment (CP)

Typical Column Properties

Table listing typical column properties: w (28 in), c (1.5 in), beta\_1 (0.8), f'\_c (5000 psi), f\_y (60 ksi), f'\_t (90 ksi), f'\_e (7500 psi), f'\_y (75 ksi), f'\_u (112.5 ksi), k (1), C\_1 (1), C\_2 (1), n (7.195).

Parameters Below Assume P/A\_g f'\_c is less than 0.6, s/d is less than or equal to 0.5, rho is greater than or equal to 0.006, and V\_u/(V\_u + J) is less than 0.6. Check these values prior to using this portion of the table.

PCA Column Models should use expected behaviours

Main data table with columns for COLID, I\_n, Longit Reinf, Yield Zone Reinforcing, Non-Yield Zone Reinforcing, Top/Bottom V DESIGN, rho, P DESIGN MAX, P DESIGN MIN, Axial Q\_u/A\_g f'\_c, X/E\_f, Flexure-Axial M DESIGN MAX, M DESIGN MIN, Mu/M, and Flexure-Axial DCR MAX, DCR MIN, P MAX, P MIN. Rows include various column IDs like 6A01, 6A02, etc.

Linear Static Procedure										Nonlinear Static Procedure									
Acceptance		Bottom		Top		Shear		Yield Zone		Non-Yield Zone		Hinge Rotation		Column Hinge		Shear Acceptance			
P <sub>min</sub>	P <sub>max</sub>	κ <sub>CL,yield</sub>	κ <sub>CL,non-yield</sub>	J	Q <sub>G</sub>	Q <sub>E</sub>	Acceptance	Acceptance	Acceptance	Acceptance	Top (radians)	Bottom (radians)	θ <sub>allow,Top</sub> (radians)	θ <sub>allow,Bottom</sub> (radians)	Rotation Acceptance	V <sub>DES</sub> (kips)	V <sub>E</sub> (kips)	Acceptance	
FAIL	FAIL	408.0	408.0	2.0	6.00	453.0	PASS	PASS	0	0.04666	0	0.0343	0.0342	PASS	FAIL	99.5	93.50	PASS	
FAIL	FAIL	408.0	408.0	2.0	11.39	373.9	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	66.6	55.18	PASS	
FAIL	FAIL	408.0	408.0	2.0	10.18	366.2	PASS	PASS	0.021006	0	0.0350	0.0350	PASS	PASS	PASS	42.2	32.05	PASS	
FAIL	FAIL	408.0	408.0	2.0	10.79	357.6	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	22.5	11.66	PASS	
PASS	PASS	408.0	408.0	2.0	10.78	313.8	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	2.4	-8.35	PASS	
PASS	PASS	408.0	408.0	2.0	11.18	260.1	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	16.8	5.63	PASS	
PASS	PASS	408.0	408.0	2.0	10.35	191.7	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	9.8	-0.60	PASS	
PASS	PASS	408.0	408.0	2.0	13.81	89.9	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	16.5	2.70	PASS	
PASS	FAIL	469.5	375.6	2.0	0.16	602.8	PASS	PASS	0	0.039742	0	0.0303	0.0302	PASS	FAIL	118.9	118.76	PASS	
PASS	PASS	469.5	375.6	2.0	0.02	635.7	PASS	PASS	0.00032	0	0.0316	0.0315	PASS	PASS	PASS	109.8	109.74	PASS	
PASS	PASS	469.5	375.6	2.0	0.32	600.7	PASS	PASS	0.021037	0	0.0329	0.0328	PASS	PASS	PASS	82.1	81.81	PASS	
PASS	PASS	469.5	375.6	2.0	0.45	584.1	PASS	PASS	0	0	0.0342	0.0341	PASS	PASS	PASS	42.0	41.52	PASS	
PASS	PASS	469.5	469.5	2.0	0.63	529.8	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	24.4	23.73	PASS	
PASS	PASS	469.5	469.5	2.0	0.71	448.6	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	17.5	16.82	PASS	
PASS	PASS	469.5	469.5	2.0	0.85	333.1	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	7.4	6.59	PASS	
PASS	PASS	469.5	469.5	2.0	0.94	186.3	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	2.1	1.20	PASS	
PASS	FAIL	469.5	375.6	2.0	0.02	607.5	PASS	PASS	0	0.039663	0	0.0303	0.0302	PASS	FAIL	118.4	118.37	PASS	
PASS	PASS	469.5	375.6	2.0	0.00	637.5	PASS	PASS	0.000318	0	0.0316	0.0315	PASS	PASS	PASS	109.4	109.36	PASS	
PASS	PASS	469.5	375.6	2.0	0.01	632.6	PASS	PASS	0.02099	0	0.0329	0.0328	PASS	PASS	PASS	81.4	81.42	PASS	
PASS	PASS	469.5	375.6	2.0	0.01	569.8	PASS	PASS	0	0	0.0342	0.0340	PASS	PASS	PASS	43.0	42.97	PASS	
PASS	PASS	469.5	469.5	2.0	0.01	523.1	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	23.8	23.83	PASS	
PASS	PASS	469.5	469.5	2.0	0.01	444.8	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	18.8	18.75	PASS	
PASS	PASS	469.5	469.5	2.0	0.02	333.8	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	10.1	10.11	PASS	
PASS	PASS	469.5	469.5	2.0	0.02	193.1	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	5.8	5.74	PASS	
PASS	FAIL	469.5	375.6	2.0	0.01	606.7	PASS	PASS	0	0.039664	0	0.0303	0.0302	PASS	FAIL	118.2	118.14	PASS	
PASS	PASS	469.5	375.6	2.0	0.00	637.0	PASS	PASS	0.000318	0	0.0316	0.0315	PASS	PASS	PASS	109.1	109.06	PASS	
PASS	PASS	469.5	375.6	2.0	0.01	632.5	PASS	PASS	0.020994	0	0.0329	0.0328	PASS	PASS	PASS	81.6	81.58	PASS	
PASS	PASS	469.5	375.6	2.0	0.01	570.0	PASS	PASS	0	0	0.0342	0.0340	PASS	PASS	PASS	43.0	43.00	PASS	
PASS	PASS	469.5	469.5	2.0	0.02	523.3	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	24.0	23.93	PASS	
PASS	PASS	469.5	469.5	2.0	0.02	445.0	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	18.7	18.73	PASS	
PASS	PASS	469.5	469.5	2.0	0.02	333.9	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	10.1	10.10	PASS	
PASS	PASS	469.5	469.5	2.0	0.04	193.1	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	5.7	5.64	PASS	
PASS	FAIL	469.5	375.6	2.0	0.24	608.7	PASS	PASS	0	0.039705	0	0.0303	0.0302	PASS	FAIL	118.7	118.42	PASS	
PASS	PASS	469.5	375.6	2.0	0.03	639.8	PASS	PASS	0.000319	0	0.0316	0.0315	PASS	PASS	PASS	109.7	109.67	PASS	
PASS	PASS	469.5	375.6	2.0	0.19	604.0	PASS	PASS	0.020985	0	0.0329	0.0328	PASS	PASS	PASS	82.0	81.78	PASS	
PASS	PASS	469.5	375.6	2.0	0.27	586.5	PASS	PASS	0	0	0.0342	0.0341	PASS	PASS	PASS	42.6	42.36	PASS	
PASS	PASS	469.5	469.5	2.0	0.39	530.9	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	23.9	23.48	PASS	
PASS	PASS	469.5	469.5	2.0	0.43	448.8	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	19.6	19.17	PASS	
PASS	PASS	469.5	469.5	2.0	0.56	333.0	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	11.4	10.84	PASS	
PASS	PASS	469.5	469.5	2.0	0.53	186.3	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	7.0	6.52	PASS	
FAIL	FAIL	408.0	408.0	2.0	4.02	453.0	PASS	PASS	0	0.040453	0	0.0343	0.0342	PASS	FAIL	94.4	90.35	PASS	
FAIL	FAIL	408.0	408.0	2.0	7.61	373.9	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	80.9	73.31	PASS	
FAIL	FAIL	408.0	408.0	2.0	6.79	366.2	PASS	PASS	0.021225	0	0.0350	0.0350	PASS	PASS	PASS	46.1	39.30	PASS	
FAIL	FAIL	408.0	408.0	2.0	7.20	357.6	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	24.4	17.23	PASS	
PASS	PASS	408.0	408.0	2.0	7.17	313.8	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	15.6	8.39	PASS	
PASS	PASS	408.0	408.0	2.0	7.50	260.1	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	10.9	3.38	PASS	
PASS	PASS	408.0	408.0	2.0	6.70	191.7	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	13.7	6.99	PASS	
PASS	PASS	408.0	408.0	2.0	9.89	89.9	PASS	PASS	0	0	0.0350	0.0350	PASS	PASS	PASS	17.2	7.36	PASS	

**Section 4a: LSP Moment Frame Joint Assessment (IO)**

## Typical Joint Details

w	28	in, Column Dimension
A <sub>j</sub>	784	in <sup>2</sup>
C <sub>1</sub>	1	
C <sub>2</sub>	1	
f <sub>c</sub> '	5000	psi, Lower Bound Compressive Strength of Concrete
f <sub>y</sub>	60	ksi, Lower Bound Reinforcing Yield Strength
f <sub>c</sub> '	7500	psi, Expected Compressive Strength of Concrete
f <sub>y</sub>	75	ksi, Expected Reinforcing Yield Strength
k	1	Knowledge Factor

Joint ID	ΣM <sub>nc</sub> /ΣM <sub>nb</sub>	Offset Rigidities		Linear Static Procedure							
		Col	Beam	Joint Shear		J	Q <sub>UF</sub> (V <sub>j</sub> )	Condition	Int/Ext	Q <sub>CL</sub> (V <sub>n</sub> )	Joint Shear Acceptance
		(k)	(k)	(V <sub>B,L</sub> + V <sub>B,R</sub> )*h/2l <sub>c</sub>	(V <sub>E,L</sub> + V <sub>E,R</sub> )*h/2l <sub>c</sub>						
6A01	1.50	1	0	0.02	0.1	1.0	345.8909	Other (Main)	Exterior	665.2	PASS
6A02	1.50	1	0	0.01	0.1	1.0	330.3343	Other (Main)	Exterior	665.2	PASS
6A03	1.50	1	0	0.02	0.1	1.0	324.4839	Other (Main)	Exterior	665.2	PASS
6A04	1.86	1	0	0.02	0.1	1.0	329.7102	Other (Main)	Exterior	665.2	PASS
6A05	1.87	1	0	0.02	0.1	1.0	307.4043	Other (Main)	Exterior	665.2	PASS
6A06	1.87	1	0	0.01	0.1	1.0	275.1684	Other (Main)	Exterior	665.2	PASS
6A07	2.45	1	0	0.01	0.1	1.0	247.6867	Other (Main)	Exterior	665.2	PASS
6A08	1.53	1	0	0.02	0.1	1.0	161.8461	Other (Roof)	Exterior	443.5	PASS
6B01	1.32	1	0	0.14	0.2	1.0	333.1019	2 Opp (Main)	Interior	831.6	PASS
6B02	1.34	1	0	0.15	0.2	1.0	313.4034	2 Opp (Main)	Interior	831.6	PASS
6B03	1.34	1	0	0.15	0.2	1.0	311.5452	2 Opp (Main)	Interior	831.6	PASS
6B04	1.69	1	0	0.17	0.2	1.0	272.7417	2 Opp (Main)	Interior	831.6	PASS
6B05	1.69	1	0	0.15	0.2	1.0	254.2773	2 Opp (Main)	Interior	831.6	PASS
6B06	1.69	1	0	0.12	0.2	1.0	245.1504	2 Opp (Main)	Interior	831.6	PASS
6B07	1.98	1	0	0.08	0.1	1.0	261.3192	2 Opp (Main)	Interior	831.6	PASS
6B08	1.11	0.5	0.5	0.07	0.1	1.0	188.5971	2 Opp (Roof)	Interior	665.2	PASS
6C01	1.34	1	0	0.12	0.2	1.0	326.5103	2 Opp (Main)	Interior	831.6	PASS
6C02	1.34	1	0	0.13	0.2	1.0	318.7567	2 Opp (Main)	Interior	831.6	PASS
6C03	1.34	1	0	0.12	0.2	1.0	311.5656	2 Opp (Main)	Interior	831.6	PASS
6C04	1.69	1	0	0.14	0.2	1.0	272.9039	2 Opp (Main)	Interior	831.6	PASS
6C05	1.69	1	0	0.13	0.2	1.0	254.0147	2 Opp (Main)	Interior	831.6	PASS
6C06	1.69	1	0	0.10	0.2	1.0	245.1651	2 Opp (Main)	Interior	831.6	PASS
6C07	1.98	1	0	0.07	0.1	1.0	261.323	2 Opp (Main)	Interior	831.6	PASS
6C08	1.11	0.5	0.5	0.07	0.1	1.0	188.5971	2 Opp (Roof)	Interior	665.2	PASS
6D01	1.34	1	0	0.12	0.2	1.0	323.7672	2 Opp (Main)	Interior	831.6	PASS
6D02	1.34	1	0	0.14	0.2	1.0	313.3216	2 Opp (Main)	Interior	831.6	PASS
6D03	1.34	1	0	0.13	0.2	1.0	311.5578	2 Opp (Main)	Interior	831.6	PASS
6D04	1.69	1	0	0.14	0.2	1.0	272.4791	2 Opp (Main)	Interior	831.6	PASS
6D05	1.69	1	0	0.13	0.2	1.0	253.9141	2 Opp (Main)	Interior	831.6	PASS
6D06	1.69	1	0	0.10	0.2	1.0	245.1655	2 Opp (Main)	Interior	831.6	PASS
6D07	1.98	1	0	0.07	0.1	1.0	261.324	2 Opp (Main)	Interior	831.6	PASS
6D08	1.11	0.5	0.5	0.06	0.1	1.0	188.6	2 Opp (Roof)	Interior	665.2	PASS
6E01	1.34	1	0	0.13	0.2	1.0	341.3914	2 Opp (Main)	Interior	831.6	PASS
6E02	1.34	1	0	0.14	0.2	1.0	323.8933	2 Opp (Main)	Interior	831.6	PASS
6E03	1.34	1	0	0.14	0.2	1.0	319.034	2 Opp (Main)	Interior	831.6	PASS
6E04	1.69	1	0	0.15	0.2	1.0	290.0687	2 Opp (Main)	Interior	831.6	PASS
6E05	1.69	1	0	0.14	0.2	1.0	267.4509	2 Opp (Main)	Interior	831.6	PASS
6E06	1.69	1	0	0.11	0.2	1.0	245.1586	2 Opp (Main)	Interior	831.6	PASS
6E07	1.98	1	0	0.08	0.1	1.0	261.3217	2 Opp (Main)	Interior	831.6	PASS
6E08	1.11	0.5	0.5	0.07	0.1	1.0	188.5987	2 Opp (Roof)	Interior	665.2	PASS
6F01	2.42	1	0	0.12	0.1	1.0	413.6228	Other (Main)	Exterior	665.2	PASS
6F02	2.42	1	0	0.13	0.1	1.0	396.9453	Other (Main)	Exterior	665.2	PASS
6F03	2.42	1	0	0.12	0.1	1.0	396.9545	Other (Main)	Exterior	665.2	PASS
6F04	3.05	1	0	0.14	0.2	1.0	313.3021	Other (Main)	Exterior	665.2	PASS
6F05	3.05	1	0	0.12	0.1	1.0	313.341	Other (Main)	Exterior	665.2	PASS
6F06	3.05	1	0	0.10	0.1	1.0	313.396	Other (Main)	Exterior	665.2	PASS
6F07	3.05	1	0	0.06	0.1	1.0	313.4709	Other (Main)	Exterior	665.2	PASS
6F08	1.53	1	0	0.05	0.1	1.0	271.6025	Other (Roof)	Exterior	443.5	PASS



**Section 4b: LSP Moment Frame Joint Assessment (LS)**

Typical Joint Details

w	28	in, Column Dimension
A <sub>j</sub>	784	in <sup>2</sup>
C <sub>1</sub>	1	
C <sub>2</sub>	1	
f <sub>c</sub> '	5000	psi, Lower Bound Compressive Strength of Concrete
f <sub>y</sub>	60	ksi, Lower Bound Reinforcing Yield Strength
f <sub>c</sub> '	7500	psi, Expected Compressive Strength of Concrete
f <sub>y</sub>	75	ksi, Expected Reinforcing Yield Strength
k	1	Knowledge Factor

Joint ID	ΣM <sub>nc</sub> /ΣM <sub>nb</sub>	Offset Rigidities		Linear Static Procedure							Joint Shear Acceptance
		Col	Beam	Joint Shear		J	Q <sub>UF</sub> (V <sub>j</sub> )	Condition	Int/Ext	Q <sub>CL</sub> (V <sub>n</sub> )	
				(V <sub>g,L</sub> + V <sub>g,R</sub> )*h/2l <sub>c</sub>	(V <sub>e,L</sub> + V <sub>e,R</sub> )*h/2l <sub>c</sub>						
6A01	1.50	1	0	0.02	0.1	2.0	345.9189	Other (Main)	Exterior	665.2	PASS
6A02	1.50	1	0	0.01	0.1	2.0	330.367	Other (Main)	Exterior	665.2	PASS
6A03	1.50	1	0	0.02	0.1	2.0	324.5175	Other (Main)	Exterior	665.2	PASS
6A04	1.86	1	0	0.01	0.1	2.0	329.7532	Other (Main)	Exterior	665.2	PASS
6A05	1.87	1	0	0.01	0.1	2.0	307.4511	Other (Main)	Exterior	665.2	PASS
6A06	1.87	1	0	0.01	0.1	2.0	275.2145	Other (Main)	Exterior	665.2	PASS
6A07	2.45	1	0	0.01	0.1	2.0	247.7316	Other (Main)	Exterior	665.2	PASS
6A08	1.53	1	0	0.01	0.1	2.0	161.9028	Other (Roof)	Exterior	443.5	PASS
6B01	1.32	1	0	0.19	0.3	2.0	333.1118	2 Opp (Main)	Interior	831.6	PASS
6B02	1.34	1	0	0.20	0.3	2.0	313.421	2 Opp (Main)	Interior	831.6	PASS
6B03	1.34	1	0	0.19	0.3	2.0	311.5697	2 Opp (Main)	Interior	831.6	PASS
6B04	1.69	1	0	0.21	0.3	2.0	272.7894	2 Opp (Main)	Interior	831.6	PASS
6B05	1.69	1	0	0.17	0.2	2.0	254.3521	2 Opp (Main)	Interior	831.6	PASS
6B06	1.69	1	0	0.12	0.2	2.0	245.2394	2 Opp (Main)	Interior	831.6	PASS
6B07	1.98	1	0	0.07	0.1	2.0	261.3978	2 Opp (Main)	Interior	831.6	PASS
6B08	1.11	0.5	0.5	0.06	0.1	2.0	188.6772	2 Opp (Roof)	Interior	665.2	PASS
6C01	1.34	1	0	0.15	0.3	2.0	326.5467	2 Opp (Main)	Interior	831.6	PASS
6C02	1.34	1	0	0.17	0.3	2.0	318.7918	2 Opp (Main)	Interior	831.6	PASS
6C03	1.34	1	0	0.16	0.3	2.0	311.6103	2 Opp (Main)	Interior	831.6	PASS
6C04	1.69	1	0	0.17	0.3	2.0	272.9774	2 Opp (Main)	Interior	831.6	PASS
6C05	1.69	1	0	0.14	0.2	2.0	254.1021	2 Opp (Main)	Interior	831.6	PASS
6C06	1.69	1	0	0.10	0.2	2.0	245.2579	2 Opp (Main)	Interior	831.6	PASS
6C07	1.98	1	0	0.07	0.1	2.0	261.4018	2 Opp (Main)	Interior	831.6	PASS
6C08	1.11	0.5	0.5	0.06	0.1	2.0	188.6781	2 Opp (Roof)	Interior	665.2	PASS
6D01	1.34	1	0	0.16	0.3	2.0	323.7986	2 Opp (Main)	Interior	831.6	PASS
6D02	1.34	1	0	0.18	0.3	2.0	313.3505	2 Opp (Main)	Interior	831.6	PASS
6D03	1.34	1	0	0.17	0.3	2.0	311.5964	2 Opp (Main)	Interior	831.6	PASS
6D04	1.69	1	0	0.17	0.3	2.0	272.5521	2 Opp (Main)	Interior	831.6	PASS
6D05	1.69	1	0	0.14	0.2	2.0	254.0015	2 Opp (Main)	Interior	831.6	PASS
6D06	1.69	1	0	0.10	0.2	2.0	245.2584	2 Opp (Main)	Interior	831.6	PASS
6D07	1.98	1	0	0.06	0.1	2.0	261.4027	2 Opp (Main)	Interior	831.6	PASS
6D08	1.11	0.5	0.5	0.06	0.1	2.0	188.6809	2 Opp (Roof)	Interior	665.2	PASS
6E01	1.34	1	0	0.16	0.3	2.0	341.4208	2 Opp (Main)	Interior	831.6	PASS
6E02	1.34	1	0	0.18	0.3	2.0	323.9227	2 Opp (Main)	Interior	831.6	PASS
6E03	1.34	1	0	0.17	0.3	2.0	319.0726	2 Opp (Main)	Interior	831.6	PASS
6E04	1.69	1	0	0.18	0.3	2.0	290.1365	2 Opp (Main)	Interior	831.6	PASS
6E05	1.69	1	0	0.15	0.2	2.0	267.5355	2 Opp (Main)	Interior	831.6	PASS
6E06	1.69	1	0	0.11	0.2	2.0	245.2506	2 Opp (Main)	Interior	831.6	PASS
6E07	1.98	1	0	0.07	0.1	2.0	261.399	2 Opp (Main)	Interior	831.6	PASS
6E08	1.11	0.5	0.5	0.07	0.1	2.0	188.6762	2 Opp (Roof)	Interior	665.2	PASS
6F01	2.42	1	0	0.17	0.2	2.0	413.6071	Other (Main)	Exterior	665.2	PASS
6F02	2.42	1	0	0.18	0.2	2.0	396.9378	Other (Main)	Exterior	665.2	PASS
6F03	2.42	1	0	0.17	0.2	2.0	396.9543	Other (Main)	Exterior	665.2	PASS
6F04	3.05	1	0	0.18	0.2	2.0	313.3175	Other (Main)	Exterior	665.2	PASS
6F05	3.05	1	0	0.14	0.2	2.0	313.3804	Other (Main)	Exterior	665.2	PASS
6F06	3.05	1	0	0.10	0.1	2.0	313.4496	Other (Main)	Exterior	665.2	PASS
6F07	3.05	1	0	0.06	0.1	2.0	313.5118	Other (Main)	Exterior	665.2	PASS
6F08	1.53	1	0	0.05	0.1	2.0	271.6332	Other (Roof)	Exterior	443.5	PASS



**Section 4c: LSP Moment Frame Joint Assessment (CP)**

Typical Joint Details

w	28	in, Column Dimension
A <sub>j</sub>	784	in <sup>2</sup>
C <sub>1</sub>	1	
C <sub>2</sub>	1	
f <sub>c</sub> '	5000	psi, Lower Bound Compressive Strength of Concrete
f <sub>y</sub>	60	ksi, Lower Bound Reinforcing Yield Strength
f <sub>c</sub> '	7500	psi, Expected Compressive Strength of Concrete
f <sub>y</sub>	75	ksi, Expected Reinforcing Yield Strength
K	1	Knowledge Factor

Joint ID	ΣM <sub>nc</sub> /ΣM <sub>nb</sub>	Offset Rigidities		Linear Static Procedure							Joint Shear Acceptance
		Col	Beam	Joint Shear		J	Q <sub>UF</sub> (V <sub>j</sub> ) (k)	Condition	Int/Ext	Q <sub>CL</sub> (V <sub>n</sub> ) (k)	
		(V <sub>e,L</sub> + V <sub>e,R</sub> )*h/2l <sub>c</sub>	(V <sub>e,L</sub> + V <sub>e,R</sub> )*h/2l <sub>c</sub>								
6A01	1.50	1	0	0.02	0.1	2.0	345.9129	Other (Main)	Exterior	665.2	PASS
6A02	1.50	1	0	0.02	0.1	2.0	330.3592	Other (Main)	Exterior	665.2	PASS
6A03	1.50	1	0	0.02	0.1	2.0	324.5079	Other (Main)	Exterior	665.2	PASS
6A04	1.86	1	0	0.02	0.2	2.0	329.7419	Other (Main)	Exterior	665.2	PASS
6A05	1.87	1	0	0.01	0.1	2.0	307.4433	Other (Main)	Exterior	665.2	PASS
6A06	1.87	1	0	0.01	0.1	2.0	275.21	Other (Main)	Exterior	665.2	PASS
6A07	2.45	1	0	0.01	0.1	2.0	247.7316	Other (Main)	Exterior	665.2	PASS
6A08	1.53	1	0	0.01	0.1	2.0	161.9033	Other (Roof)	Exterior	443.5	PASS
6B01	1.32	1	0	0.21	0.3	2.0	333.0807	2 Opp (Main)	Interior	831.6	PASS
6B02	1.34	1	0	0.23	0.3	2.0	313.3753	2 Opp (Main)	Interior	831.6	PASS
6B03	1.34	1	0	0.23	0.3	2.0	311.5179	2 Opp (Main)	Interior	831.6	PASS
6B04	1.69	1	0	0.25	0.3	2.0	272.7292	2 Opp (Main)	Interior	831.6	PASS
6B05	1.69	1	0	0.19	0.3	2.0	254.3103	2 Opp (Main)	Interior	831.6	PASS
6B06	1.69	1	0	0.13	0.2	2.0	245.2144	2 Opp (Main)	Interior	831.6	PASS
6B07	1.98	1	0	0.07	0.1	2.0	261.3937	2 Opp (Main)	Interior	831.6	PASS
6B08	1.11	0.5	0.5	0.07	0.1	2.0	188.6734	2 Opp (Roof)	Interior	665.2	PASS
6C01	1.34	1	0	0.17	0.3	2.0	326.5037	2 Opp (Main)	Interior	831.6	PASS
6C02	1.34	1	0	0.20	0.3	2.0	318.7418	2 Opp (Main)	Interior	831.6	PASS
6C03	1.34	1	0	0.18	0.3	2.0	311.566	2 Opp (Main)	Interior	831.6	PASS
6C04	1.69	1	0	0.19	0.3	2.0	272.9306	2 Opp (Main)	Interior	831.6	PASS
6C05	1.69	1	0	0.16	0.3	2.0	254.0675	2 Opp (Main)	Interior	831.6	PASS
6C06	1.69	1	0	0.11	0.2	2.0	245.2359	2 Opp (Main)	Interior	831.6	PASS
6C07	1.98	1	0	0.07	0.1	2.0	261.3978	2 Opp (Main)	Interior	831.6	PASS
6C08	1.11	0.5	0.5	0.06	0.1	2.0	188.6742	2 Opp (Roof)	Interior	665.2	PASS
6D01	1.34	1	0	0.19	0.3	2.0	323.7525	2 Opp (Main)	Interior	831.6	PASS
6D02	1.34	1	0	0.21	0.3	2.0	313.2984	2 Opp (Main)	Interior	831.6	PASS
6D03	1.34	1	0	0.20	0.3	2.0	311.5487	2 Opp (Main)	Interior	831.6	PASS
6D04	1.69	1	0	0.19	0.3	2.0	272.5051	2 Opp (Main)	Interior	831.6	PASS
6D05	1.69	1	0	0.16	0.3	2.0	253.9668	2 Opp (Main)	Interior	831.6	PASS
6D06	1.69	1	0	0.11	0.2	2.0	245.2365	2 Opp (Main)	Interior	831.6	PASS
6D07	1.98	1	0	0.07	0.1	2.0	261.3988	2 Opp (Main)	Interior	831.6	PASS
6D08	1.11	0.5	0.5	0.06	0.1	2.0	188.677	2 Opp (Roof)	Interior	665.2	PASS
6E01	1.34	1	0	0.19	0.3	2.0	341.3792	2 Opp (Main)	Interior	831.6	PASS
6E02	1.34	1	0	0.21	0.3	2.0	323.8724	2 Opp (Main)	Interior	831.6	PASS
6E03	1.34	1	0	0.20	0.3	2.0	319.0258	2 Opp (Main)	Interior	831.6	PASS
6E04	1.69	1	0	0.21	0.3	2.0	290.0861	2 Opp (Main)	Interior	831.6	PASS
6E05	1.69	1	0	0.17	0.3	2.0	267.4987	2 Opp (Main)	Interior	831.6	PASS
6E06	1.69	1	0	0.12	0.2	2.0	245.2275	2 Opp (Main)	Interior	831.6	PASS
6E07	1.98	1	0	0.07	0.1	2.0	261.3949	2 Opp (Main)	Interior	831.6	PASS
6E08	1.11	0.5	0.5	0.07	0.1	2.0	188.6722	2 Opp (Roof)	Interior	665.2	PASS
6F01	2.42	1	0	0.19	0.2	2.0	413.5835	Other (Main)	Exterior	665.2	PASS
6F02	2.42	1	0	0.20	0.2	2.0	396.901	Other (Main)	Exterior	665.2	PASS
6F03	2.42	1	0	0.20	0.2	2.0	396.9106	Other (Main)	Exterior	665.2	PASS
6F04	3.05	1	0	0.22	0.2	2.0	313.2663	Other (Main)	Exterior	665.2	PASS
6F05	3.05	1	0	0.17	0.2	2.0	313.3451	Other (Main)	Exterior	665.2	PASS
6F06	3.05	1	0	0.11	0.1	2.0	313.4284	Other (Main)	Exterior	665.2	PASS
6F07	3.05	1	0	0.06	0.1	2.0	313.5078	Other (Main)	Exterior	665.2	PASS
6F08	1.53	1	0	0.05	0.1	2.0	271.6291	Other (Roof)	Exterior	443.5	PASS

