SEISMIC PERFORMANCE COMPARISON OF A FIXED-BASE
VERSUS A BASE-ISOLATED OFFICE BUILDING

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Master of Science in Architecture with a Specialization in Architectural Engineering

by

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ABSTRACT

Seismic Performance Comparison of a Fixed-Base versus a Base-Isolated Office Building

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The topic of this thesis is base isolation. The purpose of this thesis is to offer a relative understanding of the seismic performance enhancements that a typical 12-story steel office building can achieve through the implementation of base isolation technology. To reach this understanding, the structures of a fixed-base office building and a base-isolated office building of similar size and layout are designed, their seismic performance is compared, and a cost-benefit analysis is completed. The base isolation system that is utilized is composed of Triple Friction Pendulum (TFP) bearings.

The work of this thesis is divided into four phases. First, in the building selection phase, the structural systems (SMF and SCBF), layout, location (San Diego, CA), and design parameters of the buildings are selected. Then, in the design phase, each structure is designed using modal response spectrum analysis in ETABS. In the analysis phase, nonlinear time history analyses at DBE and MCE levels are conducted in PERFORM-3D to obtain the related floor accelerations and interstory drifts. Finally, in the performance assessment phase, probable damage costs are computed using fragility curves and FEMA P-58 methodology in PACT. Damage costs are computed for each building and seismic demand level and the results are compared.

Keywords: base isolation, Triple Friction Pendulum (TFP), seismic performance, non-linear time history analysis, ETABS, PERFORM-3D, PACT, fragility curve, FEMA P-58

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¹ KPFF can be contacted through the firm's website, www.kpff.com

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LIST OF NOMENCLATURE

A	= area (in^2)
A_f	= area of flange (in^2)
A_T	= tributary area (ft^2)
B_D	= numerical coefficient for effective damping equal to β_D
B_M	= numerical coefficient for effective damping equal to β_M
B_1, B_2	= moment magnification factors
C	= capacity, used in demand over capacity (D/C) ratio
C_d	= deflection amplification factor
C_{RS}	= mapped value of the risk coefficient at short periods
C_{RX}	= location of center of rigidity in x-direction (in)
C_{RY}	= location of center of rigidity in y-direction (in)
C_{R1}	= mapped value of the risk coefficient at a period of 1 second
C_s	= seismic response coefficient
C_t	= approximate fundamental period coefficient
C_u	= coefficient for upper limit on calculated period
C_v	= web shear coefficient
C_{vx}	= vertical distribution factor for distribution of seismic forces
D	= damage value of component (story drift ratio or acceleration in g)
D	= dead load
D	= demand, used in demand over capacity (D/C) ratio
D_D	= design displacement of the isolation system (in)
DL	= member deformation at which strength loss is initiated (in or radians)

D_M	= maximum displacement of the isolation system (in)
DR	= member deformation at which strength loss is terminated (in or radians)
DS	= boundary element displacement capacity of isolation bearing (in)
DS	= damage state value of component (story drift ratio or acceleration in g)
D_{TD}	= total design displacement of the isolation system, including torsion (in)
D_{TM}	= total maximum displacement of the isolation system, including torsion (in)
DU	= ultimate member deformation (in or radians)
DX	= failure member deformation (in or radians)
DX	= deformation capacity of isolation bearing (in)
D_y	= yield displacement of the isolation system (at which sliding on the outer plates is initiated) (in)
E	= modulus of elasticity (ksi)
E_{loop}	= energy dissipated in an isolator unit during a hysteretic cycle (kip-in)
F_a	= short-period site coefficient
F_D	= lateral force of the bearing at which D_D of the isolation system is reached (kips)
F_f	= friction force needed to cause sliding in a friction pendulum bearing (kips)
F_i	= portion of the seismic base shear induced at level i (kips)
F_M	= lateral force of the bearing at which D_M of the isolation system is reached (kips)
F_{PGA}	= site coefficient for PGA
FR	= residual member strength (kips or kip-in)
FU	= ultimate member strength (kips or kip-in)
F_v	= long-period site coefficient
F_x	= lateral seismic force (kips)

F_Y	= yield member strength (kips or kip-in)
F_y	= yield strength of steel (ksi)
F_{ye}	= expected yield strength of steel (ksi)
F^+	= maximum positive force in an isolation bearing during a single cycle of testing at a displacement of Δ^+ (kips)
F^-	= minimum negative force in an isolation bearing during a single cycle of testing at a displacement of Δ^- (kips)
H	= height of isolation bearing
I	= moment of inertia (in^4)
I_e	= seismic importance factor
K	= stiffness (kips/in)
K_C	= compression stiffness of isolation bearing (kips/in)
K_{eff}	= effective stiffness of isolation bearing (kips/in)
K_{LL}	= live load element factor
K_O	= elastic stiffness of isolation bearing (kips/in)
K_S	= boundary element stiffness of isolation bearing (kips/in)
K_T	= tensile stiffness of isolation bearing (kips/in)
K_1, K_2	= effective length factor
L	= live load
L	= length of isolation bearing
L_o	= unreduced live load
L_r	= roof live load
M	= mass ($\text{kip-sec}^2/\text{in}$)

M_{CE} = expected flexural strength of a member or joint (kip-in)
 MMI = mass moment of inertia (kip-in-sec²)
 M_{ta} = accidental torsional moment (kip-feet)
 M_x = strong-axis bending strength (kip-in)
 M_y = weak-axis bending strength (kip-in)
 P = axial force in member (kips)
 P_c = axial compressive strength (kips)
 PGA = mapped MCE peak ground acceleration (g)
 PGA_M = MCE peak ground acceleration adjusted for Site Class effects (g)
 P_n = nominal axial strength (kips)
 P_r = required second-order axial strength (kips)
 P_t = axial tensile strength (kips)
 PU = deformation capacity (in or radians)
 P_x = total vertical design load at and above level x (kips)
 P_y = axial yield strength (kips)
 P_{ye} = expected yield axial strength of a member (kips)
 $P()$ = probability ratio function for fragility curve damage states
 Q = generalized force in a component
 Q_{CE} = component expected strength
 Q_E = seismic load
 Q_y = yield strength of a component
 R = response modification coefficient
 R = radius of curvature of the concave plates of a TFP bearing (in)

R_1, R_2 = live load reduction factors

R_1, R_2 = radii of curvature of the outer concave plates of a TFP bearing (in)

R_3, R_4 = radii of curvature of the inner concave plates of a TFP bearing (in)

R_{eff} = effective radius of curvature of the concave plates of a TFP bearing (in)

R_I = isolated response modification coefficient

R_t = expected/minimum tensile strength ratio

R_y = expected/minimum yield stress ratio

S = snow load

S_a = spectral response acceleration (g)

S_{DS} = the DBE, 5% damped spectral response acceleration at short periods (g)

S_{D1} = the DBE, 5% damped spectral response acceleration at a period of 1 second (g)

S_{MS} = the MCE, 5% damped spectral response acceleration at short periods (g)

S_{M1} = the MCE, 5% damped spectral response acceleration at a period of 1 second (g)

S_s = mapped, 5% damped spectral response acceleration at short periods (g)

S_1 = mapped, 5% damped spectral response acceleration at a period of 1 second (g)

T = period of structure (sec)

$T_0 = 0.2S_{D1}/S_{DS}$

T_a = approximate fundamental period (sec)

T_D = effective period of the isolated structure at the design displacement (sec)

T_{eff} = effective period of bearing (sec)

T_L = long-period transition period (sec)

T_M = effective period of the isolated structure at the maximum displacement (sec)

T_o = period of the original fixed-base structure (sec)

T_S	= S_{D1}/S_{DS}
T_1	= first mode period
V	= base shear (kips)
V_b	= seismic design shear force below the isolation system (kips)
V_n	= nominal shear strength (kips)
V_{nx}	= strong-axis shear force (kips)
V_{ny}	= weak-axis shear force (kips)
V_s	= seismic design shear force above the isolation system (kips)
V_t	= base shear from the required modal combination (kips)
V_x	= seismic design story shear (kips)
W	= weight (kips)
X_c	= centroid distance in x-direction (in)
Y_c	= centroid distance in y-direction (in)
Z	= plastic section modulus (in ³)
a	= parameter used to measure deformation capacity in component load-deformation curves
b	= parameter used to measure deformation capacity in component load-deformation curves
b_f	= flange width (in)
c	= parameter used to measure residual strength
d_i	= floor elastic displacement associated with f_i (in)
d_1, d_2	= sliding displacement capacities on the outer slide plates of a TFP bearing (in)
d_3, d_4	= sliding displacement capacities on the inner slide plates of a TFP bearing (in)

f_i	= floor lateral force (kips)
g	= gravitational acceleration (in/sec ²)
h	= distance from inside of compression flange to inside of tension flange (in)
h_i, h_x	= height from the base to level i or x (feet)
h_n	= structural height (in)
h_{sx}	= story height below level x (in)
h_1, h_2	= height of the outer slide plates of a TFP bearing (in)
h_3, h_4	= height of the inner slide plates of a TFP bearing (in)
k	= vertical distribution of seismic forces exponent related to structure period
l	= length of structural member (in)
r	= governing radius of gyration (in)
t_f	= flange thickness (in)
t_w	= web thickness (in)
w_i, w_x	= floor weight (kips)
x	= approximate fundamental period coefficient
α	= design philosophy factor (1.0 for LRFD)
α	= P yield surface parameter for P-M interaction
α	= Rayleigh damping parameter for mass
β	= M yield surface parameter for P-M interaction
β	= Rayleigh damping parameter for stiffness
β_D	= effective damping of the isolation system at the design displacement
β_c	= definition and construction quality assurance factor
β_{eff}	= effective damping of the isolation system

β_M	= effective damping of the isolation system at the maximum displacement
β_m	= modeling dispersion factor
β_q	= model quality and completeness factor
γ	= M yield surface parameter for M-M interaction
Δ	= story drift
Δ	= generalized deformation
Δ^+	= maximum positive displacement of an bearing during each cycle of testing (in)
Δ^-	= minimum negative displacement of an bearing during each cycle of testing (in)
θ	= generalized deformation (radians)
θ	= stability coefficient for P-Delta effects
θ_y	= generalized yield deformation (radians)
μ	= friction coefficient of a sliding surface of a TFP bearing
μ_1, μ_2	= friction coefficients of the outer sliding surfaces of a TFP bearing
μ_3, μ_4	= friction coefficients of the inner sliding surfaces of a TFP bearing
ρ	= axial compression ratio (kips ⁻¹)
ρ	= structural redundancy factor
ΣE_D	= total energy dissipated in the isolation system during a cycle at D_D (kip-in)
ΣE_M	= total energy dissipated in the isolation system during a cycle at D_M (kip-in)
ΣF_D	= sum, for all bearings, of the lateral force at which D_D is reached (kips)
ΣF_M	= sum, for all bearings, of the lateral force at which D_M is reached (kips)
ΣK_D	= effective stiffness of the isolation system at D_D (kips/in)
ΣK_M	= effective stiffness of the isolation system at D_M (kips/in)
τ_b	= stiffness reduction factor

ϕ = LRFD resistance factor

Ω_o = overstrength factor

$\$C$ = repair cost

1.0 INTRODUCTION

The topic of this thesis is base isolation. Base isolation is a proven technology for the seismic design of structures. The system reduces the likelihood of structural and non-structural damage to a building subjected to seismic forces. As a result of the use of base isolation, lives and property have been saved.

Base isolation technology is used primarily in critical facilities such as hospitals, museums, and emergency response centers, where the benefits of protecting the structure and its property from seismic damage far exceed the cost of implementing the system. However, despite base isolation's proven benefits, the technology is underutilized. Although tall, flexible, and non-critical facilities such as office buildings are not the most ideal candidates for base isolation, they may still achieve an optimal seismic design by using the technology. Therefore, in order to increase the quality and prevalence of base-isolated structures, there is a need to study the technology's seismic performance enhancements and cost effectiveness for projects on which the system is infrequently used.

1.1 Purpose

The purpose of this thesis is to offer a relative understanding of the seismic performance enhancements that a typical 12-story steel office building can achieve through the implementation of base isolation technology. To reach this understanding, the structures of a fixed-base office building and a base-isolated office building of similar size and layout are designed, their seismic performance is compared, and a cost-benefit

analysis is completed. To a greater extent, this study demonstrates the feasibility and cost effectiveness of implementing base isolation on tall, flexible, and non-critical structures.

As a result of this thesis, building owners and construction industry professionals can recognize the benefits of implementing base isolation on a wider range of projects, thereby creating the potential for a significant increase in the technology's use.

1.2 Scope

This study analyzes a fixed-base and a base-isolated 12-story steel office building for an assumed San Diego location. The superstructure of each building has a lateral system composed of special moment frames (SMF) in the longitudinal direction and special concentrically braced frames (SCBF) in the transverse direction. Also, Triple Friction Pendulum (TFP) bearings are used for the isolation system in the base-isolated structure. The seismic performance of each structure is analyzed and compared for both design basis earthquake (DBE) and maximum considered earthquake (MCE) seismic demand levels. 7 ground motions are scaled to each seismic demand level to perform nonlinear time history analyses on both structures. The resulting floor accelerations and interstory drifts are used to determine the levels of structural and non-structural damage inflicted on each building. A cost-benefit analysis is then performed, which takes into account the cost of the isolation system and the damage costs of the structures and their components.

1.3 Overview of Procedure

The procedure of this study is divided into 4 phases, which are briefly described below:

1. Building Selection Phase
 - Building layout selected
 - Structural and isolation systems chosen
 - Project site selected
2. Design Phase
 - Lateral systems of fixed-base and base-isolated structures designed
 - Isolation system designed
 - Code used: ASCE 7-10
 - Method: modal response spectrum analysis run in ETABS
3. Analysis Phase
 - Maximum floor accelerations and interstory drifts analyzed
 - Structural performance analyzed via plastic hinge deformations (at Immediate Occupancy, Life Safety, and Collapse Prevention levels)
 - Method: nonlinear time history analysis run in PERFORM-3D
 - Ground motions: 7 records scaled to DBE and MCE levels
4. Performance Assessment Phase
 - Damage costs computed in PACT
 - Method: fragility curves and FEMA P-58 procedures and data used
 - Cost-benefit analysis of implementing base isolation completed

2.0 BACKGROUND

This section introduces background knowledge that is relevant to this study.

2.1 Fundamentals of Base Isolation

Base isolation technology works by separating or greatly reducing the lateral movement of a building's superstructure from the movement of the ground/foundation during a seismic event. To allow for this difference in lateral movement while still supporting the weight of the superstructure, base isolation bearings are designed to be very flexible laterally while being stiff vertically. This base condition is in contrast to a

Figure A: Fixed-Base vs. Base-Isolated Building

Source: Nelson 2010

typical fixed-base structure, in which the connections between the superstructure and its base/foundation are rigid and translation of the superstructure is resisted in all directions. The difference between these two base conditions is illustrated in Figure A above. The ultimate purpose of a base isolation system is to reduce the seismic forces exerted onto a building's superstructure. This reduction in seismic forces is achieved in part by

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reducing the superstructure's spectral accelerations. These accelerations are reduced both by increasing the effective fundamental period of the isolated structure and through damping caused by energy dissipated within the isolation bearings. The effect that both of these approaches have on spectral acceleration is shown in Figure B below.

Figure B: Effect of Seismic Isolation on Spectral Acceleration

Source: Symans 2003

The effective period of the superstructure is increased due to the reduced lateral stiffness of the isolation system. Base isolation is most effective in stiff structures with small periods. The period and stiffness of a structure are inversely correlated, as shown by Equation 1 below:

$$T = 2\pi \sqrt{\frac{M}{K}} \quad \downarrow K \Rightarrow \uparrow T \quad \text{Eq. 1}$$

Where: T = Period [sec.]
 M = Mass [kips-sec.²/in.]
 K = Stiffness [kips/in.]

Damping occurs via energy dissipated by the partially restricted movement (and resulting nonlinear behavior) of the seismic isolation bearings, usually through inelastic deformation or friction. In the case of sliding isolation bearings, which are the type used in this study, friction is the force that causes energy dissipation. The friction force of sliding isolation bearings will be discussed in further detail in the next section. The amount of energy dissipated by an isolation bearing is determined by the area of the loops resulting from tracing the nonlinear force-displacement behavior of the bearing. This behavior for a typical cycle of loading of an isolation bearing is illustrated in Figure C below.

Figure C: Nonlinear Force-Displacement Relationship of Isolation Bearings

Source: Symans 2003

The area of the loop (E_{loop}) is equal to the amount of damping or energy dissipated in the cycle. A cycle is defined as the movement of the bearing from a given maximum positive displacement (Δ^+), to maximum negative displacement (Δ^-), and back to Δ^+ ; or vice-versa. The damping ratio (β_{eff}) is based on E_{loop} and the effective stiffness (K_{eff}) of the cycle. The equations for calculating β_{eff} and K_{eff} are given in Figure C above, and

they can also be found in Chapter 17 of ASCE 7-10. These effective properties are used to design the isolation system and determine parameters such as the effective period of the isolated structure. The design of the isolation system used in this study is discussed in section “4.0 Design Phase.”

2.2 Friction Pendulum (FP) Bearings

The type of base isolation technology that is used in this study is the Triple Friction Pendulum (TFP) bearing. It is a more advanced modification of the single Friction Pendulum (FP) bearing. Both of these bearings are sliding type bearings, and they were each developed by Earthquake Protection Systems (EPS).² Pictures of the TFP and FP bearings are shown in Figures D and E below.

Figure D: Triple Friction Pendulum Bearing

Source: EPS 2011

Figure E: Friction Pendulum Bearing

Source: EPS 2011

² EPS can be reached at the company's website, www.earthquakeprotection.com

To make TFP bearing isolation technology easier to understand, this thesis will first describe the FP bearing. A cross-section of the FP bearing is shown in Figure F below.