**Section 5a: LSP & NSP Shear Wall Assessment (IO)**

## Typical Wall Properties

$f'_c$	5000	psi, Lower Bound Concrete Compressive Strength
$f_y$	60	ksi, Lower Bound Reinforcing Steel Yield Strength
$f_u$	90	ksi, Lower Bound Reinforcing Steel Ultimate Strength
$f'_c$	7500	psi, Expected Concrete Compressive Strength
$f_y$	75	ksi, Expected Reinforcing Steel Yield Strength
$f_u$	112.5	ksi, Expected Reinforcing Steel Ultimate Strength
$t_w$	16	in, Shear Wall Thickness
$h_w$	106	ft, Shear Wall Height
$l_w$	22.33	ft, Shear wall Length
Slender	Wall Slenderness (ASCE 41-06 Section C6.7.1)	
$w$	28	in, Boundary Element Width
$A_g$	4960	kips, Nominal Axial Capacity (No Eccentricity, Concrete Only)
$\kappa$	1.0	Knowledge Factor
$C_1$	1.0	
$C_2$	1.0	

**Exterior Shear Wall**

Level	Elev (ft)	$h_{FE}$ (ft)	Horiz Reinf		Vert Reinf		Boundary Element		Effectiveness																		
			$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Size	Spc (in)	Size	Spc (in)	$A_{st}^1$ (in <sup>2</sup> )	Size	Spc (in)	Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	$P > .35P_o^2$	$s_h > 18"l^2$	$s_v > 18"l^2$	$M_{N,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{N,Y}$ (kips)	Control <sup>7</sup>	$J^*P_{DES,MAX}$ (kips)	$J^*P_{DES,MIN}$ (kips)	$M_{DES/m}$ (k-ft)
Level 08	106	13	111.7	80.6	5402.2	#5	18.0	#5	18.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	23270.4	415.6	151.3	1088.9	Flexure	111.7	80.6	2701.1
Level 07	93	13	248.7	165.7	15632.6	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	787.0	151.3	1158.1	Flexure	248.7	165.7	7816.3
Level 06	80	13	385.8	250.8	29830.9	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1092.2	151.3	1158.1	Flexure	385.8	250.8	14915.4
Level 05	67	13	522.8	336.0	47178.0	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1334.4	151.3	1158.1	Flexure	522.8	336.0	23589.0
Level 04	54	13	659.9	421.1	66902.3	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	1517.2	151.3	1158.1	Flexure	659.9	421.1	33451.1
Level 03	41	13	796.9	506.2	88286.6	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	1645.0	151.3	1158.1	Flexure	796.9	506.2	44143.3
Level 02	28	13	934.0	591.3	110680.8	#5	15.0	#5	15.0	20.32	#5	4.0	YES	18880.9	28062.3	0.673	NO	NO	NO	52017.3	1722.6	151.3	1158.1	Flexure	934.0	591.3	55396.4
Level 01	15	15	1074.6	679.4	137046.6	#5	10.0	#5	10.0	29.32	#5	4.0	YES	19307.4	28587.3	0.675	NO	NO	NO	65401.3	1757.7	174.6	1365.8	Flexure	1074.6	679.4	81212.8

**Interior Shear Wall**

Level	Elev (ft)	$h_{FE}$ (ft)	Horiz Reinf		Vert Reinf		Boundary Element		Effectiveness																		
			$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Size	Spc (in)	Size	Spc (in)	$A_{st}^1$ (in <sup>2</sup> )	Size	Spc (in)	Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	$P > .35P_o^2$	$s_h > 18"l^2$	$s_v > 18"l^2$	$M_{N,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{N,Y}$ (kips)	Control <sup>7</sup>	$J^*P_{DES,MAX}$ (kips)	$J^*P_{DES,MIN}$ (kips)	$M_{DES/m}$ (k-ft)
Level 08	106	13	190.2	134.1	4986.2	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	383.6	89.8	1088.9	Flexure	190.2	134.1	2493.1
Level 07	93	13	426.7	273.5	14428.9	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	726.4	89.8	1088.9	Flexure	426.7	273.5	7214.5
Level 06	80	13	663.1	413.0	27533.9	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1008.1	89.8	1158.1	Flexure	663.1	413.0	13767.0
Level 05	67	13	899.6	552.4	43545.3	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1231.6	89.8	1158.1	Flexure	899.6	552.4	21772.7
Level 04	54	13	1136.0	691.9	61750.8	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1400.4	89.8	1158.1	Flexure	1136.0	691.9	30875.4
Level 03	41	13	1372.5	831.3	81488.6	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1518.3	89.8	1158.1	Flexure	1372.5	831.3	40744.3
Level 02	28	13	1608.9	970.8	102158.4	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1590.0	89.8	1158.1	Flexure	1608.9	970.8	51255.4
Level 01	15	15	1849.0	1113.2	126494.1	#5	15.0	#5	10.0	15.24	#5	5.0	YES	18640.2	27766.0	0.671	NO	NO	NO	38815.0	1622.4	103.6	1158.1	Flexure	1849.0	1113.2	74959.4

- Area of Longitudinal Steel in Boundary Elements
- If "NO", Shear Wall may not be taken as effective in resisting seismic forces per ASCE 41-06 Section 6.7.1.1
- Nominal Moment Capacity taken from PCA Column Analysis corresponding to minimum design Axial Load
- Nominal Moment Capacity at P=0 taken from PCA Column Analysis, including only boundary reinforcing.  $\Phi = 1.0$ , and expected strengths are used.
- Nominal Moment Capacity at P=0 is not required to exceed Moment Demand, as the compressive force is assumed to be 0 and therefore no interaction effects are taken into account.
- Maximum shear force calculated based on nominal flexural capacity of base of wall.
- Wall is considered controlled by flexure if the maximum shear force value is less than the nominal shear capacity. This indicates that the wall will fail in flexure prior to reaching the shear capacity.

Linear Static Procedure					NSP (IO, LS, and CP)							
DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Axial-Flexural Acceptance		Shear Acceptance	IO (rad)	LS Hinge Rotation (rad)		CP (rad)	IO	LS Hinge Acceptance		CP
0.111	0.112	PASS	PASS	PASS	0	0	0	0	PASS	PASS	PASS	
0.283	0.291	PASS	PASS	PASS	0	0	0	0	PASS	PASS	PASS	
0.514	0.539	PASS	PASS	PASS	0	0	0	0	PASS	PASS	PASS	
0.776	0.827	PASS	PASS	PASS	0	0.000628	0.001557	0	PASS	PASS	PASS	
0.715	0.752	PASS	PASS	PASS	0	0	0	0	PASS	PASS	PASS	
0.918	0.975	PASS	PASS	PASS	0.002879	0.005502	0.007161	0	PASS	PASS	PASS	
0.910	0.961	PASS	PASS	PASS	0	0.001483	0.003534	0	PASS	PASS	PASS	
1.075	1.130	FAIL	FAIL	PASS	0	0	0	0	PASS	PASS	PASS	
DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Axial-Flexural Acceptance		Shear Acceptance	IO (rad)	LS Hinge Rotation (rad)		CP (rad)	IO	LS Hinge Acceptance		CP
0.074	0.075	PASS	PASS	PASS	0	0	0	0	PASS	PASS	PASS	
0.199	0.208	PASS	PASS	PASS	0	0	0	0	PASS	PASS	PASS	
0.341	0.363	PASS	PASS	PASS	0	0	0	0	PASS	PASS	PASS	
0.510	0.554	PASS	PASS	PASS	0	0	0	0	PASS	PASS	PASS	
0.686	0.759	PASS	PASS	PASS	0	0	0	0	PASS	PASS	PASS	
0.862	0.969	PASS	PASS	PASS	0	0	0.00046	0	PASS	PASS	PASS	
1.035	1.181	FAIL	FAIL	PASS	0.00221	0.003867	0.005177	0	PASS	PASS	PASS	
1.325	1.511	FAIL	FAIL	PASS	0.000671	0.002613	0.004256	0	PASS	PASS	PASS	

**Section 5b: LSP Shear Wall Assessment (LS)**

Typical Wall Properties

$f'_c$	5000	psi, Lower Bound Concrete Compressive Strength
$f_y$	60	ksi, Lower Bound Reinforcing Steel Yield Strength
$f_u$	90	ksi, Lower Bound Reinforcing Steel Ultimate Strength
$f'_c$	7500	psi, Expected Concrete Compressive Strength
$f_y$	75	ksi, Expected Reinforcing Steel Yield Strength
$f_u$	112.5	ksi, Expected Reinforcing Steel Ultimate Strength
$t_w$	16	in, Shear Wall Thickness
$h_w$	106	ft, Shear Wall Height
$l_w$	22.33	ft, Shear wall Length
Slender	Wall Slenderness (ASCE 41-06 Section C6.7.1)	
$w$	28	in, Boundary Element Width
$A_g$	4960	kips, Nominal Axial Capacity (No Eccentricity, Concrete Only)
$k$	1.0	Knowledge Factor
$C_1$	1.0	
$C_2$	1.0	

**Exterior Shear Wall**

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{n,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)	Control <sup>7</sup>	$J^*P_{DES,MAX}$ (kips)	$J^*P_{DES,MIN}$ (kips)	
						Size	Spc (in)	Size	Spc (in)	$A_{st}^{-1}$ (in <sup>2</sup> )	Size					Spc (in)	$P > .35P_o^{*2}$	$s_h > 18''^{*2}$								$s_v > 18''^{*2}$
Level 08	106	13	111.7	80.6	9402.6	#5	18.0	#5	18.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	23270.4	723.3	151.3	1088.9	Flexure	223.4	161.2
Level 07	93	13	248.7	165.7	27215.5	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1370.2	151.3	1158.1	Flexure	497.5	331.4
Level 06	80	13	385.8	250.8	51944.1	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1902.2	151.3	1158.1	Flexure	771.6	501.7
Level 05	67	13	522.8	336.0	82164.4	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	2324.6	151.3	1158.1	Flexure	1045.7	671.9
Level 04	54	13	659.9	421.1	116532.0	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	2643.7	151.3	1158.1	Flexure	1319.7	842.2
Level 03	41	13	796.9	506.2	153796.0	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	2866.5	151.3	1158.1	Flexure	1593.8	1012.4
Level 02	28	13	934.0	591.3	192820.5	#5	15.0	#5	15.0	20.32	#5	4.0	YES	18880.9	28062.3	0.673	NO	NO	NO	52017.3	3001.9	151.3	1158.1	Flexure	1867.9	1182.7
Level 01	15	15	1074.6	679.4	238762.9	#5	10.0	#5	10.0	29.32	#5	4.0	YES	19307.4	28587.3	0.675	NO	NO	NO	65401.3	3062.8	174.6	1365.8	Flexure	2149.2	1358.8

**Interior Shear Wall**

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{n,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)	Control <sup>7</sup>	$J^*P_{DES,MAX}$ (kips)	$J^*P_{DES,MIN}$ (kips)	
						Size	Spc (in)	Size	Spc (in)	$A_{st}^{-1}$ (in <sup>2</sup> )	Size					Spc (in)	$P > .35P_o^{*2}$	$s_h > 18''^{*2}$								$s_v > 18''^{*2}$
Level 08	106	13	190.2	134.1	8699.5	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	669.2	89.8	1088.9	Flexure	380.5	268.1
Level 07	93	13	426.7	273.5	25174.2	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	1267.3	89.8	1088.9	Flexure	853.4	547.0
Level 06	80	13	663.1	413.0	48038.6	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1758.8	89.8	1158.1	Flexure	1326.3	825.9
Level 05	67	13	899.6	552.4	75973.8	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	2148.9	89.8	1158.1	Flexure	1799.2	1104.9
Level 04	54	13	1136.0	691.9	107737.0	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	2443.3	89.8	1158.1	Flexure	2272.1	1383.8
Level 03	41	13	1372.5	831.3	142173.6	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	2649.0	89.8	1158.1	Flexure	2745.0	1662.7
Level 02	28	13	1608.9	970.8	178236.4	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	2774.1	89.8	1158.1	Flexure	3217.9	1941.6
Level 01	15	15	1849.0	1113.2	220695.0	#5	15.0	#5	10.0	15.24	#5	5.0	YES	18640.2	27766.0	0.671	NO	NO	NO	38815.0	2830.6	103.6	1158.1	Flexure	3698.0	2226.4

- Area of Longitudinal Steel in Boundary Elements
- If "NO", Shear Wall may not be taken as effective in resisting seismic forces per ASCE 41-06 Section 6.7.1.1
- Nominal Moment Capacity taken from PCA Column Analysis corresponding to minimum design Axial Load
- Nominal Moment Capacity at P=0 taken from PCA Column Analysis, including only boundary reinforcing.  $\Phi = 1.0$ , and expected strengths are used.
- Nominal Moment Capacity at P=0 is not required to exceed Moment Demand, as the compressive force is assumed to be 0 and therefore no interaction effects are taken into account.
- Maximum shear force calculated based on nominal flexural capacity of base of wall.
- Wall is considered controlled by flexure if the maximum shear force value is less than the nominal shear capacity. This indicates that the wall will fail in flexure prior to reaching the shear capacity.





M <sub>DES</sub> /m (k-ft)	Axial-Flexure		Axial-Flexural		Shear
	DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Acceptance		Acceptance
2350.6	0.092	0.094	PASS	PASS	PASS
7276.0	0.241	0.255	PASS	PASS	PASS
17314.7	0.527	0.574	PASS	PASS	PASS
27388.1	0.770	0.859	PASS	PASS	PASS
38844.0	0.733	0.801	PASS	PASS	PASS
51265.3	0.923	1.022	PASS	FAIL	PASS
64334.5	0.926	1.018	PASS	FAIL	PASS
79587.6	0.934	1.018	PASS	FAIL	PASS
M <sub>DES</sub> /m (k-ft)	DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Axial-Flexural		Shear
2174.9	0.061	0.063	PASS	PASS	PASS
6438.1	0.159	0.172	PASS	PASS	PASS
15497.9	0.331	0.369	PASS	PASS	PASS
25324.6	0.494	0.567	PASS	PASS	PASS
35912.3	0.645	0.758	PASS	PASS	PASS
47391.2	0.790	0.948	PASS	PASS	PASS
59620.6	0.929	1.133	PASS	FAIL	PASS
73565.0	1.005	1.224	FAIL	FAIL	PASS



## Section 5c: LSP Shear Wall Assessment (CP)

## Typical Wall Properties

$f'_c$	5000	psi, Lower Bound Concrete Compressive Strength
$f_y$	60	ksi, Lower Bound Reinforcing Steel Yield Strength
$f_u$	90	ksi, Lower Bound Reinforcing Steel Ultimate Strength
$f'_c$	7500	psi, Expected Concrete Compressive Strength
$f_y$	75	ksi, Expected Reinforcing Steel Yield Strength
$f_u$	112.5	ksi, Expected Reinforcing Steel Ultimate Strength
$t_w$	16	in, Shear Wall Thickness
$h_w$	106	ft, Shear Wall Height
$l_w$	22.33	ft, Shear wall Length
Slender		Wall Slenderness (ASCE 41-06 Section C6.7.1)
$w$	28	in, Boundary Element Width
$A_g$	4960	kips, Nominal Axial Capacity (No Eccentricity, Concrete Only)
$\kappa$	1.0	Knowledge Factor
$C_1$	1.0	
$C_2$	1.0	

## Exterior Shear Wall

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{h,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{h,V}$ (kips)	Control <sup>7</sup>	$J^*P_{DES,MAX}$ (kips)	$J^*P_{DES,MIN}$ (kips)
						Size	Spc (in)	Size	Spc (in)	Size	Spc (in)					$A_{se}^1$ (in <sup>2</sup> )	$P > .35P_o^{2,2}$	$s_h > 18''^{2,2}$							
Level 08	106	13	111.7	80.6	13222.0	#5	18.0	#5	18.0	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	23270.4	1017.1	151.3	1088.9	Flexure	223.4	161.2
Level 07	93	13	248.7	165.7	38270.6	#5	15.0	#5	15.0	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1926.8	151.3	1158.1	Flexure	497.5	331.4
Level 06	80	13	385.8	250.8	73044.2	#5	15.0	#5	15.0	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	2674.9	151.3	1158.1	Flexure	771.6	501.7
Level 05	67	13	522.8	336.0	115540.2	#5	15.0	#5	15.0	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	3268.9	151.3	1158.1	Flexure	1045.7	671.9
Level 04	54	13	659.9	421.1	163868.1	#5	15.0	#5	15.0	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	3717.5	151.3	1158.1	Flexure	1319.7	842.2
Level 03	41	13	796.9	506.2	216269.0	#5	15.0	#5	15.0	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	4030.8	151.3	1158.1	Flexure	1593.8	1012.4
Level 02	28	13	934.0	591.3	271145.4	#5	15.0	#5	15.0	#5	4.0	YES	18880.9	28062.3	0.673	NO	NO	NO	52017.3	4221.3	151.3	1158.1	Flexure	1867.9	1182.7
Level 01	15	15	1074.6	679.4	335750.0	#5	10.0	#5	10.0	#5	4.0	YES	19307.4	28587.3	0.675	NO	NO	NO	65401.3	4307.0	174.6	1365.8	Flexure	2149.2	1358.8

## Interior Shear Wall

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{h,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{h,V}$ (kips)	Control <sup>7</sup>	$J^*P_{DES,MAX}$ (kips)	$J^*P_{DES,MIN}$ (kips)
						Size	Spc (in)	Size	Spc (in)	Size	Spc (in)					$A_{se}^1$ (in <sup>2</sup> )	$P > .35P_o^{2,2}$	$s_h > 18''^{2,2}$							
Level 08	106	13	190.2	134.1	12233.3	#5	18.0	#5	18.0	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	941.0	89.8	1088.9	Flexure	380.5	268.1
Level 07	93	13	426.7	273.5	35400.2	#5	18.0	#5	18.0	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	1782.1	89.8	1088.9	Flexure	853.4	547.0
Level 06	80	13	663.1	413.0	67552.2	#5	15.0	#5	15.0	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	2473.2	89.8	1158.1	Flexure	1326.3	825.9
Level 05	67	13	899.6	552.4	106834.9	#5	15.0	#5	15.0	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	3021.7	89.8	1158.1	Flexure	1799.2	1104.9
Level 04	54	13	1136.0	691.9	151500.6	#5	15.0	#5	15.0	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	3435.8	89.8	1158.1	Flexure	2272.1	1383.8
Level 03	41	13	1372.5	831.3	199925.5	#5	15.0	#5	15.0	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	3725.0	89.8	1158.1	Flexure	2745.0	1662.7
Level 02	28	13	1608.9	970.8	250637.2	#5	15.0	#5	15.0	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	3900.9	89.8	1158.1	Flexure	3217.9	1941.6
Level 01	15	15	1849.0	1113.2	310342.8	#5	15.0	#5	10.0	#5	5.0	YES	18640.2	27766.0	0.671	NO	NO	NO	38815.0	3980.4	103.6	1158.1	Flexure	3698.0	2226.4

1. Area of Longitudinal Steel in Boundary Elements

2. If "NO", Shear Wall may not be taken as effective in resisting seismic forces per ASCE 41-06 Section 6.7.1.1

3. Nominal Moment Capacity taken from PCA Column Analysis corresponding to minimum design Axial Load

4. Nominal Moment Capacity at P=0 taken from PCA Column Analysis, including only boundary reinforcing.  $\Phi = 1.0$ , and expected strengths are used.

5. Nominal Moment Capacity at P=0 is not required to exceed Moment Demand, as the compressive force is assumed to be 0 and therefore no interaction effects are taken into account.

6. Maximum shear force calculated based on nominal flexural capacity of base of wall.

7. Wall is considered controlled by flexure if the maximum shear force value is less than the nominal shear capacity. This indicates that the wall will fail in flexure prior to reaching the shear capacity.



M <sub>DES</sub> /m (k-ft)	Axial-Flexure		Axial-Flexural		Shear
	DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Acceptance		Acceptance
2203.7	0.086	0.088	PASS	PASS	PASS
9567.6	0.317	0.336	PASS	PASS	PASS
18261.0	0.556	0.605	PASS	PASS	PASS
28885.0	0.812	0.907	PASS	PASS	PASS
40967.0	0.773	0.845	PASS	PASS	PASS
54067.2	0.974	1.079	PASS	FAIL	PASS
67858.8	0.978	1.074	PASS	FAIL	PASS
81393.9	0.934	1.018	PASS	FAIL	PASS
M <sub>DES</sub> /m (k-ft)	DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Axial-Flexural		Shear
2038.9	0.057	0.059	PASS	PASS	PASS
8586.8	0.212	0.229	PASS	PASS	PASS
16888.0	0.361	0.402	PASS	PASS	PASS
26708.7	0.521	0.597	PASS	PASS	PASS
37875.1	0.680	0.799	PASS	PASS	PASS
49981.4	0.833	0.999	PASS	PASS	PASS
62906.8	0.979	1.195	PASS	FAIL	PASS
75234.6	1.029	1.252	FAIL	FAIL	PASS



Beam Flexure Acceptance												Beam Shear Acceptance			
M+				M-				J		kQ <sub>CL</sub>		Left		Right	
Middle	Right	Left	Right	Q <sub>0, left</sub>	Q <sub>0, right</sub>	Q <sub>E, left</sub>	Q <sub>E, right</sub>	J <sub>left</sub>	J <sub>right</sub>	kQ <sub>CL, left</sub>	kQ <sub>CL, right</sub>	Left	Right	Left	Right
				(k)	(k)	(k)	(k)			(k)	(k)				
PASS	PASS	PASS	PASS	25.4	26.6	97.3	97.3	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	26.0	25.9	91.6	91.6	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	26.0	26.0	92.7	92.7	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.9	26.0	91.6	91.6	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	26.6	25.4	97.3	97.3	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.8	96.5	96.5	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.8	93.6	93.6	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.7	95.0	95.0	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.8	25.7	93.6	93.6	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.8	25.7	96.5	96.5	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.8	25.7	89.9	89.9	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.8	86.5	86.5	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.7	87.9	87.9	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.8	25.7	86.5	86.5	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.8	89.9	89.9	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	26.0	25.5	81.9	81.9	1.00	1.00	151.4	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.8	77.7	77.7	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.7	77.7	77.7	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	25.8	25.7	77.7	77.7	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	25.5	26.0	81.9	81.9	1.00	1.00	151.7	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	26.1	25.3	71.0	71.0	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.8	67.9	67.9	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.7	67.9	67.9	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	25.8	25.7	67.9	67.9	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	25.3	26.1	71.0	71.0	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	26.2	25.3	58.8	58.8	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.8	56.5	56.5	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.7	56.5	56.5	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	25.8	25.7	56.5	56.5	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	25.3	26.2	58.8	58.8	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	26.4	25.1	43.5	43.5	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.8	42.3	42.3	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.7	25.7	42.3	42.3	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.8	25.7	42.3	42.3	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	25.1	26.4	43.5	43.5	1.00	1.00	151.4	151.4	PASS	PASS		
PASS	PASS	PASS	PASS	21.2	20.8	24.6	24.6	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	20.9	21.0	23.5	23.5	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	21.0	21.0	23.4	23.4	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	21.0	20.9	23.5	23.5	1.00	1.00	151.7	151.7	PASS	PASS		
PASS	PASS	PASS	PASS	20.8	21.2	24.6	24.6	1.00	1.00	151.7	151.7	PASS	PASS		



Q <sub>G,left</sub> (k)	Q <sub>G,right</sub> (k)	Q <sub>L,left</sub> (k)	Q <sub>L,right</sub> (k)	J <sub>left</sub>	J <sub>right</sub>	KQ <sub>CL,left</sub> (k)	KQ <sub>CL,right</sub> (k)	Beam Shear Acceptance	
								Left	Right
25.4	26.6	169.8	169.8	2.00	2.00	151.4	151.4	PASS	PASS
26.0	25.9	159.8	159.8	2.00	2.00	151.4	151.4	PASS	PASS
26.0	26.0	161.7	161.7	2.00	2.00	151.4	151.4	PASS	PASS
25.9	26.0	159.8	159.8	2.00	2.00	151.4	151.4	PASS	PASS
26.6	25.4	169.8	169.8	2.00	2.00	151.4	151.4	PASS	PASS
25.7	25.8	168.3	168.3	2.00	2.00	151.4	151.4	PASS	PASS
25.7	25.8	163.3	163.3	2.00	2.00	151.4	151.4	PASS	PASS
25.7	25.7	165.7	165.7	2.00	2.00	151.4	151.4	PASS	PASS
25.8	25.7	163.3	163.3	2.00	2.00	151.4	151.4	PASS	PASS
25.8	25.7	168.3	168.3	2.00	2.00	151.4	151.4	PASS	PASS
25.8	25.7	156.9	156.9	2.00	2.00	151.4	151.4	PASS	PASS
25.7	25.8	150.9	150.9	2.00	2.00	151.4	151.4	PASS	PASS
25.7	25.7	153.3	153.3	2.00	2.00	151.4	151.4	PASS	PASS
25.8	25.7	150.9	150.9	2.00	2.00	151.4	151.4	PASS	PASS
25.7	25.8	156.9	156.9	2.00	2.00	151.4	151.4	PASS	PASS
26.0	25.5	142.8	142.8	2.00	2.00	151.4	151.7	PASS	PASS
25.7	25.8	135.6	135.6	2.00	2.00	151.7	151.7	PASS	PASS
25.7	25.7	135.6	135.6	2.00	2.00	151.7	151.7	PASS	PASS
25.8	25.7	135.6	135.6	2.00	2.00	151.7	151.7	PASS	PASS
25.5	26.0	142.8	142.8	2.00	2.00	151.7	151.4	PASS	PASS
26.1	25.3	123.8	123.8	2.00	2.00	151.7	151.7	PASS	PASS
25.7	25.8	118.4	118.4	2.00	2.00	151.7	151.7	PASS	PASS
25.7	25.7	118.5	118.5	2.00	2.00	151.7	151.7	PASS	PASS
25.8	25.7	118.4	118.4	2.00	2.00	151.7	151.7	PASS	PASS
25.3	26.1	123.8	123.8	2.00	2.00	151.7	151.7	PASS	PASS
26.2	25.3	102.6	102.6	2.00	2.00	151.7	151.7	PASS	PASS
25.7	25.8	98.5	98.5	2.00	2.00	151.7	151.7	PASS	PASS
25.7	25.7	98.5	98.5	2.00	2.00	151.7	151.7	PASS	PASS
25.8	25.7	98.5	98.5	2.00	2.00	151.7	151.7	PASS	PASS
25.3	26.2	102.6	102.6	2.00	2.00	151.7	151.7	PASS	PASS
26.4	25.1	75.7	75.7	2.00	2.00	151.4	151.4	PASS	PASS
25.7	25.8	73.8	73.8	2.00	2.00	151.4	151.4	PASS	PASS
25.7	25.7	73.7	73.7	2.00	2.00	151.4	151.4	PASS	PASS
25.8	25.7	73.8	73.8	2.00	2.00	151.4	151.4	PASS	PASS
25.1	26.4	75.7	75.7	2.00	2.00	151.4	151.4	PASS	PASS
21.2	20.8	42.8	42.8	2.00	2.00	151.7	151.7	PASS	PASS
20.9	21.0	40.9	40.9	2.00	2.00	151.7	151.7	PASS	PASS
21.0	21.0	40.8	40.8	2.00	2.00	151.7	151.7	PASS	PASS
21.0	20.9	40.9	40.9	2.00	2.00	151.7	151.7	PASS	PASS
20.8	21.2	42.8	42.8	2.00	2.00	151.7	151.7	PASS	PASS



Section 6c: (Response Spectrum) LDP Moment Frame Beam Assessment (CP)

Typical Beam Properties

Table listing beam properties: b\_w (24 in), h (32 in), b\_e (27 in), beta\_1 (0.8), c (1.5 in), #4 (3), f'\_c (5000 psi), f\_y (60 ksi), f\_u (90 ksi), f'\_c (7500 psi), f\_y (75 ksi), f\_u (113 ksi), k (1), C\_1 (1), C\_2 (1).

Parameters Below Assume that 0 < rho / rho\_min < 5, and Transverse Reinforcing Conforms. Check these values prior to using this portion of the table

Main assessment table with columns for BEAM ID, Existing Beam Reinforcing (M+, M-), VDES (left, middle, right), S\_eL, S\_eR, Conform? (left, middle, right), M+ (m\_left, m\_middle, m\_right), M- (m\_left, m\_middle, m\_right), and Beam Flexure Acc (Left, Middle, Right).



Acceptance		M										Beam Shear Acceptance	
Left	Right	Q <sub>u,left</sub>	Q <sub>u,right</sub>	Q <sub>u,left</sub>	Q <sub>u,right</sub>	J <sub>left</sub>	J <sub>right</sub>	KQ <sub>u,left</sub>	KQ <sub>u,right</sub>	Left	Right		
		(k)	(k)	(k)	(k)			(k)	(k)				
FAIL	FAIL	25.4	26.6	238.6	238.6	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	26.0	25.9	224.6	224.6	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	26.0	26.0	227.2	227.2	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	25.9	26.0	224.6	224.6	2.00	2.00	151.4	151.4	PASS	PASS		
FAIL	FAIL	26.6	25.4	238.6	238.6	2.00	2.00	151.4	151.4	PASS	PASS		
FAIL	FAIL	25.7	25.8	236.7	236.7	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	25.7	25.8	229.6	229.6	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	25.7	25.7	232.9	232.9	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	25.8	25.7	229.6	229.6	2.00	2.00	151.4	151.4	PASS	PASS		
FAIL	FAIL	25.8	25.7	236.7	236.7	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	25.8	25.7	220.4	220.4	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	25.7	25.8	212.0	212.0	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	25.7	25.7	215.4	215.4	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	25.8	25.7	212.0	212.0	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	25.7	25.8	220.4	220.4	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	26.0	25.5	200.6	200.6	2.00	2.00	151.4	151.7	PASS	PASS		
PASS	PASS	25.7	25.8	190.4	190.4	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	25.7	25.7	190.5	190.5	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	25.8	25.7	190.4	190.4	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	25.5	26.0	200.6	200.6	2.00	2.00	151.7	151.4	PASS	PASS		
PASS	PASS	26.1	25.3	174.0	174.0	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	25.7	25.8	166.5	166.5	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	25.7	25.7	166.5	166.5	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	25.8	25.7	166.5	166.5	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	25.3	26.1	174.0	174.0	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	26.2	25.3	144.1	144.1	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	25.7	25.8	138.4	138.4	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	25.7	25.7	138.3	138.3	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	25.8	25.7	138.4	138.4	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	25.3	26.2	144.1	144.1	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	26.4	25.1	106.0	106.0	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	25.7	25.8	103.3	103.3	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	25.7	25.7	103.1	103.1	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	25.8	25.7	103.3	103.3	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	25.1	26.4	106.0	106.0	2.00	2.00	151.4	151.4	PASS	PASS		
PASS	PASS	21.2	20.8	59.6	59.6	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	20.9	21.0	57.0	57.0	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	21.0	21.0	56.8	56.8	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	21.0	20.9	57.0	57.0	2.00	2.00	151.7	151.7	PASS	PASS		
PASS	PASS	20.8	21.2	59.6	59.6	2.00	2.00	151.7	151.7	PASS	PASS		



Linear Dynamic Procedure													
Flexure-Axial Acceptance													
Bottom	Top	Bottom	Top	Bottom	Top	Bottom				Shear		Yield Zone Shear	Non-Yield Zone
DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	DCR <sub>MIN</sub>	P <sub>max</sub>	P <sub>max</sub>	P <sub>min</sub>	P <sub>min</sub>			J	Q <sub>g</sub>	Q <sub>g</sub>	Acceptance	Shear Acceptance
							kQ <sub>CL,yield</sub>	kQ <sub>CL,non-yield</sub>		(k)	(k)		
0.610	0.579	1.536	PASS	PASS	PASS	FAIL	408.0	408.0	1.0	6.00	161.7	PASS	PASS
0.283	0.675	0.553	PASS	PASS	PASS	PASS	408.0	408.0	1.0	11.39	121.2	PASS	PASS
0.271	0.598	0.475	PASS	PASS	PASS	PASS	408.0	408.0	1.0	10.18	114.0	PASS	PASS
0.268	0.544	0.406	PASS	PASS	PASS	PASS	408.0	408.0	1.0	10.79	108.2	PASS	PASS
0.243	0.476	0.320	PASS	PASS	PASS	PASS	408.0	408.0	1.0	10.78	94.7	PASS	PASS
0.230	0.411	0.255	PASS	PASS	PASS	PASS	408.0	408.0	1.0	11.18	81.5	PASS	PASS
0.207	0.357	0.191	PASS	PASS	PASS	PASS	408.0	408.0	1.0	10.35	68.5	PASS	PASS
0.135	0.262	0.070	PASS	PASS	PASS	PASS	408.0	408.0	1.0	13.81	38.1	PASS	PASS
0.668	0.350	0.711	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.16	238.5	PASS	PASS
0.424	0.444	0.450	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.02	235.0	PASS	PASS
0.374	0.426	0.396	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.32	210.6	PASS	PASS
0.357	0.411	0.380	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.45	196.6	PASS	PASS
0.320	0.383	0.340	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.63	173.9	PASS	PASS
0.285	0.346	0.297	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.71	149.2	PASS	PASS
0.232	0.298	0.237	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.85	119.8	PASS	PASS
0.143	0.230	0.142	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.94	79.6	PASS	PASS
0.685	0.340	0.724	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.02	240.1	PASS	PASS
0.427	0.441	0.450	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.00	234.8	PASS	PASS
0.387	0.440	0.407	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.01	218.1	PASS	PASS
0.342	0.404	0.363	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.01	191.4	PASS	PASS
0.316	0.377	0.336	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.01	172.3	PASS	PASS
0.281	0.341	0.294	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.01	148.0	PASS	PASS
0.229	0.295	0.236	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.02	119.4	PASS	PASS
0.140	0.227	0.142	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.02	79.3	PASS	PASS
0.685	0.340	0.724	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.01	240.1	PASS	PASS
0.427	0.441	0.450	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.00	234.8	PASS	PASS
0.387	0.440	0.407	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.01	218.1	PASS	PASS
0.342	0.404	0.363	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.01	191.4	PASS	PASS
0.317	0.377	0.336	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.02	172.3	PASS	PASS
0.281	0.342	0.294	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.02	148.0	PASS	PASS
0.229	0.296	0.236	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.02	119.4	PASS	PASS
0.140	0.227	0.142	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.04	79.3	PASS	PASS
0.668	0.351	0.710	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.24	238.5	PASS	PASS
0.425	0.443	0.450	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.03	235.0	PASS	PASS
0.373	0.425	0.397	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.19	210.6	PASS	PASS
0.355	0.410	0.381	PASS	PASS	PASS	PASS	469.5	375.6	1.0	0.27	196.6	PASS	PASS
0.318	0.381	0.342	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.39	173.9	PASS	PASS
0.282	0.344	0.299	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.43	149.2	PASS	PASS
0.230	0.295	0.240	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.56	119.8	PASS	PASS
0.140	0.226	0.146	PASS	PASS	PASS	PASS	469.5	469.5	1.0	0.53	79.6	PASS	PASS
0.593	0.501	1.582	PASS	PASS	PASS	FAIL	408.0	408.0	1.0	4.02	161.7	PASS	PASS
0.241	0.592	0.648	PASS	PASS	PASS	PASS	408.0	408.0	1.0	7.61	121.2	PASS	PASS
0.236	0.523	0.546	PASS	PASS	PASS	PASS	408.0	408.0	1.0	6.79	114.0	PASS	PASS
0.228	0.472	0.478	PASS	PASS	PASS	PASS	408.0	408.0	1.0	7.20	108.2	PASS	PASS
0.201	0.407	0.387	PASS	PASS	PASS	PASS	408.0	408.0	1.0	7.17	94.7	PASS	PASS
0.182	0.342	0.322	PASS	PASS	PASS	PASS	408.0	408.0	1.0	7.50	81.5	PASS	PASS
0.156	0.298	0.253	PASS	PASS	PASS	PASS	408.0	408.0	1.0	6.70	68.5	PASS	PASS
0.065	0.171	0.146	PASS	PASS	PASS	PASS	408.0	408.0	1.0	9.89	38.1	PASS	PASS



Section 7b: LDP (Response Spectrum) Moment Frame Column Assessment (LS)

Typical Column Properties

Table with 2 columns: Property Name and Value. Properties include width (w), cover (c), effective depth (d), concrete strength (f'c), yield strength (fy), ultimate strength (fu), expected concrete strength (fc'), expected yield strength (fy'), and ultimate strength (fu').

Parameters Below Assume P/A\_g f'c is less than 0.6, s/d is less than or equal to 0.5, p is greater than or equal to 0.006, and V\_u/(V\_u/d) is less than 0.6. Check these values prior to using this portion of the table

PCA Column Models should use expected behaviours

Main data table with columns for COL ID, I\_n, Longit Reinf, Yield Zone, Non-Yield Zone, Top Bottom, P\_DESIGN, Q\_c/A\_g f'c, E\_x E\_y, J^P\_DESIGN, and Flexure-Axial (Top Bottom) for both M and V.



Linear Dynamic Procedure														
Flexure-Axial Acceptance														
Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom	k <sub>Q<sub>c</sub>,yield</sub>		J	Shear		Yield Zone Shear Acceptance	Non-Yield Zone Shear Acceptance
DCR <sub>MAX</sub>	DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	DCR <sub>MIN</sub>	P <sub>max</sub>	P <sub>max</sub>	P <sub>min</sub>	P <sub>min</sub>	(k)	(k)		Q <sub>c</sub>	Q <sub>c</sub>		
0.236	0.734	0.685	1.898	PASS	PASS	PASS	FAIL	408.0	408.0	2.0	6.00	282.1	PASS	PASS
0.297	0.310	0.826	0.720	PASS	PASS	PASS	PASS	408.0	408.0	2.0	11.39	211.5	PASS	PASS
0.281	0.280	0.750	0.632	PASS	PASS	PASS	PASS	408.0	408.0	2.0	10.18	198.9	PASS	PASS
0.287	0.275	0.696	0.554	PASS	PASS	PASS	PASS	408.0	408.0	2.0	10.79	188.7	PASS	PASS
0.284	0.252	0.617	0.446	PASS	PASS	PASS	PASS	408.0	408.0	2.0	10.78	165.2	PASS	PASS
0.283	0.246	0.537	0.364	PASS	PASS	PASS	PASS	408.0	408.0	2.0	11.18	142.1	PASS	PASS
0.299	0.231	0.471	0.277	PASS	PASS	PASS	PASS	408.0	408.0	2.0	10.35	119.4	PASS	PASS
0.224	0.151	0.333	0.115	PASS	PASS	PASS	PASS	408.0	408.0	2.0	13.81	66.4	PASS	PASS
0.444	0.899	0.497	1.008	PASS	PASS	PASS	FAIL	469.5	375.6	2.0	0.16	416.0	PASS	PASS
0.568	0.576	0.629	0.638	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.02	410.0	PASS	PASS
0.546	0.510	0.604	0.562	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.32	367.3	PASS	PASS
0.527	0.490	0.581	0.537	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.45	342.8	PASS	PASS
0.494	0.442	0.539	0.480	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.63	303.2	PASS	PASS
0.454	0.394	0.486	0.418	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.71	260.3	PASS	PASS
0.398	0.322	0.417	0.333	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.85	208.9	PASS	PASS
0.310	0.197	0.320	0.200	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.94	138.7	PASS	PASS
0.439	0.935	0.473	1.008	PASS	PASS	PASS	FAIL	469.5	375.6	2.0	0.02	418.7	PASS	PASS
0.574	0.587	0.615	0.629	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.00	409.7	PASS	PASS
0.577	0.534	0.615	0.569	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.01	380.4	PASS	PASS
0.529	0.476	0.564	0.507	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.01	333.9	PASS	PASS
0.496	0.442	0.527	0.469	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.01	300.5	PASS	PASS
0.455	0.392	0.477	0.411	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.01	258.1	PASS	PASS
0.400	0.320	0.412	0.330	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	208.1	PASS	PASS
0.311	0.195	0.316	0.198	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	138.1	PASS	PASS
0.439	0.935	0.473	1.008	PASS	PASS	PASS	FAIL	469.5	375.6	2.0	0.01	418.7	PASS	PASS
0.574	0.587	0.615	0.629	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.00	409.7	PASS	PASS
0.577	0.534	0.615	0.569	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.01	380.4	PASS	PASS
0.528	0.476	0.564	0.507	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.01	333.9	PASS	PASS
0.496	0.442	0.527	0.469	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	300.5	PASS	PASS
0.455	0.392	0.477	0.411	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	258.1	PASS	PASS
0.400	0.320	0.412	0.330	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	208.1	PASS	PASS
0.311	0.195	0.316	0.198	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.04	138.1	PASS	PASS
0.443	0.900	0.498	1.008	PASS	PASS	PASS	FAIL	469.5	375.6	2.0	0.24	416.0	PASS	PASS
0.568	0.576	0.628	0.637	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.03	410.0	PASS	PASS
0.547	0.509	0.604	0.563	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.19	367.3	PASS	PASS
0.528	0.489	0.580	0.538	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.27	342.8	PASS	PASS
0.495	0.440	0.538	0.481	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.39	303.2	PASS	PASS
0.456	0.392	0.484	0.420	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.43	260.3	PASS	PASS
0.400	0.319	0.415	0.335	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.56	208.9	PASS	PASS
0.313	0.195	0.317	0.202	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.53	138.7	PASS	PASS
0.256	0.722	0.630	1.931	PASS	PASS	PASS	FAIL	408.0	408.0	2.0	4.02	282.1	PASS	PASS
0.320	0.283	0.766	0.789	PASS	PASS	PASS	PASS	408.0	408.0	2.0	7.61	211.5	PASS	PASS
0.304	0.259	0.694	0.685	PASS	PASS	PASS	PASS	408.0	408.0	2.0	6.79	198.9	PASS	PASS
0.312	0.251	0.641	0.608	PASS	PASS	PASS	PASS	408.0	408.0	2.0	7.20	188.7	PASS	PASS
0.310	0.226	0.564	0.498	PASS	PASS	PASS	PASS	408.0	408.0	2.0	7.17	165.2	PASS	PASS
0.315	0.215	0.483	0.416	PASS	PASS	PASS	PASS	408.0	408.0	2.0	7.50	142.1	PASS	PASS
0.332	0.196	0.424	0.326	PASS	PASS	PASS	PASS	408.0	408.0	2.0	6.70	119.4	PASS	PASS
0.287	0.099	0.260	0.176	PASS	PASS	PASS	PASS	408.0	408.0	2.0	9.89	66.4	PASS	PASS



Procedure											
Axial Acceptance			Shear			Yield Zone Shear Acceptance		Non-Yield Zone Shear Acceptance			
Bottom	Top	Bottom	$kQ_{CL,yield}$	$kQ_{CL,non-yield}$	J	$Q_s$	$Q_s$				
$P_{max}$	$P_{min}$	$P_{min}$	(k)	(k)		(k)	(k)				
FAIL	FAIL	FAIL	408.0	408.0	2.0	6.00	395.9	PASS	PASS		
PASS	FAIL	FAIL	408.0	408.0	2.0	11.39	297.3	PASS	PASS		
PASS	FAIL	PASS	408.0	408.0	2.0	10.18	279.3	PASS	PASS		
PASS	PASS	PASS	408.0	408.0	2.0	10.79	264.6	PASS	PASS		
PASS	PASS	PASS	408.0	408.0	2.0	10.78	231.8	PASS	PASS		
PASS	PASS	PASS	408.0	408.0	2.0	11.18	199.6	PASS	PASS		
PASS	PASS	PASS	408.0	408.0	2.0	10.35	166.9	PASS	PASS		
PASS	PASS	PASS	408.0	408.0	2.0	13.81	91.9	PASS	PASS		
FAIL	PASS	FAIL	469.5	375.6	2.0	0.16	584.0	PASS	PASS		
PASS	PASS	PASS	469.5	375.6	2.0	0.02	576.3	PASS	PASS		
PASS	PASS	PASS	469.5	375.6	2.0	0.32	516.0	PASS	PASS		
PASS	PASS	PASS	469.5	375.6	2.0	0.45	481.1	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.63	425.8	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.71	365.7	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.85	292.5	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.94	192.7	PASS	PASS		
FAIL	PASS	FAIL	469.5	375.6	2.0	0.02	587.8	PASS	PASS		
PASS	PASS	PASS	469.5	375.6	2.0	0.00	575.9	PASS	PASS		
PASS	PASS	PASS	469.5	375.6	2.0	0.01	534.3	PASS	PASS		
PASS	PASS	PASS	469.5	375.6	2.0	0.01	468.5	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.01	422.0	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.01	362.6	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.02	291.4	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.02	192.0	PASS	PASS		
FAIL	PASS	FAIL	469.5	375.6	2.0	0.01	587.8	PASS	PASS		
PASS	PASS	PASS	469.5	375.6	2.0	0.00	575.9	PASS	PASS		
PASS	PASS	PASS	469.5	375.6	2.0	0.01	534.3	PASS	PASS		
PASS	PASS	PASS	469.5	375.6	2.0	0.01	468.5	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.02	422.0	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.02	362.6	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.02	291.4	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.04	192.0	PASS	PASS		
FAIL	PASS	FAIL	469.5	375.6	2.0	0.24	584.0	PASS	PASS		
PASS	PASS	PASS	469.5	375.6	2.0	0.03	576.3	PASS	PASS		
PASS	PASS	PASS	469.5	375.6	2.0	0.19	516.0	PASS	PASS		
PASS	PASS	PASS	469.5	375.6	2.0	0.27	481.1	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.39	425.8	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.43	365.7	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.56	292.5	PASS	PASS		
PASS	PASS	PASS	469.5	469.5	2.0	0.53	192.7	PASS	PASS		
FAIL	FAIL	FAIL	408.0	408.0	2.0	4.02	395.9	PASS	PASS		
PASS	FAIL	FAIL	408.0	408.0	2.0	7.61	297.3	PASS	PASS		
PASS	FAIL	PASS	408.0	408.0	2.0	6.79	279.3	PASS	PASS		
PASS	PASS	PASS	408.0	408.0	2.0	7.20	264.6	PASS	PASS		
PASS	PASS	PASS	408.0	408.0	2.0	7.17	231.8	PASS	PASS		
PASS	PASS	PASS	408.0	408.0	2.0	7.50	199.6	PASS	PASS		
PASS	PASS	PASS	408.0	408.0	2.0	6.70	166.9	PASS	PASS		
PASS	PASS	PASS	408.0	408.0	2.0	9.89	91.9	PASS	PASS		



**Section 8a: LDP (Response Spectrum) Moment Frame Joint Assessment (IO)**

Typical Joint Details

w	28	in, Column Dimension
A <sub>j</sub>	784	in <sup>2</sup>
C <sub>1</sub>	1	
C <sub>2</sub>	1	
f <sub>c</sub> '	5000	psi, Lower Bound Compressive Strength of Concrete
f <sub>y</sub>	60	ksi, Lower Bound Reinforcing Yield Strength
f <sub>c</sub> '	7500	psi, Expected Compressive Strength of Concrete
f <sub>y</sub>	75	ksi, Expected Reinforcing Yield Strength
k	1	Knowledge Factor

**Linear Dynamic Procedure**

Joint ID	Offset Rigidities		Joint Shear							Joint Shear Acceptance	
	$\Sigma M_{nc}/\Sigma M_{nb}$	Col	Beam	$(V_{g,L} + V_{g,R}) * h / 2l_c$ (k)	$(V_{e,L} + V_{e,R}) * h / 2l_c$ (k)	J	Q <sub>UF</sub> (V <sub>j</sub> ) (k)	Condition	Int/Ext		Q <sub>CL</sub> (V <sub>n</sub> ) (k)
6A01	1.50	1	0	2.61	10.0	1.0	521.5851	Other (Main)	Exterior	665.2	PASS
6A02	1.50	1	0	2.90	10.9	1.0	500.8888	Other (Main)	Exterior	665.2	PASS
6A03	1.50	1	0	2.92	10.2	1.0	501.6173	Other (Main)	Exterior	665.2	PASS
6A04	1.87	1	0	2.94	9.2	1.0	397.4045	Other (Main)	Exterior	665.2	PASS
6A05	1.87	1	0	2.95	8.0	1.0	394.8705	Other (Main)	Exterior	665.2	PASS
6A06	1.87	1	0	2.96	6.6	1.0	396.2369	Other (Main)	Exterior	665.2	PASS
6A07	2.45	1	0	2.98	4.9	1.0	342.7877	Other (Main)	Exterior	665.2	PASS
6A08	1.53	1	0	4.78	5.5	1.0	261.385	Other (Roof)	Exterior	443.5	PASS
6B01	1.32	1	0	5.41	19.4	1.0	446.5803	2 Opp (Main)	Interior	831.6	PASS
6B02	1.34	1	0	5.81	21.5	1.0	434.6259	2 Opp (Main)	Interior	831.6	PASS
6B03	1.34	1	0	5.80	19.9	1.0	436.1793	2 Opp (Main)	Interior	831.6	PASS
6B04	1.68	1	0	5.78	18.0	1.0	339.4614	2 Opp (Main)	Interior	831.6	PASS
6B05	1.69	1	0	5.76	15.7	1.0	342.498	2 Opp (Main)	Interior	831.6	PASS
6B06	1.69	1	0	5.76	13.0	1.0	345.169	2 Opp (Main)	Interior	831.6	PASS
6B07	1.98	1	0	5.73	9.7	1.0	293.3558	2 Opp (Main)	Interior	831.6	PASS
6B08	1.11	0.5	0.5	9.42	10.8	1.0	168.5351	2 Opp (Roof)	Interior	665.2	PASS
6C01	1.34	1	0	5.34	19.0	1.0	449.773	2 Opp (Main)	Interior	831.6	PASS
6C02	1.34	1	0	5.81	21.3	1.0	434.7977	2 Opp (Main)	Interior	831.6	PASS
6C03	1.34	1	0	5.81	19.7	1.0	436.3986	2 Opp (Main)	Interior	831.6	PASS
6C04	1.69	1	0	5.82	17.6	1.0	340.5703	2 Opp (Main)	Interior	831.6	PASS
6C05	1.69	1	0	5.82	15.3	1.0	342.7914	2 Opp (Main)	Interior	831.6	PASS
6C06	1.69	1	0	5.82	12.7	1.0	345.3743	2 Opp (Main)	Interior	831.6	PASS
6C07	1.98	1	0	5.82	9.6	1.0	293.4057	2 Opp (Main)	Interior	831.6	PASS
6C08	1.11	0.5	0.5	9.49	10.6	1.0	168.7302	2 Opp (Roof)	Interior	665.2	PASS
6D01	1.34	1	0	5.34	19.0	1.0	449.773	2 Opp (Main)	Interior	831.6	PASS
6D02	1.34	1	0	5.81	21.3	1.0	434.7977	2 Opp (Main)	Interior	831.6	PASS
6D03	1.34	1	0	5.81	19.7	1.0	436.3986	2 Opp (Main)	Interior	831.6	PASS
6D04	1.69	1	0	5.82	17.6	1.0	340.5703	2 Opp (Main)	Interior	831.6	PASS
6D05	1.69	1	0	5.82	15.3	1.0	342.7914	2 Opp (Main)	Interior	831.6	PASS
6D06	1.69	1	0	5.82	12.7	1.0	345.3743	2 Opp (Main)	Interior	831.6	PASS
6D07	1.98	1	0	5.82	9.6	1.0	293.4057	2 Opp (Main)	Interior	831.6	PASS
6D08	1.11	0.5	0.5	9.49	10.6	1.0	168.7302	2 Opp (Roof)	Interior	665.2	PASS
6E01	1.32	1	0	5.41	19.4	1.0	446.5803	2 Opp (Main)	Interior	831.6	PASS
6E02	1.34	1	0	5.81	21.5	1.0	434.6259	2 Opp (Main)	Interior	831.6	PASS
6E03	1.34	1	0	5.80	19.9	1.0	436.1793	2 Opp (Main)	Interior	831.6	PASS
6E04	1.68	1	0	5.78	18.0	1.0	339.4614	2 Opp (Main)	Interior	831.6	PASS
6E05	1.69	1	0	5.76	15.7	1.0	342.498	2 Opp (Main)	Interior	831.6	PASS
6E06	1.69	1	0	5.76	13.0	1.0	345.169	2 Opp (Main)	Interior	831.6	PASS
6E07	1.98	1	0	5.73	9.7	1.0	293.3558	2 Opp (Main)	Interior	831.6	PASS
6E08	1.11	0.5	0.5	9.42	10.8	1.0	168.5351	2 Opp (Roof)	Interior	665.2	PASS
6F01	1.50	1	0	2.61	10.0	1.0	521.5851	Other (Main)	Exterior	665.2	PASS
6F02	1.50	1	0	2.90	10.9	1.0	500.8888	Other (Main)	Exterior	665.2	PASS
6F03	1.50	1	0	2.92	10.2	1.0	501.6173	Other (Main)	Exterior	665.2	PASS
6F04	1.87	1	0	2.94	9.2	1.0	397.4045	Other (Main)	Exterior	665.2	PASS
6F05	1.87	1	0	2.95	8.0	1.0	394.8705	Other (Main)	Exterior	665.2	PASS
6F06	1.87	1	0	2.96	6.6	1.0	396.2369	Other (Main)	Exterior	665.2	PASS
6F07	2.45	1	0	2.98	4.9	1.0	342.7877	Other (Main)	Exterior	665.2	PASS
6F08	1.53	1	0	4.78	5.5	1.0	261.385	Other (Roof)	Exterior	443.5	PASS