Figure F: Cross-Section of Friction Pendulum Bearing

Source: Okamura and Fujita 2007

A FP bearing is composed of a slider body which slides across a spherical concave surface. The slider body has a bearing material which is attached to a friction slider. The bearing material is usually made out of polytetrafluoroethylene (PTFE) plastic, more commonly known by the brand name Teflon. The spherical concave surface is made out of stainless steel. This curved surface induces a natural restoring force that re-centers the slider to its original position. A vertical restrainer is located at the circumference of the curved surface to restrict sliding in rare seismic events. The friction force needed to cause sliding (F_f) is a function of the vertical load on the bearing (W) and the coefficient of friction (μ). The coefficient of friction is determined by the interaction between the bearing material and the stainless steel of the spherical concave

plate. The coefficient of friction controls the level of damping or energy absorbed by the bearing. The radius of curvature (R) of the spherical plate determines the effective period (T_{eff}) of the bearing, and thus the period of the isolated structure (Okamura and Fujita 2007). Equations 2 and 3, shown below, illustrate the determination of F_f and T_{eff} :

$$F_f = \mu W \quad \text{Eq. 2}$$

Where:
 F_f = Friction force [kips]
 μ = Coefficient of friction
 W = Vertical load on bearing [kips]

$$T_{eff} = 2\pi \sqrt{\frac{R}{g}} \quad \text{Eq. 3}$$

Where:
 T_{eff} = Effective period of bearing [sec.]
 R = Radius of curvature [in.]
 g = Gravitational acceleration $\left[\frac{\text{in.}}{\text{sec.}^2}\right]$

Note that Equation 3 is an estimation that assumes a frictionless surface in order to calculate T_{eff} . The equations to find the actual effective periods (taking into account the effective stiffness and therefore friction) can be found in Chapter 17 of ASCE 7-10.

EPS produces concave plates with standard radii of curvature. Currently, the sizes available for the radius of curvature R include: 39, 61, 88, 120, 156, and 244 inches. EPS also states that the standard values for the bearings' dynamic coefficients of friction range between 0.03 – 0.12 (EPS 2003).

2.3 Triple Friction Pendulum (TFP) Bearings

The Triple Friction Pendulum (TFP) bearing differs from the single Friction Pendulum (FP) bearing in that there are 3 friction pendulum mechanisms existing in each bearing instead of just 1 mechanism. These mechanisms are activated at different stages as the seismic demand gets stronger. The 3 mechanisms are achieved by using 4 concave surfaces in a single bearing, with sliding occurring on two of the surfaces at a given time. An image of the TFP's disassembled parts and a cross section of a TFP bearing are shown in Figure G below and Figure H on page 11, respectively.

Figure G: Disassembled Triple Friction Pendulum Bearing

Source: EPS 2011

Figure H: Triple Friction Pendulum Bearing Cross Section

Source: Fenz and Constantinou 2007

Usually the two inner concave surfaces have the same values for their coefficients of friction ($\mu_2 = \mu_3$) and radii of curvature ($R_2 = R_3$). The outer concave surfaces may have different μ and R values from each other, although it is not uncommon for the same values to be used. For this case study, it is assumed that $\mu_1 = \mu_4$ and $R_1 = R_4$. The plates are designed such that the coefficients of the friction of the inner slide plates are lower ($\mu_2 = \mu_3 < \mu_1 = \mu_4$), thus allowing sliding to begin with them. The inner slide plates are also designed with smaller radii of curvature ($R_2 = R_3 < R_1 = R_4$), causing the effective stiffness of the inner plates to be larger, which is ideal for service level earthquakes. Sliding is designed to occur on the outer plates when their larger friction forces are reached at design basis earthquake (DBE) and maximum considered earthquake (MCE) levels, resulting in lower bearing displacements. The larger radii of curvature of the outer plates cause a larger effective period, which reduces lateral forces exerted onto the superstructure.

In order to calculate the effective period of a given pendulum mechanism of the TFP bearing, the effective radius of curvature must be calculated. Since the effective radius of the pendulum mechanism equals the sum of both effective radii of the activated slide plates, the effective radius of curvature of each slide plate must first be calculated. For instance, for slide plate 2 the effective radius would be equal to Equation 4 below:

$$R_{\text{eff}2} = R_2 - h_2 \quad \text{Eq. 4}$$

Where: $R_{\text{eff}2}$ = Effective radius of slide plate 2 [in.]
 R_2 = Radius of slide plate 2 [in.]
 h_2 = Height of slide plate 2 [in.]

The effective radius of the inner pendulum would then be equal to Equation 5 below:

$$R_{\text{eff,pend}2} = R_{\text{eff}2} + R_{\text{eff}3} \quad \text{Eq. 5}$$

Where: $R_{\text{eff,pend}2}$ = Effective radius of inner pendulum [in.]
 $R_{\text{eff}2}$ = Effective radius of slide plate 2 [in.]
 $R_{\text{eff}3}$ = Effective radius of slide plate 3 [in.]

Finally, the effective period can be found by Equation 6 as follows:

$$T_{\text{eff,pend}2} = 2\pi \sqrt{\frac{R_{\text{eff,pend}2}}{g}} \quad \text{Eq. 6}$$

Where: $T_{\text{eff,pend}2}$ = Effective period of inner pendulum [sec.]
 $R_{\text{eff,pend}2}$ = Effective radius of inner pendulum [in.]
 g = Gravitational acceleration $\left[\frac{\text{in.}}{\text{sec.}^2}\right]$

The effective radius and effective period of the outer pendulum mechanism may be found in the same manner as shown above.

Figure I: Sliding Regime Force-Displacement Relationship of TFP Bearing
 Source: Fenz and Constantinou 2007

The force-displacement relationship of the inner pendulum mechanism and outer pendulum mechanism are shown in Figure I above. Note how the stiffness of each mechanism is related to the weight on the bearing (W) and the radii of curvatures of the activated concave surfaces. Also note how each mechanism is activated once the friction force of its sliding plates is reached. The stiff, small displacement service level event (SLE) behavior of the inner pendulum mechanism is shown in relation to the flexible, large displacement (DBE and MCE) behavior of the outer pendulum mechanism. Note that a third mechanism could be designed specifically for the MCE level by using different R and μ values for each of the outer plates. The ability to have multiple mechanisms in a single bearing and design each mechanism for a given seismic demand

level is the primary benefit of using TFP bearings. Also, using multiple concave plates in a bearing instead of a single-plate FP bearing reduces the total required bearing displacement, which decreases the size of each bearing and reduces cost. Another benefit is that compared to lead-rubber bearings, TFP bearings have superior performance at service level (43-year return period) earthquakes due to their ability to be engaged and utilized at low coefficient of friction values.

3.0 BUILDING SELECTION PHASE

The first phase of this study was the building selection phase. The purpose of this phase was to determine the type of building that would be designed and analyzed, including its structural systems, layout, isolation system, and site location.

3.1 Site

A San Diego location was chosen for the project site, as shown in Figure J below, because 7 sets of ground motion records were already selected and scaled for the location from a previous project. These records were subsequently used for the nonlinear time history analyses in this study. The site's soil classification was assumed to be unknown.

Site Coordinates 32.87856°N, 117.22578°W

Figure J: Project Design Site Location

Source: USGS 2012

3.2 The Office Building

The office building that was designed and analyzed in this study was a hypothetical 220-foot by 110-foot, 12-story steel office building. An office building was selected because it is not a building type that is typically base-isolated, so it was an ideal candidate to study the cost effectiveness of implementing the technology. The building was modified from an actual fixed-base 12-story steel office building located in Los Angeles. This particular building was chosen partly because the construction drawings were readily available. The steel superstructure had a lateral system composed of special moment frames (SMF) in the longitudinal direction and special concentrically braced frames (SCBF) in the transverse direction, and the same structural systems were used for both of the office buildings designed for this study. The modifications that were made from the original structure's typical floor plan are illustrated in Figure K on page 18.

These modifications include

- Reshaping the floor slab/plan from partially curvilinear to fully rectangular
- Changing the brace location to make the plan more regular and to align the center of rigidity with the center of mass
- Relocating the outer gravity columns to align with the lateral system gridlines
- Using a typical floor plan layout for the roof plan layout

The only modification made to the elevations aside from those affected by the changes noted above was removing the penthouse on the 13th floor. This change is

illustrated in the elevations, shown in Figures L and M on pages 19 and 20, respectively.

Some characteristics of the elevations include

- 12 stories at 13.33 feet (160 inches) for a total building height of 160 feet
- X-type brace configuration used for SCBF

Figure K: Modified Office Building Typical Floor Plan

Source: KPFF 2012, self-produced

Figure L: Modified Office Building SMF Elevation

Source: KPFF 2012, self-produced

Note from Figure L that no modifications were made to the layout of the original SMF elevation. The members shown were those of the prototype structure, and they were later redesigned during the design phase of this study.

Note from Figures L and M that the following section types were used for the lateral systems:

- SMF
 - Columns: W-shapes
 - Beams: W-shapes
- SCBF
 - Columns: W-shapes
 - Beams: W-shapes
 - Braces: HSS square tubes

The W-shapes were assumed to be American Society for Testing and Materials (ASTM) A992 steel ($F_y = 50$ ksi), and the HSS sections were assumed to be ASTM A500 Grade B steel ($F_y = 46$ ksi) (Table 2-3, AISC 360-05). Also note from Figure M that the 13th story penthouse was removed from the original SCBF elevation.

The final plan and elevations used for this study are shown in Figures N, O and P on pages 21, 22 and 23, respectively:

Figure M: Modified Office Building SCBF Elevation
Source: KPFF 2012, self-produced

Figure N: Key Plan
Source: self-produced

Figure O: SMF Elevation
Source: self-produced using ETABS

Figure P: SCBF Elevation
Source: self-produced using ETABS

3.3 Isolation System

The type of base isolation bearing that was used for the base-isolated office building is the Triple Friction Pendulum (TFP) bearing. This bearing is a sliding-type bearing that functions as a combination of multiple single Friction Pendulum (FP) bearings, as described earlier in section “2.0 Background.” An isolation bearing was placed beneath every column (including lateral and gravity systems), for a total of 32 bearings. Two variations of the TFP bearings were designed:

- “flexible” bearings beneath the central columns (12 bearings total)
- “stiff” bearings beneath the perimeter columns (20 bearings total)

The perimeter bearings were designed with a higher lateral stiffness than the central bearings in order to reduce the effect of torsional isolated modes. A rigid reinforced concrete slab was used for the isolation platform (which supported the superstructure and rested above the isolation bearings). The slab weight was estimated and the tributary weight acting on each bearing was accounted for, as described later in section “4.0 Design Phase.” For the purposes of this study, only the isolation bearings and moat size were designed. The other components of the isolation system (aside from the reinforced concrete slab isolation platform) were not considered.

4.0 DESIGN PHASE

In the design phase, the lateral systems of the fixed-base and base-isolated structures as well as the isolation system of the isolated structure were designed for gravity and seismic loads. Each structure was designed according to ASCE 7-10 standards, using modal response spectrum analysis. Both structures were modeled and analyzed in ETABS in order to carry out the design.

4.1 Design Phase Procedure

An overview of the design phase procedure is shown below:

1. Determine design load combinations, criteria, and requirements according to Chapters 2, 4, 11, 12, and 17 in ASCE 7-10 and AISC 341-06.
2. Generate design basis earthquake (DBE) and maximum considered earthquake (MCE) response spectra.
3. Create the unit load take-off to determine gravity loads; calculate masses.
4. Create a structural model in ETABS using preliminary members for the lateral system. Input mass, material properties from AISC 360-05, response spectra, design parameters, loads, and load combinations, etc.
5. Perform the modal response spectrum analysis:
 - Obtain preliminary modes, interstory drifts, and forces
 - Verify/adjust the lateral load combinations to requirements; reiterate the procedure above as needed
6. Design the lateral system with ETABS ‘Steel Design/Check of Structure’ tool:
 - Analyze and reiterate the above procedure with the new structural members until all capacities meet the minimum force and drift demands
 - Repeat steps 4 and 5 until reiteration is no longer needed

4.2 ASCE 7-10 Design Criteria

The code used for the design of the structures and isolation system is *ASCE 7-10: Minimum Design Loads for Buildings and Other Structures*. The general procedure for designing each structure involved using load and resistance factor design (LRFD) load combinations to check the capacity of the structures for dead, live, and earthquake loads. The following design parameters were obtained from Chapters 11 and 12 in the code:³

- Idealize diaphragms as rigid diaphragms
- No horizontal or vertical irregularities, $\rho = 1.0$
- Soil Classification: Site Class D “Stiff Soil”
- Seismic Design Category: D
- Site Risk Category: Risk Category III – “Substantial Hazard”
- Seismic Importance Factor: $I_e = 1.25$
- Design story drift limit: $\Delta = (0.015)(\text{story height})$
- Recommended procedure: modal response spectrum analysis

Chapters 12 and 17 of ASCE 7-10 require a design story drift limit of 0.015 for both fixed-base and base-isolated structures, respectively. However, the base-isolated structure in this study was designed for a stricter design story drift limit range of 0.007 - 0.010. This story drift limit range was recommended by Earthquake Protection Systems (EPS) as a means to optimize the lateral stiffness and seismic performance of base-

³ Refer to Appendix A on page 160 for documentation of design parameter decisions

isolated structures. As explained earlier, base isolation is most effective for short, stiff buildings with small periods. The intent of the strict design drift limit requirement was to keep the superstructure (structure above the isolation platform) mostly elastic, with very minor to no yielding.

The design coefficients and factors in Table 1 below were obtained from Chapters 12 and 17 of ASCE 7-10 and were used to design the structures in this study:⁴

Table 1: Design Coefficients and Factors for SFRS

Source: self-produced

Seismic Force-Resisting System	(Table 12.2-1, ASCE 7-10)			(17.5.4.2, ASCE 7-10)
	Response Modification Coefficient R	Overstrength Factor Ω_o	Deflection Amplification Factor C_d	Isolated Response Modification Coefficient R_I
Steel special moment moment frames (SMF)	8	3	5.5	2
Steel special concentrically braced frames (SCBF)	6	2	5	2

Note from Table 1 above that the Isolated Response Modification Coefficient ($R_I = 2$) is less than the fixed-base Response Modification Coefficients ($R = 8$ and 6). ASCE 7-10 places an upper-bound limit of 2 for R_I in order to adjust the relatively lower elastic seismic demand forces of the base-isolated superstructure to design values that are nearly equal to or greater than those of a fixed-base structure. The intent of the relatively low value of R_I is to keep the superstructure essentially elastic for a design basis earthquake (DBE) level event (i.e., the superstructure should incur little to no permanent deformation from a DBE event).

⁴ Refer to Appendix C on page 167 for documentation of R_I values

Table 2 below shows the building mass data that were calculated and used for this study:⁵

Table 2: Floor, Roof, and Isolation Bearing Masses

Source: self-produced

Floor, Roof, and Isolation Bearing Masses					
Floors 1-12 (includes isolation platform)		Roof		Isolation Bearings	
$X_c =$	1320 in	$X_c =$	1320 in	# bearings =	32
$Y_c =$	660 in	$Y_c =$	660 in	Bearing W =	8.346 kip
W =	2140.6 kip	W =	2249.5 kip	Bearing M =	0.0216 kip-sec ² /in
M =	5.540 kip-sec ² /in	M =	5.822 kip-sec ² /in	Total M =	0.6905 kip-sec ² /in
MMI =	4220233 kip-in-sec ²	MMI =	4370504 kip-in-sec ²	MMI =	0.18431 kip-in-sec ²

Note that for simplicity the isolation platform was assumed to have the same mass properties as all of the other floors. All of the isolation bearings had the same mass properties since the same size was used for each bearing.

The load combinations shown in Table 3 on page 29 were determined using Chapters 2 and 12 of ASCE 7-10 and were used for designing the fixed-base and base-isolated structures:⁶

⁵ Refer to Appendices G and H on pages 185 and 186 for documentation of building mass calculations

⁶ Refer to Appendices E and F on pages 183 and 184 for documentation of load take-off calculations, and refer to Appendices B and C on pages 163 and 167 for load combination calculations

Table 3: LRFD Design Load Combinations

Source: self-produced

Fixed-Base Gravity-Only Load Combos	Base-Isolated Gravity-Only Load Combos
To Solve for Forces:	To Solve for Forces:
COMBO 1.) $(1.4)D$	COMBO 1.) $(1.4)D$
COMBO 2.) $(1.2)D + (1.6)L + (0.5)L_r$	COMBO 2.) $(1.2)D + (1.6)L + (0.5)L_r$
COMBO 3.) $(1.2)D + (1.6)L_r + L$	COMBO 3.) $(1.2)D + (1.6)L_r + L$
Fixed-Base Lateral Load Combos	Base-Isolated Lateral Load Combos
To Solve for Forces:	To Solve for Forces:
<u>SMF (U1 Direction)</u>	<u>SMF (U1 Direction)</u>
F: COMBO 5.) $(1.361)D + (0.2884)Q_E + L$	F: COMBO 5.) $(1.361)D + (0.6250)Q_E + L$
F: COMBO 7.) $(0.739)D + (0.2884)Q_E$	F: COMBO 7.) $(0.739)D + (0.6250)Q_E$
<u>SCBF (U2 Direction)</u>	<u>SCBF (U2 Direction)</u>
F: COMBO 5.) $(1.361)D + (0.3524)Q_E + L$	F: COMBO 5.) $(1.361)D + (0.6250)Q_E + L$
F: COMBO 7.) $(0.739)D + (0.3524)Q_E$	F: COMBO 7.) $(0.739)D + (0.6250)Q_E$
To Solve for Deflections:	To Solve for Deflections:
<u>SMF (U1 Direction)</u>	<u>SMF (U1 Direction)</u>
D: COMBO 5.) $(1.361)D + (1.0120)Q_E + L$	D: COMBO 5.) $(1.361)D + (1.0000)Q_E + L$
D: COMBO 7.) $(0.739)D + (1.0120)Q_E$	D: COMBO 7.) $(0.739)D + (1.0000)Q_E$
<u>SCBF (U2 Direction)</u>	<u>SCBF (U2 Direction)</u>
D: COMBO 5.) $(1.361)D + (0.8612)Q_E + L$	D: COMBO 5.) $(1.361)D + (1.0000)Q_E + L$
D: COMBO 7.) $(0.739)D + (0.8612)Q_E$	D: COMBO 7.) $(0.739)D + (1.0000)Q_E$

Note: D = dead load, L = live load, L_r = roof live load, Q_E = seismic load

The seismic load factors shown in the lateral load combinations in Table 3 above were used for the final design of the structures. Earlier trial factors were used and adjusted as the members of the structure were redesigned, the structure's modal behavior and resulting base shears obtained from modal response spectrum analysis changed, and scale factors were readjusted.