**Section 8b: LDP (Response Spectrum) Moment Frame Joint Assessment (LS)**

Typical Joint Details

w	28	in, Column Dimension
A <sub>j</sub>	784	in <sup>2</sup>
C <sub>1</sub>	1	
C <sub>2</sub>	1	
f <sub>c</sub> '	5000	psi, Lower Bound Compressive Strength of Concrete
f <sub>y</sub>	60	ksi, Lower Bound Reinforcing Yield Strength
f <sub>c</sub> '	7500	psi, Expected Compressive Strength of Concrete
f <sub>y</sub>	75	ksi, Expected Reinforcing Yield Strength
K	1	Knowledge Factor

Linear Dynamic Procedure											
Joint ID	ΣM <sub>nc</sub> /ΣM <sub>nb</sub>	Offset Rigidities		Joint Shear		J	Q <sub>UF</sub> (V <sub>j</sub> )	Condition	Int/Ext	Q <sub>CL</sub> (V <sub>n</sub> )	Joint Shear Acceptance
		Col	Beam	(V <sub>g,L</sub> + V <sub>g,R</sub> )*h/2l <sub>c</sub>	(V <sub>e,L</sub> + V <sub>e,R</sub> )*h/2l <sub>c</sub>						
6A01	1.50	1	0	2.61	17.5	2.0	522.8653	Other (Main)	Exterior	665.2	PASS
6A02	1.50	1	0	2.90	19.0	2.0	502.2804	Other (Main)	Exterior	665.2	PASS
6A03	1.50	1	0	2.92	17.7	2.0	502.9153	Other (Main)	Exterior	665.2	PASS
6A04	1.87	1	0	2.94	16.1	2.0	398.5865	Other (Main)	Exterior	665.2	PASS
6A05	1.87	1	0	2.95	14.0	2.0	395.8944	Other (Main)	Exterior	665.2	PASS
6A06	1.87	1	0	2.96	11.6	2.0	397.0858	Other (Main)	Exterior	665.2	PASS
6A07	2.45	1	0	2.98	8.6	2.0	343.4174	Other (Main)	Exterior	665.2	PASS
6A08	1.53	1	0	4.78	9.7	2.0	262.0998	Other (Roof)	Exterior	443.5	PASS
6B01	1.32	1	0	5.41	33.9	2.0	449.0656	2 Opp (Main)	Interior	831.6	PASS
6B02	1.34	1	0	5.81	37.4	2.0	437.3675	2 Opp (Main)	Interior	831.6	PASS
6B03	1.34	1	0	5.80	34.7	2.0	438.7259	2 Opp (Main)	Interior	831.6	PASS
6B04	1.68	1	0	5.78	31.4	2.0	341.7656	2 Opp (Main)	Interior	831.6	PASS
6B05	1.69	1	0	5.76	27.3	2.0	344.5012	2 Opp (Main)	Interior	831.6	PASS
6B06	1.69	1	0	5.76	22.7	2.0	346.8328	2 Opp (Main)	Interior	831.6	PASS
6B07	1.98	1	0	5.73	16.9	2.0	294.5989	2 Opp (Main)	Interior	831.6	PASS
6B08	1.11	0.5	0.5	9.42	18.9	2.0	169.932	2 Opp (Roof)	Interior	665.2	PASS
6C01	1.34	1	0	5.34	33.1	2.0	452.1974	2 Opp (Main)	Interior	831.6	PASS
6C02	1.34	1	0	5.81	37.1	2.0	437.5173	2 Opp (Main)	Interior	831.6	PASS
6C03	1.34	1	0	5.81	34.3	2.0	438.9155	2 Opp (Main)	Interior	831.6	PASS
6C04	1.69	1	0	5.82	30.6	2.0	342.8149	2 Opp (Main)	Interior	831.6	PASS
6C05	1.69	1	0	5.82	26.7	2.0	344.7504	2 Opp (Main)	Interior	831.6	PASS
6C06	1.69	1	0	5.82	22.2	2.0	347.0041	2 Opp (Main)	Interior	831.6	PASS
6C07	1.98	1	0	5.82	16.7	2.0	294.6314	2 Opp (Main)	Interior	831.6	PASS
6C08	1.11	0.5	0.5	9.49	18.4	2.0	170.0934	2 Opp (Roof)	Interior	665.2	PASS
6D01	1.34	1	0	5.34	33.1	2.0	452.1974	2 Opp (Main)	Interior	831.6	PASS
6D02	1.34	1	0	5.81	37.1	2.0	437.5173	2 Opp (Main)	Interior	831.6	PASS
6D03	1.34	1	0	5.81	34.3	2.0	438.9155	2 Opp (Main)	Interior	831.6	PASS
6D04	1.69	1	0	5.82	30.6	2.0	342.8149	2 Opp (Main)	Interior	831.6	PASS
6D05	1.69	1	0	5.82	26.7	2.0	344.7504	2 Opp (Main)	Interior	831.6	PASS
6D06	1.69	1	0	5.82	22.2	2.0	347.0041	2 Opp (Main)	Interior	831.6	PASS
6D07	1.98	1	0	5.82	16.7	2.0	294.6314	2 Opp (Main)	Interior	831.6	PASS
6D08	1.11	0.5	0.5	9.49	18.4	2.0	170.0934	2 Opp (Roof)	Interior	665.2	PASS
6E01	1.32	1	0	5.41	33.9	2.0	449.0656	2 Opp (Main)	Interior	831.6	PASS
6E02	1.34	1	0	5.81	37.4	2.0	437.3675	2 Opp (Main)	Interior	831.6	PASS
6E03	1.34	1	0	5.80	34.7	2.0	438.7259	2 Opp (Main)	Interior	831.6	PASS
6E04	1.68	1	0	5.78	31.4	2.0	341.7656	2 Opp (Main)	Interior	831.6	PASS
6E05	1.69	1	0	5.76	27.3	2.0	344.5012	2 Opp (Main)	Interior	831.6	PASS
6E06	1.69	1	0	5.76	22.7	2.0	346.8328	2 Opp (Main)	Interior	831.6	PASS
6E07	1.98	1	0	5.73	16.9	2.0	294.5989	2 Opp (Main)	Interior	831.6	PASS
6E08	1.11	0.5	0.5	9.42	18.9	2.0	169.932	2 Opp (Roof)	Interior	665.2	PASS
6F01	1.50	1	0	2.61	17.5	2.0	522.8653	Other (Main)	Exterior	665.2	PASS
6F02	1.50	1	0	2.90	19.0	2.0	502.2804	Other (Main)	Exterior	665.2	PASS
6F03	1.50	1	0	2.92	17.7	2.0	502.9153	Other (Main)	Exterior	665.2	PASS
6F04	1.87	1	0	2.94	16.1	2.0	398.5865	Other (Main)	Exterior	665.2	PASS
6F05	1.87	1	0	2.95	14.0	2.0	395.8944	Other (Main)	Exterior	665.2	PASS
6F06	1.87	1	0	2.96	11.6	2.0	397.0858	Other (Main)	Exterior	665.2	PASS
6F07	2.45	1	0	2.98	8.6	2.0	343.4174	Other (Main)	Exterior	665.2	PASS
6F08	1.53	1	0	4.78	9.7	2.0	262.0998	Other (Roof)	Exterior	443.5	PASS



## Section 8c: LDP (Response Spectrum) Moment Frame Joint Assessment (CP)

## Typical Joint Details

w	28	in, Column Dimension
A <sub>j</sub>	784	in <sup>2</sup>
C <sub>1</sub>	1	
C <sub>2</sub>	1	
f <sub>c</sub> '	5000	psi, Lower Bound Compressive Strength of Concrete
f <sub>y</sub>	60	ksi, Lower Bound Reinforcing Yield Strength
f <sub>c</sub> '	7500	psi, Expected Compressive Strength of Concrete
f <sub>y</sub>	75	ksi, Expected Reinforcing Yield Strength
K	1	Knowledge Factor

Joint ID	ΣM <sub>nc</sub> /ΣM <sub>nb</sub>	Offset Rigidities		Linear Static Procedure							Joint Shear Acceptance
		Col	Beam	Joint Shear		J	Q <sub>UF</sub> (V <sub>j</sub> )	Condition	Int/Ext	Q <sub>CL</sub> (V <sub>n</sub> )	
		(k)	(k)	(V <sub>g,L</sub> + V <sub>g,R</sub> )*h/2l <sub>c</sub>	(V <sub>e,L</sub> + V <sub>e,R</sub> )*h/2l <sub>c</sub>						
6A01	1.50	1	0	2.61	24.6	2.0	519.3229	Other (Main)	Exterior	665.2	PASS
6A02	1.50	1	0	2.90	26.7	2.0	498.4216	Other (Main)	Exterior	665.2	PASS
6A03	1.50	1	0	2.92	24.9	2.0	499.3294	Other (Main)	Exterior	665.2	PASS
6A04	1.87	1	0	2.94	22.6	2.0	395.3261	Other (Main)	Exterior	665.2	PASS
6A05	1.87	1	0	2.95	19.6	2.0	393.0597	Other (Main)	Exterior	665.2	PASS
6A06	1.87	1	0	2.96	16.3	2.0	394.7421	Other (Main)	Exterior	665.2	PASS
6A07	2.45	1	0	2.98	12.0	2.0	341.7114	Other (Main)	Exterior	665.2	PASS
6A08	1.53	1	0	4.78	13.4	2.0	260.2063	Other (Roof)	Exterior	443.5	PASS
6B01	1.32	1	0	5.41	47.7	2.0	442.1886	2 Opp (Main)	Interior	831.6	PASS
6B02	1.34	1	0	5.81	52.6	2.0	429.765	2 Opp (Main)	Interior	831.6	PASS
6B03	1.34	1	0	5.80	48.8	2.0	431.6898	2 Opp (Main)	Interior	831.6	PASS
6B04	1.68	1	0	5.78	44.1	2.0	335.4088	2 Opp (Main)	Interior	831.6	PASS
6B05	1.69	1	0	5.76	38.4	2.0	338.9548	2 Opp (Main)	Interior	831.6	PASS
6B06	1.69	1	0	5.76	31.9	2.0	342.2383	2 Opp (Main)	Interior	831.6	PASS
6B07	1.98	1	0	5.73	23.6	2.0	291.228	2 Opp (Main)	Interior	831.6	PASS
6B08	1.11	0.5	0.5	9.42	26.3	2.0	166.2221	2 Opp (Roof)	Interior	665.2	PASS
6C01	1.34	1	0	5.34	46.5	2.0	445.4887	2 Opp (Main)	Interior	831.6	PASS
6C02	1.34	1	0	5.81	52.2	2.0	429.9758	2 Opp (Main)	Interior	831.6	PASS
6C03	1.34	1	0	5.81	48.3	2.0	431.9604	2 Opp (Main)	Interior	831.6	PASS
6C04	1.69	1	0	5.82	43.0	2.0	336.622	2 Opp (Main)	Interior	831.6	PASS
6C05	1.69	1	0	5.82	37.6	2.0	339.3257	2 Opp (Main)	Interior	831.6	PASS
6C06	1.69	1	0	5.82	31.2	2.0	342.5027	2 Opp (Main)	Interior	831.6	PASS
6C07	1.98	1	0	5.82	23.3	2.0	291.3048	2 Opp (Main)	Interior	831.6	PASS
6C08	1.11	0.5	0.5	9.49	25.7	2.0	166.4655	2 Opp (Roof)	Interior	665.2	PASS
6D01	1.34	1	0	5.34	46.5	2.0	445.4887	2 Opp (Main)	Interior	831.6	PASS
6D02	1.34	1	0	5.81	52.2	2.0	429.9758	2 Opp (Main)	Interior	831.6	PASS
6D03	1.34	1	0	5.81	48.3	2.0	431.9604	2 Opp (Main)	Interior	831.6	PASS
6D04	1.69	1	0	5.82	43.0	2.0	336.622	2 Opp (Main)	Interior	831.6	PASS
6D05	1.69	1	0	5.82	37.6	2.0	339.3257	2 Opp (Main)	Interior	831.6	PASS
6D06	1.69	1	0	5.82	31.2	2.0	342.5027	2 Opp (Main)	Interior	831.6	PASS
6D07	1.98	1	0	5.82	23.3	2.0	291.3048	2 Opp (Main)	Interior	831.6	PASS
6D08	1.11	0.5	0.5	9.49	25.7	2.0	166.4655	2 Opp (Roof)	Interior	665.2	PASS
6E01	1.32	1	0	5.41	47.7	2.0	442.1886	2 Opp (Main)	Interior	831.6	PASS
6E02	1.34	1	0	5.81	52.6	2.0	429.765	2 Opp (Main)	Interior	831.6	PASS
6E03	1.34	1	0	5.80	48.8	2.0	431.6898	2 Opp (Main)	Interior	831.6	PASS
6E04	1.68	1	0	5.78	44.1	2.0	335.4088	2 Opp (Main)	Interior	831.6	PASS
6E05	1.69	1	0	5.76	38.4	2.0	338.9548	2 Opp (Main)	Interior	831.6	PASS
6E06	1.69	1	0	5.76	31.9	2.0	342.2383	2 Opp (Main)	Interior	831.6	PASS
6E07	1.98	1	0	5.73	23.6	2.0	291.228	2 Opp (Main)	Interior	831.6	PASS
6E08	1.11	0.5	0.5	9.42	26.3	2.0	166.2221	2 Opp (Roof)	Interior	665.2	PASS
6F01	1.50	1	0	2.61	24.6	2.0	519.3229	Other (Main)	Exterior	665.2	PASS
6F02	1.50	1	0	2.90	26.7	2.0	498.4216	Other (Main)	Exterior	665.2	PASS
6F03	1.50	1	0	2.92	24.9	2.0	499.3294	Other (Main)	Exterior	665.2	PASS
6F04	1.87	1	0	2.94	22.6	2.0	395.3261	Other (Main)	Exterior	665.2	PASS
6F05	1.87	1	0	2.95	19.6	2.0	393.0597	Other (Main)	Exterior	665.2	PASS
6F06	1.87	1	0	2.96	16.3	2.0	394.7421	Other (Main)	Exterior	665.2	PASS
6F07	2.45	1	0	2.98	12.0	2.0	341.7114	Other (Main)	Exterior	665.2	PASS
6F08	1.53	1	0	4.78	13.4	2.0	260.2063	Other (Roof)	Exterior	443.5	PASS

**Section 9a: LDP (Response Spectrum) Shear Wall Assessment (IO)**

## Typical Wall Properties

$f'_c$	5000	psi, Lower Bound Concrete Compressive Strength
$f_y$	60	ksi, Lower Bound Reinforcing Steel Yield Strength
$f_u$	90	ksi, Lower Bound Reinforcing Steel Ultimate Strength
$f'_c$	7500	psi, Expected Concrete Compressive Strength
$f_y$	75	ksi, Expected Reinforcing Steel Yield Strength
$f_u$	112.5	ksi, Expected Reinforcing Steel Ultimate Strength
$t_w$	16	in, Shear Wall Thickness
$h_w$	106	ft, Shear Wall Height
$l_w$	22.33	ft, Shear wall Length
Slender		Wall Slenderness (ASCE 41-06 Section C6.7.1)
w	28	in, Boundary Element Width
$A_b$	4960	kips, Nominal Axial Capacity (No Eccentricity, Concrete Only)
k	1.0	Knowledge Factor
$C_1$	1.0	
$C_2$	1.0	

**Exterior Shear Wall**

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element			Effectiveness			$M_{n,P=0}^{4.5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)				
						Size	Spc (in)	Size	Spc (in)	$A_{st}^{-1}$ (in <sup>2</sup> )	Size	Spc (in)	Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)					$\Phi_{AXIAL}$	$P > .35P_0^{?2}$	$S_h > 18''^{?2}$	$S_v > 18''^{?2}$
Level 08	106	13	111.7	80.6	5833.0	#5	18.0	#5	18.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	23270.4	454.3	151.3	1088.9
Level 07	93	13	248.7	165.7	16045.2	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	790.8	151.3	1158.1
Level 06	80	13	385.8	250.8	28926.5	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1008.3	151.3	1158.1
Level 05	67	13	522.8	336.0	43444.8	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1168.4	151.3	1158.1
Level 04	54	13	659.9	421.1	59298.1	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	1324.4	151.3	1158.1
Level 03	41	13	796.9	506.2	76597.5	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	1483.4	151.3	1158.1
Level 02	28	13	934.0	591.3	95373.4	#5	15.0	#5	15.0	20.32	#5	4.0	YES	18880.9	28062.3	0.673	NO	NO	NO	52017.3	1612.0	151.3	1158.1
Level 01	15	15	1074.6	679.4	118431.8	#5	10.0	#5	10.0	29.32	#5	4.0	YES	19307.4	28587.3	0.675	NO	NO	NO	65401.3	1679.2	174.6	1365.8

**Interior Shear Wall**

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element			Effectiveness			$M_{n,P=0}^{4.5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)				
						Size	Spc (in)	Size	Spc (in)	$A_{st}^{-1}$ (in <sup>2</sup> )	Size	Spc (in)	Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)					$\Phi_{AXIAL}$	$P > .35P_0^{?2}$	$S_h > 18''^{?2}$	$S_v > 18''^{?2}$
Level 08	106	13	190.2	134.1	5544.2	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	431.4	89.8	1088.9
Level 07	93	13	426.7	273.5	14990.3	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	734.0	89.8	1088.9
Level 06	80	13	663.1	413.0	26765.1	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	931.4	89.8	1158.1
Level 05	67	13	899.6	552.4	40078.0	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1084.6	89.8	1158.1
Level 04	54	13	1136.0	691.9	54703.7	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1229.2	89.8	1158.1
Level 03	41	13	1372.5	831.3	70670.4	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1370.0	89.8	1158.1
Level 02	28	13	1608.9	970.8	87968.1	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1489.4	89.8	1158.1
Level 01	15	15	1849.0	1113.2	109253.0	#5	15.0	#5	10.0	15.24	#5	5.0	YES	18640.2	27766.0	0.671	NO	NO	NO	38815.0	1561.8	103.6	1158.1

1. Area of Longitudinal Steel in Boundary Elements

2. If "NO", Shear Wall may not be taken as effective in resisting seismic forces per ASCE 41-06 Section 6.7.1.1

3. Nominal Moment Capacity taken from PCA Column Analysis corresponding to minimum design Axial Load

4. Nominal Moment Capacity at P=0 taken from PCA Column Analysis, including only boundary reinforcing.  $\Phi = 1.0$ , and expected strengths are used.

5. Nominal Moment Capacity at P=0 is not required to exceed Moment Demand, as the compressive force is assumed to be 0 and therefore no interaction effects are taken into account.

6. Maximum shear force calculated based on nominal flexural capacity of base of wall.

7. Wall is considered controlled by flexure if the maximum shear force value is less than the nominal shear capacity. This indicates that the wall will fail in flexure prior to reaching the shear capacity.



Control <sup>7</sup>	J*P <sub>DES,MAX</sub> (kips)	J*P <sub>DES,MIN</sub> (kips)	M <sub>DES</sub> /m (k-ft)	DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Axial-Flexural Acceptance		Shear Acceptance
Flexure	111.7	80.6	2916.5	0.120	0.121	PASS	PASS	PASS
Flexure	248.7	165.7	8022.6	0.290	0.299	PASS	PASS	PASS
Flexure	385.8	250.8	14463.2	0.498	0.522	PASS	PASS	PASS
Flexure	522.8	336.0	21722.4	0.715	0.762	PASS	PASS	PASS
Flexure	659.9	421.1	29649.0	0.634	0.666	PASS	PASS	PASS
Flexure	796.9	506.2	38298.8	0.797	0.845	PASS	PASS	PASS
Flexure	934.0	591.3	47731.9	0.784	0.828	PASS	PASS	PASS
Flexure	1074.6	679.4	70181.8	0.929	0.977	PASS	PASS	PASS

Control <sup>7</sup>	J*P <sub>DES,MAX</sub> (kips)	J*P <sub>DES,MIN</sub> (kips)	M <sub>DES</sub> /m (k-ft)	DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Axial-Flexural Acceptance		Shear Acceptance
Flexure	190.2	134.1	2772.1	0.082	0.083	PASS	PASS	PASS
Flexure	426.7	273.5	7495.2	0.207	0.216	PASS	PASS	PASS
Flexure	663.1	413.0	13382.6	0.331	0.353	PASS	PASS	PASS
Flexure	899.6	552.4	20039.0	0.469	0.510	PASS	PASS	PASS
Flexure	1136.0	691.9	27351.9	0.608	0.672	PASS	PASS	PASS
Flexure	1372.5	831.3	35335.2	0.748	0.840	PASS	PASS	PASS
Flexure	1608.9	970.8	44126.2	0.891	1.016	PASS	FAIL	PASS
Flexure	1849.0	1113.2	64742.5	1.144	1.304	FAIL	FAIL	PASS

**Section 9b: LDP (Response Spectrum) Shear Wall Assessment (LS)**

## Typical Wall Properties

$f_c'$	5000	psi, Lower Bound Concrete Compressive Strength
$f_y$	60	ksi, Lower Bound Reinforcing Steel Yield Strength
$f_u$	90	ksi, Lower Bound Reinforcing Steel Ultimate Strength
$f_c'$	7500	psi, Expected Concrete Compressive Strength
$f_y$	75	ksi, Expected Reinforcing Steel Yield Strength
$f_u$	112.5	ksi, Expected Reinforcing Steel Ultimate Strength
$t_w$	16	in, Shear Wall Thickness
$h_w$	106	ft, Shear Wall Height
$l_w$	22.33	ft, Shear wall Length
		Slender Wall Slenderness (ASCE 41-06 Section C6.7.1)
$w$	28	in, Boundary Element Width
$A_g$	4960	kips, Nominal Axial Capacity (No Eccentricity, Concrete Only)
$\kappa$	1.0	Knowledge Factor
$C_1$	1.0	
$C_2$	1.0	

**Exterior Shear Wall**

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{n,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)	
						Size	Spc (in)	Size	Spc (in)	Size	Spc (in)					$P_{>.35P_0}^{2,2}$	$S_h > 18''^{2,2}$	$S_v > 18''^{2,2}$					
Level 08	106	13	111.7	80.6	10160.9	#5	18.0	#5	18.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	23270.4	791.4	151.3	1088.9
Level 07	93	13	248.7	165.7	27959.6	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1378.3	151.3	1158.1
Level 06	80	13	385.8	250.8	50424.7	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1758.4	151.3	1158.1
Level 05	67	13	522.8	336.0	75759.8	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	2038.2	151.3	1158.1
Level 04	54	13	659.9	421.1	103432.5	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	2310.3	151.3	1158.1
Level 03	41	13	796.9	506.2	133625.4	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	2586.7	151.3	1158.1
Level 02	28	13	934.0	591.3	166382.6	#5	15.0	#5	15.0	20.32	#5	4.0	YES	18880.9	28062.3	0.673	NO	NO	NO	52017.3	2810.0	151.3	1158.1
Level 01	15	15	1074.6	679.4	206595.1	#5	10.0	#5	10.0	29.32	#5	4.0	YES	19307.4	28587.3	0.675	NO	NO	NO	65401.3	2926.7	174.6	1365.8

**Interior Shear Wall**

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{n,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)	
						Size	Spc (in)	Size	Spc (in)	Size	Spc (in)					$P_{>.35P_0}^{2,2}$	$S_h > 18''^{2,2}$	$S_v > 18''^{2,2}$					
Level 08	106	13	190.2	134.1	9655.1	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	751.2	89.8	1088.9
Level 07	93	13	426.7	273.5	26118.0	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	1279.2	89.8	1088.9
Level 06	80	13	663.1	413.0	46655.4	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1624.2	89.8	1158.1
Level 05	67	13	899.6	552.4	69888.6	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1891.9	89.8	1158.1
Level 04	54	13	1136.0	691.9	95418.4	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	2144.0	89.8	1158.1
Level 03	41	13	1372.5	831.3	123284.9	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	2388.9	89.8	1158.1
Level 02	28	13	1608.9	970.8	153463.6	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	2596.4	89.8	1158.1
Level 01	15	15	1849.0	1113.2	190582.9	#5	15.0	#5	10.0	15.24	#5	5.0	YES	18640.2	27766.0	0.671	NO	NO	NO	38815.0	2721.8	103.6	1158.1

1. Area of Longitudinal Steel in Boundary Elements

2. If "NO", Shear Wall may not be taken as effective in resisting seismic forces per ASCE 41-06 Section 6.7.1.1

3. Nominal Moment Capacity taken from PCA Column Analysis corresponding to minimum design Axial Load

4. Nominal Moment Capacity at P=0 taken from PCA Column Analysis, including only boundary reinforcing.  $\Phi = 1.0$ , and expected strengths are used.

5. Nominal Moment Capacity at P=0 is not required to exceed Moment Demand, as the compressive force is assumed to be 0 and therefore no interaction effects are taken into account.

6. Maximum shear force calculated based on nominal flexural capacity of base of wall.

7. Wall is considered controlled by flexure if the maximum shear force value is less than the nominal shear capacity. This indicates that the wall will fail in flexure prior to reaching the shear capacity.



Control <sup>7</sup>	J*P <sub>DES,MAX</sub> (kips)	J*P <sub>DES,MIN</sub> (kips)	M <sub>DES</sub> /m (k-ft)	Axial-Flexure		Axial-Flexural		Shear
				DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Acceptance		
Flexure	223.4	161.2	2540.2	0.099	0.102	PASS	PASS	PASS
Flexure	497.5	331.4	7501.7	0.249	0.263	PASS	PASS	PASS
Flexure	771.6	501.7	16264.3	0.495	0.539	PASS	PASS	PASS
Flexure	1045.7	671.9	25253.3	0.710	0.792	PASS	PASS	PASS
Flexure	1319.7	842.2	34477.5	0.651	0.711	PASS	PASS	PASS
Flexure	1593.8	1012.4	44541.8	0.802	0.889	PASS	PASS	PASS
Flexure	1867.9	1182.7	55513.5	0.799	0.879	PASS	PASS	PASS
Flexure	2149.2	1358.8	68865.0	0.808	0.882	PASS	PASS	PASS

Control <sup>7</sup>	J*P <sub>DES,MAX</sub> (kips)	J*P <sub>DES,MIN</sub> (kips)	M <sub>DES</sub> /m (k-ft)	DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Axial-Flexural		Shear
						Acceptance		
Flexure	380.5	268.1	2413.8	0.067	0.070	PASS	PASS	PASS
Flexure	853.4	547.0	6713.3	0.166	0.179	PASS	PASS	PASS
Flexure	1326.3	825.9	14045.8	0.300	0.334	PASS	PASS	PASS
Flexure	1799.2	1104.9	23296.2	0.454	0.521	PASS	PASS	PASS
Flexure	2272.1	1383.8	31806.1	0.571	0.672	PASS	PASS	PASS
Flexure	2745.0	1662.7	41095.0	0.685	0.822	PASS	PASS	PASS
Flexure	3217.9	1941.6	51334.0	0.799	0.976	PASS	PASS	PASS
Flexure	3698.0	2226.4	63527.6	0.868	1.057	PASS	FAIL	PASS

**Section 9c: LDP (Response Spectrum) Shear Wall Assessment (CP)**

## Typical Wall Properties

$f_c'$	5000	psi, Lower Bound Concrete Compressive Strength
$f_y$	60	ksi, Lower Bound Reinforcing Steel Yield Strength
$f_u$	90	ksi, Lower Bound Reinforcing Steel Ultimate Strength
$f_c'$	7500	psi, Expected Concrete Compressive Strength
$f_y$	75	ksi, Expected Reinforcing Steel Yield Strength
$f_u$	112.5	ksi, Expected Reinforcing Steel Ultimate Strength
$t_w$	16	in, Shear Wall Thickness
$h_w$	106	ft, Shear Wall Height
$l_w$	22.33	ft, Shear wall Length
Slender		Wall Slenderness (ASCE 41-06 Section C6.7.1)
$w$	28	in, Boundary Element Width
$A_g$	4960	kips, Nominal Axial Capacity (No Eccentricity, Concrete Only)
$\kappa$	1.0	Knowledge Factor
$C_1$	1.0	
$C_2$	1.0	

**Exterior Shear Wall**

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{n,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)	
						Size	Spc (in)	Size	Spc (in)	$A_{st}^1$ (in <sup>2</sup> )	Size					Spc (in)	$P_{>.35P_0}^{2,3}$	$s_h > 18''^{2,3}$					$s_v > 18''^{2,3}$
Level 08	106	13	111.7	80.6	14133.4	#5	18.0	#5	18.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	23270.4	1101.0	151.3	1088.9
Level 07	93	13	248.7	165.7	38988.9	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1924.9	151.3	1158.1
Level 06	80	13	385.8	250.8	70509.8	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	2465.9	151.3	1158.1
Level 05	67	13	522.8	336.0	106211.6	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	2865.7	151.3	1158.1
Level 04	54	13	659.9	421.1	145289.1	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	3246.9	151.3	1158.1
Level 03	41	13	796.9	506.2	187883.5	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	3626.3	151.3	1158.1
Level 02	28	13	934.0	591.3	233965.2	#5	15.0	#5	15.0	20.32	#5	4.0	YES	18880.9	28062.3	0.673	NO	NO	NO	52017.3	3929.6	151.3	1158.1
Level 01	15	15	1074.6	679.4	290370.9	#5	10.0	#5	10.0	29.32	#5	4.0	YES	19307.4	28587.3	0.675	NO	NO	NO	65401.3	4087.2	174.6	1365.8

**Interior Shear Wall**

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{n,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)	
						Size	Spc (in)	Size	Spc (in)	$A_{st}^1$ (in <sup>2</sup> )	Size					Spc (in)	$P_{>.35P_0}^{2,3}$	$s_h > 18''^{2,3}$					$s_v > 18''^{2,3}$
Level 08	106	13	190.2	134.1	13402.3	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	1043.0	89.8	1088.9
Level 07	93	13	426.7	273.5	36385.9	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	1785.7	89.8	1088.9
Level 06	80	13	663.1	413.0	65222.1	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	2277.4	89.8	1158.1
Level 05	67	13	899.6	552.4	97978.8	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	2658.4	89.8	1158.1
Level 04	54	13	1136.0	691.9	134029.2	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	3011.5	89.8	1158.1
Level 03	41	13	1372.5	831.3	173339.1	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	3348.6	89.8	1158.1
Level 02	28	13	1608.9	970.8	215797.7	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	3630.3	89.8	1158.1
Level 01	15	15	1849.0	1113.2	267861.1	#5	15.0	#5	10.0	15.24	#5	5.0	YES	18640.2	27766.0	0.671	NO	NO	NO	38815.0	3798.7	103.6	1158.1

1. Area of Longitudinal Steel in Boundary Elements

2. If "NO", Shear Wall may not be taken as effective in resisting seismic forces per ASCE 41-06 Section 6.7.1.1

3. Nominal Moment Capacity taken from PCA Column Analysis corresponding to minimum design Axial Load

4. Nominal Moment Capacity at P=0 taken from PCA Column Analysis, including only boundary reinforcing.  $\Phi = 1.0$ , and expected strengths are used.

5. Nominal Moment Capacity at P=0 is not required to exceed Moment Demand, as the compressive force is assumed to be 0 and therefore no interaction effects are taken into account.

6. Maximum shear force calculated based on nominal flexural capacity of base of wall.

7. Wall is considered controlled by flexure if the maximum shear force value is less than the nominal shear capacity. This indicates that the wall will fail in flexure prior to reaching the shear capacity.



Control <sup>7</sup>	J*P <sub>DES,MAX</sub> (kips)	J*P <sub>DES,MIN</sub> (kips)	M <sub>DES</sub> /m (k-ft)	Axial-Flexure		Axial-Flexural Acceptance		Shear Acceptance
				DCR <sub>MAX</sub>	DCR <sub>MIN</sub>			
Flexure	223.4	161.2	2355.6	0.092	0.095	PASS	PASS	PASS
Flexure	497.5	331.4	9747.2	0.323	0.342	PASS	PASS	PASS
Flexure	771.6	501.7	17627.4	0.536	0.584	PASS	PASS	PASS
Flexure	1045.7	671.9	26552.9	0.747	0.833	PASS	PASS	PASS
Flexure	1319.7	842.2	36322.3	0.685	0.749	PASS	PASS	PASS
Flexure	1593.8	1012.4	46970.9	0.846	0.937	PASS	PASS	PASS
Flexure	1867.9	1182.7	58553.8	0.843	0.927	PASS	PASS	PASS
Flexure	2149.2	1358.8	70392.9	0.826	0.901	PASS	PASS	PASS

Control <sup>7</sup>	J*P <sub>DES,MAX</sub> (kips)	J*P <sub>DES,MIN</sub> (kips)	M <sub>DES</sub> /m (k-ft)	DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Axial-Flexural Acceptance		Shear Acceptance
Flexure	380.5	268.1	2233.7	0.062	0.064	PASS	PASS	PASS
Flexure	853.4	547.0	8851.4	0.219	0.236	PASS	PASS	PASS
Flexure	1326.3	825.9	16305.5	0.348	0.388	PASS	PASS	PASS
Flexure	1799.2	1104.9	24494.7	0.478	0.548	PASS	PASS	PASS
Flexure	2272.1	1383.8	33507.3	0.602	0.707	PASS	PASS	PASS
Flexure	2745.0	1662.7	43334.8	0.722	0.867	PASS	PASS	PASS
Flexure	3217.9	1941.6	54162.5	0.843	1.029	PASS	FAIL	PASS
Flexure	3698.0	2226.4	64936.0	0.887	1.080	PASS	FAIL	PASS





Q <sub>G,left</sub> (k)	Q <sub>G,right</sub> (k)	Q <sub>L,left</sub> (k)	Q <sub>L,right</sub> (k)	J <sub>left</sub>	J <sub>right</sub>	KQ <sub>CL,left</sub> (k)	KQ <sub>CL,right</sub> (k)	Beam Shear Acceptance	
								Left	Right
25.4	26.6	73.2	73.2	1.00	1.00	151.4	151.4	PASS	PASS
26.0	25.9	68.8	68.8	1.00	1.00	151.4	151.4	PASS	PASS
26.0	26.0	69.6	69.6	1.00	1.00	151.4	151.4	PASS	PASS
25.9	26.0	68.8	68.8	1.00	1.00	151.4	151.4	PASS	PASS
26.6	25.4	73.2	73.2	1.00	1.00	151.4	151.4	PASS	PASS
25.7	25.8	71.1	71.1	1.00	1.00	151.4	151.4	PASS	PASS
25.7	25.8	68.8	68.8	1.00	1.00	151.4	151.4	PASS	PASS
25.7	25.7	69.8	69.8	1.00	1.00	151.4	151.4	PASS	PASS
25.8	25.7	68.8	68.8	1.00	1.00	151.4	151.4	PASS	PASS
25.8	25.7	71.1	71.1	1.00	1.00	151.4	151.4	PASS	PASS
25.8	25.7	68.5	68.5	1.00	1.00	151.4	151.4	PASS	PASS
25.7	25.8	65.7	65.7	1.00	1.00	151.4	151.4	PASS	PASS
25.7	25.7	66.8	66.8	1.00	1.00	151.4	151.4	PASS	PASS
25.8	25.7	65.7	65.7	1.00	1.00	151.4	151.4	PASS	PASS
25.7	25.8	68.5	68.5	1.00	1.00	151.4	151.4	PASS	PASS
26.0	25.5	66.1	66.1	1.00	1.00	151.4	151.7	PASS	PASS
25.7	25.8	62.6	62.6	1.00	1.00	151.7	151.7	PASS	PASS
25.7	25.7	62.6	62.6	1.00	1.00	151.7	151.7	PASS	PASS
25.8	25.7	62.6	62.6	1.00	1.00	151.7	151.7	PASS	PASS
25.5	26.0	66.1	66.1	1.00	1.00	151.7	151.4	PASS	PASS
26.1	25.3	60.7	60.7	1.00	1.00	151.7	151.7	PASS	PASS
25.7	25.8	57.8	57.8	1.00	1.00	151.7	151.7	PASS	PASS
25.7	25.7	57.9	57.9	1.00	1.00	151.7	151.7	PASS	PASS
25.8	25.7	57.8	57.8	1.00	1.00	151.7	151.7	PASS	PASS
25.3	26.1	60.7	60.7	1.00	1.00	151.7	151.7	PASS	PASS
26.2	25.3	53.5	53.5	1.00	1.00	151.7	151.7	PASS	PASS
25.7	25.8	51.1	51.1	1.00	1.00	151.7	151.7	PASS	PASS
25.7	25.7	51.1	51.1	1.00	1.00	151.7	151.7	PASS	PASS
25.8	25.7	51.1	51.1	1.00	1.00	151.7	151.7	PASS	PASS
25.3	26.2	53.5	53.5	1.00	1.00	151.7	151.7	PASS	PASS
26.4	25.1	43.6	43.6	1.00	1.00	151.4	151.4	PASS	PASS
25.7	25.8	42.1	42.1	1.00	1.00	151.4	151.4	PASS	PASS
25.7	25.7	42.1	42.1	1.00	1.00	151.4	151.4	PASS	PASS
25.8	25.7	42.1	42.1	1.00	1.00	151.4	151.4	PASS	PASS
25.1	26.4	43.6	43.6	1.00	1.00	151.4	151.4	PASS	PASS
21.2	20.8	26.7	26.7	1.00	1.00	151.7	151.7	PASS	PASS
20.9	21.0	25.0	25.0	1.00	1.00	151.7	151.7	PASS	PASS
21.0	21.0	25.1	25.1	1.00	1.00	151.7	151.7	PASS	PASS
21.0	20.9	25.0	25.0	1.00	1.00	151.7	151.7	PASS	PASS
20.8	21.2	26.7	26.7	1.00	1.00	151.7	151.7	PASS	PASS

Section 10b. LDP (Time History) Moment Frame Beam Assessment (LS)

Typical Beam Properties

Table with 2 columns: Property Name and Value. Properties include beam width (bw), height (h), flange extension (bf), cover (c), reinforcement size (#), number of legs, strength values (fc', fy, fu), and knowledge factor (k).

Parameters Below Assume that  $\phi_p/\phi_{bal} < 5$ , and Transverse Reinforcing Conforms. Check these values prior to using this portion of the table

Main data table with columns: BEAM ID, Existing Beam Reinforcing (Left/Right), M+, M-, VDES, left, middle, right, S, L, R, Conform?, and various Q and mkQ values for M+, M-, and M+ categories. The right side includes 'Beam Flexure Acceptance' with Pass/Fail indicators and Q values.

None

J <sub>left</sub>	J <sub>right</sub>	K <sub>Q<sub>C</sub>Left</sub> (k)	K <sub>Q<sub>C</sub>Right</sub> (k)	Beam Shear Acceptance	
				Left	Right
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS





J <sub>left</sub>	J <sub>right</sub>	kQ <sub>C,Left</sub> (k)	kQ <sub>C,Right</sub> (k)	Beam Shear Acceptance	
				Left	Right
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.4	151.4	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS
2.00	2.00	151.7	151.7	PASS	PASS

Section 11a. LDP (Time History) Moment Frame Column Assessment (IO)

Typical Column Properties

Table with 2 columns: Property Name and Value. Properties include width (w), cover (c), concrete strength (f'c), steel yield strength (fy), steel ultimate strength (fu), expected concrete strength (f'c), expected steel yield strength (fy), expected steel ultimate strength (fu), knowledge factor (kappa), and shear reduction factor (eta).

Parameters Below Assume P/Ag is less than 0.6, s/d is less than or equal to 0.5, p is greater than or equal to 0.006, and Vu/(Vd\*k) is less than 0.6. Check these values prior to using this portion of the table

PCA Column Models should use expected behaviours

Main data table with columns for COL ID, dimensions, reinforcement, and various strength/behavior parameters. Includes sub-columns for Yield Zone Reinforcing, Non-Yield Zone Reinforcing, Axial, and Flexure-Axial properties.







Section 11b. LDP (Time History) Moment Frame Column Assessment (LS)

Typical Column Properties

Table with 2 columns: Property Name (w, c, beta\_1, f'\_c, f\_y, f\_u, f'\_e, f\_y, f\_u, kappa, C\_1, C\_2, eta) and Value (28 in, 1.5 in, 0.8, 5000 psi, 60 ksi, 90 ksi, 7500 psi, 75 ksi, 112.5 ksi, 1, 1, 1, 7.195)

Parameters Below Assume P/A\_g f'\_c is less than 0.6, s/d is less than or equal to 0.5, rho is greater than or equal to 0.006, and V\_u/(V\_u/k) is less than 0.6. Check these values prior to using this portion of the table

PCA Column Models should use expected behaviours

Large data table with columns: COL ID, l\_c, Longit Reinf, Yield Zone Reinforcing, Non-Yield Zone Reinforcing, Top Bottom V DESIGN, rho, Top Bottom P DESIGN, Axial Q\_u/A\_g f'\_c, X E\_Lg, Top Bottom P DESIGN, Top Bottom J P DESIGN, Top Bottom M DESIGN, Top Bottom M\_u/m, Top Bottom M\_u/m. Rows represent different column models (e.g., 6A01, 6A02, etc.).

Bottom M <sub>u</sub> /m (k-ft)	Flexure-Axial Acceptance								kQ <sub>CL</sub> yield (k)	kQ <sub>CLnon-yield</sub> (k)	J	Q <sub>c</sub> (k)	Shear Q <sub>c</sub> (k)	Yield Zone Shear Acceptance	Non-Yield Zone Shear Acceptance
	Top DCR <sub>MAX</sub>	Bottom DCR <sub>MAX</sub>	Top DCR <sub>MIN</sub>	Bottom DCR <sub>MIN</sub>	Top P <sub>max</sub>	Bottom P <sub>max</sub>	Top P <sub>min</sub>	Bottom P <sub>min</sub>							
1030.6	0.405	0.707	1.211	1.953	PASS	PASS	FAIL	FAIL	408.0	408.0	2.0	6.00	333.1	PASS	PASS
496.9	0.354	0.348	1.047	0.885	PASS	PASS	FAIL	PASS	408.0	408.0	2.0	11.39	255.9	PASS	PASS
490.9	0.318	0.332	0.914	0.823	PASS	PASS	PASS	PASS	408.0	408.0	2.0	10.18	244.2	PASS	PASS
476.5	0.314	0.327	0.847	0.755	PASS	PASS	PASS	PASS	408.0	408.0	2.0	10.79	237.7	PASS	PASS
423.4	0.306	0.312	0.743	0.637	PASS	PASS	PASS	PASS	408.0	408.0	2.0	10.78	216.9	PASS	PASS
376.8	0.309	0.314	0.648	0.541	PASS	PASS	PASS	PASS	408.0	408.0	2.0	11.18	195.0	PASS	PASS
306.0	0.337	0.311	0.567	0.423	PASS	PASS	PASS	PASS	408.0	408.0	2.0	10.35	169.1	PASS	PASS
139.6	0.271	0.210	0.393	0.190	PASS	PASS	PASS	PASS	408.0	408.0	2.0	13.81	98.9	PASS	PASS
1395.0	0.624	0.850	0.731	0.995	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.16	486.4	PASS	PASS
981.0	0.627	0.616	0.727	0.713	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.02	477.6	PASS	PASS
871.2	0.582	0.564	0.674	0.649	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.32	427.6	PASS	PASS
832.5	0.566	0.557	0.651	0.635	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.45	406.0	PASS	PASS
763.7	0.546	0.532	0.618	0.596	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.63	375.3	PASS	PASS
684.4	0.522	0.504	0.575	0.549	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.71	337.8	PASS	PASS
565.0	0.477	0.442	0.511	0.465	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.85	285.7	PASS	PASS
338.8	0.356	0.284	0.373	0.289	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.94	187.3	PASS	PASS
1429.7	0.630	0.887	0.685	0.964	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.02	487.9	PASS	PASS
987.2	0.646	0.634	0.696	0.683	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.00	477.7	PASS	PASS
913.3	0.630	0.608	0.672	0.648	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.01	447.6	PASS	PASS
803.7	0.571	0.550	0.612	0.589	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.01	394.6	PASS	PASS
751.9	0.550	0.532	0.588	0.569	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.01	368.5	PASS	PASS
677.2	0.525	0.503	0.553	0.531	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.01	332.9	PASS	PASS
561.4	0.481	0.439	0.498	0.456	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	282.7	PASS	PASS
340.9	0.361	0.282	0.368	0.289	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	186.7	PASS	PASS
1429.8	0.630	0.887	0.685	0.964	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.01	487.8	PASS	PASS
987.3	0.646	0.634	0.696	0.683	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.00	477.7	PASS	PASS
913.2	0.630	0.608	0.672	0.648	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.01	447.7	PASS	PASS
803.6	0.571	0.550	0.612	0.589	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.01	394.7	PASS	PASS
751.7	0.550	0.532	0.588	0.569	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	368.5	PASS	PASS
677.0	0.525	0.504	0.554	0.531	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	333.0	PASS	PASS
561.1	0.481	0.440	0.498	0.456	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	282.8	PASS	PASS
340.5	0.361	0.283	0.368	0.288	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.04	186.9	PASS	PASS
1394.1	0.623	0.850	0.732	0.994	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.24	486.8	PASS	PASS
980.5	0.628	0.617	0.726	0.712	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.03	477.5	PASS	PASS
873.1	0.583	0.563	0.672	0.651	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.19	426.7	PASS	PASS
835.2	0.568	0.556	0.648	0.638	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.27	404.8	PASS	PASS
767.5	0.548	0.529	0.615	0.600	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.39	373.6	PASS	PASS
688.9	0.525	0.501	0.571	0.552	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.43	335.9	PASS	PASS
570.1	0.481	0.438	0.507	0.469	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.56	283.5	PASS	PASS
344.5	0.360	0.279	0.368	0.294	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.53	185.0	PASS	PASS
1046.7	0.423	0.696	1.157	1.984	PASS	PASS	FAIL	FAIL	408.0	408.0	2.0	4.02	335.1	PASS	PASS
535.4	0.377	0.323	0.984	0.953	PASS	PASS	PASS	PASS	408.0	408.0	2.0	7.61	259.7	PASS	PASS
524.4	0.340	0.311	0.855	0.879	PASS	PASS	PASS	PASS	408.0	408.0	2.0	6.79	247.6	PASS	PASS
513.0	0.337	0.304	0.789	0.813	PASS	PASS	PASS	PASS	408.0	408.0	2.0	7.20	241.3	PASS	PASS
460.5	0.331	0.287	0.686	0.693	PASS	PASS	PASS	PASS	408.0	408.0	2.0	7.17	220.5	PASS	PASS
415.2	0.339	0.285	0.592	0.596	PASS	PASS	PASS	PASS	408.0	408.0	2.0	7.50	198.7	PASS	PASS
343.6	0.370	0.277	0.516	0.475	PASS	PASS	PASS	PASS	408.0	408.0	2.0	6.70	172.8	PASS	PASS
184.3	0.329	0.159	0.323	0.250	PASS	PASS	PASS	PASS	408.0	408.0	2.0	9.89	102.9	PASS	PASS

Section 11c. LDP (Time History) Moment Frame Column Assessment (CP)

Typical Column Properties table with parameters: w, c, beta1, fc, fr, fu, fc', fy, fs, kappa, C1, C2, eta

Parameters Below Assume P/Ag is less than 0.6, s/d is less than or equal to 0.5, p is greater than or equal to 0.006, and Vn/(Vn+k) is less than 0.6. Check these values prior to using this portion of the table

PCA Column Models should use expected behaviours

Main data table with columns for COL ID, geometry (l, #, size, spc), reinforcement (Yield Zone, Non-Yield Zone), axial forces (Top, Bottom), and flexure-axial moments (Top, Bottom) in both kips and M\_k/ft.

Bottom M <sub>u</sub> /m (k-ft)	Top DCR <sub>MAX</sub>	Bottom DCR <sub>MAX</sub>	Top DCR <sub>MIN</sub>	Bottom DCR <sub>MIN</sub>	Flexure-Axial Acceptance				kQ <sub>CL</sub> yield (k)	kQ <sub>CL,non-yield</sub> (k)	J	Q <sub>c</sub> (k)	Shear Q <sub>c</sub> (k)	Yield Zone Shear Acceptance	Non-Yield Zone Shear Acceptance
					Top P <sub>max</sub>	Bottom P <sub>max</sub>	Top P <sub>min</sub>	Bottom P <sub>min</sub>							
1208.4	0.612	1.065	2.358	3.802	PASS	FAIL	FAIL	FAIL	408.0	408.0	2.0	6.00	468.7	PASS	PASS
586.8	0.488	0.471	1.795	1.534	PASS	PASS	FAIL	FAIL	408.0	408.0	2.0	11.39	360.2	PASS	PASS
579.0	0.412	0.423	1.416	1.292	PASS	PASS	FAIL	FAIL	408.0	408.0	2.0	10.18	343.5	PASS	PASS
562.7	0.381	0.391	1.211	1.098	PASS	PASS	FAIL	FAIL	408.0	408.0	2.0	10.79	334.3	PASS	PASS
500.5	0.346	0.345	0.993	0.868	PASS	PASS	PASS	PASS	408.0	408.0	2.0	10.78	305.0	PASS	PASS
446.2	0.337	0.334	0.818	0.699	PASS	PASS	PASS	PASS	408.0	408.0	2.0	11.18	274.2	PASS	PASS
363.2	0.367	0.329	0.684	0.525	PASS	PASS	PASS	PASS	408.0	408.0	2.0	10.35	237.7	PASS	PASS
169.6	0.311	0.225	0.454	0.234	PASS	PASS	PASS	PASS	408.0	408.0	2.0	13.81	139.2	PASS	PASS
1626.9	0.739	1.008	0.878	1.193	PASS	FAIL	PASS	FAIL	469.5	375.6	2.0	0.16	684.2	PASS	PASS
1148.1	0.736	0.724	0.872	0.853	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.02	671.6	PASS	PASS
1019.9	0.679	0.659	0.806	0.776	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.32	601.2	PASS	PASS
974.7	0.658	0.649	0.775	0.758	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.45	570.8	PASS	PASS
894.4	0.632	0.617	0.733	0.708	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.63	527.5	PASS	PASS
801.7	0.605	0.584	0.680	0.649	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.71	474.8	PASS	PASS
662.0	0.555	0.514	0.603	0.550	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.85	401.5	PASS	PASS
397.3	0.415	0.331	0.437	0.340	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.94	263.0	PASS	PASS
1679.4	0.747	1.054	0.810	1.138	PASS	FAIL	PASS	FAIL	469.5	375.6	2.0	0.02	686.1	PASS	PASS
1156.9	0.764	0.751	0.820	0.803	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.00	671.8	PASS	PASS
1069.0	0.742	0.717	0.789	0.759	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.01	629.5	PASS	PASS
940.7	0.670	0.646	0.718	0.690	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.01	555.0	PASS	PASS
880.0	0.644	0.623	0.690	0.668	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.01	518.2	PASS	PASS
792.6	0.614	0.589	0.650	0.622	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.01	468.1	PASS	PASS
657.0	0.562	0.514	0.584	0.534	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	397.6	PASS	PASS
398.9	0.422	0.331	0.432	0.338	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	262.6	PASS	PASS
1679.5	0.748	1.054	0.810	1.138	PASS	FAIL	PASS	FAIL	469.5	375.6	2.0	0.01	686.0	PASS	PASS
1157.0	0.764	0.751	0.820	0.803	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.00	671.8	PASS	PASS
1068.9	0.742	0.717	0.789	0.759	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.01	629.5	PASS	PASS
940.6	0.670	0.646	0.718	0.690	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.01	555.0	PASS	PASS
879.9	0.643	0.623	0.690	0.668	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	518.3	PASS	PASS
792.4	0.614	0.589	0.650	0.622	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	468.2	PASS	PASS
656.8	0.562	0.514	0.584	0.534	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.02	397.7	PASS	PASS
398.6	0.422	0.331	0.432	0.338	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.04	262.8	PASS	PASS
1626.2	0.739	1.009	0.879	1.193	PASS	FAIL	PASS	FAIL	469.5	375.6	2.0	0.24	684.6	PASS	PASS
1147.7	0.736	0.724	0.871	0.853	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.03	671.6	PASS	PASS
1021.5	0.681	0.658	0.805	0.777	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.19	600.3	PASS	PASS
977.0	0.660	0.647	0.773	0.759	PASS	PASS	PASS	PASS	469.5	375.6	2.0	0.27	569.5	PASS	PASS
897.6	0.635	0.615	0.730	0.710	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.39	525.8	PASS	PASS
805.4	0.608	0.582	0.677	0.652	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.43	472.9	PASS	PASS
666.2	0.558	0.510	0.600	0.553	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.56	399.2	PASS	PASS
402.0	0.419	0.327	0.433	0.344	PASS	PASS	PASS	PASS	469.5	469.5	2.0	0.53	260.7	PASS	PASS
1221.8	0.631	1.053	2.288	3.846	PASS	FAIL	FAIL	FAIL	408.0	408.0	2.0	4.02	470.7	PASS	PASS
618.9	0.510	0.447	1.715	1.616	PASS	PASS	FAIL	FAIL	408.0	408.0	2.0	7.61	364.0	PASS	PASS
607.0	0.432	0.403	1.350	1.355	PASS	PASS	FAIL	FAIL	408.0	408.0	2.0	6.79	346.9	PASS	PASS
593.0	0.401	0.371	1.151	1.157	PASS	PASS	FAIL	FAIL	408.0	408.0	2.0	7.20	337.9	PASS	PASS
531.5	0.366	0.325	0.938	0.922	PASS	PASS	PASS	PASS	408.0	408.0	2.0	7.17	308.7	PASS	PASS
478.2	0.360	0.312	0.766	0.570	PASS	PASS	PASS	PASS	408.0	408.0	2.0	7.50	277.9	PASS	PASS
394.5	0.392	0.303	0.640	0.570	PASS	PASS	PASS	PASS	408.0	408.0	2.0	6.70	241.3	PASS	PASS
206.8	0.357	0.185	0.394	0.285	PASS	PASS	PASS	PASS	408.0	408.0	2.0	9.89	143.2	PASS	PASS