4.2.1 ASCE 7-10 – Response Spectrum (USGS)

In order to generate seismic load demands to design the structures, a design basis earthquake (DBE) level response spectrum was generated, based on the project's site location, soil type, etc. A DBE level seismic event is classified as having a uniform 10% probability of exceedence in 50 years, which equals a return period of 475 years (ASCE 7-10). A maximum considered earthquake (MCE) level spectrum was generated and scaled down to find the DBE spectrum, with $S_{a-DBE} = (2/3)S_{a-MCE}$, where S_a = spectral acceleration. An MCE level seismic event is classified as having a uniform 2% probability of exceedence in 50 years, which equals a return period of 2,475 years (ASCE 7-10). The MCE level spectrum was used to determine the maximum displacement of the isolation system, and thereby design the displacement capacity of the isolation system. DBE and MCE seismic levels were also used to gauge seismic performance levels of the structure, which is covered in the analysis phase of this thesis. The DBE and MCE response spectra used for this study are shown in Figure Q below:⁷

Figure Q: MCE and DBE Response Spectra

Source: USGS 2012

⁷ Refer to Appendix D on page 176 for documentation of response spectrum calculations

4.2.2 ASCE 7-10 – Modal Response Spectrum Analysis

According to section 12.9 of ASCE 7-10, modal response spectrum analysis involves finding the natural mode shapes of the structure and including “a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass” in each horizontal orthogonal direction. The response of the structure is to be found for each of the mode shapes and combined, relative to each mode’s mass participation value, to give a total response value. For the design of a fixed-base structure, a DBE spectrum (with 5% critical damping) is used for the analysis. According to section 17.6.3.3 of ASCE 7-10, to design an isolated structure the isolated modes must be assigned a damping value “not greater than the effective damping of the isolation system or 30 percent of critical, whichever is less.”

The isolation system in this study had an equivalent viscous damping ratio of 22% at design displacement.⁸ In order model the equivalent viscous damping ratios for the isolated modes during modal response spectrum analysis, the damping ratios were assigned directly to the isolation bearing elements. This method is explained in more detail in section “4.3 ETABS.”

⁸ Refer to section 4.5.1 on page 54 for documentation of the base isolation DBE damping value

4.3 ETABS

To model and design both of the structures using modal response spectrum analysis, a 3D digital model of each structure was created in ETABS. ETABS is a building analysis software program developed by Computers & Structures, Inc. (CSI). This section of the study will illustrate the modeling and analysis decisions (input) for creating the structural models in ETABS, including material properties, structural member section types, connectivity data, loading, mass, and design criteria.

4.3.1 ETABS – Model Structural Input

In order to create the structural models, the standard material properties in ETABS were used for this study, including A992 steel, A500 Grade B steel, and concrete. Units were in SI kip-in format. A992 steel was assigned to all steel structural members except for the HSS braces, which were assigned A500 Grade B steel. Concrete was assigned to the concrete slabs. The masses and weights of the materials were zeroed out and instead accounted for directly; the weights were included in the assigned loading and the masses were entered as equivalent point masses attached to the diaphragms.

The default section property values in ETABS were used for all the wide-flange steel members. Default section properties were not available in ETABS for hollow structural sections (HSS), so they were taken from the AISC Steel Construction Manual and input directly into ETABS. Property modifiers (for area, moment of inertia, mass, and weight) were not assigned to any of the structural sections.

All the typical floors were assigned a section property named “SLAB1”. The section was assigned as a shell and the membrane and bending thickness of the slab were entered as 0.001 inches in order to make the membrane and bending properties effectively equal to zero. This small stiffness assignment was done to model only the slab’s lateral rigidity, which was accounted for by the rigid diaphragm assignments. All the points on each floor were modeled as a rigid diaphragm.

In ETABS, rigid end zone lengths are determined by assigning a factor to the column depth for beams or to the beam depth for columns. A rigid end zone factor of 0.5 was used for this study since it was typical for most steel frame structures. The ends of the frame members were kept fully rigid, while the braces were assigned end releases for both bending degrees of freedom on each end. In order to properly model the relative stiffness of the members’ connections to the base, the moment frame members were assigned fully rigid support restraints while the braced frame members were assigned pinned (fixed only in translation) support restraints. Figure R on page 34 shows how the member end releases and support restraints were modeled in ETABS.

Figure R: ETABS Illustration of Structural Member End Release Properties
Source: ETABS 2011

As mentioned earlier, masses were modeled in ETABS as equivalent point masses located at the center of mass and attached to each rigid diaphragm. Point masses were calculated for the roof and typical floor slabs, and they accounted for the tributary mass of the exterior partitions.⁹ The mass properties are shown in Figures S, T, and U on page 35:

⁹ Refer to Appendices G and H on pages 185 and 186 for documentation of point mass calculations

Masses in Global Directions	
Direction X, Y	5.82
Direction Z	0.

Mom. of Inertia in Global Directions	
Rotation about X	0.
Rotation about Y	0.
Rotation about Z	4370504

Figure S: ETABS Roof Point Masses
Source: ETABS 2011

Masses in Global Directions	
Direction X, Y	5.54
Direction Z	0.

Mom. of Inertia in Global Directions	
Rotation about X	0.
Rotation about Y	0.
Rotation about Z	4220233

Figure T: ETABS Floor Point Masses
Source: ETABS 2011

Figure U: ETABS Illustration of Restraints and Point Masses
Source: ETABS 2011

4.3.2 ETABS – Model Isolation System Input

A separate slab, “SLAB2,” was modeled for the isolation platform in the base-isolated model. Again, units were in SI kip-in format. The slab was designated as a thick plate (having translational and rotational stiffness in all directions) with a bending thickness of 24 inches in order to form a properly rigid bending connection to the isolation bearings. The membrane bending thickness was entered as 0.001 inches. It was important for the isolation platform to be rigid rotationally in order to resist the large bending moments imposed by the large lateral forces of the isolation bearings, as demonstrated by the free body diagram of the isolation bearing element shown in Figure V below. For simplicity, the same point mass values were assumed for the isolation platform as for the typical floor slabs (5.54 kip-sec²/in translational and 44220233 kip-in-sec² rotational).¹⁰

Figure V: Friction Pendulum Bearing Element Free Body Diagram

Source: CSI 2011

¹⁰ Refer to Appendix H on page 186 for documentation of the isolation platform mass calculations

Figure W: ETABS Friction Pendulum Bearing Assignments

Source: ETABS 2011

Figure W above shows the layout of the friction pendulum bearings, which had different types (A through F) modeled in ETABS based on their unique lateral properties. These lateral properties were determined by the varying tributary weights acting upon each bearing.¹¹ The typical properties shared by all the bearings (mass, rotational inertia, and vertical stiffness) are shown in Figures X and Y on page 38:

¹¹ Refer to Appendix I on page 187 for calculations of friction pendulum bearing properties and weights

Property Name FPA		Type Isolator2	
Total Mass and Weight			
Mass	0.0216	Rotational Inertia 1	0.1843
Weight	0.	Rotational Inertia 2	0.
		Rotational Inertia 3	0.
Directional Properties		P-Delta Parameters	
Direction	NonLinear	Advanced...	
☒ U1	☐	Modify/Show for U1...	
☒ U2	☒	Modify/Show for U2...	
☒ U3	☒	Modify/Show for U3...	
☐ R1	☐	Modify/Show for R1...	
☐ R2	☐	Modify/Show for R2...	
☐ R3	☐	Modify/Show for R3...	
		OK	
		Cancel	

Figure X: ETABS Typical Friction Pendulum Bearing Properties
Source: ETABS 2011

Identification	
Property Name	FPA
Direction	U1
Type	Isolator2
NonLinear	No
Linear Properties	
Effective Stiffness	126147.
Effective Damping	0.

Figure Y: ETABS Typical Friction Pendulum Bearing Vertical Properties
Source: ETABS 2011

Friction pendulum bearing FPA was located at the inside of the structure, while all the other FP bearings were located along the perimeter. In order to reduce torsional modes, FPA was made more flexible laterally by having a friction coefficient of 0.02, while the perimeter bearings were made stiffer by having a friction coefficient equal to 0.03. For simplicity, the friction coefficients were assumed to have the same value for both fast and slow velocities. The rate parameter determined the rate of change between the fast and slow coefficients of friction for a given velocity, so it had no effect and was taken as unity.¹² The lateral properties are shown in the Figures Z, AA, and BB below:

Identification		Identification	
Property Name	FPA	Property Name	FPB
Direction	U2	Direction	U2
Type	Isolator2	Type	Isolator2
NonLinear	Yes	NonLinear	Yes
Linear Properties		Linear Properties	
Effective Stiffness	4.17	Effective Stiffness	4.99
Effective Damping	0.22	Effective Damping	0.22
Shear Deformation Location		Shear Deformation Location	
Distance from End-J	0.	Distance from End-J	0.
Nonlinear Properties		Nonlinear Properties	
Stiffness	55.33	Stiffness	40.41
Friction Coefficient, Slow	0.02	Friction Coefficient, Slow	0.03
Friction Coefficient, Fast	0.02	Friction Coefficient, Fast	0.03
Rate Parameter	1.	Rate Parameter	1.
Radius of Sliding Surface	303.	Radius of Sliding Surface	303.

Figure Z: ETABS Friction Pendulum Bearing Type A and B Lateral Properties

Source: ETABS 2011

¹² Refer to Appendix I on page 187 for calculations of friction pendulum bearing properties

Identification		Identification	
Property Name	FPC	Property Name	FPD
Direction	U2	Direction	U2
Type	Isolator2	Type	Isolator2
NonLinear	Yes	NonLinear	Yes
Linear Properties		Linear Properties	
Effective Stiffness	4.55	Effective Stiffness	4.34
Effective Damping	0.22	Effective Damping	0.22
Shear Deformation Location		Shear Deformation Location	
Distance from End-J	0.	Distance from End-J	0.
Nonlinear Properties		Nonlinear Properties	
Stiffness	36.82	Stiffness	35.09
Friction Coefficient, Slow	0.03	Friction Coefficient, Slow	0.03
Friction Coefficient, Fast	0.03	Friction Coefficient, Fast	0.03
Rate Parameter	1.	Rate Parameter	1.
Radius of Sliding Surface	303.	Radius of Sliding Surface	303.

Figure AA: ETABS Friction Pendulum Bearing Type C and D Lateral Properties

Source: ETABS 2011

As mentioned above, although all the outer/stiff bearings (FPB, FPC, FPD, FPE, and FPF) had the same friction coefficients and radii of curvature, their varying effective weights affected their lateral stiffness. The different weights acting on each bearing directly correlated with their friction force, which in turn impacted their effective and elastic stiffness.

The effective damping (0.22 or 22%) was a product of all the bearings making up the isolation system. This value was the average damping value of all the isolation bearings, and the same effective damping value was input for all the bearings.

Alternatively, each individual bearing's damping value could have been input separately;

however, the same average value was chosen for simplicity and because the difference between the two methods was insignificant.

Identification	Property Name	Value	Identification	Property Name	Value		
Identification	Property Name	FPE	Identification	Property Name	FPF		
	Direction	U2		Identification	Direction	U2	
	Type	Isolator2			Identification	Type	Isolator2
	NonLinear	Yes				Identification	NonLinear
Linear Properties		Linear Properties					
Effective Stiffness	6.72	Effective Stiffness	3.36				
Effective Damping	0.22	Effective Damping	0.22				
Shear Deformation Location		Shear Deformation Location	Shear Deformation Location				
Distance from End-J	0.	Distance from End-J	0.				
Nonlinear Properties		Nonlinear Properties					
Stiffness	54.36	Stiffness	27.18				
Friction Coefficient, Slow	0.03	Friction Coefficient, Slow	0.03				
Friction Coefficient, Fast	0.03	Friction Coefficient, Fast	0.03				
Rate Parameter	1.	Rate Parameter	1.				
Radius of Sliding Surface	303.	Radius of Sliding Surface	303.				

Figure BB: ETABS Friction Pendulum Bearing Type E and F Lateral Properties
Source: ETABS 2011

4.3.3 ETABS – Loading and Analysis Input

In order to perform the gravity, modal, and modal response spectrum analyses in ETABS, the loads were first entered. The distributed and point loads were found with tributary area and were in units of lb-ft. For the moment frames, distributed dead loads of 1,240 lb/ft and 1,180 lb/ft were calculated for the floors and roof, respectively.

Point loads on the left edge of the moment frames were larger than the right edge, due to the larger tributary area from the 40-foot span to the edge of the structure, compared to the 30-foot span on the opposite end. The floors had dead point loads of 24,800 lb and 18,600 lb on each end, while the roof had dead point loads of 23,600 lb and 18,600 lb.

No live load reduction was assigned in ETABS due to the live loads already being reduced by hand and entered into the models.¹³ The moment frames had distributed reduced live loads of 1,066 lb/ft on the floors. The roof distributed reduced live loads equaled 240 lb/ft. Floor point reduced live loads equaled 18,440 lb and 13,830 lb, while roof point reduced live loads equaled 4,800 lb and 3,600 lb.

The braced frames had distributed dead loads of 620 lb/ft and 590 lb/ft for the floors and roof, respectively. It had point dead loads on each edge of the frames, which equaled 37,200 lb on the floors and 35,400 lb. on the roof. The reduced distributed live loads equaled 633 lb/ft and 1,350 lb/ft on the floors and roof, respectively. The reduced point live loads equaled 25,200 lb on the floors and 27,600 lb on the roof.

¹³ Refer to Appendices E and F on pages 183 and 184 for calculations of live load reductions

Once the loads were entered into ETABS, the design load combinations were created as was shown in Table 3 on page 29. In order to perform the modal analysis, 12 modes were calculated, which was enough to capture the 90 percent cumulative mass modal participation requirement in the x-translational, y-translational, and z-torsional directions. The analysis included residual-mass modes.

The DBE and MCE response spectra were entered into ETABS and were assigned the typical 5% (0.05) damping ratio. Response spectrum functions were created for each direction, using the complete quadratic combination (CQC) modal combination method, and the square root of the sum of squares (SRSS) directional combination method. Also, the response spectrum functions' acceleration units were converted from g to in/sec² via the 386.4 scale factor.

An ETABS analysis was conducted to find the isolation bearing design displacement (with the effective damping values of 22% assigned directly to the isolation bearings). After comparing the results to the expected hand calculation value of $D_D = 13.46$ inches, it was found that the ETABS analysis displacements differed by about 2 inches. The results are shown in Table 4 below:

Table 4: Isolation Bearing Design Displacement Results Comparison
Source: self-produced

	D_{Dx} [in.]	D_{Dy} [in.]
Hand Calculations	13.46	13.46
(% Difference)	(0%)	(0%)
Damped Bearings	15.19	15.85
(% Difference)	(12.9%)	(17.8%)

The purpose of assigning 22% damping to the bearings was to take into account the equivalent damping of the isolation bearings at design displacement for the isolated modes (typically modes 1-3 for a 3-dimensional model of a structure: x-translational, y-translational, and z-torsional). After the third isolated mode, the higher modes tended to behave more like the modes of the fixed-base structure, and the damping of the base isolation bearings had less of an effect on the superstructure's modal behavior.

4.3.4 ETABS – Steel Design Check Input

The final ETABS input needed to perform the analyses and design the structural members in each building was the steel design check input. Load combinations (that were specified earlier) were assigned to design the special moment frame (SMF) and special concentrically braced frame (SCBF) systems for strength and deflection requirements. The SMF was assigned its appropriate U1/x-direction modal response spectrum analysis load combinations, and the SCBF was assigned its appropriate load combinations for the U2/y-direction.¹⁴ Load combinations 1-3 were the governing load and resistance factor design (LRFD) gravity load combinations.

Point bracing was assigned along the third-length points of the SMF beams, where the gravity beams framed into the moment frame beams. Similarly, point bracing was assigned at the halfway points along the SCBF beams where the lateral braces intersected. Uniform bracing was assigned along the top of the SCBF beams due to shear studs connecting the beams to the metal decking. The studs were assumed to occur at a close enough interval such that uniform bracing would be deemed appropriate. The point bracing and uniform bracing were assigned in order to account for restraint against lateral torsional buckling during the design of the steel frames.

Next, special seismic data was input into ETABS. As determined in section “4.2 ASCE 7-10 Design Criteria,” a rho factor of 1 and design category D were specified. The lateral force resisting system and system overstrength factors of 3 and 2 for the SMF

¹⁴ Refer to Appendices B and C on pages 163 and 167 for calculations of load combinations

and SCBF, respectively, were entered by default in the steel frame design preferences interface.

The ETABS steel frame design preference interfaces for the SMF and SCBF were used to specify the design coefficients and factors, shown in Table 1 on page 27, as well as the analysis preferences for the design of each lateral system. The design method chosen was the direct analysis method. This method used general second order analysis with the stiffness reduction factor (τ_b), which varied with the axial force in each member (CSI 2010). These were the default analysis settings in ETABS. The related equations for these settings are shown below.

$$EI^* = 0.8\tau_b EI \quad \text{Eq. 7}$$

$$EA^* = 0.8EA \quad \text{Eq. 8}$$

$$\tau_b = \begin{cases} 1.0 & \text{for } \frac{\alpha P_r}{P_y} \leq 0.5 \\ 4 \left(\frac{\alpha P_r}{P_y} \right) \left(1 - \frac{\alpha P_r}{P_y} \right) & \text{for } \frac{\alpha P_r}{P_y} \geq 0.5 \end{cases} \quad \text{Eq. 9}$$

B_1 and B_2 not used

$K_2 = 1$ (used for P_n)

Where: E = Modulus of Elasticity [kips/in²]
 I = Moment of Inertia [in⁴]
 A = Area [in²]
 τ_b = Stiffness Reduction Factor
 α = Design philosophy factor (1.0 for LRFD)
 P_r = Required second-order axial strength [kips]
 P_y = Axial yield strength [kips]
 B_1, B_2 = Moment magnification factors
 K_2 = Effective length factor

The typical LRFD resistance factors (ϕ) for each loading type were also specified in the steel frame design preferences interface. Conservatively, the doubler plate was not assumed to be plug welded, which neglected the cumulative column web and doubler plate thickness for the minimum thickness local buckling check of the panel zone. By default, the submerged arc welding (SAW) process was chosen for the welding of the hollow structural sections (HSS), which allowed the design wall thickness of the HSS members to be unreduced (CSI 2010). Dead load, live load, and total deflections were considered using the standard deflection ratio limits. Once the steel frame design preferences interface was completed, all the required data was input into ETABS and the structures were ready to be designed.