**Section 12a. LDP (Time History) Moment Frame Joint Assessment (IO)**

Typical Joint Details

w	28	in, Column Dimension
A <sub>j</sub>	784	in <sup>2</sup>
C <sub>1</sub>	1	
C <sub>2</sub>	1	
f <sub>c</sub> '	5000	psi, Lower Bound Compressive Strength of Concrete
f <sub>y</sub>	60	ksi, Lower Bound Reinforcing Yield Strength
f <sub>c</sub> '	7500	psi, Expected Compressive Strength of Concrete
f <sub>y</sub>	75	ksi, Expected Reinforcing Yield Strength
K	1	Knowledge Factor

		Linear Static Procedure									
Joint ID	ΣM <sub>nc</sub> /ΣM <sub>nb</sub>	Offset Rigidities		Joint Shear		J	Q <sub>UF</sub> (V <sub>j</sub> ) (k)	Condition	Int/Ext	Q <sub>CL</sub> (V <sub>n</sub> ) (k)	Joint Shear Acceptance
		Col	Beam	(V <sub>g,L</sub> + V <sub>g,R</sub> )*h/2l <sub>c</sub> (k)	(V <sub>e,L</sub> + V <sub>e,R</sub> )*h/2l <sub>c</sub> (k)						
6A01	1.50	1	0	2.61	7.5	1.0	524.067	Other (Main)	Exterior	665.2	PASS
6A02	1.50	1	0	2.90	8.0	1.0	503.7549	Other (Main)	Exterior	665.2	PASS
6A03	1.50	1	0	2.92	7.7	1.0	504.0401	Other (Main)	Exterior	665.2	PASS
6A04	1.87	1	0	2.94	7.5	1.0	399.189	Other (Main)	Exterior	665.2	PASS
6A05	1.87	1	0	2.95	6.8	1.0	396.0336	Other (Main)	Exterior	665.2	PASS
6A06	1.87	1	0	2.96	6.0	1.0	396.8342	Other (Main)	Exterior	665.2	PASS
6A07	2.45	1	0	2.98	4.9	1.0	342.7736	Other (Main)	Exterior	665.2	PASS
6A08	1.53	1	0	4.78	6.0	1.0	260.8943	Other (Roof)	Exterior	443.5	PASS
6B01	1.32	1	0	5.41	14.6	1.0	451.4115	2 Opp (Main)	Interior	831.6	PASS
6B02	1.34	1	0	5.81	15.8	1.0	440.2923	2 Opp (Main)	Interior	831.6	PASS
6B03	1.34	1	0	5.80	15.2	1.0	440.9484	2 Opp (Main)	Interior	831.6	PASS
6B04	1.68	1	0	5.78	14.5	1.0	342.956	2 Opp (Main)	Interior	831.6	PASS
6B05	1.69	1	0	5.76	13.4	1.0	344.7949	2 Opp (Main)	Interior	831.6	PASS
6B06	1.69	1	0	5.76	11.8	1.0	346.3715	2 Opp (Main)	Interior	831.6	PASS
6B07	1.98	1	0	5.73	9.7	1.0	293.369	2 Opp (Main)	Interior	831.6	PASS
6B08	1.11	0.5	0.5	9.42	11.7	1.0	167.6885	2 Opp (Roof)	Interior	665.2	PASS
6C01	1.34	1	0	5.34	14.2	1.0	454.4965	2 Opp (Main)	Interior	831.6	PASS
6C02	1.34	1	0	5.81	15.7	1.0	440.4376	2 Opp (Main)	Interior	831.6	PASS
6C03	1.34	1	0	5.81	15.0	1.0	441.1282	2 Opp (Main)	Interior	831.6	PASS
6C04	1.69	1	0	5.82	14.1	1.0	343.9852	2 Opp (Main)	Interior	831.6	PASS
6C05	1.69	1	0	5.82	13.1	1.0	345.0579	2 Opp (Main)	Interior	831.6	PASS
6C06	1.69	1	0	5.82	11.5	1.0	346.5811	2 Opp (Main)	Interior	831.6	PASS
6C07	1.98	1	0	5.82	9.5	1.0	293.4565	2 Opp (Main)	Interior	831.6	PASS
6C08	1.11	0.5	0.5	9.49	11.3	1.0	168.0006	2 Opp (Roof)	Interior	665.2	PASS
6D01	1.34	1	0	5.34	14.2	1.0	454.4965	2 Opp (Main)	Interior	831.6	PASS
6D02	1.34	1	0	5.81	15.7	1.0	440.4376	2 Opp (Main)	Interior	831.6	PASS
6D03	1.34	1	0	5.81	15.0	1.0	441.1282	2 Opp (Main)	Interior	831.6	PASS
6D04	1.69	1	0	5.82	14.1	1.0	343.9852	2 Opp (Main)	Interior	831.6	PASS
6D05	1.69	1	0	5.82	13.1	1.0	345.0579	2 Opp (Main)	Interior	831.6	PASS
6D06	1.69	1	0	5.82	11.5	1.0	346.5811	2 Opp (Main)	Interior	831.6	PASS
6D07	1.98	1	0	5.82	9.5	1.0	293.4565	2 Opp (Main)	Interior	831.6	PASS
6D08	1.11	0.5	0.5	9.49	11.3	1.0	168.0006	2 Opp (Roof)	Interior	665.2	PASS
6E01	1.32	1	0	5.41	14.6	1.0	451.4115	2 Opp (Main)	Interior	831.6	PASS
6E02	1.34	1	0	5.81	15.8	1.0	440.2923	2 Opp (Main)	Interior	831.6	PASS
6E03	1.34	1	0	5.80	15.2	1.0	440.9484	2 Opp (Main)	Interior	831.6	PASS
6E04	1.68	1	0	5.78	14.5	1.0	342.956	2 Opp (Main)	Interior	831.6	PASS
6E05	1.69	1	0	5.76	13.4	1.0	344.7949	2 Opp (Main)	Interior	831.6	PASS
6E06	1.69	1	0	5.76	11.8	1.0	346.3715	2 Opp (Main)	Interior	831.6	PASS
6E07	1.98	1	0	5.73	9.7	1.0	293.369	2 Opp (Main)	Interior	831.6	PASS
6E08	1.11	0.5	0.5	9.42	11.7	1.0	167.6885	2 Opp (Roof)	Interior	665.2	PASS
6F01	1.50	1	0	2.61	7.5	1.0	524.067	Other (Main)	Exterior	665.2	PASS
6F02	1.50	1	0	2.90	8.0	1.0	503.7549	Other (Main)	Exterior	665.2	PASS
6F03	1.50	1	0	2.92	7.7	1.0	504.0401	Other (Main)	Exterior	665.2	PASS
6F04	1.87	1	0	2.94	7.5	1.0	399.189	Other (Main)	Exterior	665.2	PASS
6F05	1.87	1	0	2.95	6.8	1.0	396.0336	Other (Main)	Exterior	665.2	PASS
6F06	1.87	1	0	2.96	6.0	1.0	396.8342	Other (Main)	Exterior	665.2	PASS
6F07	2.45	1	0	2.98	4.9	1.0	342.7736	Other (Main)	Exterior	665.2	PASS
6F08	1.53	1	0	4.78	6.0	1.0	260.8943	Other (Roof)	Exterior	443.5	PASS



**Section 12b. LDP (Time History) Moment Frame Joint Assessment (LS)**

Typical Joint Details

w	28	in, Column Dimension
A <sub>j</sub>	784	in <sup>2</sup>
C <sub>1</sub>	1	
C <sub>2</sub>	1	
f <sub>c</sub> '	5000	psi, Lower Bound Compressive Strength of Concrete
f <sub>y</sub>	60	ksi, Lower Bound Reinforcing Yield Strength
f <sub>c</sub> '	7500	psi, Expected Compressive Strength of Concrete
f <sub>y</sub>	75	ksi, Expected Reinforcing Yield Strength
K	1	Knowledge Factor

Joint ID	ΣM <sub>nc</sub> /ΣM <sub>nb</sub>	Offset Rigidities		Linear Static Procedure							Joint Shear Acceptance
		Col	Beam	Joint Shear		J	Q <sub>UF</sub> (V <sub>j</sub> ) (k)	Condition	Int/Ext	Q <sub>CL</sub> (V <sub>n</sub> ) (k)	
		(V <sub>e,L</sub> + V <sub>e,R</sub> )*h/2l <sub>c</sub> (k)	(V <sub>e,L</sub> + V <sub>e,R</sub> )*h/2l <sub>c</sub> (k)								
6A01	1.50	1	0	2.61	13.2	2.0	525.0242	Other (Main)	Exterior	665.2	PASS
6A02	1.50	1	0	2.90	14.0	2.0	504.7746	Other (Main)	Exterior	665.2	PASS
6A03	1.50	1	0	2.92	13.5	2.0	505.0221	Other (Main)	Exterior	665.2	PASS
6A04	1.87	1	0	2.94	13.0	2.0	400.1365	Other (Main)	Exterior	665.2	PASS
6A05	1.87	1	0	2.95	12.0	2.0	396.9036	Other (Main)	Exterior	665.2	PASS
6A06	1.87	1	0	2.96	10.5	2.0	397.6017	Other (Main)	Exterior	665.2	PASS
6A07	2.45	1	0	2.98	8.6	2.0	343.3985	Other (Main)	Exterior	665.2	PASS
6A08	1.53	1	0	4.78	10.5	2.0	261.6609	Other (Roof)	Exterior	443.5	PASS
6B01	1.32	1	0	5.41	25.5	2.0	453.268	2 Opp (Main)	Interior	831.6	PASS
6B02	1.34	1	0	5.81	27.6	2.0	442.2988	2 Opp (Main)	Interior	831.6	PASS
6B03	1.34	1	0	5.80	26.5	2.0	442.873	2 Opp (Main)	Interior	831.6	PASS
6B04	1.68	1	0	5.78	25.4	2.0	344.8011	2 Opp (Main)	Interior	831.6	PASS
6B05	1.69	1	0	5.76	23.4	2.0	346.4945	2 Opp (Main)	Interior	831.6	PASS
6B06	1.69	1	0	5.76	20.6	2.0	347.8718	2 Opp (Main)	Interior	831.6	PASS
6B07	1.98	1	0	5.73	16.9	2.0	294.5977	2 Opp (Main)	Interior	831.6	PASS
6B08	1.11	0.5	0.5	9.42	20.4	2.0	169.1732	2 Opp (Roof)	Interior	665.2	PASS
6C01	1.34	1	0	5.34	24.9	2.0	456.3062	2 Opp (Main)	Interior	831.6	PASS
6C02	1.34	1	0	5.81	27.3	2.0	442.4255	2 Opp (Main)	Interior	831.6	PASS
6C03	1.34	1	0	5.81	26.1	2.0	443.0284	2 Opp (Main)	Interior	831.6	PASS
6C04	1.69	1	0	5.82	24.7	2.0	345.781	2 Opp (Main)	Interior	831.6	PASS
6C05	1.69	1	0	5.82	22.8	2.0	346.7174	2 Opp (Main)	Interior	831.6	PASS
6C06	1.69	1	0	5.82	20.2	2.0	348.047	2 Opp (Main)	Interior	831.6	PASS
6C07	1.98	1	0	5.82	16.6	2.0	294.6635	2 Opp (Main)	Interior	831.6	PASS
6C08	1.11	0.5	0.5	9.49	19.8	2.0	169.4377	2 Opp (Roof)	Interior	665.2	PASS
6D01	1.34	1	0	5.34	24.9	2.0	456.3062	2 Opp (Main)	Interior	831.6	PASS
6D02	1.34	1	0	5.81	27.3	2.0	442.4255	2 Opp (Main)	Interior	831.6	PASS
6D03	1.34	1	0	5.81	26.1	2.0	443.0284	2 Opp (Main)	Interior	831.6	PASS
6D04	1.69	1	0	5.82	24.7	2.0	345.781	2 Opp (Main)	Interior	831.6	PASS
6D05	1.69	1	0	5.82	22.8	2.0	346.7174	2 Opp (Main)	Interior	831.6	PASS
6D06	1.69	1	0	5.82	20.2	2.0	348.047	2 Opp (Main)	Interior	831.6	PASS
6D07	1.98	1	0	5.82	16.6	2.0	294.6635	2 Opp (Main)	Interior	831.6	PASS
6D08	1.11	0.5	0.5	9.49	19.8	2.0	169.4377	2 Opp (Roof)	Interior	665.2	PASS
6E01	1.32	1	0	5.41	25.5	2.0	453.268	2 Opp (Main)	Interior	831.6	PASS
6E02	1.34	1	0	5.81	27.6	2.0	442.2988	2 Opp (Main)	Interior	831.6	PASS
6E03	1.34	1	0	5.80	26.5	2.0	442.873	2 Opp (Main)	Interior	831.6	PASS
6E04	1.68	1	0	5.78	25.4	2.0	344.8011	2 Opp (Main)	Interior	831.6	PASS
6E05	1.69	1	0	5.76	23.4	2.0	346.4945	2 Opp (Main)	Interior	831.6	PASS
6E06	1.69	1	0	5.76	20.6	2.0	347.8718	2 Opp (Main)	Interior	831.6	PASS
6E07	1.98	1	0	5.73	16.9	2.0	294.5977	2 Opp (Main)	Interior	831.6	PASS
6E08	1.11	0.5	0.5	9.42	20.4	2.0	169.1732	2 Opp (Roof)	Interior	665.2	PASS
6F01	1.50	1	0	2.61	13.2	2.0	525.0242	Other (Main)	Exterior	665.2	PASS
6F02	1.50	1	0	2.90	14.0	2.0	504.7746	Other (Main)	Exterior	665.2	PASS
6F03	1.50	1	0	2.92	13.5	2.0	505.0221	Other (Main)	Exterior	665.2	PASS
6F04	1.87	1	0	2.94	13.0	2.0	400.1365	Other (Main)	Exterior	665.2	PASS
6F05	1.87	1	0	2.95	12.0	2.0	396.9036	Other (Main)	Exterior	665.2	PASS
6F06	1.87	1	0	2.96	10.5	2.0	397.6017	Other (Main)	Exterior	665.2	PASS
6F07	2.45	1	0	2.98	8.6	2.0	343.3985	Other (Main)	Exterior	665.2	PASS
6F08	1.53	1	0	4.78	10.5	2.0	261.6609	Other (Roof)	Exterior	443.5	PASS



### Section 12c. LDP (Time History) Moment Frame Joint Assessment (CP)

Typical Joint Details

w	28	in, Column Dimension
A <sub>j</sub>	784	in <sup>2</sup>
C <sub>1</sub>	1	
C <sub>2</sub>	1	
f <sub>c</sub> '	5000	psi, Lower Bound Compressive Strength of Concrete
f <sub>y</sub>	60	ksi, Lower Bound Reinforcing Yield Strength
f <sub>c</sub> '	7500	psi, Expected Compressive Strength of Concrete
f <sub>y</sub>	75	ksi, Expected Reinforcing Yield Strength
K	1	Knowledge Factor

				Linear Static Procedure							
Joint ID	ΣM <sub>nc</sub> /ΣM <sub>nb</sub>	Offset Rigidities		Joint Shear		J	Q <sub>UF</sub> (V <sub>j</sub> ) (k)	Condition	Int/Ext	Q <sub>CL</sub> (V <sub>n</sub> ) (k)	Joint Shear Acceptance
		Col	Beam	(V <sub>e,L</sub> + V <sub>e,R</sub> )*h/2l <sub>c</sub> (k)	(V <sub>e,L</sub> + V <sub>e,R</sub> )*h/2l <sub>c</sub> (k)						
6A01	1.50	1	0	2.61	18.5	2.0	522.3625	Other (Main)	Exterior	665.2	PASS
6A02	1.50	1	0	2.90	19.7	2.0	501.9394	Other (Main)	Exterior	665.2	PASS
6A03	1.50	1	0	2.92	19.0	2.0	502.2918	Other (Main)	Exterior	665.2	PASS
6A04	1.87	1	0	2.94	18.3	2.0	397.5021	Other (Main)	Exterior	665.2	PASS
6A05	1.87	1	0	2.95	16.8	2.0	394.4844	Other (Main)	Exterior	665.2	PASS
6A06	1.87	1	0	2.96	14.8	2.0	395.4589	Other (Main)	Exterior	665.2	PASS
6A07	2.45	1	0	2.98	12.1	2.0	341.661	Other (Main)	Exterior	665.2	PASS
6A08	1.53	1	0	4.78	14.8	2.0	259.5293	Other (Roof)	Exterior	443.5	PASS
6B01	1.32	1	0	5.41	35.8	2.0	448.1057	2 Opp (Main)	Interior	831.6	PASS
6B02	1.34	1	0	5.81	38.7	2.0	436.7199	2 Opp (Main)	Interior	831.6	PASS
6B03	1.34	1	0	5.80	37.2	2.0	437.5216	2 Opp (Main)	Interior	831.6	PASS
6B04	1.68	1	0	5.78	35.6	2.0	339.6711	2 Opp (Main)	Interior	831.6	PASS
6B05	1.69	1	0	5.76	32.8	2.0	341.769	2 Opp (Main)	Interior	831.6	PASS
6B06	1.69	1	0	5.76	29.0	2.0	343.683	2 Opp (Main)	Interior	831.6	PASS
6B07	1.98	1	0	5.73	23.7	2.0	291.1812	2 Opp (Main)	Interior	831.6	PASS
6B08	1.11	0.5	0.5	9.42	28.7	2.0	165.0451	2 Opp (Roof)	Interior	665.2	PASS
6C01	1.34	1	0	5.34	34.9	2.0	451.2743	2 Opp (Main)	Interior	831.6	PASS
6C02	1.34	1	0	5.81	38.4	2.0	436.8982	2 Opp (Main)	Interior	831.6	PASS
6C03	1.34	1	0	5.81	36.7	2.0	437.7449	2 Opp (Main)	Interior	831.6	PASS
6C04	1.69	1	0	5.82	34.7	2.0	340.7877	2 Opp (Main)	Interior	831.6	PASS
6C05	1.69	1	0	5.82	32.0	2.0	342.1032	2 Opp (Main)	Interior	831.6	PASS
6C06	1.69	1	0	5.82	28.3	2.0	343.9538	2 Opp (Main)	Interior	831.6	PASS
6C07	1.98	1	0	5.82	23.3	2.0	291.3073	2 Opp (Main)	Interior	831.6	PASS
6C08	1.11	0.5	0.5	9.49	27.7	2.0	165.4422	2 Opp (Roof)	Interior	665.2	PASS
6D01	1.34	1	0	5.34	34.9	2.0	451.2743	2 Opp (Main)	Interior	831.6	PASS
6D02	1.34	1	0	5.81	38.4	2.0	436.8982	2 Opp (Main)	Interior	831.6	PASS
6D03	1.34	1	0	5.81	36.7	2.0	437.7449	2 Opp (Main)	Interior	831.6	PASS
6D04	1.69	1	0	5.82	34.7	2.0	340.7877	2 Opp (Main)	Interior	831.6	PASS
6D05	1.69	1	0	5.82	32.0	2.0	342.1032	2 Opp (Main)	Interior	831.6	PASS
6D06	1.69	1	0	5.82	28.3	2.0	343.9538	2 Opp (Main)	Interior	831.6	PASS
6D07	1.98	1	0	5.82	23.3	2.0	291.3073	2 Opp (Main)	Interior	831.6	PASS
6D08	1.11	0.5	0.5	9.49	27.7	2.0	165.4422	2 Opp (Roof)	Interior	665.2	PASS
6E01	1.32	1	0	5.41	35.8	2.0	448.1057	2 Opp (Main)	Interior	831.6	PASS
6E02	1.34	1	0	5.81	38.7	2.0	436.7199	2 Opp (Main)	Interior	831.6	PASS
6E03	1.34	1	0	5.80	37.2	2.0	437.5216	2 Opp (Main)	Interior	831.6	PASS
6E04	1.68	1	0	5.78	35.6	2.0	339.6711	2 Opp (Main)	Interior	831.6	PASS
6E05	1.69	1	0	5.76	32.8	2.0	341.769	2 Opp (Main)	Interior	831.6	PASS
6E06	1.69	1	0	5.76	29.0	2.0	343.683	2 Opp (Main)	Interior	831.6	PASS
6E07	1.98	1	0	5.73	23.7	2.0	291.1812	2 Opp (Main)	Interior	831.6	PASS
6E08	1.11	0.5	0.5	9.42	28.7	2.0	165.0451	2 Opp (Roof)	Interior	665.2	PASS
6F01	1.50	1	0	2.61	18.5	2.0	522.3625	Other (Main)	Exterior	665.2	PASS
6F02	1.50	1	0	2.90	19.7	2.0	501.9394	Other (Main)	Exterior	665.2	PASS
6F03	1.50	1	0	2.92	19.0	2.0	502.2918	Other (Main)	Exterior	665.2	PASS
6F04	1.87	1	0	2.94	18.3	2.0	397.5021	Other (Main)	Exterior	665.2	PASS
6F05	1.87	1	0	2.95	16.8	2.0	394.4844	Other (Main)	Exterior	665.2	PASS
6F06	1.87	1	0	2.96	14.8	2.0	395.4589	Other (Main)	Exterior	665.2	PASS
6F07	2.45	1	0	2.98	12.1	2.0	341.661	Other (Main)	Exterior	665.2	PASS
6F08	1.53	1	0	4.78	14.8	2.0	259.5293	Other (Roof)	Exterior	443.5	PASS

**Section 13a. LDP (Time History) Shear Wall Assessment (IO)**

## Typical Wall Properties

$f'_c$	5000	psi, Lower Bound Concrete Compressive Strength
$f_y$	60	ksi, Lower Bound Reinforcing Steel Yield Strength
$f_u$	90	ksi, Lower Bound Reinforcing Steel Ultimate Strength
$f'_c$	7500	psi, Expected Concrete Compressive Strength
$f_y$	75	ksi, Expected Reinforcing Steel Yield Strength
$f_u$	112.5	ksi, Expected Reinforcing Steel Ultimate Strength
$t_w$	16	in, Shear Wall Thickness
$h_w$	106	ft, Shear Wall Height
$l_w$	22.33	ft, Shear wall Length
Slender		Wall Slenderness (ASCE 41-06 Section C6.7.1)
w	28	in, Boundary Element Width
$A_B$	4960	kips, Nominal Axial Capacity (No Eccentricity, Concrete Only)
k	1.0	Knowledge Factor
$C_1$	1.0	
$C_2$	1.0	

**Exterior Shear Wall**

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		$A_{st}^{-1}$ (in <sup>2</sup> )	Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{n,P=0}^{4.5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)
						Size	Spc (in)	Size	Spc (in)		Size	Spc (in)					$P > .35P_0^2$	$S_h > 18''^2$	$S_v > 18''^2$				
Level 08	106	13	111.7	80.6	4864.0	#5	18.0	#5	18.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	23270.4	377.4	151.3	1088.9
Level 07	93	13	248.7	165.7	12997.8	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	626.2	151.3	1158.1
Level 06	80	13	385.8	250.8	22541.5	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	747.2	151.3	1158.1
Level 05	67	13	522.8	336.0	32364.7	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	811.5	151.3	1158.1
Level 04	54	13	659.9	421.1	42124.4	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	911.1	151.3	1158.1
Level 03	41	13	796.9	506.2	52206.2	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	1065.2	151.3	1158.1
Level 02	28	13	934.0	591.3	63727.2	#5	15.0	#5	15.0	20.32	#5	4.0	YES	18880.9	28062.3	0.673	NO	NO	NO	52017.3	1194.4	151.3	1158.1
Level 01	15	15	1074.6	679.4	80183.8	#5	10.0	#5	10.0	29.32	#5	4.0	YES	19307.4	28587.3	0.675	NO	NO	NO	65401.3	1287.9	174.6	1365.8

**Interior Shear Wall**

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		$A_{st}^{-1}$ (in <sup>2</sup> )	Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{n,P=0}^{4.5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)
						Size	Spc (in)	Size	Spc (in)		Size	Spc (in)					$P > .35P_0^2$	$S_h > 18''^2$	$S_v > 18''^2$				
Level 08	106	13	190.2	134.1	4486.1	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	348.1	89.8	1088.9
Level 07	93	13	426.7	273.5	11988.1	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	577.6	89.8	1088.9
Level 06	80	13	663.1	413.0	20790.3	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	689.2	89.8	1158.1
Level 05	67	13	899.6	552.4	29850.4	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	748.5	89.8	1158.1
Level 04	54	13	1136.0	691.9	38851.8	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	840.3	89.8	1158.1
Level 03	41	13	1372.5	831.3	48150.4	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	982.4	89.8	1158.1
Level 02	28	13	1608.9	970.8	58776.4	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1101.6	89.8	1158.1
Level 01	15	15	1849.0	1113.2	73954.5	#5	15.0	#5	10.0	15.24	#5	5.0	YES	18640.2	27766.0	0.671	NO	NO	NO	38815.0	1187.9	103.6	1158.1

1. Area of Longitudinal Steel in Boundary Elements

2. If "NO", Shear Wall may not be taken as effective in resisting seismic forces per ASCE 41-06 Section 6.7.1.1

3. Nominal Moment Capacity taken from PCA Column Analysis corresponding to minimum design Axial Load

4. Nominal Moment Capacity at P=0 taken from PCA Column Analysis, including only boundary reinforcing.  $\Phi = 1.0$ , and expected strengths are used.

5. Nominal Moment Capacity at P=0 is not required to exceed Moment Demand, as the compressive force is assumed to be 0 and therefore no interaction effects are taken into account.

6. Maximum shear force calculated based on nominal flexural capacity of base of wall.

7. Wall is considered controlled by flexure if the maximum shear force value is less than the nominal shear capacity. This indicates that the wall will fail in flexure prior to reaching the shear capacity.





Control <sup>7</sup>	J*P <sub>DES,MAX</sub> (kips)	J*P <sub>DES,MIN</sub> (kips)	M <sub>DES</sub> /m (k-ft)	DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Axial-Flexural Acceptance		Shear Acceptance
Flexure	111.7	80.6	2432.0	0.100	0.101	PASS	PASS	PASS
Flexure	248.7	165.7	6498.9	0.235	0.242	PASS	PASS	PASS
Flexure	385.8	250.8	11270.7	0.388	0.407	PASS	PASS	PASS
Flexure	522.8	336.0	16182.4	0.532	0.567	PASS	PASS	PASS
Flexure	659.9	421.1	21062.2	0.509	0.540	PASS	PASS	PASS
Flexure	796.9	506.2	26103.1	0.612	0.655	PASS	PASS	PASS
Flexure	934.0	591.3	31886.3	0.574	0.610	PASS	PASS	PASS
Flexure	1074.6	679.4	47516.3	0.629	0.661	PASS	PASS	PASS

Control <sup>7</sup>	J*P <sub>DES,MAX</sub> (kips)	J*P <sub>DES,MIN</sub> (kips)	M <sub>DES</sub> /m (k-ft)	DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Axial-Flexural Acceptance		Shear Acceptance
Flexure	190.2	134.1	2243.1	0.066	0.067	PASS	PASS	PASS
Flexure	426.7	273.5	5994.0	0.165	0.173	PASS	PASS	PASS
Flexure	663.1	413.0	10395.1	0.257	0.274	PASS	PASS	PASS
Flexure	899.6	552.4	14925.2	0.349	0.380	PASS	PASS	PASS
Flexure	1136.0	691.9	19425.9	0.432	0.477	PASS	PASS	PASS
Flexure	1372.5	831.3	24075.2	0.509	0.572	PASS	PASS	PASS
Flexure	1608.9	970.8	29465.5	0.595	0.679	PASS	PASS	PASS
Flexure	1849.0	1113.2	43824.9	0.774	0.883	PASS	PASS	PASS

**Section 13b. LDP (Time History) Shear Wall Assessment (LS)**

## Typical Wall Properties

$f_c'$	5000	psi, Lower Bound Concrete Compressive Strength
$f_y$	60	ksi, Lower Bound Reinforcing Steel Yield Strength
$f_u$	90	ksi, Lower Bound Reinforcing Steel Ultimate Strength
$f_c'$	7500	psi, Expected Concrete Compressive Strength
$f_y$	75	ksi, Expected Reinforcing Steel Yield Strength
$f_u$	112.5	ksi, Expected Reinforcing Steel Ultimate Strength
$t_w$	16	in, Shear Wall Thickness
$h_w$	106	ft, Shear Wall Height
$l_w$	22.33	ft, Shear wall Length
Slender		Wall Slenderness (ASCE 41-06 Section C6.7.1)
$w$	28	in, Boundary Element Width
$A_g$	4960	kips, Nominal Axial Capacity (No Eccentricity, Concrete Only)
$\kappa$	1.0	Knowledge Factor
$C_1$	1.0	
$C_2$	1.0	

**Exterior Shear Wall**

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{N,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)	
						Size	Spc (in)	Size	Spc (in)	Size	Spc (in)					$P > .35P_o^{?2}$	$s_h > 18''^{?2}$	$s_v > 18''^{?2}$					
Level 08	106	13	111.7	80.6	8492.4	#5	18.0	#5	18.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	23270.4	658.9	151.3	1088.9
Level 07	93	13	248.7	165.7	22693.8	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1093.4	151.3	1158.1
Level 06	80	13	385.8	250.8	39356.6	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1304.6	151.3	1158.1
Level 05	67	13	522.8	336.0	56507.6	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1416.9	151.3	1158.1
Level 04	54	13	659.9	421.1	73547.7	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	1590.7	151.3	1158.1
Level 03	41	13	796.9	506.2	91150.2	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	1859.7	151.3	1158.1
Level 02	28	13	934.0	591.3	111265.5	#5	15.0	#5	15.0	20.32	#5	4.0	YES	18880.9	28062.3	0.673	NO	NO	NO	52017.3	2085.4	151.3	1158.1
Level 01	15	15	1074.6	679.4	139998.1	#5	10.0	#5	10.0	29.32	#5	4.0	YES	19307.4	28587.3	0.675	NO	NO	NO	65401.3	2248.6	174.6	1365.8

**Interior Shear Wall**

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{N,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)	
						Size	Spc (in)	Size	Spc (in)	Size	Spc (in)					$P > .35P_o^{?2}$	$s_h > 18''^{?2}$	$s_v > 18''^{?2}$					
Level 08	106	13	190.2	134.1	7832.6	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	607.7	89.8	1088.9
Level 07	93	13	426.7	273.5	20930.7	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	1008.5	89.8	1088.9
Level 06	80	13	663.1	413.0	36299.1	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1203.2	89.8	1158.1
Level 05	67	13	899.6	552.4	52117.7	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1306.9	89.8	1158.1
Level 04	54	13	1136.0	691.9	67833.9	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1467.2	89.8	1158.1
Level 03	41	13	1372.5	831.3	84068.9	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1715.3	89.8	1158.1
Level 02	28	13	1608.9	970.8	102621.6	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1923.3	89.8	1158.1
Level 01	15	15	1849.0	1113.2	129122.0	#5	15.0	#5	10.0	15.24	#5	5.0	YES	18640.2	27766.0	0.671	NO	NO	NO	38815.0	2073.9	103.6	1158.1

1. Area of Longitudinal Steel in Boundary Elements

2. If "NO", Shear Wall may not be taken as effective in resisting seismic forces per ASCE 41-06 Section 6.7.1.1

3. Nominal Moment Capacity taken from PCA Column Analysis corresponding to minimum design Axial Load

4. Nominal Moment Capacity at P=0 taken from PCA Column Analysis, including only boundary reinforcing.  $\Phi = 1.0$ , and expected strengths are used.

5. Nominal Moment Capacity at P=0 is not required to exceed Moment Demand, as the compressive force is assumed to be 0 and therefore no interaction effects are taken into account.

6. Maximum shear force calculated based on nominal flexural capacity of base of wall.

7. Wall is considered controlled by flexure if the maximum shear force value is less than the nominal shear capacity. This indicates that the wall will fail in flexure prior to reaching the shear capacity.



Control <sup>7</sup>	J*P <sub>DES,MAX</sub> (kips)	J*P <sub>DES,MIN</sub> (kips)	M <sub>DES</sub> /m (k-ft)	Axial-Flexure		Axial-Flexural Acceptance		Shear Acceptance
				DCR <sub>MAX</sub>	DCR <sub>MIN</sub>			
Flexure	223.4	161.2	2123.1	0.083	0.085	PASS	PASS	PASS
Flexure	497.5	331.4	5673.4	0.188	0.199	PASS	PASS	PASS
Flexure	771.6	501.7	10226.0	0.311	0.339	PASS	PASS	PASS
Flexure	1045.7	671.9	15424.8	0.434	0.484	PASS	PASS	PASS
Flexure	1319.7	842.2	21780.3	0.457	0.505	PASS	PASS	PASS
Flexure	1593.8	1012.4	30383.4	0.604	0.679	PASS	PASS	PASS
Flexure	1867.9	1182.7	37123.7	0.577	0.641	PASS	PASS	PASS
Flexure	2149.2	1358.8	46666.0	0.548	0.597	PASS	PASS	PASS

Control <sup>7</sup>	J*P <sub>DES,MAX</sub> (kips)	J*P <sub>DES,MIN</sub> (kips)	M <sub>DES</sub> /m (k-ft)	DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Axial-Flexural Acceptance		Shear Acceptance
Flexure	380.5	268.1	1958.2	0.055	0.056	PASS	PASS	PASS
Flexure	853.4	547.0	5232.7	0.129	0.140	PASS	PASS	PASS
Flexure	1326.3	825.9	9074.8	0.194	0.216	PASS	PASS	PASS
Flexure	1799.2	1104.9	13554.8	0.264	0.303	PASS	PASS	PASS
Flexure	2272.1	1383.8	18944.9	0.340	0.400	PASS	PASS	PASS
Flexure	2745.0	1662.7	26508.0	0.442	0.530	PASS	PASS	PASS
Flexure	3217.9	1941.6	34327.2	0.534	0.652	PASS	PASS	PASS
Flexure	3698.0	2226.4	43040.7	0.588	0.716	PASS	PASS	PASS



## Section 13c. LDP (Time History) Shear Wall Assessment (CP)

## Typical Wall Properties

$f_c'$	5000	psi, Lower Bound Concrete Compressive Strength
$f_y$	60	ksi, Lower Bound Reinforcing Steel Yield Strength
$f_u$	90	ksi, Lower Bound Reinforcing Steel Ultimate Strength
$f_c'$	7500	psi, Expected Concrete Compressive Strength
$f_y$	75	ksi, Expected Reinforcing Steel Yield Strength
$f_u$	112.5	ksi, Expected Reinforcing Steel Ultimate Strength
$t_w$	16	in, Shear Wall Thickness
$h_w$	106	ft, Shear Wall Height
$l_w$	22.33	ft, Shear wall Length
		Slender Wall Slenderness (ASCE 41-06 Section C6.7.1)
$w$	28	in, Boundary Element Width
$A_g$	4960	kips, Nominal Axial Capacity (No Eccentricity, Concrete Only)
$k$	1.0	Knowledge Factor
$C_1$	1.0	
$C_2$	1.0	

## Exterior Shear Wall

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{N,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)	
						Size	Spc (in)	Size	Spc (in)	Size	Spc (in)					$P > .35P_o^{?2}$	$s_h > 18''^{?2}$	$s_v > 18''^{?2}$					
Level 08	106	13	111.7	80.6	11939.4	#5	18.0	#5	18.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	23270.4	925.6	151.3	1088.9
Level 07	93	13	248.7	165.7	31913.1	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1537.6	151.3	1158.1
Level 06	80	13	385.8	250.8	55345.2	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1834.6	151.3	1158.1
Level 05	67	13	522.8	336.0	79463.9	#5	15.0	#5	15.0	6.32	#5	4.0	YES	18217.5	27245.7	0.669	NO	NO	NO	25170.5	1992.6	151.3	1158.1
Level 04	54	13	659.9	421.1	103426.4	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	2237.0	151.3	1158.1
Level 03	41	13	796.9	506.2	128179.9	#5	15.0	#5	15.0	12.64	#5	4.0	YES	18517.0	27614.3	0.671	NO	NO	NO	40400.4	2612.1	151.3	1158.1
Level 02	28	13	934.0	591.3	156467.1	#5	15.0	#5	15.0	20.32	#5	4.0	YES	18880.9	28062.3	0.673	NO	NO	NO	52017.3	2929.0	151.3	1158.1
Level 01	15	15	1074.6	679.4	196636.1	#5	10.0	#5	10.0	29.32	#5	4.0	YES	19307.4	28587.3	0.675	NO	NO	NO	65401.3	3158.4	174.6	1365.8

## Interior Shear Wall

Level	Elev (ft)	$h_{seg}$ (ft)	$P_{DES,MAX}$ (kips)	$P_{DES,MIN}$ (kips)	$M_{DES,MAX}$ (k-ft)	Horiz Reinf		Vert Reinf		Boundary Element		Confined?	$P_{CL}$ (kips)	$P_{CE}$ (kips)	$\Phi_{AXIAL}$	Effectiveness			$M_{N,P=0}^{4,5}$ (k-ft)	$V_{DES}$ (kips)	$V_{MAX}^6$ (kips)	$V_{n,y}$ (kips)	
						Size	Spc (in)	Size	Spc (in)	Size	Spc (in)					$P > .35P_o^{?2}$	$s_h > 18''^{?2}$	$s_v > 18''^{?2}$					
Level 08	106	13	190.2	134.1	11011.9	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	853.7	89.8	1088.9
Level 07	93	13	426.7	273.5	29433.8	#5	18.0	#5	18.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	32018.5	1418.1	89.8	1088.9
Level 06	80	13	663.1	413.0	51045.6	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1692.1	89.8	1158.1
Level 05	67	13	899.6	552.4	73290.5	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	1837.8	89.8	1158.1
Level 04	54	13	1136.0	691.9	95391.5	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	2063.2	89.8	1158.1
Level 03	41	13	1372.5	831.3	118222.0	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	2409.2	89.8	1158.1
Level 02	28	13	1608.9	970.8	144311.6	#5	15.0	#5	15.0	9.48	#5	5.0	YES	18367.2	27430.0	0.670	NO	NO	NO	33870.6	2701.5	89.8	1158.1
Level 01	15	15	1849.0	1113.2	181359.9	#5	15.0	#5	10.0	15.24	#5	5.0	YES	18640.2	27766.0	0.671	NO	NO	NO	38815.0	2913.0	103.6	1158.1

1. Area of Longitudinal Steel in Boundary Elements

2. If "NO", Shear Wall may not be taken as effective in resisting seismic forces per ASCE 41-06 Section 6.7.1.1

3. Nominal Moment Capacity taken from PCA Column Analysis corresponding to minimum design Axial Load

4. Nominal Moment Capacity at P=0 taken from PCA Column Analysis, including only boundary reinforcing.  $\Phi = 1.0$ , and expected strengths are used.

5. Nominal Moment Capacity at P=0 is not required to exceed Moment Demand, as the compressive force is assumed to be 0 and therefore no interaction effects are taken into account.

6. Maximum shear force calculated based on nominal flexural capacity of base of wall.

7. Wall is considered controlled by flexure if the maximum shear force value is less than the nominal shear capacity. This indicates that the wall will fail in flexure prior to reaching the shear capacity.

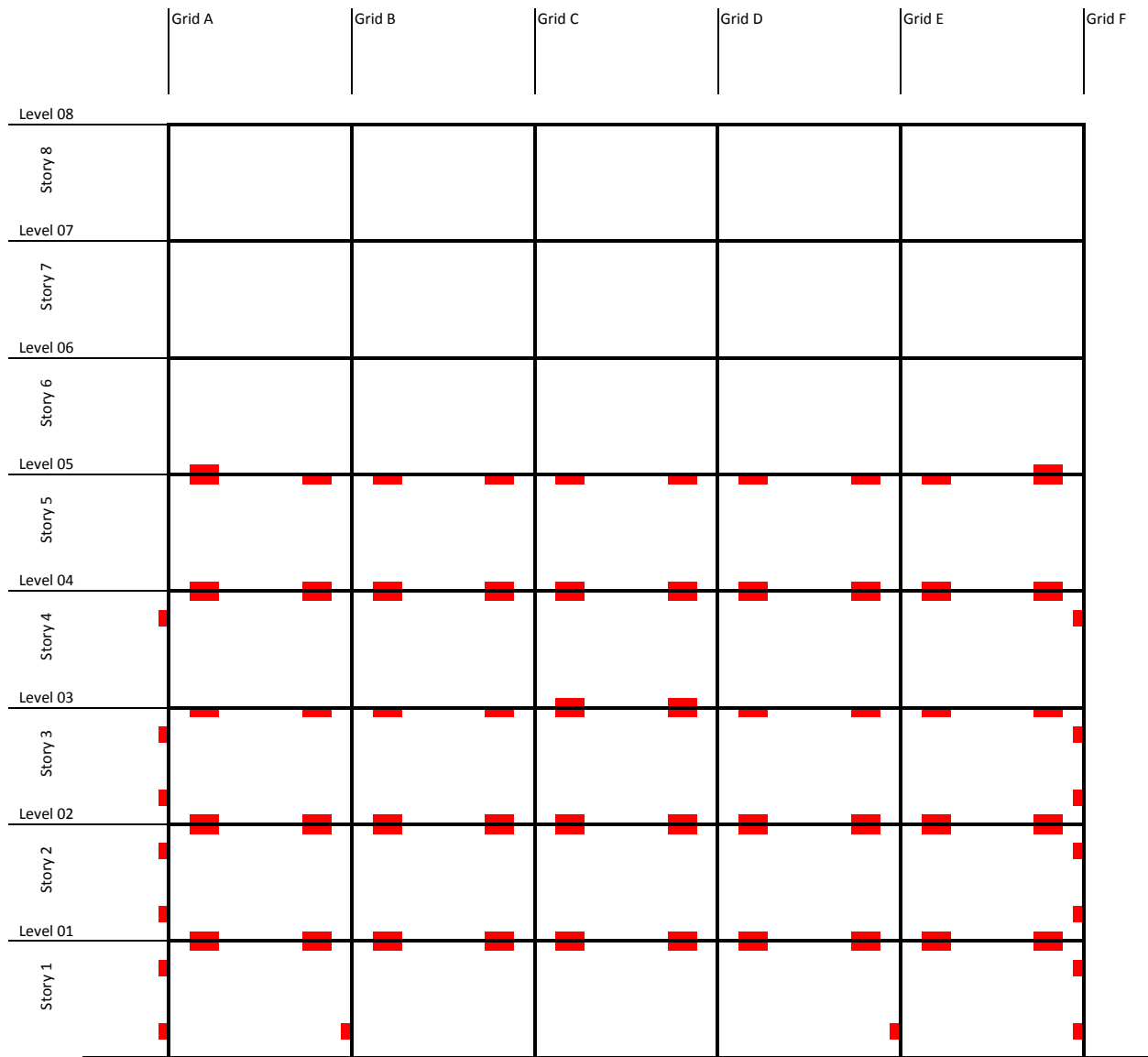


Control <sup>7</sup>	J*P <sub>DES,MAX</sub> (kips)	J*P <sub>DES,MIN</sub> (kips)	M <sub>DES</sub> /m (k-ft)	Axial-Flexure		Axial-Flexural Acceptance		Shear Acceptance
				DCR <sub>MAX</sub>	DCR <sub>MIN</sub>			
Flexure	223.4	161.2	1989.9	0.078	0.080	PASS	PASS	PASS
Flexure	497.5	331.4	6474.7	0.215	0.227	PASS	PASS	PASS
Flexure	771.6	501.7	13836.3	0.421	0.459	PASS	PASS	PASS
Flexure	1045.7	671.9	19866.0	0.559	0.623	PASS	PASS	PASS
Flexure	1319.7	842.2	25856.6	0.542	0.600	PASS	PASS	PASS
Flexure	1593.8	1012.4	32045.0	0.637	0.716	PASS	PASS	PASS
Flexure	1867.9	1182.7	39158.6	0.608	0.676	PASS	PASS	PASS
Flexure	2149.2	1358.8	47669.4	0.559	0.610	PASS	PASS	PASS

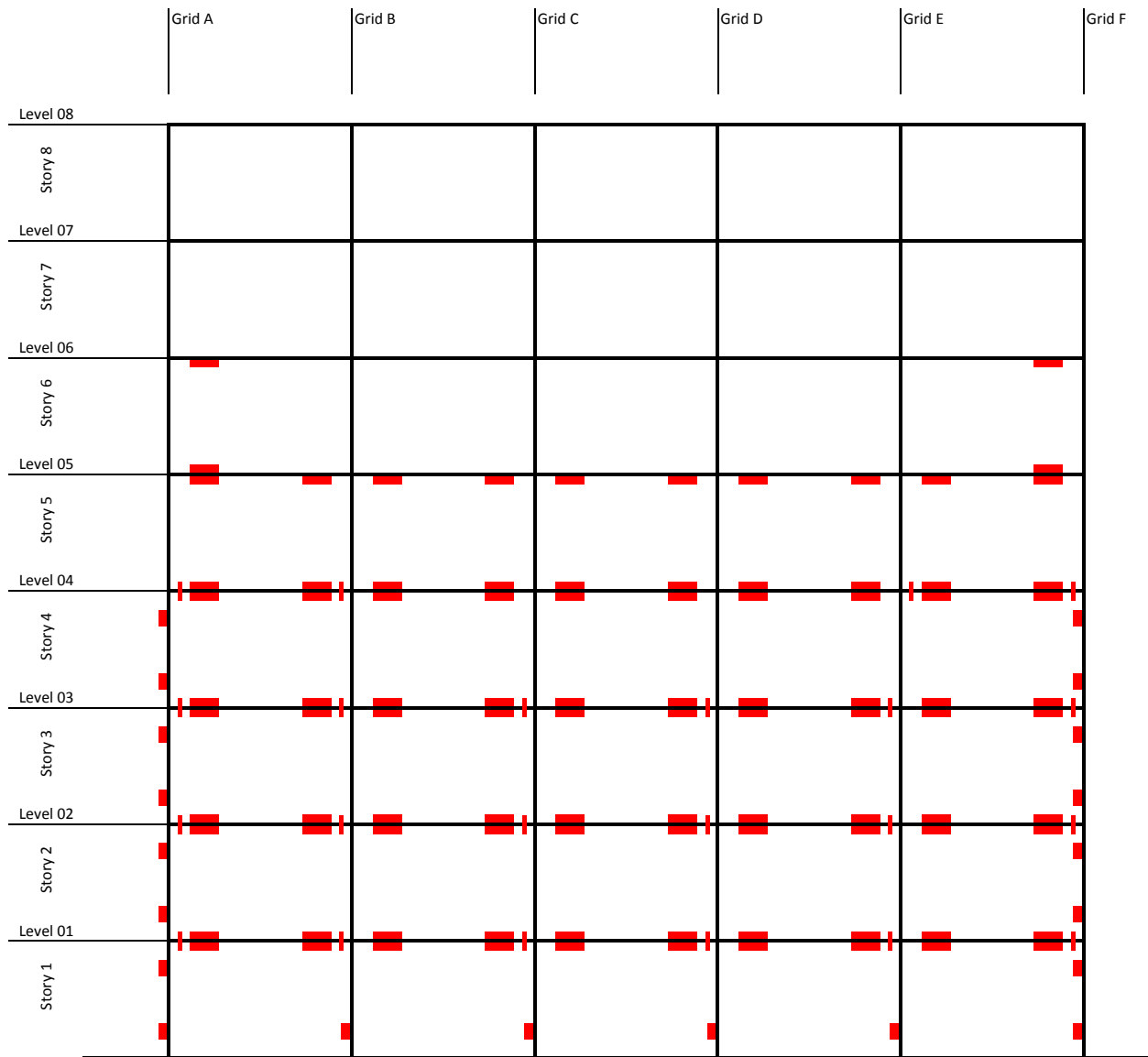
  

Control <sup>7</sup>	J*P <sub>DES,MAX</sub> (kips)	J*P <sub>DES,MIN</sub> (kips)	M <sub>DES</sub> /m (k-ft)	DCR <sub>MAX</sub>	DCR <sub>MIN</sub>	Axial-Flexural Acceptance		Shear Acceptance
Flexure	380.5	268.1	1835.3	0.051	0.053	PASS	PASS	PASS
Flexure	853.4	547.0	5529.7	0.137	0.148	PASS	PASS	PASS
Flexure	1326.3	825.9	11550.1	0.247	0.275	PASS	PASS	PASS
Flexure	1799.2	1104.9	18322.6	0.357	0.410	PASS	PASS	PASS
Flexure	2272.1	1383.8	23847.9	0.428	0.504	PASS	PASS	PASS
Flexure	2745.0	1662.7	29555.5	0.493	0.591	PASS	PASS	PASS
Flexure	3217.9	1941.6	36220.4	0.564	0.688	PASS	PASS	PASS
Flexure	3698.0	2226.4	43966.0	0.601	0.732	PASS	PASS	PASS

## **Appendix E – LSP and LDP Demand-Capacity Ratio Elevations**

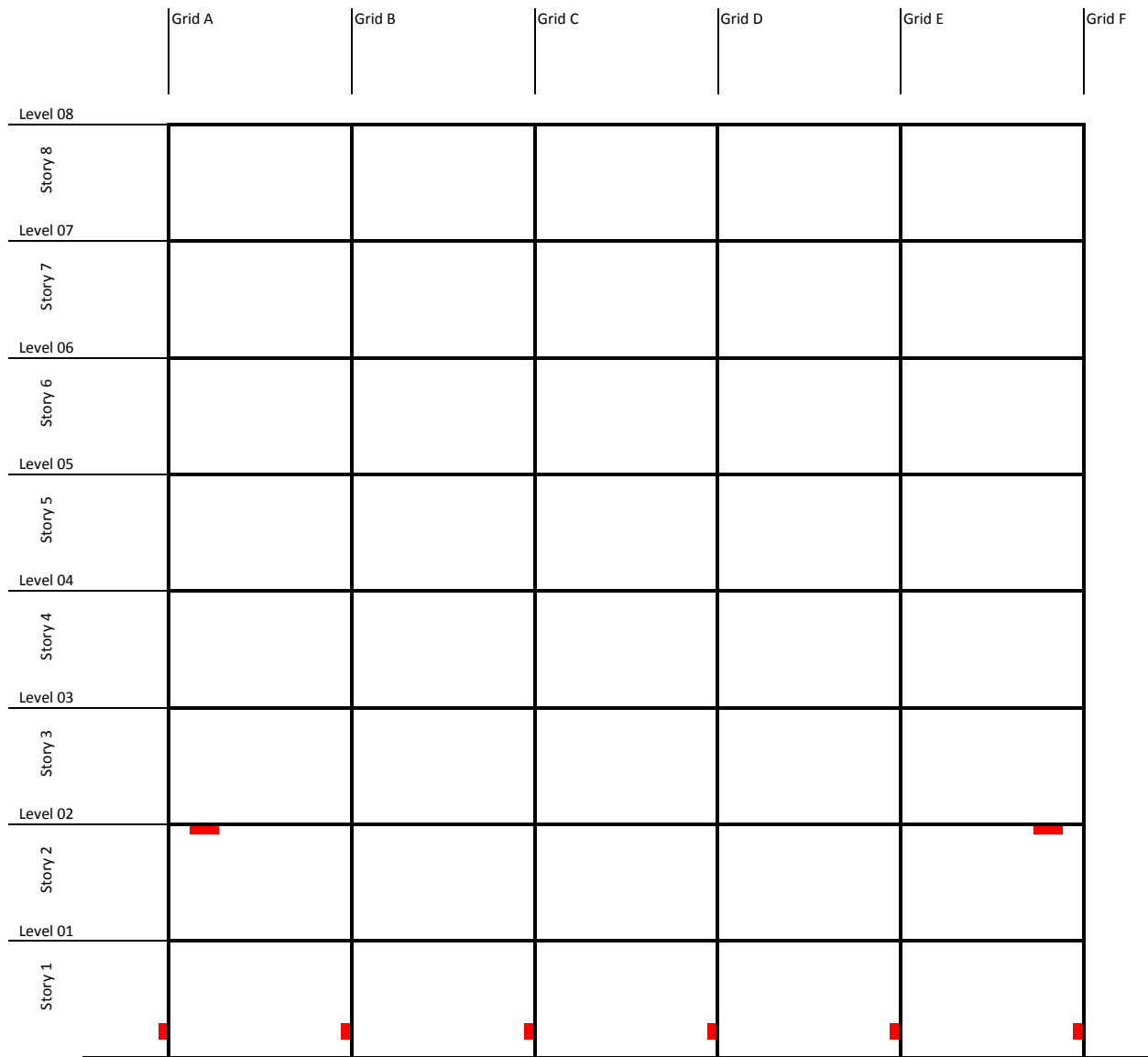


LSP Moment Frame Failure Elevation (LS)