4.4 Design of Fixed-Base Structure

The fixed-base structure was designed using modal response spectrum analysis and the ‘Steel Design/Check of Structure’ tool in ETABS. The analysis resulted in a final lateral system with the structural sections shown in Figures CC and DD below.

Figure CC: Sections of Designed Fixed-Base SMF

Source: ETABS 2011

Figure DD: Sections of Designed Fixed-Base SCBF

Source: ETABS 2011

The special moment frame (SMF) resulted in W27x columns and W30x beams. For constructability, the column splices were placed every 2 stories and the same sections were used for the SMF columns on each floor. Figures CC and DD above also illustrate the color-coded demand-capacity ratios (ranging in severity from less than 0.50 in blue to more than 0.95 in red) of the structural members from the 'Steel Design/Check of Structure' tool in ETABS. The design of the SMF was determined by drift limits and relative stiffness limits. The outer columns easily passed the relative stiffness requirements, while the inner W27x columns were governed by the W30x beams attached on each side. Specifically, the inner columns sustained demand-capacity ratios of over 0.90 and in some cases 0.95, as shown by the color coding in Figure CC.

The special concentrically braced frame (SCBF) resulted in W18x columns, W16x beams, and braces including HSS8x8x and HSS7x7x sections. The design of the SCBF was determined by the strength limits of the braces and the relative stiffness limits of the frame members. The braces were designed such that they were likely to yield before the frame members (i.e. the braces sustained higher demand-capacity ratios than the corresponding frame members on each level). The majority of the braces reached demand-capacity ratios of over 0.70, and many of the frame members also surpassed ratios of 0.70. Therefore, the structural sections that were selected closely met their design criteria, and the fixed-base structure was efficiently designed.

4.4.1 Design of Fixed-Base Structure – Modal

This section contains the final modal information used to design the fixed-base structure via modal response spectrum analysis. The modes and cumulative mass participation ratios of the designed fixed-base structure are shown in Table 5 below.

Table 5: Fixed-Base Modes (ETABS Modal Analysis)

Source: self-produced

Fixed-Base Modes (ETABS Modal Analysis)

Mode	Period [Sec.]	Governing Direction	Cumulative Mass Participation [%]		
			X	Y	RZ
1	3.224	X	79.93	-	-
2	2.723	Y	-	67.56	-
3	2.648	RZ	-	-	72.23
4	1.053	X-2	89.97	-	-
5	0.794	RZ-2	-	-	88.73
6	0.730	Y-2	-	88.11	-
7	0.601	X-3	93.76	-	-
8	0.428	RZ-3	-	-	94.12
9	0.405	X-4	95.83	-	-
10	0.378	Y-3	-	94.35	-

X = U1 (SMF); Y = U2 (SCBF); RZ = Torsional

Note from the 10 modes shown above that the minimum requirement of 90 percent cumulative mass participation was reached in each modal direction (x-translational, y-translational, and z-torsional). Figure EE on page 52 shows the mode shapes of the fixed-base structure. As expected, the first two modes were translational in the x and y directions, and the third mode was torsional about the z-direction.

Figure EE: Fixed-Base Mode Shapes (ETABS Modal Analysis)

Source: ETABS 2011, self-produced

4.4.2 Design of Fixed-Base Structure – Drifts and Forces

Table 6 below shows the drifts of the fixed-base structure resulting from modal response spectrum analysis. Note that the design drift in the x-direction (U1 – SMF) was 0.0148, which closely met the design drift limit of 0.0150. The proximity of the values was due to the design of the SMF being governed by the drift limit instead of member force limits, which is common when designing moment frames due to their greater flexibility compared to braced frames. The design drift in the y-direction (U2 – SCBF) was 0.0120. The design of the SCBF was governed by strength instead of drift limits, so the drifts easily met the design drift limit.

Table 6: Fixed-Base Drifts (ETABS)

Source: self-produced

Fixed-Base Drifts (ETABS)

Story	X (U1 - SMF)	Y (U2 - SCBF)
12	0.0050	0.0112
11	0.0073	0.0116
10	0.0090	0.0120 ←
9	0.0104	0.0110
8	0.0113	0.0110
7	0.0124	0.0097
6	0.0133	0.0096
5	0.0141	0.0083
4	0.0144	0.0074
3	0.0148 ←	0.0058
2	0.0141	0.0057
1	0.0087	0.0037

4.5 Design of the Base-Isolated Office Building

The first step of designing the base-isolated office building was to design the base isolation system. The isolation system was designed to meet a target base-isolated effective period and effective viscous damping ratio for a given design displacement. Then, the maximum displacement of the isolation bearings was calculated to determine the moat size (the gap size between the building façade and concrete retaining wall at the base of the building). Lastly, the superstructure was designed to meet the required seismic performance criteria.

4.5.1 Design of the Isolation System – Bearings

As described in section “2.1 Fundamentals of Base Isolation,” the design goal of the base isolation system was to reduce the spectral accelerations and forces incurred in the superstructure. This goal was accomplished by both increasing the effective period of the structure via base isolation and damping seismic energy via friction caused by the friction pendulum bearings. In order to increase the effective period, the isolation system was designed to be flexible laterally. This flexibility was achieved in the friction pendulum bearings primarily by using concave plates with large radii of curvature.

The values of the coefficients of friction can vary with the production of each bearing as well as the velocity of the bearing, which in turn affects the effective stiffness of the bearing. To account for these differences, upper-bound and lower-bound properties are typically determined through cyclical testing of each bearing at given displacements. The upper-bound properties (with higher coefficients of friction and

effective stiffness values) are used to conservatively determine the design lateral forces incurred within the isolation system and superstructure at a DBE level event. The lower-bound properties (with lower coefficients of friction and effective stiffness values) are used to conservatively determine the maximum bearing displacements at an MCE level event. The isolation bearing design procedure used for this study involved an iterative process, described below:

1. Place an isolation bearing under each column and calculate gravity loads.
2. Choose a target viscous damping ratio (β_D) and effective period (T_D) for an assumed design displacement (D_D) and a given spectral acceleration value at a period of 1 second (S_{D1}). Note that T_D is recommended to be taken as 3 times the fixed-base period (T_o)
3. Choose initial isolation bearing properties, including
 - Effective radii of curvature of the concave spherical plates (R_{eff})
 - Upper-bound friction coefficients (μ)
4. Choose a stiff variation of the isolation bearing to be placed at the perimeter columns, with upper-bound friction coefficient values roughly 1/3 larger than the flexible/inner bearing values. This variation is done to reduce the influence of the torsional base-isolated modes.
5. Calculate and match the isolation bearing properties to the target isolation system properties, reiterating as needed

By following the procedure outlined above, the DBE upper-bound bearings were designed according to their target design criteria. The DBE target design criteria included a viscous damping ratio of 22% and an effective period of 4.50 seconds. The spectral acceleration value at a period of 1 second (S_{D1}) was given as 0.446 g. After a trial and error process, it was found that using bearings with larger radii of curvature and smaller coefficients of friction led to isolation system properties that approached the target values. Therefore, the inner/flexible upper-bound (DBE) bearing was designed with a coefficient of friction of 0.02, while the outer/stiff upper-bound (DBE) bearing was designed with a coefficient of friction of 0.03. These were the lowest practical upper-bound friction coefficient values that simultaneously differed by roughly 1/3, as suggested. The outer concave plates were designed with a radius of curvature (R) of 156 inches, which was the second largest size available according to EPS. The largest available plate size, with a radius of 244 inches, was deemed impractical for this project due to its relatively oversized footprint and higher cost (EPS 2003).

The final iterations of the upper-bound isolation system properties led to a design with a DBE level bearing displacement (D_D) of 13.46 inches. Through these trial bearing properties, it was found that reducing the friction coefficients had a larger impact on improving the seismic performance of the isolated structure when compared to increasing the radius of curvature of the outer spherical concave plates. However, both techniques led to an improvement in seismic performance. The greater effectiveness of lowering the friction coefficients was attributed to the structure being tall and flexible. As illustrated

in section “2.1 Fundamentals of Base Isolation,” increasing the effective period of a structure via base isolation (by increasing the radius of curvature of the plates) has diminishing returns as the period of the original structure increases. The final upper-bound bearing properties are shown in Figures FF and GG on page 58, and Tables 7 and 8 on page 59.¹⁵

¹⁵ Refer to Appendix I on page 187 for documentation of isolation system and bearing calculations

Figure FF: Force-Displacement of DBE (Upper-Bound) Inner/Flexible Bearing
Source: self-produced

Figure GG: Force-Displacement of DBE (Upper-Bound) Outer/Stiff Bearing
Source: self-produced

Table 7: DBE (Upper Bound) Isolation Bearing Properties

Source: self-produced

Isolation Bearing Properties		
	DBE (Upper Bound)	
	Flexible/Inner	Stiff/Outer
# Bearings =	12	20
$\mu_2 = \mu_3 =$	0.01	0.01
$\mu_1 = \mu_4 =$	0.02	0.03
R_{eff2} (in) =	33	33
R_{eff1} (in) =	303	303
K_{eff}/W (in ⁻¹) =	0.00471	0.00537
D_D (in) =	13.46	13.46
$F_D/W =$	0.0633	0.0722
D_V (in) =	0.33	0.66
β_{eff} (%) =	18.54	23.31

Table 8: DBE (Upper Bound) Isolation System Properties

Source: self-produced

Isolation System Properties	
DBE (Upper Bound)	
S_{D1} (g) =	0.466
D_D (in) =	13.46
ΣF_D (kips) =	1920
ΣK_D (kips/in) =	142.6
T_D (sec) =	4.47
ΣE_D (kip-in) =	35060
$\Sigma \beta_D$ (%) =	21.59
$B_D =$	1.515

Figure HH and Table 9 below illustrate the geometry of the Triple Friction Pendulum (TFP) bearings used for this study.

Figure HH: Triple Friction Pendulum Diagram

Source: Zekioglu et al. 2009

Table 9: Geometry of Triple Friction Pendulum Bearing

Source: self-produced

Geometry of the Triple-Friction Pendulum Bearing (in)						
$R_1 =$	156	$d_1 =$	28	$h_1 =$	4.5	$L =$ 72
$R_2 =$	20	$d_2 =$	3	$h_2 =$	3.5	$H =$ 13
$R_3 =$	20	$d_3 =$	3	$h_3 =$	3.5	
$R_4 =$	156	$d_4 =$	28	$h_4 =$	4.5	

Note that the same geometry was used for all the bearings. From a manufacturing standpoint, the only difference between the bearings was their friction coefficient values. Figures II and JJ on page 61, and Tables 10 and 11 on page 62 summarize the lower-bound (MCE) design properties of the TFP bearings.¹⁶

¹⁶ Refer to Appendix I on page 187 for documentation of isolation system and bearing calculations

Figure II: Force-Displacement of MCE (Lower-Bound) Inner/Flexible Bearing
Source: self-produced

Figure JJ: Force-Displacement of MCE (Lower-Bound) Outer/Stiff Bearing
Source: self-produced

Table 10: MCE (Lower Bound) Isolation Bearing Properties

Source: self-produced

Isolation Bearing Properties		
	MCE (Lower Bound)	
	Flexible/Inner	Stiff/Outer
# Bearings =	12	20
$\mu_2 = \mu_3 =$	0.01	0.01
$\mu_1 = \mu_4 =$	0.0164	0.0245
R_{eff2} (in) =	33	33
R_{eff1} (in) =	303	303
K_{eff}/W (in ⁻¹) =	0.00387	0.00413
D_M (in) =	27.58	27.58
$F_M/W =$	0.1067	0.1139
D_y (in) =	0.21	0.48
β_{eff} (%) =	9.30	12.58

Table 11: MCE (Lower Bound) Isolation System Properties

Source: self-produced

Isolation System Properties	
MCE (Lower Bound)	
S_{M1} (g) =	0.699
D_M (in) =	27.58
ΣF_M (kips) =	3103
ΣK_M (kips/in) =	112.5
T_M (sec) =	5.04
ΣE_M (kip-in) =	61000
$\Sigma \beta_M$ (%) =	11.34
$B_M =$	1.25

The lower-bound (MCE) bearing properties were determined in a similar manner as that of the upper-bound (DBE) properties.¹⁷ A plus or minus 10% variation from the nominal friction coefficient values to the upper-bound and lower-bound properties was

¹⁷ Refer to Appendix I on page 187 for documentation of isolation system and bearing calculations

assumed. Since the nominal properties were not needed for design, the lower-bound properties were found directly from the upper-bound properties by applying a ratio of 0.9/1.1, or about 0.818 to the coefficient of friction values. This procedure resulted in flexible/inner and stiff/outer lower-bound friction coefficient values of 0.0164 and 0.0245, respectively. The lower-bound target effective period and viscous damping ratio equaled 5.0 seconds and 11%, respectively. The final iterations of the lower-bound isolation system properties led to a design with an MCE level bearing displacement (D_M) of 27.58 inches.

As explained in section “2.1 Fundamentals of Base Isolation,” TFP bearings are versatile in that they may be designed to have different lateral stiffness values at DBE and MCE events through the use of sequentially activated outer plates with differing properties. Although models have been developed (parallel and series models) to capture the tri-linear behavior of the TFP bearings in ETABS, at the time of this writing there are no known tri-linear TFP models for PERFORM-3D (Sarlis and Constantinou 2010). Due to this fact, a bilinear model was used for this study (i.e. the same properties were used for both outer concave plates in each TFP bearing). However, this caused the bearings in this study to have a less than optimal design for MCE events. Ideally, each bearing’s outer plates should be designed with differing properties such that its lateral stiffness increases when an MCE event is reached. The seismic performance results from this study would therefore be improved if tri-linear TFP bearing models for PERFORM-3D were available and implemented.

4.5.2 Design of the Isolation System – Moat

The moat size of the isolation system was determined by the maximum bearing displacement expected for an MCE event, using the lower-bound properties of the isolation bearings. Conservatively, the maximum of the hand-calculated value and the average value of 7 time histories (from PERFORM-3D) was used, while taking into account the effects of torsion. The results are shown in Table 12 below. The moat size was calculated to be 36 inches, or 3 feet.¹⁸

Table 12: MCE (Lower Bound) Maximum Bearing Displacements

Source: self-produced

	D_{M-max} (in)	D_{TM} (in)
Hand Calculations	27.58	30.34
Average of 7 Time Histories	31.70	34.87

$$D_{TM} = 1.1 * D_{M-max} \quad (17.5.3.5, ASCE 7-10)$$

- $D_{TM} = 34.87$ inches governs
- Round moat size up to **36 inches = 3 feet**

The maximum bearing displacements from each of the 7 MCE time histories were obtained by the bearings' radial displacement time history plots in PERFORM-3D. The maximum radial displacement out of all the bearings was used for each ground motion record. An illustration of one of the bearing's MCE time history plots for radial displacement is shown in Figure KK on page 65.

¹⁸ Refer to Appendix I on page 187 for documentation of isolation system, bearing, and moat calculations

Figure KK: Radial Displacement Time History Plot of TFP Bearing (MCE-183)
Source: PERFORM-3D 2011

The maximum radial displacement for the bearing in Figure KK was 33.4 inches, which was found by using the Pythagorean theorem and the plotted Cartesian coordinates. The maximum bearing displacements for the illustrated time history (MCE-183) and all the other time histories were within a similar range to the average maximum displacement of 31.7 inches. The bearings with the largest displacements for each ground motion tended to be the bearings located at the outer corners of the structure.

None of the maximum bearing displacements from the time histories surpassed the bearing radius or moat size of 3 feet, which indicated that there was no seismic damage inflicted on the isolation system. An example of a hysteresis plot of the same bearing illustrated in Figure KK is shown in Figure LL below.

Figure LL: SMF-Direction Hysteresis Plot of TFP Bearing (MCE-183)

Source: PERFORM-3D 2011

The hysteretic behavior illustrated in Figure LL was typical for most of the bearings subjected to MCE level ground motions. The hysteretic behavior for the

bearings subjected to DBE level ground motions was also similar, but the bearings had smaller maximum displacements due to the smaller intensity of the seismic loading. The hysteresis plots showed that the bearings behaved in a bilinear manner as expected, with slight variations in lateral stiffness and force due to the varying vertical forces acting on each bearing throughout the ground motion records. As explained earlier, the lateral stiffness of TFP bearings depends on the friction coefficient and the vertical force acting on the bearing. The conformity of the hysteretic behavior also illustrated that the bearings were properly absorbing energy and therefore damping the seismic response as expected. Once the hysteresis plots and time history plots were shown to be in agreement and they indicated that the bearings behaved as expected, the design of the isolation system, including the moat, was complete.

4.5.3 Design of the Base-Isolated Structure

The base-isolated structure was also designed using modal response spectrum analysis and the 'Steel Design/Check of Structure' tool in ETABS. The analysis resulted in a final lateral system with the structural sections shown in Figures MM and NN below.

Figure MM: Sections of Designed Base-Isolated SMF

Source: ETABS 2011

Figure NN: Sections of Designed Base-Isolated SCBF
Source: ETABS 2011

The special moment frames (SMF) resulted in the same structural members as those in the fixed-base structure, with W27x columns and W30x beams. Again, for constructability, the column splices were placed every 2 stories and the same sections were used for the SMF columns on each floor. Similar color-coded demand-capacity ratios (ranging in severity from less than 0.50 in blue to more than 0.95 in red) of the structural members were reached as those in the fixed-base structure. The same structural members were used for the SMF of both structures, partly because the designs of the SMFs were determined by drift limits and relative stiffness limits for both cases. The major differences between the two structures was that a stricter design drift limit was used for the base-isolated structure (ranging between 0.007-0.010 in lieu of the fixed-base design drift limit of 0.0015), and the base-isolated superstructure incurred smaller story drifts than the fixed-base structure at the design basis earthquake (DBE) force level. The smaller interstory drifts and stricter drift limits of the base-isolated structure led to similar demand-capacity ratios as the fixed-base structure when using the same structural members. Similar to the fixed-base structure, the outer columns of the base-isolated structure also easily passed the relative stiffness requirements, while the inner W27x columns were governed by the W30x beams attached on each side. Again, the inner columns sustained demand-capacity ratios of over 0.90 and in some cases 0.95, as shown by the color-coding in Figure MM.