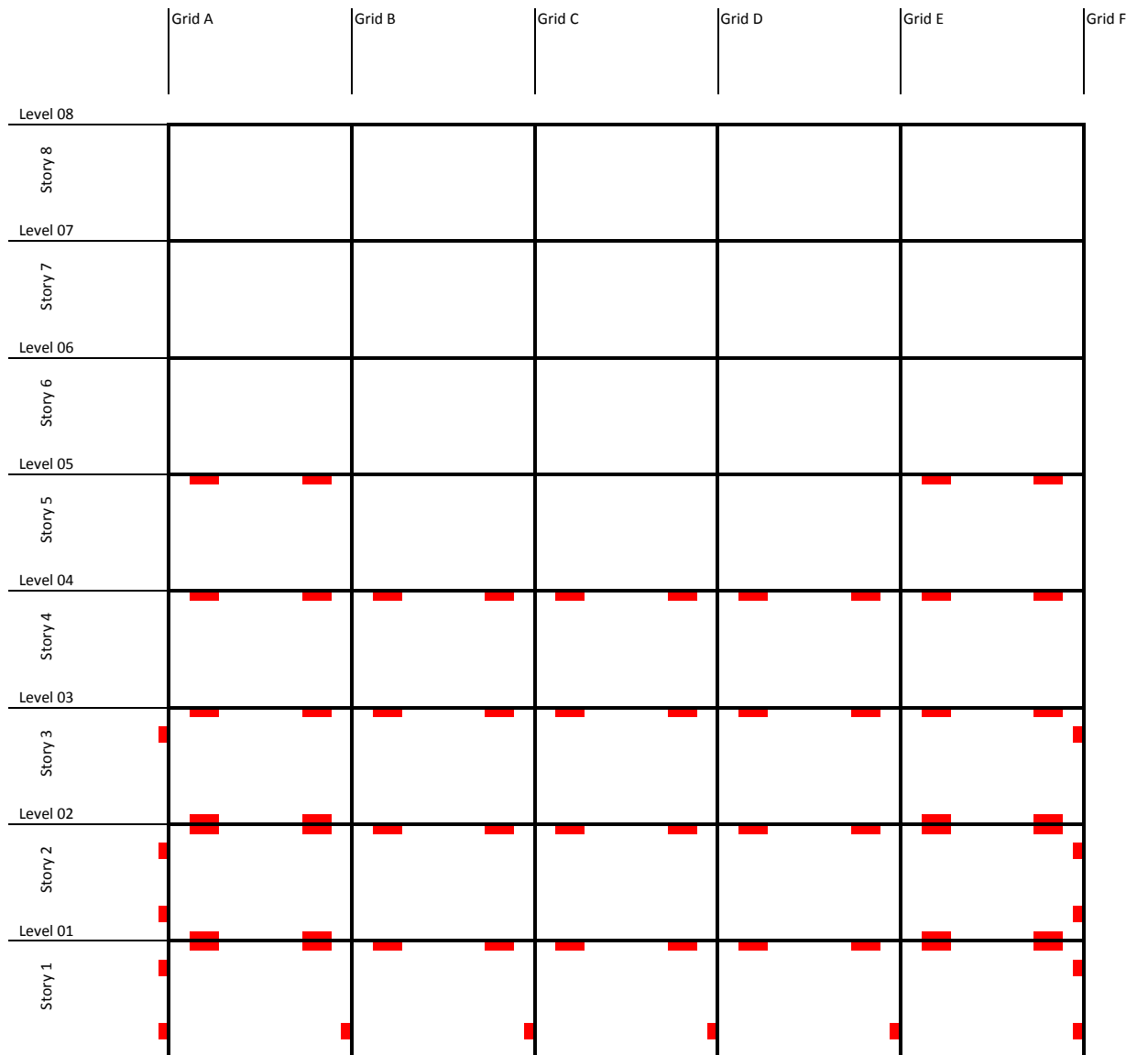


LSP Moment Frame Failure Elevation (CP)

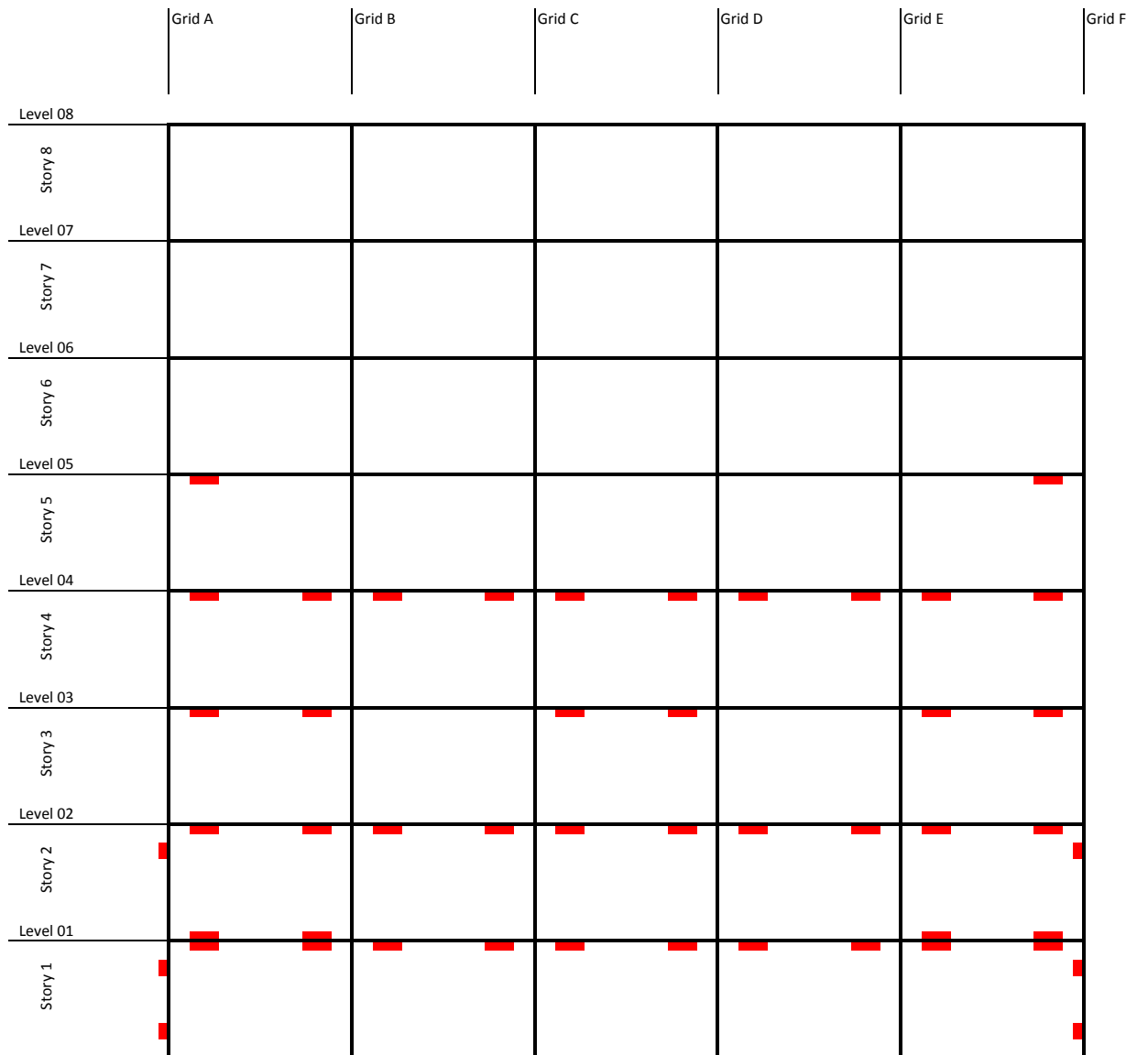




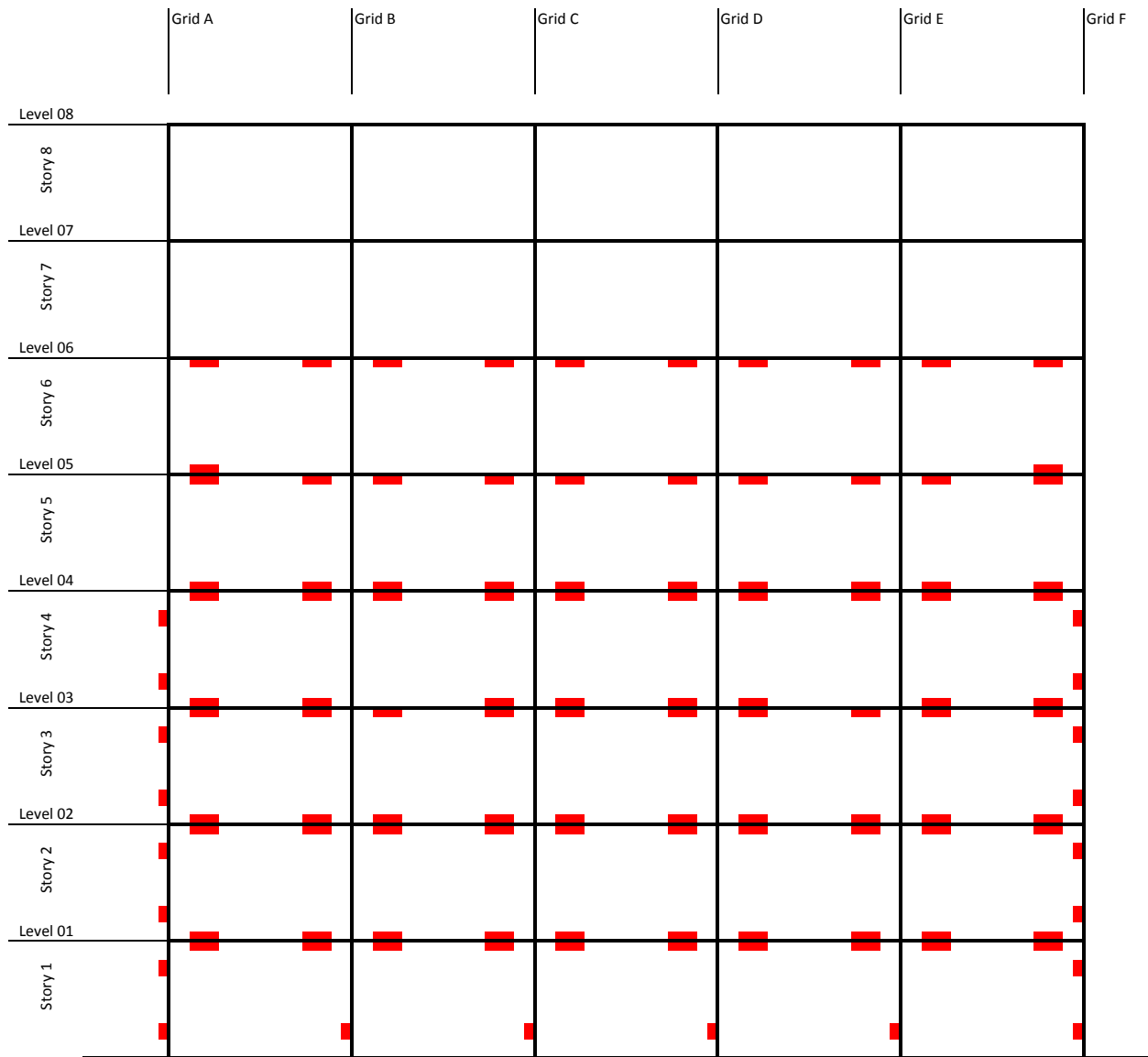
LDP (RESP) Moment Frame Failure Elevation (LS)



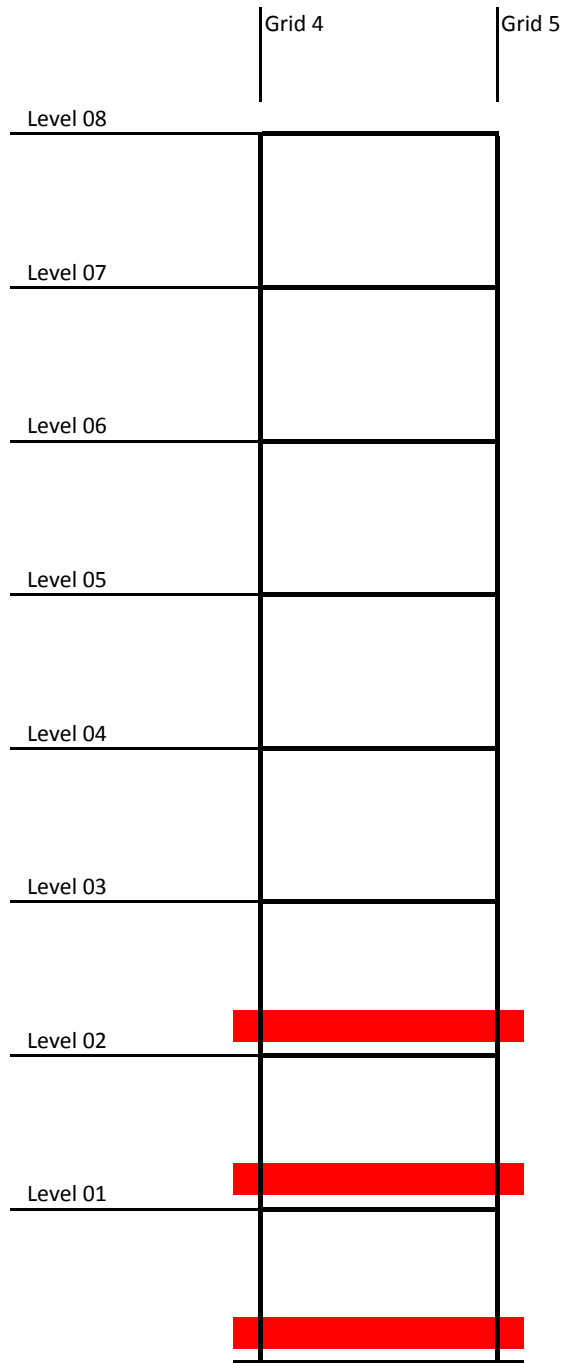
LDP (RESP) Moment Frame Failure Elevation (CP)



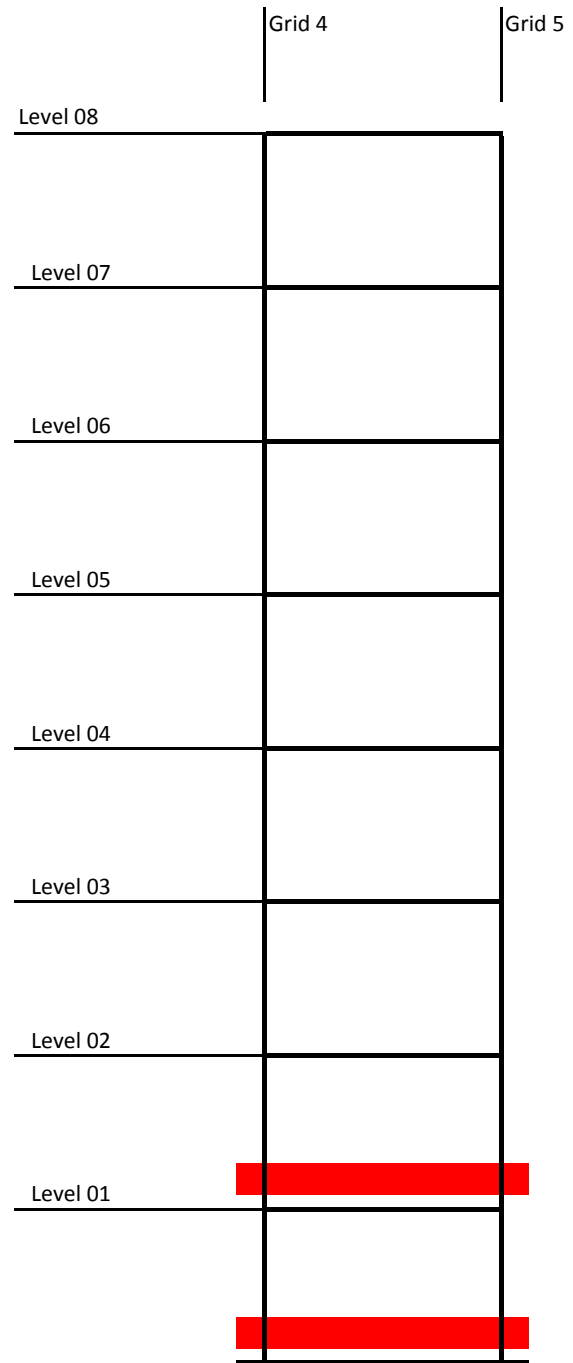
LDP (THA) Moment Frame Failure Elevation (LS)



LDP (THA) Moment Frame Failure Elevation (CP)

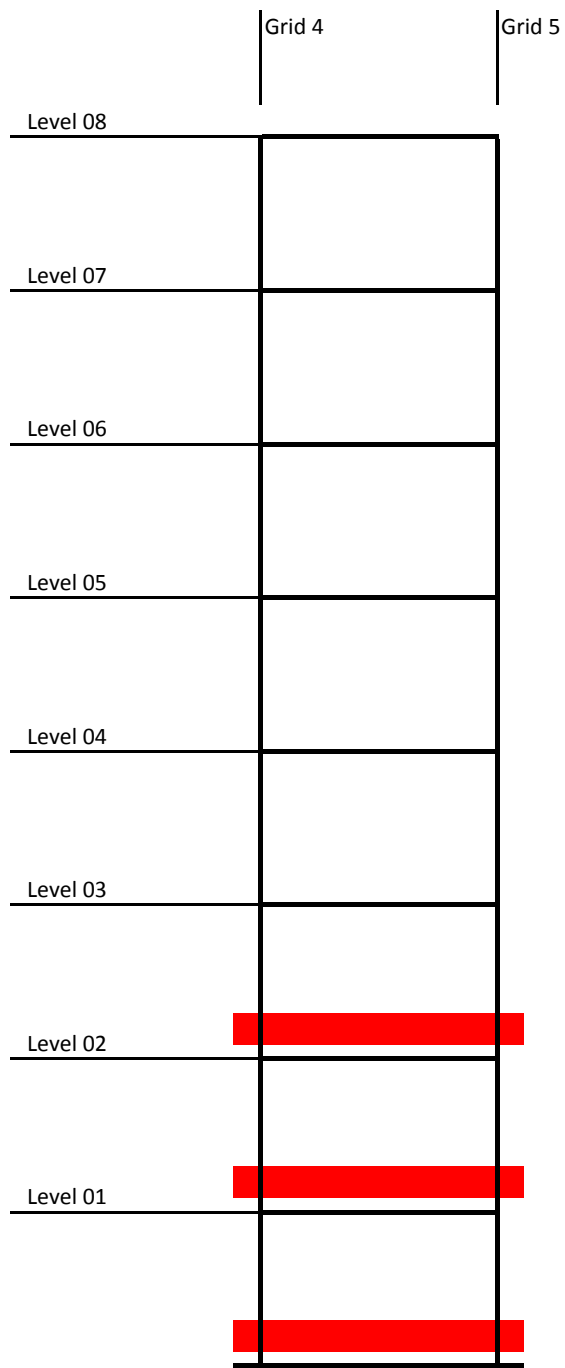


Exterior SW

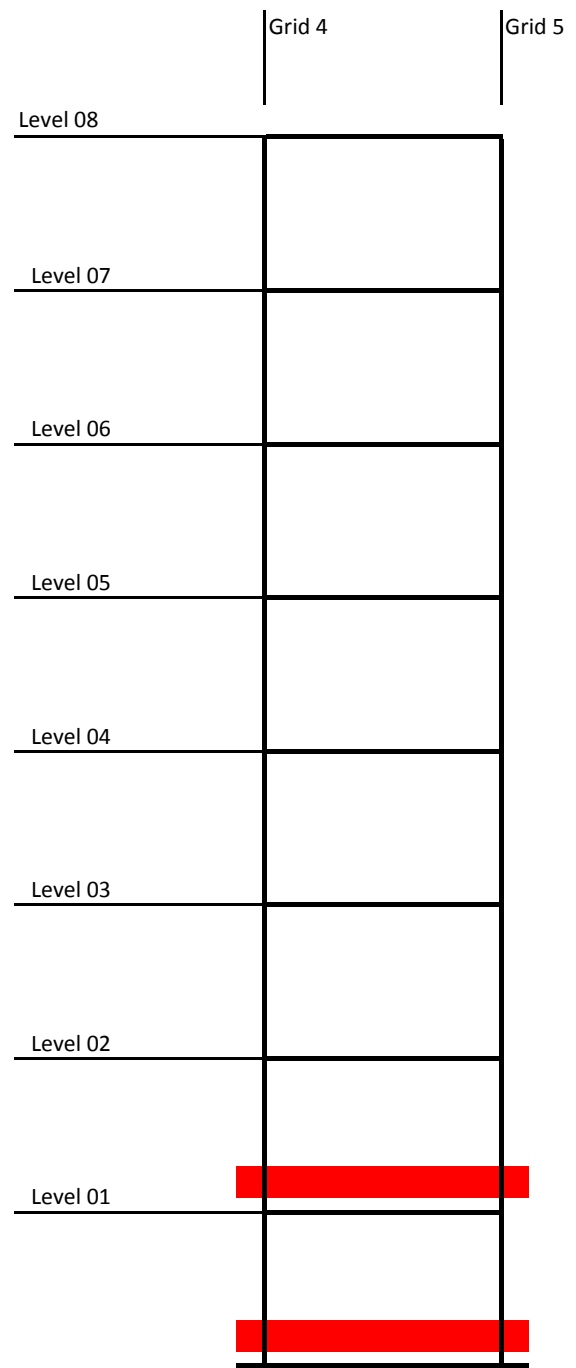


Interior SW

**LSP Shear Wall Failure Elevations (LS)**



Exterior SW



Interior SW

**LSP Shear Wall Failure Elevations (CP)**

	Grid 4	Grid 5
Level 08		
Level 07		
Level 06		
Level 05		
Level 04		
Level 03		
Level 02		
Level 01		

Exterior SW

	Grid 4	Grid 5
Level 08		
Level 07		
Level 06		
Level 05		
Level 04		
Level 03		
Level 02		
Level 01		



Interior SW

**LDP (RESP) Shear Wall Failure Elevations (LS)**

	Grid 4	Grid 5
Level 08		
Level 07		
Level 06		
Level 05		
Level 04		
Level 03		
Level 02		
Level 01		

Exterior SW

	Grid 4	Grid 5
Level 08		
Level 07		
Level 06		
Level 05		
Level 04		
Level 03		
Level 02		
Level 01		

Interior SW

**LDP (RESP) Shear Wall Failure Elevations (CP)**



	Grid 4	Grid 5
Level 08		
Level 07		
Level 06		
Level 05		
Level 04		
Level 03		
Level 02		
Level 01		

Exterior SW

	Grid 4	Grid 5
Level 08		
Level 07		
Level 06		
Level 05		
Level 04		
Level 03		
Level 02		
Level 01		

Interior SW

**LDP (THA) Shear Wall Failure Elevations (LS)**

	Grid 4	Grid 5
Level 08		
Level 07		
Level 06		
Level 05		
Level 04		
Level 03		
Level 02		
Level 01		

Exterior SW

	Grid 4	Grid 5
Level 08		
Level 07		
Level 06		
Level 05		
Level 04		
Level 03		
Level 02		
Level 01		

Interior SW

**LDP (THA) Shear Wall Failure Elevations (CP)**

## **Appendix F – Time History Record Scaling Calculations**



### Time History Record Scaling Factors and Calculations

Key	System	Perform	?/IO	SF	Total SF
MF-IO	MF	IO	1	0.954208	0.95420826
MF-LS	MF	LS	1.74	0.954208	1.6640144
MF-CP	MF	CP	2.45	0.954208	2.34002025
SW-IO	SW	IO	1	0.698597	0.69859676
SW-LS	SW	LS	1.74	0.698597	1.21826139
SW-CP	SW	CP	2.45	0.698597	1.71318008

ID	Record	dt	N	Scale of Original	tstop	TH Scale Factors for Lateral System and Limit State					
						MF-IO	MF-LS	MF-CP	SW-IO	SW-LS	SW-CP
TH01	120111	0.01	2999	0.65082281	29.98	20.0	34.9	49.0	14.6	25.5	35.9
TH02	120112	0.01	2999	0.65082281	29.98	20.0	34.9	49.0	14.6	25.5	35.9
TH03	120121	0.01	1999	0.83152823	19.98	25.5	44.6	62.7	18.7	32.6	45.9
TH04	120122	0.01	1999	0.83152823	19.98	25.5	44.6	62.7	18.7	32.6	45.9
TH05	120411	0.01	5590	0.62910197	55.89	19.3	33.7	47.4	14.2	24.7	34.7
TH06	120412	0.01	5590	0.62910197	55.89	19.3	33.7	47.4	14.2	24.7	34.7
TH07	120521	0.01	4531	1.09153986	45.3	33.5	58.5	82.2	24.6	42.8	60.2
TH08	120522	0.01	4531	1.09153986	45.3	33.5	58.5	82.2	24.6	42.8	60.2
TH09	120611	0.01	9992	1.31146275	99.91	40.3	70.3	98.8	29.5	51.4	72.3
TH10	120612	0.01	9992	1.31146275	99.91	40.3	70.3	98.8	29.5	51.4	72.3
TH11	120621	0.005	7807	1.01385396	39.03	31.2	54.3	76.4	22.8	39.8	55.9
TH12	120622	0.005	7807	1.01385396	39.03	31.2	54.3	76.4	22.8	39.8	55.9
TH13	120711	0.01	4096	1.03408514	40.95	31.8	55.4	77.9	23.3	40.6	57.0
TH14	120712	0.01	4096	1.03408514	40.95	31.8	55.4	77.9	23.3	40.6	57.0
TH15	120721	0.01	4096	1.099025	40.95	33.8	58.9	82.8	24.7	43.1	60.6
TH16	120722	0.01	4096	1.099025	40.95	33.8	58.9	82.8	24.7	43.1	60.6
TH17	120811	0.005	5437	0.68831589	27.18	21.1	36.9	51.9	15.5	27.0	38.0
TH18	120812	0.005	5437	0.68831589	27.18	21.1	36.9	51.9	15.5	27.0	38.0
TH19	120821	0.005	6000	1.36020085	29.995	41.8	72.9	102.5	30.6	53.4	75.0
TH20	120822	0.005	6000	1.36020085	29.995	41.8	72.9	102.5	30.6	53.4	75.0
TH21	120911	0.02	2200	0.98651955	43.98	30.3	52.9	74.3	22.2	38.7	54.4
TH22	120912	0.02	2200	0.98651955	43.98	30.3	52.9	74.3	22.2	38.7	54.4
TH23	120921	0.0025	11186	1.14898349	27.9625	35.3	61.6	86.6	25.8	45.1	63.4
TH24	120922	0.0025	11186	1.14898349	27.9625	35.3	61.6	86.6	25.8	45.1	63.4
TH25	121011	0.005	7991	1.08851067	39.95	33.4	58.3	82.0	24.5	42.7	60.0
TH26	121012	0.005	7991	1.08851067	39.95	33.4	58.3	82.0	24.5	42.7	60.0
TH27	121021	0.005	7989	0.88027055	39.94	27.0	47.2	66.3	19.8	34.5	48.6
TH28	121022	0.005	7989	0.88027055	39.94	27.0	47.2	66.3	19.8	34.5	48.6
TH29	121111	0.02	2676	0.78716503	53.5	24.2	42.2	59.3	17.7	30.9	43.4
TH30	121112	0.02	2300	0.78716503	45.98	24.2	42.2	59.3	17.7	30.9	43.4
TH31	121211	0.005	8000	0.86966825	39.995	26.7	46.6	65.5	19.6	34.1	48.0
TH32	121212	0.005	8000	0.86966825	39.995	26.7	46.6	65.5	19.6	34.1	48.0
TH33	121221	0.01	2230	1.17435536	22.29	36.1	62.9	88.5	26.4	46.1	64.8
TH34	121222	0.01	2230	1.17435536	22.29	36.1	62.9	88.5	26.4	46.1	64.8
TH35	121321	0.02	1800	0.81997941	35.98	25.2	43.9	61.8	18.4	32.2	45.2
TH36	121322	0.02	1800	0.81997941	35.98	25.2	43.9	61.8	18.4	32.2	45.2
TH37	121411	0.005	18000	0.41044173	89.995	12.6	22.0	30.9	9.2	16.1	22.6
TH38	121412	0.005	18000	0.41044173	89.995	12.6	22.0	30.9	9.2	16.1	22.6
TH39	121421	0.005	18000	0.95946044	89.995	29.5	51.4	72.3	21.6	37.6	52.9
TH40	121422	0.005	18000	0.95946044	89.995	29.5	51.4	72.3	21.6	37.6	52.9
TH41	121511	0.01	2800	2.09615629	27.99	64.4	112.3	157.9	47.2	82.2	115.6
TH42	121512	0.01	2800	2.09615629	27.99	64.4	112.3	157.9	47.2	82.2	115.6
TH43	121711	0.005	7269	1.44009628	36.34	44.2	77.2	108.5	32.4	56.5	79.4
TH44	121712	0.005	7269	1.44009628	36.34	44.2	77.2	108.5	32.4	56.5	79.4