The special concentrically braced frames (SCBF) also resulted in the same structural members as those in the fixed-base structure, with W18x columns, W16x beams, and braces including HSS8x8x and HSS7x7x sections. The design of the SCBF was again determined by the strength limits of the braces and the relative stiffness limits of the frame members. Similar demand-capacity ratios were achieved for both structures due to the use of similar design base shears. The base-isolated superstructure incurred a smaller nominal base shear at DBE than the fixed-base structure, however the design base shears were scaled down to comparable values. Similar design base shears were achieved in the SCBF direction due to the isolated response modification coefficient ($R_I = 2$) being smaller than the fixed-base response modification coefficient ($R = 6$). Similar to the fixed-base structure, the majority of the braces were designed such that they were likely to yield before the frame members (i.e. the braces sustained higher demand-capacity ratios than the corresponding frame members on each level). The majority of the braces also reached demand-capacity ratios of over 0.70, and many of the frame members surpassed ratios of 0.70 as well. Therefore, the structural sections that were selected closely met their design criteria, and the base-isolated structure was efficiently designed.

Since the same structural members were used for both the fixed-base and base-isolated structures, this thesis also encompasses the seismic performance enhancements that would result from retrofitting the fixed-base 12-story steel office building with the same base isolation system in this study.

4.5.4 Design of the Base-Isolated Structure – Modal

This section contains the final modal information used to design the base-isolated structure via modal response spectrum analysis. The modes and cumulative mass participation ratios of the designed base-isolated structure are shown in Table 13 below.

Table 13: Base-Isolated Modes (ETABS Modal Analysis)

Source: self-produced

Base-Isolated Modes (ETABS Modal Analysis)					
Mode	Period [Sec.]	Governing Direction	Cumulative Mass Participation [%]		
			X	Y	RZ
1	5.328	X	97.04	-	-
2	5.024	Y	-	94.84	-
3	4.562	RZ	-	-	97.04
4	1.577	Y-2	-	99.95	-
5	1.565	X-2	99.74	-	-
6	1.439	RZ-2	-	-	99.90
7	0.818	X-3	99.96	-	-
8	0.594	RZ-3	-	-	99.98
9	0.535	Y-3	-	99.99	-
10	0.526	X-4	99.99	-	-

X = U1 (SMF); Y = U2 (SCBF); RZ = Torsional

Note from the 10 modes shown above that the minimum requirement of 90 percent cumulative mass participation was reached in each modal direction (x-translational, y-translational, and z-torsional), just as it was in the fixed-base structure. Figure OO on page 73 shows the mode shapes of the base-isolated structure. Again, as expected, the first two modes were translational in the x and y directions, and the third mode was torsional about the z-direction.

Figure OO: Base-Isolated Mode Shapes (ETABS Modal Analysis)

Source: ETABS 2011, self-produced

4.5.5 Design of the Base-Isolated Structure – Drifts and Forces

Table 14 below shows the drifts of the base-isolated structure, resulting from modal response spectrum analysis. Note that the design drift in the x-direction (U1 – SMF) was 0.0089, which met the isolated design drift limit range of 0.0070-0.0100. The proximity of the values was due to the design of the SMF being governed by the drift limit instead of member force limits, which is common when designing moment frames due to their greater flexibility compared to braced frames. The design drift in the y-direction (U2 – SCBF) was 0.0069. The design of the SCBF was governed by strength instead of drift limits, so the design drift easily met the design drift limit.

Table 14: Base-Isolated Drifts (ETABS)

Source: self-produced

Base-Isolated Drifts (ETABS)

Story	X (U1 - SMF)	Y (U2 - SCBF)
12	0.0019	0.0064
11	0.0030	0.0066
10	0.0041	0.0069 ←
9	0.0051	0.0067
8	0.0059	0.0069
7	0.0067	0.0063
6	0.0075	0.0064
5	0.0082	0.0057
4	0.0085	0.0051
3	0.0089 ←	0.0039
2	0.0086	0.0038
1	0.0058	0.0022

4.6 Verification Checks of the Design Models

The following checks were performed to verify that the structures were properly modeled in ETABS for the design phase:¹⁹

- Isolation Bearing Overturning/Uplift Check²⁰

No tension/uplift forces occurred within any of the bearings in the base-isolated model during any of the load combinations, including a time history of the DBE-182 (El Centro) ground motion.

- Maximum Displacement Check (Restrainer at D_{TM})

All bearings in the base-isolated model were checked for maximum bearing displacement for all the load combinations, and it was found that none of the bearings reached the displacement limit of $D_{TM} = 34.87$ inches.

- Period/Drift Hand Calculation Check²¹

The fundamental period of the fixed structure ($T = 3.22$ sec.) was verified with the following equation (Eq. 15.4-6, ASCE 7-05):

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^n w_i d_i^2}{g \sum_{i=1}^n f_i d_i^2}} = 3.21 \text{ sec.} \approx 3.22 \text{ sec.} \quad \text{Eq. 10}$$

Where:

- T = Period [sec.]
- w_i = Floor weight [kips]
- d_i = Floor elastic displacement associated with f_i [in.]
- g = Gravitational acceleration [in/sec²]
- f_i = Floor lateral force [kips]

¹⁹ Refer to section 5.5 on page 105 for checks verifying similarity between the ETABS and PERFORM-3D models

²⁰ Refer to “Figure HHH: Isolator Uplift Check (DBE-182: ETABS)” on page 109

²¹ Refer to Appendix O on page 204 for documentation of period check calculations

5.0 ANALYSIS PHASE

In the analysis phase, the inter-story drifts and floor accelerations of both the fixed-base and base-isolated office buildings were found at design basis earthquake (DBE) and maximum considered earthquake (MCE) levels via nonlinear time history analyses. Also, the structural performance levels were analyzed by tracking plastic hinge rotations occurring in the structural members, according to ASCE 41-06 standards. PERFORM-3D was used to model the structures and perform the analyses.

5.1 Analysis Phase Procedure

An overview of the analysis phase procedure is shown below:

1. Select and scale 7 ground motions to DBE and MCE levels (via PEER website)
2. Model structures, loading, and analysis criteria
 - Input hinge properties according to ASCE 41-06 standards
 - Input isolation bearings
 - Input time history functions
3. Perform nonlinear time history analyses
 - Obtain floor accelerations and interstory drifts
 - Obtain plastic hinge rotations and structural performance levels
4. Verify congruence between PERFORM-3D and ETABS models

5.2 Nonlinear Time History Analysis

In order to account for the variation of the office building's response throughout the duration of each earthquake ground motion, a time history analysis was required. A time history analysis involves simulating earthquake ground motion accelerations that vary over time, and tracking the structure's response to these ground motions. The seismic response parameters that were considered for this study were

- floor accelerations
- interstory drifts
- plastic hinge rotations
- isolation bearing displacements and forces

The time history analyses performed in this study were nonlinear due to the curvilinear behavior of the isolation bearings and the plastic hinge rotations of the structural members. The nonlinear behavior of the isolation bearings (force vs. displacement) was tracked over time via hysteresis loops. The area inside the hysteresis loops was equal to the amount of energy dissipated by the bearings.

5.2.1 Nonlinear Time History Analysis – Procedure (ASCE 7-10)

The procedure for conducting a nonlinear time history analysis involves selecting a minimum of 3 ground motions and scaling those ground motions to a given response spectrum.. If a minimum of 7 ground motion records are selected (as was the case for this study) the average of the response values from all the ground motions may be used in lieu of the maximum value.

5.2.2 Nonlinear Time History Analysis – Scaling of Ground Motions (PEER)

In order to perform the nonlinear time history analyses, 7 ground motions were selected and scaled to both the DBE and MCE response spectrum levels for a San Diego location. The ground motions and their scale factors are given in Table 15 below. These ground motions were selected from a previous structural engineering project near the same San Diego location used for this study. Therefore, the soil conditions were assumed to be the same, and the selected ground motions were deemed appropriate for this study.

Table 15: Scale Factors for EQ Ground Motions
Source: self-produced

Event	Year	Station	NGA#	DBE ScaleF	MCE ScaleF
Imperial Valley-06	1979	El Centro Array #7	182	0.7014	1.0520
Imperial Valley-06	1979	El Centro Array #8	183	1.0541	1.5812
Superstition Hills-02	1987	El Centro Imp. Co. Cent	721	1.1627	1.7440
Landers	1992	Yermo Fire Station	900	1.3041	1.9562
Northridge-01	1994	Sylmar - Olive View Med FF	1086	0.5775	0.8662
Kocaeli- Turkey	1999	Duzce	1158	0.8435	1.2653
Duzce- Turkey	1999	Duzce	1605	0.7525	1.1288

In order to scale the ground motions, the beta version of the PEER Ground Motion Database tool was used. This online tool linearly scales the acceleration amplitudes of fault-normal and fault-parallel ground motion records to a defined response spectrum over a defined period range, using the geometric mean of the records. The weight function used in this study uniformly scaled the amplitudes of the records between a period range between 5.3 – 1.0 seconds. This period range included the primary modes of the base-isolated and fixed-base structures (ranging between 5.3 – 2.4 seconds) as well

as periods as low as 1 second to account for higher modal effects. As can be seen in Figure PP below, the geometric mean (red line) of the ground motions closely correlated to the DBE target spectrum (black line) over the defined period range (green lines). The same weight function was used to scale ground motions to the MCE target spectrum.

Figure PP: Weight Function Scaling of Ground Motions to DBE Spectrum

Source: PEER 2010

5.3 Nonlinear Hinge Properties (ASCE 41-06)

This section of the thesis describes the nonlinear hinge properties used in the study.

5.3.1 Plastic Deformation of Structural Components

When structural members or connections exceed their material yield limit, they experience plastic behavior. Plastic behavior is classified as the state between yielding and failure (crushing, rupture, etc.) where, as a stress is applied to the material and it deforms, permanent deformation of the material will remain once the stress is released. Figure QQ shown below demonstrates a typical stress-strain curve for steel under tensile loading.

Figure QQ: Typical Steel Stress vs. Strain Curve

Source: Sandt 1999

The flat portion of the graph (zone 2, between points A and B) is the material's yield zone, where no additional stress is required for the material to experience further deformation. After the steel yields, it reaches its plastic limit (point B) where the material begins to regain stiffness and requires additional stress to deform further. This portion of the diagram (zone 3), is the strain-hardening zone. At the ultimate strength of the material (point C), less stress is required to deform the material, until failure is

reached (at point D) due to rupture. The region between these two points (zone 4) is the necking zone.

When designing a structural member, the nominal (lower bound) yield strength is normally used. Conversely, when analyzing the seismic performance of a member (for yielding) the expected (upper bound) strength of the member is used. The expected and yield stresses are related by expected/yield strength ratios. The ratios used for this study are shown in Table 16 below.

Table 16: Required Strength Ratios
Source: self-produced

Seismic Force-Resisting System	(Table I-6-1, AISC 327-05)	
	Expected/Minimum Yield Stress Ratio R_y	Expected/Minimum Tensile Strength Ratio R_t
Steel special moment moment frames (SMF)	1.1	1.1
Steel special concetrically braced frames (SCBF)	1.4	1.3

Once the steel has yielded, the relationship of the material's stress-strain curve becomes curvilinear. Accurately capturing this curvilinear behavior requires nonlinear analysis (as opposed to approximate linear-elastic methods), which is one reason why this study involved nonlinear time history analyses. In nonlinear structural analysis, one method of determining the stress in a connection or part of a structural steel member that has yielded is to assume that a plastic hinge has formed. This is the method that was chosen for this study. For plastic hinges, all of the yielding is assumed to occur locally at a single point along the length of each yielding zone of the structural member.

The degree of plastic deformation that a structural member is able to withstand before failure is known as ductility. Structural performance levels are established for given degrees of strain/rotation of structural members, based on the expected ductility of the members. The specific criteria used to gauge the structural performance levels of the structural members in this study are discussed in the next section.

5.3.2 Guidelines for Analyzing Plastic Deformation

The code ASCE-41, entitled *Seismic Rehabilitation of Existing Buildings*, categorizes structural performance levels by degrees of plastic deformation. The listed categories that were considered for this thesis, in increasing deformation/damage were

- Immediate Occupancy (S-1): Light damage, no permanent drifts
- Life Safety (S-3): Moderate damage, some permanent drifts
- Collapse Prevention (S-5): Severe damage, larger permanent drifts

These performance levels are illustrated in the steel “back-bone curve” (force vs. deformation plot) in Figure RR below.

Figure RR: Deformation Acceptance Criteria

Source: ASCE 41-06

Chapter 5 of ASCE 41 describes the criteria for analyzing steel components and elements. These structural strength criteria values form the “back-bone curve” of the material, as shown in Figure SS below. Note from Tables 17 and 18 below that the criteria are determined by the type of component, type of loading action, and member dimensions (to account for slenderness).

Figure SS: Force-Deformation Relation for Steel Elements or Components

Source: ASCE 41-06

Table 17: Nonlinear Modeling Parameters and Acceptance Criteria – Structural Steel

Source: ASCE 41-06

Component/Action	Modeling Parameters			Acceptance Criteria ¹⁴				
	Plastic Rotation Angle, Radians	Residual Strength Ratio		Plastic Rotation Angle, Radians				
				Primary			Secondary	
				<i>a</i>	<i>b</i>	<i>c</i>	IO	LS
Beams—Flexure								
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_{ye}}}$	$9q_y$	$11q_y$	0.6	$1q_y$	$6q_y$	$8q_y$	$9q_y$	$11q_y$
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{640}{\sqrt{F_{ye}}}$	$4q_y$	$6q_y$	0.2	$0.25q_y$	$2q_y$	$3q_y$	$3q_y$	$4q_y$
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lower resulting value shall be used							
Columns—Flexure ^{2,7}								
For $P/P_{cl} < 0.2$								
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{300}{\sqrt{F_{ye}}}$	$9q_y$	$11q_y$	0.6	$1q_y$	$6q_y$	$8q_y$	$9q_y$	$11q_y$
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{460}{\sqrt{F_{ye}}}$	$4q_y$	$6q_y$	0.2	$0.25q_y$	$2q_y$	$3q_y$	$3q_y$	$4q_y$
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lower resulting value shall be used							

Table 18: Nonlinear Modeling Parameters and Acceptance Criteria – Steel Braces
Source: ASCE 41-06

Component/Action	Modeling Parameters			Acceptance Criteria ⁶				
	Plastic Deformation		Residual Strength Ratio	Plastic Deformation				
				Primary		Secondary		
	a	b		c	IO	LS	CP	LS
Braces in Compression (except EBF braces) ^{1,2}								
a. Slender								
$\frac{Kl}{r} \geq 4.2\sqrt{E/F_y}$								
1. W, I, 2L In-Plane ³ , 2C In-Plane ³	0.5Δ _c	10Δ _c	0.3	0.25Δ _c	6Δ _c	8Δ _c	8Δ _c	10Δ _c
2. 2L Out-of-Plane ³ , 2C Out-of-Plane ³	0.5Δ _c	9Δ _c	0.3	0.25Δ _c	5Δ _c	7Δ _c	7Δ _c	9Δ _c
3. HSS, Pipes, Tubes	0.5Δ _c	9Δ _c	0.3	0.25Δ _c	5Δ _c	7Δ _c	7Δ _c	9Δ _c
b. Stocky ⁴								
$\frac{Kl}{r} \leq 2.1\sqrt{E/F_y}$								
1. W, I, 2L In-Plane ³ , 2C In-Plane ³	1Δ _c	8Δ _c	0.5	0.25Δ _c	5Δ _c	7Δ _c	7Δ _c	8Δ _c
2. 2L Out-of-Plane ³ , 2C Out-of-Plane ³	1Δ _c	7Δ _c	0.5	0.25Δ _c	4Δ _c	6Δ _c	6Δ _c	7Δ _c
3. HSS, Pipes, Tubes	1Δ _c	7Δ _c	0.5	0.25Δ _c	4Δ _c	6Δ _c	6Δ _c	7Δ _c
c. Intermediate								
Linear interpolation between the values for slender and stocky braces (after application of all applicable modifiers) shall be used.								
Braces in Tension (except EBF braces) ⁵								
11Δ _T	14Δ _T	0.8	0.25Δ _T	7Δ _T	9Δ _T	11Δ _T	13Δ _T	
Beams, Columns in Tension (except EBF beams, columns) ⁵								
5Δ _T	7Δ _T	1.0	0.25Δ _T	3Δ _T	5Δ _T	6Δ _T	7Δ _T	

As can be seen by Tables 17 and 18 above, and Figure TT on page 85, scale factors of yield rotations are used to define structural performance levels for flexural members (i.e. moment frames), while scale factors of yield displacements are used to define structural performance levels for axial members (i.e. braces). The majority of the structural members in this study were characterized by condition “a” in the Tables 17 and 18 (i.e. non-slender).²² Note from Figure TT that the equations defining the yield

²² Refer to Appendices J-N on pages 197-203 for documentation of hinge property calculations

rotations and strengths of the beams and columns use the expected yield stress, F_{ye} . As discussed earlier, the expected yield stress is an upper-bound, more realistic strength for what the structural components will reach before yielding. This more realistic strength is important for properly analyzing the structural members for ductility when subjected to seismic demands, as was done in this study.

$\text{Beams: } \theta_y = \frac{ZF_{ye}l_b}{6EI_b} \quad (\text{Eq. 5-1})$
$\text{Columns: } \theta_y = \frac{ZF_{ye}l_c}{6EI_c} \left(1 - \frac{P}{P_{ye}} \right) \quad (\text{Eq. 5-2})$
<p>Q_{CE} is the component expected strength. For flexural actions of beams and columns, Q_{CE} refers to the plastic moment capacity, which shall be calculated using Eqs. 5-3 and 5-4: k</p>
$\text{Beams: } Q_{CE} = M_{CE} = ZF_{ye} \quad (\text{Eq. 5-3})$
$\text{Columns: } Q_{CE} = M_{CE} = 1.18ZF_{ye} \left(1 - \frac{P}{P_{ye}} \right) \leq ZF_{ye} \quad (\text{Eq. 5-4})$

Figure TT: Force-Deformation Equations

Source: ASCE 41-06

5.4 PERFORM-3D

To model and analyze both of the office buildings using nonlinear time history analysis, a 3D digital model of each structure was created in PERFORM-3D.

PERFORM-3D is a building analysis software program developed by Computers & Structures, Inc. (CSI). This section of the study illustrates the modeling and analysis decisions (input) of the structural models in PERFORM-3D, including the structural member sections, locations, lengths, material properties, connectivity data, loads, mass, and design criteria. In particular, the input for plastic hinges, time history functions, and isolation bearings are emphasized.

5.4.1 PERFORM-3D – Model Structural Input

In order to model the fixed-base and base-isolated buildings in PERFORM-3D, first their structural data was entered into the program. Rigid horizontal diaphragms were assumed for all typical floors in both models. All nodes on each floor were constrained in both lateral directions (x and y) as well as torsionally around the vertical axis (z). Also, point masses were assigned and constrained at the center of mass location on each floor.²³ The diaphragms and masses for typical floors and the support conditions at the base were all the same as those input into the ETABS models. Note that in the base-isolated model, the isolation platform level was constrained to be rotationally rigid in all directions. This constraint was discussed in more detail in section “4.3.2 ETABS – Model Isolation System Input.”

²³ Refer to Appendices G and H on pages 185 and 186 for documentation of point mass calculations

Once the forms of the structures were modeled, the properties of the structural members were defined. The default section properties existing in PERFORM-3D were used for the structural members in this study. The types of components that used section properties in the models were inelastic components (including plastic hinges) and axial strength sections. The axial strength sections included the hollow structural section (HSS) braces in the special concentrically braced frames (SCBF).

In PERFORM-3D, the stiffness of the end-zones of each member is defined as a multiple of the member's stiffness. After attempting several trial factors, it was found that a stiffness ratio of 1.5 caused the modal behavior and drifts of the PERFORM-3D models to correlate closely with those of the ETABS models, which instead use a rigid end zone factor (equal to 0.5 for this study). At each column-beam intersection, the smaller members' dimensions were used for the end zone lengths in PERFORM-3D for the same reason as stated above. Note that the rigid end zone factor in ETABS determines the ratio of the rigid end zone length (50% in this case) that is considered fully rigid, and the remaining length of the rigid end zone is equal to the stiffness of the rest of the structural member (CSI 2005). PERFORM-3D on the other hand uses the full length of the defined end zones and multiplies the stiffness by a factor as described above. The methods in each program determine the stiffness of column-beam connections (i.e. end zones), which is particularly important for properly modeling the stiffness of moment frames and determining accurate drift values.

Documentation of typical plastic hinge property input for PERFORM-3D is shown in Figures UU and VV below, illustrating the hinge properties of a W30x116 beam (which was used for the special moment frames).

Section and Dimensions	Basic F-D Relationship	Strength Loss
<p>F = hinge moment. D = hinge rotation.</p> <div style="display: flex; align-items: center;"> <div style="margin-left: 20px;"> <p>Name: W30x116 Hinge</p> <div style="display: flex;"> <div style="border: 1px solid gray; padding: 5px; margin-right: 10px;"> <p>Positive Actions</p> <p>FY 19956</p> <p>FU 20790</p> </div> <div style="border: 1px solid gray; padding: 5px;"> <p>Negative Actions</p> <p>FY</p> <p>FU</p> </div> </div> <div style="display: flex;"> <div style="border: 1px solid gray; padding: 5px; margin-right: 10px;"> <p>Positive Deformations</p> <p>DU 0.07852</p> <p>DX 0.09597</p> </div> <div style="border: 1px solid gray; padding: 5px;"> <p>Negative Deformations</p> <p>DU</p> <p>DX</p> </div> </div> </div> </div>		
<p>Deformations = hinge rotations</p> <div style="display: flex;"> <div style="border: 1px solid gray; padding: 5px; margin-right: 10px;"> <p>Positive Deformations</p> <p>DL 0.07931</p> <p>DR 0.0809</p> <p>FR/FU 0.6</p> </div> <div style="border: 1px solid gray; padding: 5px; margin-right: 10px;"> <p>Negative Deformations</p> <p>DL</p> <p>DR</p> <p>FR/FU</p> </div> <div style="border: 1px solid gray; padding: 5px;"> <p>Strength Loss Interaction</p> <p>Interaction Factor Min = 0, Max = 1 0</p> <p>0 = no interaction. Strength loss in one direction has no effect on the strength in the other direction.</p> <p>1 = full interaction. Strength loss in one direction causes an equivalent loss in the other direction.</p> </div> </div> <div style="border: 1px solid gray; padding: 5px; margin-top: 10px;"> <p>Total Strength Loss at Point X</p> <p>☒ No ☐ Yes</p> <p>For the "Yes" option, if Point X is reached, in either the positive or negative direction, the strength and stiffness suddenly reduce to zero.</p> </div>		

Figure UU: Typical Beam Hinge Force-Displacement Input
Source: PERFORM-3D 2011

Figure VV: Typical Beam Hinge Property Deformation Capacity Input
Source: PERFORM-3D 2011

The yield and ultimate bending moments (F_Y and F_U) along with the rotations at ultimate (DU), failure (DX), and other points that form the backbone curve of the plastic hinge are defined in Figure UU above. The rotations at which strength loss begins and ends are defined by DL and DR, respectively. The ratio of residual strength (after strength loss) to ultimate strength is given by FR/F_U . Figure VV above illustrates the rotations at which Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) structural performance levels are defined. Note that all these levels (indicated by the red vertical lines on the graph) exist in successive order and before strength loss

occurs. All the values in these two figures were defined by ASCE 41-06, as explained earlier in section “5.3.2 Guidelines for Analyzing Plastic Deformation.”²⁴

Once the section properties, end zones, and plastic hinges were defined; they were combined into a single compound to form a structural member. The compound properties of a typical moment frame beam (which was a W30x116 in this case) is shown in Figure WW below, where component types are assigned and their lengths along the member are defined.

NAME W30X116 BEAM

COMPONENT LENGTHS ARE NOT TO SCALE

No.	Component Type	Component Name	Length	Propn
1	End Zone for a Beam or Column	Default End Zone	Auto	
2	Beam, Standard Steel Section	W30X116	15	
3	Moment Hinge, Rotation Type	W30X116 Hinge	0	
4	Beam, Standard Steel Section	W30X116		1
5	Moment Hinge, Rotation Type	W30X116 Hinge	0	
6	Beam, Standard Steel Section	W30X116	15	
7	End Zone for a Beam or Column	Default End Zone	Auto	

Figure WW: Typical Beam Compound Property Input

Source: PERFORM-3D 2011

While the beam hinge properties only needed to account for rotations and moments associated with major bending (around the local y-axis), the column plastic hinges took into account bending about both local axes, tension and compression deformations, and the axial forces' influence on the overall force-deformation relationship via yield surfaces (i.e. P-M plots).

²⁴ Refer to Appendices J-N on pages 197-203 for documentation of hinge strengths and deformations

The procedure for modeling the plastic hinges of the columns was similar to that for modeling the hinges of the beams, described above. Bending about axis 3 (i.e. about the local y-axis) was modeled as strong-axis bending. Units were in kip-in, and the deformations for bending were rotations (in radians) while the deformations for axial deformations were in inches. The suggested parameter values given in PERFORM-3D were used to define the shape of the yield surface. The structural performance levels (rotation values) for the column plastic hinges were modeled in a similar manner as was done for the beam hinges, except the column hinges also accounted for the weak-axis rotations via a scale factor. As was done for the beam hinges, plastic hinge values for the columns were determined by ASCE 41-06.

The column properties were modeled in a similar manner as was done for the beams. The only difference was that an end zone was placed on only the upper end of each column compound. This placement methodology was done to mimic the condition of the column-beam joints being located at the top of the beam cross sections, thereby allowing the full depth of the beams to be utilized for the end zones on the upper ends of the column elements. This method was efficient since the base columns had no end zones at their lower-ends (due to the lack of beam-column intersections), which therefore eliminated the need to use two different methods for modeling the column compound elements.

The plastic hinge properties for the braces were of the same type as those used for the columns. The compound element properties of the braces were different from those of

the other member types in that moment releases were placed at the ends of the braces in order to correctly model their primarily axial force-resisting behavior. Bending moment releases were also placed at the ends of the brace members in ETABS for the same reason as stated above. The braces were also not modeled with end zones since they had no beam-column connections.

Once the compound elements were created, they were assigned to the lines that constituted the form of the structural models. Part of this process involved assigning the orientation of the local axes of each element. The orientation of a typical column from one of the special moment frames (SMF), which had the depth of its member (axis 2) running along the direction of the moment frame lines (+ H1). Similarly, axis 2 of the braced-frame columns was oriented along the direction of the braced frame lines (+ H2). All beams were oriented with their depths running vertically (vertical up).

5.4.2 PERFORM-3D – Isolation System Input

Once the superstructure of the base-isolated structure was modeled in PERFORM-3D, the isolation system was then modeled. The first step for modeling the isolation system was to model the isolation platform, which was assigned a rigid diaphragm with all its nodes restrained in all 6 degrees of freedom, as shown in Figure XX below.

Figure XX: Isolation Platform Node Slaving Input

Source: PERFORM-3D 2011

The isolation platform nodes were slaved as full rigid links in all directions in order to model the rigid bending stiffness of the isolation platform. This bending stiffness was needed to support the large bending moments attributed to the isolation bearings, as discussed earlier in section “4.3.2 ETABS – Model Isolation System Input.” The mass assignments of the isolation platform were the same as those of the typical floors, just as they were in ETABS.

After the isolation platform was created, the isolation bearings were modeled. The same procedure was followed as was done in ETABS, in which a bearing was placed beneath every column, and the specific bearing properties were input for each unique gravity load. As explained earlier in section 4.3.2, two different bearing types were created (the outer/stiff bearing and the inner/flexible bearing) in order to reduce torsional mode effects. Figures YY and ZZ on pages 95 and 97 illustrate the typical modeling input for the two types of isolation bearings.²⁵ Note that the “Friction Pendulum” seismic isolator property was used in PERFORM-3D, which is the same as a single Friction Pendulum bearing. Since ETABS and PERFORM-3D don’t have Triple Friction Pendulum bearings, an effective bilinear model was used (as suggested by the manufacturer EPS) using the single Friction Pendulum bearings.

²⁵ Refer to Appendix I on page 187 for documentation of bearing types and property calculations

Figure YY: Inner/Flexible TFP Bearing Input

Source: PERFORM-3D 2011

Figure YY above shows the lateral property input for the inner/flexible bearing type “FP A” in PERFORM-3D. The units are in kip-inches. The same parameter values were entered as those entered in ETABS; however, no linear effective properties (such as effective stiffness and effective damping) were entered since PERFORM-3D only uses nonlinear lateral properties. KO is the elastic stiffness, DX is the deformation capacity,

and DS and KS are the boundary element displacement capacity and stiffness, respectively. A small amount of lateral stiffness was needed in the bearings for stability during gravity analysis, so the recommended lateral stiffness scale factor of 0.01 was used. As with ETABS, a single friction coefficient value was used in each bearing (i.e. the variability due to velocity was ignored) in order to promote congruence between the ETABS and PERFORM-3D models.

Figure ZZ on page 97 illustrates the lateral property input for one of the stiff/outer bearings, “FP B.” The major difference between the “stiff” and “flexible” bearing types was their friction coefficient values of 0.03 and 0.02, respectively. Note that all bearings used an effective radius of curvature of 303 inches. As explained earlier, this value is equal to the effective radius of curvature of the outer plates of the TFP bearing. The resulting stiffness of the effective radius of curvature of the inner plates and the initial sliding friction coefficient were captured with an equivalent stiffness, which was input as the elastic stiffness. An equivalent stiffness was utilized due to the need to use a bilinear model, as explained above.

The same vertical properties were used for all the bearings. The compression stiffness (KC) was calculated based on the size of the bearings, and equaled 126,147 kips/in.²⁶ Since friction-pendulum bearings have no tensile stiffness, a very small value (0.001 kips/in) was input for KT.

²⁶ Refer to Appendix I on page 187 for documentation of bearing types and property calculations

5.4.3 PERFORM-3D – Loading and Analysis Input

Once the structural data of the superstructure and isolation system was modeled in PERFORM-3D, the loading data (including time history loading) was input into the program. The same member load values were entered as those used in the ETABS models. Refer to section “4.3.3 ETABS – Loading and Analysis Input” for documentation of the loading that was input into both programs.

Interstory drifts were taken at the centers of mass and as well as the edges of the floor slabs, and the worst case interstory drifts were recorded. The methodology for assigning drifts in PERFORM-3D included specifying the global direction (H1, H2, or an angle from H1), then selecting the upper node and lower node from which the drifts were determined. The relative displacements between two points could have also been determined by following a similar procedure. However, the relative displacements were not required to be assigned at the isolation bearing nodes since the displacements were already accounted for by the bearing elements.

Once the loads and drifts were input into the models, strength and deformation limit states were defined for the plastic hinges and strength sections. Figure AAA below illustrates the typical limit state input in PERFORM-3D, particularly for the deformation Life Safety (level 2) structural performance level of the columns. Note that grouped elements were selected along with their specific component (P-M-M plastic hinge rotation type, which was used for all columns). The deformation type (hinge rotation), performance level, and corresponding demand/capacity ratio were then specified. The

actual deformations for each performance level were already specified when each plastic hinge was defined, so a demand/capacity value of 1.0 was specified for each performance level to correspond directly with its specified rotation value. Three performance levels were specified for each plastic hinge, corresponding to the Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) performance levels. The limit states of each level were assigned accordingly as shown Figure AAA below.

LIMIT STATES

Type: Deformation

Status: Saved

Name: columns LS

Deformation Conditions:

CONDITION TO BE ADDED

Element Group: MF COL 3

Element Type: Column

Component Type: P-M2-M3 Hinge, Steel Rotation Type

Deformation Type: Hinge Rotation

Performance Level: 2

Limit on D/C Ratio: 1.0

LIST OF CONDITIONS (max. 20) Click to highlight for Insert, etc. Double click to select for editing.

No.	Element Group	Element Type	Component Type	Deformation Type	Level	D/C Limit
1	MF COL 3	Column	P-M2-M3 Hinge, Steel Rotation Type	Hinge Rotation	2	1
2	MF COL 2	Column	P-M2-M3 Hinge, Steel Rotation Type	Hinge Rotation	2	1
3	MF COL 1	Column	P-M2-M3 Hinge, Steel Rotation Type	Hinge Rotation	2	1
4	BF COL 1	Column	P-M2-M3 Hinge, Steel Rotation Type	Hinge Rotation	2	1
5	BF COL 2	Column	P-M2-M3 Hinge, Steel Rotation Type	Hinge Rotation	2	1
6	BF COL 3	Column	P-M2-M3 Hinge, Steel Rotation Type	Hinge Rotation	2	1
7	BF COL 4	Column	P-M2-M3 Hinge, Steel Rotation Type	Hinge Rotation	2	1
8	BF COL 5	Column	P-M2-M3 Hinge, Steel Rotation Type	Hinge Rotation	2	1
9	BF COL 6	Column	P-M2-M3 Hinge, Steel Rotation Type	Hinge Rotation	2	1

Figure AAA: Limit State Input

Source: PERFORM-3D 2011

When all the limit states were assigned for the plastic hinges and strength sections of each component, they were grouped together into a single structural performance level group (IO, LS, or CP).

To make the model analyze the structure more accurately, an option in PERFORM-3D was selected in which the braced frames were automatically deleted for gravity analysis and then reentered for the lateral analyses. This temporary exclusion prevented gravity loads from being transferred into the braces and caused the columns to support all the gravity loads, which was in agreement with standard engineering assumptions and practice. This analytical option was conducted in ETABS as well.

Next, the load cases were assigned. For the gravity (dead + live) load case, the nodal and element/distributed loads were input for each load type group (floor dead, floor live, roof dead, and roof live loads). The factors for each load case were all specified as 1 since the structure was already designed using load combinations in ETABS and it was only to be analyzed for its seismic performance in PERFROM-3D. As stated earlier, the same loads were input as those in the ETABS models.

Similarly, the lateral load cases were input into PERFORM-3D. For the dynamic earthquake load cases, first each earthquake ground acceleration record had to be added to the PERFORM-3D database, as shown in Figure BBB on page 101. The specific earthquake illustrated below is the “DBE-182” El Cento ground motion. The text files of each ground motion (obtained from the PEER database website, as explained earlier) were added, specifying their related parameters in order to properly interpret the files. The contents of each file were a series of acceleration values given in units of gravity (g), separated by spaces. The earthquake duration and time interval were given in each file and input into PERFORM-3D.

LOAD CASES

USE THIS SECTION TO ADD AN EARTHQUAKE RECORD

Read one or more earthquake accelerograms from text files, and add to the earthquake list.
The earthquakes in the list can be used to set up earthquake load cases.

File Path:

If the file is in the USER folder, give only the file name. Otherwise give the complete file path.

Contents: Time Interval (secs): Duration (secs): Accel Unit:

Skip First: Lines. No. of Accel Values Per Line: Values Separated by: ☒ Space ☐ Comma First value at: ☒ Step 0 ☐ Step 1

Add to earthquake list as follows. Press Check to show earthquake.

Earthquake Group: File Name:

Earthquake Name:

USE THIS SECTION TO REVIEW OR DELETE AN EARTHQUAKE RECORD

Earthquake Group: File Name:

Earthquake Name =

Review earthquakes in the earthquake list, and/or delete earthquakes from the list.

Figure BBB: Input of Earthquake Record

Source: PERFORM-3D 2011

While each ground motion was being input into PERFORM-3D, it was also able to be graphed and reviewed. Figure CCC on page 102 shows the plot of the “DBE-182” El Centro fault-normal ground motion, which was input into PERFORM-3D.

Figure CCC: Review of Earthquake Record
Source: PERFORM-3D 2011

After each ground motion was input into the PERFORM-3D database, the load cases were then defined. The same total time and time steps were specified as those used when each ground motion was added. The maximum number of events in any step was set to a large value (1000) due to the large number of nonlinear components in the model and to ensure that the analysis would not be stopped. The results were specified to be saved at an interval of time steps that equaled a total time interval of about 0.05 seconds for each ground motion. The analyses were programmed to stop when the failure of a component occurred; however, no component reached failure in any of the final analyses. The fault-normal and fault-parallel ground motions were simultaneously input in the Q1

(H1) and Q2 (H2) directions, respectively. Vertical ground motions were not considered in this study, so they were not input into PERFORM-3D.

Once the load cases were defined, the analysis series were specified. For both the DBE and MCE series analysis input, the number of mode shapes to calculate was selected as 12, which was the same number of modes specified in ETABS in order to achieve 90% total modal mass participation in each degree of freedom. P-delta effects were considered in the analysis in order to achieve more realistic results.

Figure DDD: Analysis Series Damping Input
Source: PERFORM-3D 2011

Figure DDD above illustrates the typical analysis input for modal and Rayleigh damping. In accordance with standard practice, a damping value of roughly 3% was selected for the fixed-base structure. The base-isolated structure was assigned a damping

value of roughly 1%, because damping occurring in the isolation system was expected to reduce structural deformations and correlated damping in the superstructure. Rayleigh damping was specified to greatly increase damping for low spectral-period to fundamental-period ratios in order to negate higher-order modal effects, as shown in Figure DDD. Properly defining this modal damping behavior was particularly important for the proper modeling of the base-isolated structures, where the base-isolated modes are the primary modes of the structures and the higher-mode effects tend to be negligible. The default settings for Alpha-M and Beta-K damping were used, with scale factors of unity. No upper/lower bounds were specified for the structural members, so none were specified for the analysis. A normal analysis setting was selected in lieu of the “Quick’n’Dirty” analysis setting, which ignores strength loss and cyclic degradation. This setting is meant to be activated when initially creating and debugging the model, as it aids identifying problems with the model by ignoring more complex modeling features.

The typical procedure for setting up and running each analysis in PERFORM-3D was to first run the gravity analysis and then run each dynamic earthquake analysis, while superimposing the previous gravity analysis results. The same procedure was followed for both the DBE and MCE analysis series.

5.5 Verification of Models

This section illustrates the checks that were performed in order to verify consistency between the ETABS and PERFORM-3D models for each structure.

5.5.1 Verification of Models – Modal Check

Tables 19 and 20 below verify the consistency between the modes of each model.

Table 19: Fixed-Base Models Modal Verification

Source: self-produced

Fixed-Base Modes Verification

(ETABS Modal Analysis)				(PERFORM-3D Modal Analysis)				Period Difference [%]
Mode	Period [Sec.]	Direction	Cumulative Mass Participation [%]	Mode	Period [Sec.]	Direction	Cumulative Mass Participation [%]	
1	3.224	X	79.93	1	2.983	X	79.92	7.5
2	2.723	Y	67.56	2	2.442	Y	67.57	10.3
3	2.648	RZ	72.23	3	2.407	RZ	-	9.1
4	1.053	X-2	89.97	4	0.976	X-2	89.93	7.3
5	0.794	RZ-2	88.73	5	0.720	RZ-2	-	9.3
6	0.730	Y-2	88.11	6	0.655	Y-2	88.09	10.3
7	0.601	X-3	93.76	7	0.558	X-3	93.72	7.2
8	0.428	RZ-3	94.12	8	0.388	RZ-3	-	9.3
9	0.405	X-4	95.83	9	0.377	X-4	95.80	6.9
10	0.378	Y-3	94.36	10	0.339	Y-3	94.34	10.3

X = U1 (SMF); Y = U2 (SCBF); RZ = Torsional

Table 20: Base-Isolated Models Modal Verification

Source: self-produced

Base-Isolated Modes Verification

(ETABS Modal Analysis)				(PERFORM-3D Modal Analysis)				Period Difference [%]
Mode	Period [Sec.]	Direction	Cumulative Mass Participation [%]	Mode	Period [Sec.]	Direction	Cumulative Mass Participation [%]	
1	5.328	X	97.04	1	5.230	X	97.71	1.8
2	5.024	Y	94.84	2	4.949	Y	95.09	1.5
3	4.562	RZ	97.04	3	4.492	RZ	-	-
4	1.577	Y-2	99.95	4	1.480	X-2	99.82	8.5
5	1.565	X-2	99.74	5	1.443	Y-2	99.97	5.4
6	1.439	RZ-2	99.90	6	1.331	RZ-2	-	-
7	0.818	X-3	99.96	7	0.765	X-3	99.97	6.5
8	0.594	RZ-3	99.98	8	0.537	RZ-3	-	-
9	0.535	Y-3	99.99	9	0.489	X-4	99.99	9.7
10	0.526	X-4	99.99	10	0.483	Y-3	100.00	7.0

X = U1 (SMF); Y = U2 (SCBF); RZ = Torsional

Note that the modal mass participation ratios matched very closely between the two models. Also, modal periods were within a 10.3% difference between the two models. The difference between the fundamental periods of the ETABS and PERFORM-3D models was due to the difference in lateral stiffness calculated by each program. Figure EEE below shows the story displacements of the ETABS and PERFORM-3D fixed-base models in the special moment frame (SMF) direction, based on loading obtained by the equivalent lateral force procedure as described in ASCE 7-10. These story displacements were used to verify the fundamental period of each model by hand.²⁷

Figure EEE: ETABS vs. PERFORM-3D Model Verification: Fixed-Base Drifts
Source: self-produced

²⁷ Refer to Appendix O on page 204 for documentation of fundamental period check calculations

Note that PERFORM-3D calculated smaller lateral displacement values than ETABS for the same load case, which implied that PERFORM-3D analyzed the structures with a methodology that resulted in stiffer stiffness matrices than ETABS.

5.5.2 Verification of Models – Drift Check

The drift results from the ETABS modal response spectrum analysis and average response values from the nonlinear time history analysis were obtained and compared. As shown in Figure FFF below depicting drifts in the SMF/x-direction, the results closely correlated, which verified consistency between the ETABS and PERFORM-3D models.

Figure FFF: ETABS vs. PERFORM-3D Model Verification: Drifts

Source: self-produced

5.5.3 Verification of Models – Isolator Uplift Check

The base-isolated PEFORM-3D model was checked for isolation bearing uplift.

It was found that no uplift occurred in any of the bearings during any of the seismic ground motions. Also, the minimum and maximum forces occurring in each bearing during the DBE-182 (El Centro) ground motion was recorded and compared for both the base-isolated ETABS and PERFORM-3D models. Figures GGG and HHH below illustrate the similar values and consistency between the two models for the DBE-182 ground motion.

Figure GGG: Isolator Uplift Check (DBE-182: PERFORM-3D)

Source: self-produced

Figure HHH: Isolator Uplift Check (DBE-182: ETABS)

Source: self-produced

5.5.4 Verification of Models – Equivalent Damping Check

The fixed-base model was assigned a modal damping value equal to the effective damping of the isolation system (22%) in order to verify that the resulting floor accelerations and interstory drift values were within a close range to the base-isolated model. The results of each ground motion's interstory drifts and floor accelerations in the special moment frame direction are shown in Figures III and JJJ on pages 110 and 111, respectively. As can be seen by both figures, the base-isolated and 22% damped fixed-base response values correlated with each other fairly well, especially towards the upper half of the structure where the different base conditions had less of an effect on the response.

Figure III: ETABS vs. PERFORM-3D Model Verification: Damped Drifts

Source: self-produced

The base condition caused the greatest disparity between the two models' acceleration response values near the base, as shown in Figure JJJ. This disparity was to be expected since assigning additional modal damping to the fixed-base structure should have had no effect on these acceleration values at the base, and the ground motion

acceleration records of the same intensity were applied for all conditions. However, the upper-half of the damped and isolated structures' response values matched closely and confirmed the correlation between the models.

**Figure JJJ: ETABS vs. PERFORM-3D Model Verification:
Damped Accelerations**
Source: self-produced

6.0 PERFORMANCE ASSESSMENT PHASE

In the performance assessment phase, the floor accelerations and interstory drifts obtained from the nonlinear time history analyses in the analysis phase were used to assess the seismic performance of the structures via fragility curves and FEMA P-58 methodology. The accompanying FEMA P-58 calculation tool, PACT, was used to compute probable damage costs for each base condition and seismic demand level, and the results were compared.

6.1 Performance Assessment Phase Procedure

An overview of the performance assessment phase procedure is shown below:

1. Enter building data and seismic response values into PACT, including
 - Quantity and types of components in the office buildings via the PACT component quantification tool
 - Total value of each building
 - Fragility curves of the components
 - Floor accelerations and interstory drifts for each ground motion
2. Perform seismic performance assessment using PACT, and generate probable damage costs
3. Compare seismic performance and probable damage costs for each base condition and seismic demand level
4. Perform cost-benefit analysis for implementing base isolation in the office building

6.2 FEMA P-58 Methodology

The purpose of the Applied Technology Council's report "Seismic Performance Assessment of Buildings" (prepared for the Federal Emergency Management Agency (FEMA) in September 2012) was to establish a standardized methodology for determining the seismic damage that a given building and its components would be expected to experience during a given seismic event, as well as the correlated damage costs. These performance levels and damage costs are determined primarily through the use of fragility curves, which are functions relating statistical probabilities of surpassing component damage states for given demand levels. An alternate and more simplistic way to view fragility curves is as a damage ratio of a component; i.e. a gauge of how much a component is damaged for a given demand level. Costs of replacement or repair are assigned to each damage state for each component, and the types and quantities of components (along with their fragility curves) typical for a given building type are determined. With the floor accelerations and interstory drifts being known, the probable damage states of the components and their correlating probable damage costs can be computed, thereby allowing total probable damage costs of a building to be computed for given seismic demand levels.

6.3 PACT

PACT is the performance assessment calculation tool provided by FEMA P-58. This section illustrates how the program was used for this study.

6.3.1 PACT – Input of Building Information

The first step in PACT was to enter the basic information of the office building into the program. This data included the region cost multiplier and date cost multiplier, which linearly scaled the damage cost results based on ratios of how much the results vary from Northern California region and 2011 date values, respectively (FEMA P-58 2012). Since this was a comparative study and since the project's region (Southern California) was expected to yield cost values similar to the Northern California region, the region cost multiplier was taken as 1. The project was analyzed to be concurrent with the time of this study (early 2013), so an inflation rate of 3% was calculated based on the Consumer Price Index data provided by the U.S. Department of Labor and Bureau of Labor Statistics (BLS 2013). Based on this calculation, a date cost multiplier of 1.03 was used for this study.

A random seed value was chosen, which ensured that the same values for the performance assessment results were obtained each time the same analysis was executed, even though the results were based on random probability of the damage occurring for the given input. The results varied for each seed value, although they were still comparatively close. The particular seed value was arbitrary as long as it was a number greater than zero (which would have caused different results to be obtained each time the analysis was performed). The program's default value of 5 was therefore chosen for this study (FEMA P-58 2012).

Next, the office building's basic information was entered into PACT, as shown in Figure KKK below.

Number of Stories: 12					
Total Replacement Cost (\$):	51,000,000	Replacement Time (days):	720.00	Total Loss Threshold (As Ratio of Total Replacement Cost)	
Core and Shell Replacement Cost (\$):	20,000,000	Max Workers per sq. ft.	0.002		1
Most Typical Defaults					
Floor Area (sq. ft.):	24,200.00	Story Height (ft.):	13.33		

Floor Num	Floor Name	Story Height (ft.):	Area (sq. ft.):	Height Factor	Hazmat Factor	Occupancy Factor
1	Floor 1	13.33	24,200.00	1	1	1.2
2	Floor 2	13.33	24,200.00	1	1	1.2
3	Floor 3	13.33	24,200.00	1	1	1.2
4	Floor 4	13.33	24,200.00	1	1	1.2
5	Floor 5	13.33	24,200.00	1.08	1	1.2
6	Floor 6	13.33	24,200.00	1.08	1	1.2
7	Floor 7	13.33	24,200.00	1.08	1	1.2
8	Floor 8	13.33	24,200.00	1.08	1	1.2
9	Floor 9	13.33	24,200.00	1.08	1	1.2
10	Floor 10	13.33	24,200.00	1.08	1	1.2
11	Floor 11	13.33	24,200.00	1.16	1	1.2
12	Floor 12	13.33	24,200.00	1.16	1	1.2
13	Roof	0.00	24,200.00	1.16	1	1.2

Figure KKK: PACT Building Information

Source: PACT 2012

All 12 floors and the roof along with their story heights and areas were input, as illustrated in the Figure KKK above. The Total Replacement Cost was estimated to be \$51 million (\$162 per square foot), equal to the total construction cost of a similar office building built in Los Angeles in 2001 for \$39 million and taking into account adjustments for inflation (KPFF 2006). The Core and Shell Replacement cost was given as 40% of the Total Replacement Cost, which was the percentage used in example problems of office buildings in the PACT implementation guide (FEMA P-58 2012). This cost ratio translated into a core and shell replacement cost of \$20 million (\$65 per square foot).

The Height Factor linearly scales damage costs to take into account the increase in cost required to repair building components on upper levels, due to added travel time, scaffolding, etc. (FEMA P-58 2012). The Height Factors recommended in the PACT implementation guide, shown in Table 21 below, were used for this study.

Table 21: PACT Height Factor Premium Values for Building Level
Source: FEMA P-58 2012

Building Level	Height Factor
Below grade levels and floors 1-4	1.00
Floors 5-10	1.08
Floor 11 and higher	1.16

The Hazmat Factor linearly scales damage costs in accordance with premiums for hazardous materials in the building. Since no hazardous materials were assumed to be located in the building, the default value of 1 was selected for the Hazmat Factor. The Occupancy Factor linearly scales cost results according to the premium associated with repairing buildings of particular occupancy types during their typical operations. For low to moderate level earthquakes, the default values for this occupied condition are recommended by the PACT implementation guide (FEMA P-58 2012). For this study, a value of 1.2 was selected for an occupied office building, based on the values given in Table 22 on page 117.

Table 22: PACT Occupancy Factors

Source: FEMA P-58 2012

Occupancy Category	Occupancy Factor	
	Unoccupied	Occupied
Commercial Office	1.0	1.2
Research	1.4	1.8
Healthcare	1.5	2.0
Education K-12	1.0	1.1
Multi-Unit Residential	1.1	1.2
Retail	1.2	1.3
Warehouse	1.1	1.1
Hospitality	1.1	1.3

6.3.2 PACT – Selecting and Quantifying Components

The PACT component quantification tool was used to determine the types and quantities of components within the office buildings. The tool populates each building with components based on pre-determined population densities of the components for each occupancy category, as shown in Table 23 below.

Table 23: Building Occupancy Component Matrix
Source: FEMA P-58 2012

BUILDING OCCUPANCY COMPONENT MATRIX								
Component	LABORATORY	HEALTHCARE	SCHOOLS	APARTMENT	OFFICE	RETAIL	WAREHOUSE	HOSPITALITY
B2022.001	0.0015	0.0014	0.0011	0.0015	0.003	0.0006	0.00001	0.0012
B3011.011	0.245	0.29	0.68	0.32	0.27	0.5	1	0.2
C1011.001a	0.00085	0.00105	0.00056	0.0012	0.001	0.0001	0.00003	0.0006
C3021.001k	0.028	0.0705	0.0788	0.212	0.042	0.06	0.0024	0.16
C3011.001a	0.0000504	0.0001269	0.00014184	0.0003816	0.0000756	0.000108	0.00000432	0.000288
C3027.001					0.75			
C3032.001a	0.85	0.8	0.95	0	0.9	0.9	0.05	0.15
C3032.001a	0.05	0.08	0.03	0.95	0.05	0.03	0.05	0.7
C3032.001a	0.08	0.08	0.02	0.05	0.03	0.02	0.9	0.08
C3032.001a	0.02	0.04	0	0	0.02	0.05	0	0.07
E2022.001					0.007			
E2022.112a	0.0003	0.0004			0.0008			
E2022.102a	0.003	0.001			0.002			
F1012.001							0.1	
C2011.001a	0.0001	0.00008	0.00007	0.00012	0.0001	0.0001	0	0.00011
D1014.011	0.000017	0.000028	0.00002	0.000034	0.000028	0.0001	0	0.000015
D2021.011a	0.00006	0.00011	0.00003	0.000106	0.000042	0.00003	0.0000048	0.00008
D3031.011a	0.00325	0.00325	0	0	0.00285	0	0	0
D3031.021a	0.00325	0.00325	0	0	0.00285	0	0	0
D3052.011a	1.25	1	0	0	0.7	0	0.2	0
D3041.101a	1.25	1			0	0	0	
D3041.012a	0.00005	0.000035	0	0	0.00002	0.00005	0	0
D3041.011a	0.0001	0.000075	0.00005	0.00005	0.000075	0.000005	0.00004	0.00005
D3041.031a	0.016	0.02	0.005	0.008	0.009	0	0.003	0.008
D3041.041a	0.002	0.005	0	0	0.005	0	0	0
D3041.041a	0.0003	0.005	0	0	0.002	0.004	0	0
D3041.041a			0.004	0.004				0.006
D5012.013a	0.00004	0.00005	0.00004	0	0.00004	0	0	0
D5011.011a	0.0001	0.0001	0.0001	0.00005	0.0001	0.0001	0.00005	0.0001
D5012.031a	0.00004	0.00006	0.00004	0.000013	0.00004	0.00003	0.00002	0.00003
D5012.021a	0.0003	0.0003	0.0003	0.0008	0.0003	0.0003	0.0001	0.0003
C3033.001	0.015	0.015	0.015	0	0.015	0.015	0	0
C3034.001	0.015	0.015	0.015	0	0.015	0.015	0.015	0
D5092.031a	0.005	0.005	0	0	0	0	0	0
D4011.021a	0.01	0.011	0.009	0.011	0.01	0.009	0.0055	0.011
D4011.031a	0.01	0.012	0.008	0.012	0.009	0.008	0.008	0.012

By entering the area of each floor and the percentage of floor area each occupancy category is assigned to, the given population densities of the components for each occupancy category may be combined to quantify the total number of components likely to be on each floor and the building as a whole.

Once the components were selected and quantified, their fragility curves were assigned for each damage state. The majority of components used for this study had fragility curves that were already defined in PACT. Examples of a typical component's fragility curves are shown in Figure LLL below.

Figure LLL: Typical Component Fragility Curve Damage States
Source: PACT 2012

Figure LLL shows the fragility curves for the damage states of the “C1011.001c” component, which is a full-height partition wall. The x-axis defines the story drift ratio (in radians). The y-axis gives the probability at each story drift ratio that the component will be damaged to the defined damage state. By default, PACT uses the story drift values corresponding to a 50% (0.5) probability of the component being damaged to each damage state for its seismic performance analyses. Note that the first damage state of the partition wall, defined by the green line, is defined by screws popping out and minor cracking. As the story drift is increased, the second damage state is reached, defined by the yellow line. This damage state is defined by moderate cracking and crushing of the gypsum wall boards. Finally, as the story drifts are further increased, the third damage state is reached. This damage state is defined by significant cracking or crushing of the gypsum wall boards.

Each damage state is defined by a median story drift ratio (which corresponds to a 50% probability of the damage level occurring) and a dispersion factor. This relationship is lognormal (FEMA P-58 2012). The dispersion factor determines the spread/variance in story drift ratio from the mean probability to the lesser and greater probabilities (between 0% and 100%). Note that each successive damage state has a larger median story drift ratio at which the damage state is reached.

Once the fragility curves of the components were defined, the directional information of the components was entered into PACT, as shown in Table 24 on page 121.

Table 24: PACT Component Fragilities

Source: PACT 2012

Category	Component	Dir. 1	Dir. 2	Population Model
A: Substructure				
B: Shell				
B10: Super Structure				
B103: Structural Steel Elements				
	B1033.011b: Special Concentric Braced Frame w HSS braces, balanced design criteria, Chevron Brace,		X	Commercial Office
	B1035.012: Post-Northridge RBS connection with welded web, beams both sides of column, beam de	X		Commercial Office
	B1035.002: Post-Northridge RBS connection with welded web, beam one side of column only, beam	X		Commercial Office
	B1031.021b: Welded column splices, Column 150 plf < W < 300 plf	X	X	Commercial Office
	B1031.011c: Steel Column Base Plates, Column W > 300 plf	X	X	Commercial Office
B20: Exterior Enclosure				
B202: Exterior Window Systems				
	B2022.001: Curtain Walls - Generic Midrise Stick-Built Curtain wall, Config: Monolithic, Lamination: U	X	X	Commercial Office
C: Interiors				
C10: Interior Construction				
C101: Partitions				
	C1011.001b: Wall Partition, Type: Gypsum with metal studs, Partial Height, Fixed Below, Lateral Brace	X	X	Commercial Office
	C1011.001c: Wall Partition, Type: Gypsum with metal studs, Full Height, Fixed Below, Slip Track Above	X	X	Commercial Office
C20: Stairs				
C201: Stairs				
	C2011.001a: Prefabricated steel stair with steel treads and landings with seismic joints that accommo	X	X	Commercial Office
C30: Interior Finishes				
C301: Wall Finishes				
	C3011.001b: Wall Partition, Type: Gypsum + Wallpaper, Partial Height, Fixed Below, Lateral Braced Al	X	X	Commercial Office
	C3011.002c: Wall Partition, Type: Gypsum + Ceramic Tile, Full Height, Fixed Below, Slip Track Above v	X	X	Commercial Office
C302: Floor Finishes, Raised Access Floors and Floor Flooding				
	C3027.002: Raised Access Floor, seismically rated.	X	X	Commercial Office
C303: Ceilings and Ceiling Lighting				
	C3034.002: Independent Pendant Lighting - seismically rated	X	X	Commercial Office
	C3032.003a: Suspended Ceiling, SDC D,E (Ip=1.0), Area (A): A < 250, Vert & Lat support	X	X	Commercial Office
	C3032.003b: Suspended Ceiling, SDC D,E (Ip=1.0), Area (A): 250 < A < 1000, Vert & Lat support	X	X	Commercial Office
	C3033.001: Recessed lighting in suspended ceiling - no independent support wires	X	X	Commercial Office
D: Services				
D10: Conveying				
D101: Elevators & Lifts				
	D1014.021: Hydraulic Elevator - Applies to most California Installations 1976 or later, most western s	X	X	Commercial Office
D20: Plumbing				
D202: Domestic Water Distribution including hot water heaters				
	D2021.013a: Cold Water Piping (dia > 2.5 inches), SDC D,E,F, PIPING FRAGILITY	X	X	Commercial Office
	D2021.013b: Cold Water Piping (dia > 2.5 inches), SDC D,E,F, BRACING FRAGILITY	X	X	Commercial Office
	D2022.023a: Hot Water Piping - Large Diameter Welded Steel - (greater than 2.5 inches in diameter),	X	X	Commercial Office
	D2022.023b: Hot Water Piping - Large Diameter Welded Steel - (greater than 2.5 inches in diameter),	X	X	Commercial Office
D203: Sanitary Waste Piping System				
	D2031.023a: Sanitary Waste Piping - Cast Iron w/bell and spigot couplings, SDC D,E,F, PIPING FRAGILI	X	X	Commercial Office
	D2031.023b: Sanitary Waste Piping - Cast Iron w/bell and spigot couplings, SDC D,E,F, BRACING FRAG	X	X	Commercial Office
D30: HVAC				
D303: Chillers, Cooling Towers and Compressors				
	D3031.013h: Chiller - Capacity: 350 to <750 Ton - Equipment that is either hard anchored or is vibrati	X	X	Commercial Office
D304: Distribution Systems including Fans, Drops & Diffusers and VAV Boxes				
	D3041.011c: HVAC Galvanized Sheet Metal Ducting less than 6 sq. ft in cross sectional area, SDC D, E,	X	X	Commercial Office
	D3041.032c: HVAC Drops / Diffusers without ceilings - supported by ducting only - No independent s	X	X	Commercial Office
	D3041.041b: Variable Air Volume (VAV) box with in-line coil, SDC C	X	X	Commercial Office
D305: Package Air Handling Units				
	D3052.013k: Air Handling Unit - Capacity: 25000 to <40000 CFM - Equipment that is either hard anch	X	X	Commercial Office
D306: Control Panels & Instrumentation				
	D3067.013c: Control Panel - Capacity: all - Equipment that is either hard anchored or is vibration isol	X	X	Commercial Office
D40: Fire Protection				
D401: Sprinklers Horizontal Distribution including Risers and Drops				
	D4011.053a: Fire Sprinkler Drop Standard Threaded Steel - Dropping into braced lay-in tile SOFT ceiling	X	X	Commercial Office
	D4011.023a: Fire Sprinkler Water Piping - Horizontal Mains and Branches - Old Style Vitaulic - Thin W	X	X	Commercial Office
D50: Electrical				
D501: Electrical Service & Distribution				
	D5012.021a: Low Voltage Switchgear - Capacity: 100 to <350 Amp - Unanchored equipment that is ne	X	X	Commercial Office
D509: Other Electrical System including generators, battery racks and chargers				
	D5092.013c: Battery Rack - Capacity: all - Equipment that is either hard anchored or is vibration isolat	X	X	Commercial Office
	D5092.011a: Battery Rack - Capacity: all - Unanchored equipment that is not vibration isolated - Equip	X	X	Commercial Office
E: Equipment & Furnishings				
E20: Furnishings				
E202: Movable Furnishings (including office furniture and equipment, etc)				
	E2022.023: Desktop electronics including computers, monitors, stereos, etc, smooth surface	X	X	Commercial Office
	E2022.001: Modular office work stations.	X	X	Commercial Office
	E2022.104b: Bookcase, 4 shelves, anchored laterally	X	X	Commercial Office
	E2022.125b: Lateral Filing Cabinet, 4 drawer, anchored laterally	X	X	Commercial Office

Table 24 illustrates the direction in which each component was applied. Most of the components were applied in both directions. However, some of the structural components were applied only in their respective directions, including the special concentrically braced frames in Direction 2 (y-direction) and the special moment frames in Direction 1 (x-direction). All the components were assigned a “Commercial Office” population model. Table 24 also illustrates how each component was divided into categories for general use and sub-categories for each different type of component.

Figures MMM, NNN, and OOO on pages 123 and 124 show how the components were sorted into structural performance groups. The performance groups were created for each story level and each direction (Direction 1, Direction 2, and Non-Directional). Here the component quantities and quantity dispersions (found earlier with the component quantification tool) were entered, along with their population model (commercial office building) and demand parameter (story drift ratio or acceleration). The quantity dispersions accounted for the degree of uncertainty of the quantities of each component. The components were selected as being correlated, which determined a single damage state for each member type in each performance group and applied the damage states to all the members of the same type. This strategy saved computation time and reduced variability among the individual members of the groups while having negligible effects on the mean results of the analyses (FEMA P-58 2012).

Direction: ☒ Direction 1 ☐ Direction 2 ☐ Non-Directional

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No.	Component Type	Unit	Performance Group Quantities	Quantity Dispersion	Fragility Correlated	Population Model	Demand Parameters
B1031.011c	Steel Column Base Plates, Column W > 300 plf	Ea.	16.00	0.00	☒	Commercial Office	Story Drift Ratio
B1031.021b	Welded column splices, Column 150 plf < W < 300 plf	Ea.	16.00	0.00	☒	Commercial Office	Story Drift Ratio
B1035.002	Post-Northridge RBS connection with welded web, beam one side of column only, beam depth >= W30	Ea.	2.00	0.00	☒	Commercial Office	Story Drift Ratio
B1035.012	Post-Northridge RBS connection with welded web, beams both sides of column, beam depth >= W30	Ea.	4.00	0.00	☒	Commercial Office	Story Drift Ratio
B2022.001	Curtain Walls - Generic Midrise Stick-Built Curtain wall, Config: Monolithic, Lamination: Unknown, Glass Type: Unknown, Details: Aspect ratio ...	Ea.	180.00	0.60	☒	Commercial Office	Story Drift Ratio
C1011.001b	Wall Partition, Type: Gypsum with metal studs, Partial Height, Fixed Below, Lateral Braced Above	Ea.	16.00	0.20	☒	Commercial Office	Story Drift Ratio
C1011.001c	Wall Partition, Type: Gypsum with metal studs, Full Height, Fixed Below, Slip Track Above with returns	Ea.	6.00	0.20	☒	Commercial Office	Story Drift Ratio
C2011.001a	Prefabricated steel stair with steel treads and landings with seismic joints that accommodate drift.	Ea.	2.00	0.30	☒	Commercial Office	Story Drift Ratio
C3011.001b	Wall Partition, Type: Gypsum + Wallpaper, Partial Height, Fixed Below, Lateral Braced Above	Ea.	1.30	0.70	☒	Commercial Office	Story Drift Ratio
C3011.002c	Wall Partition, Type: Gypsum + Ceramic Tile, Full Height, Fixed Below, Slip Track Above w/ returns friction connection	Ea.	1.30	0.70	☒	Commercial Office	Story Drift Ratio

Update Table

Figure MMM: PACT Performance Groups – Direction 1 (SMF)

Source: PACT 2012

Direction: ☒ Direction 1 ☐ Direction 2 ☐ Non-Directional

13 of 13 (Floor 1) | < >

No.	Component Type	Unit	Performance Group Quantities	Quantity Dispersion	Fragility Correlated	Population Model	Demand Parameters
B1031.011c	Steel Column Base Plates, Column W > 300 plf	Ea.	12.00	0.00	☒	Commercial Office	Story Drift Ratio
B1031.021b	Welded column splices, Column 150 plf < W < 300 plf	Ea.	12.00	0.00	☒	Commercial Office	Story Drift Ratio
B1033.011b	Special Concentric Braced Frame w HSS braces, balanced design criteria, Chevron Brace, Brace 41 PLF < w < 99 PLF	Ea.	4.00	0.00	☒	Commercial Office	Story Drift Ratio
B2022.001	Curtain Walls - Generic Midrise Stick-Built Curtain wall, Config: Monolithic, Lamination: Unknown, Glass Type: Unknown, Details: Aspect ratio ...	Ea.	90.00	0.60	☒	Commercial Office	Story Drift Ratio
C1011.001b	Wall Partition, Type: Gypsum with metal studs, Partial Height, Fixed Below, Lateral Braced Above	Ea.	8.00	0.20	☒	Commercial Office	Story Drift Ratio
C1011.001c	Wall Partition, Type: Gypsum with metal studs, Full Height, Fixed Below, Slip Track Above with returns	Ea.	3.00	0.20	☒	Commercial Office	Story Drift Ratio
C2011.001a	Prefabricated steel stair with steel treads and landings with seismic joints that accommodate drift.	Ea.	2.00	0.30	☒	Commercial Office	Story Drift Ratio
C3011.001b	Wall Partition, Type: Gypsum + Wallpaper, Partial Height, Fixed Below, Lateral Braced Above	Ea.	0.50	0.70	☒	Commercial Office	Story Drift Ratio
C3011.002c	Wall Partition, Type: Gypsum + Ceramic Tile, Full Height, Fixed Below, Slip Track Above w/ returns friction connection	Ea.	0.50	0.70	☒	Commercial Office	Story Drift Ratio

Update Table

Figure NNN: PACT Performance Groups – Direction 2 (SCBF)

Source: PACT 2012

Direction										Update Table	
Direction 1		Direction 2		Non-Directional							
14		Floor		1		of 13 (Floor 1)					
No.	Component Type	Performance Group Quantities	Unit	Fragility Correlated	Population Model	Demand Parameters					
C3027.002	Raised Access Floor, seismically rated.	Ea. 183.00	0.20	✓	Commercial Office	Acceleration					
C3032.003a	Suspended Ceiling, SDC D,E (Ip=1.0), Area (A): A < 250, Vert & Lat support	Ea. 97.00	0.00	✓	Commercial Office	Acceleration					
C3032.003b	Suspended Ceiling, SDC D,E (Ip=1.0), Area (A): 250 < A < 1000, Vert & Lat support	Ea. 10.00	0.00	✓	Commercial Office	Acceleration					
C3033.001	Recessed lighting in suspended ceiling - no independent support wires	Ea. 366.00	0.30	✓	Commercial Office	Acceleration					
C3034.002	Independent Pendant Lighting - seismically rated	Ea. 366.00	0.30	✓	Commercial Office	Acceleration					
D1014.021	Hydraulic Elevator - Applies to most California Installations 1976 or later, most western states installations postdating 1982 and most U.S installations...	Ea. 6.00	0.70	✓	Commercial Office	Acceleration					
D2021.013a	Cold Water Piping (dia > 2.5 inches), SDC D,E,F, BRACING FRAGILITY	Ea. 1.02	0.70	✓	Commercial Office	Acceleration					
D2021.013b	Cold Water Piping (dia > 2.5 inches), SDC D,E,F, BRACING FRAGILITY	Ea. 1.02	0.70	✓	Commercial Office	Acceleration					
D2022.023a	Hot Water Piping - Large Diameter Welded Steel - (greater than 2.5 inches in diameter), SDC D, E, or F, BRACING FRAGILITY	Ea. 1.02	0.70	✓	Commercial Office	Acceleration					
D2022.023b	Hot Water Piping - Large Diameter Welded Steel - (greater than 2.5 inches in diameter), SDC D, E, or F, BRACING FRAGILITY	Ea. 1.02	0.70	✓	Commercial Office	Acceleration					
D2031.023a	Sanitary Waste Piping - Cast Iron w/bell and spigot couplings, SDC D,E,F, BRACING FRAGILITY	Ea. 1.02	0.70	✓	Commercial Office	Acceleration					
D2031.023b	Sanitary Waste Piping - Cast Iron w/bell and spigot couplings, SDC D,E,F, BRACING FRAGILITY	Ea. 1.02	0.70	✓	Commercial Office	Acceleration					
D3031.013h	Chiller - Capacity: 350 to <750 Ton - Equipment that is either hard anchored or is vibration isolated with seismic snubbers/restraints - Equipment f...	Ea. 12.00	0.10	✓	Commercial Office	Acceleration					
D3041.011c	HVAC Galvanized Sheet Metal Ducting less than 6 sq. ft in cross sectional area, SDC D, E, or F	Ea. 2.32	0.20	✓	Commercial Office	Acceleration					
D3041.032c	HVAC Drops / Diffusers without ceilings - supported by ducting only - No independent safety wires, SDC D, E, or F	Ea. 21.96	0.50	✓	Commercial Office	Acceleration					
D3041.041b	Variable Air Volume (VAV) box with in-line coil, SDC C	Ea. 17.08	0.20	✓	Commercial Office	Acceleration					
D3052.013k	Air Handling Unit - Capacity: 25000 to <40000 CFM - Equipment that is either hard anchored or is vibration isolated with seismic snubbers/restral...	Ea. 1.00	0.00	✓	Commercial Office	Acceleration					
D3067.013c	Control Panel - Capacity: all - Equipment that is either hard anchored or is vibration isolated with seismic snubbers/restraints - Combined anchor...	Ea. 1.00	0.00	✓	Commercial Office	Acceleration					
D4011.023a	Fire Sprinkler Water Piping - Horizontal Mains and Branches - Old Style Vaulic - Thin Wall Steel - Poorly designed bracing, SDC D, E, or F, PIP ...	Ea. 4.88	0.10	✓	Commercial Office	Acceleration					
D4011.053a	Fire Sprinkler Drop Standard Threaded Steel - Dropping into braced lay-in tile SOFT ceiling - 6 ft. long drop maximum, SDC D, E, or F	Ea. 2.20	0.20	✓	Commercial Office	Acceleration					
D5012.021a	Low Voltage Switchgear - Capacity: 100 to <350 Amp - Unanchored equipment that is not vibration isolated - Equipment fragility only	Ea. 1.00	0.40	✓	Commercial Office	Acceleration					
D5092.011a	Battery Rack - Capacity: all - Unanchored equipment that is not vibration isolated - Equipment fragility only	Ea. 1.00	0.00	✓	Commercial Office	Acceleration					
D5092.013c	Battery Rack - Capacity: all - Equipment that is either hard anchored or is vibration isolated with seismic snubbers/restraints - Combined anchor...	Ea. 1.00	0.00	✓	Commercial Office	Acceleration					
E2022.001	Modular office work stations.	Ea. 120.00	0.20	✓	Commercial Office	Acceleration					
E2022.023	Desktop electronics including computers, monitors, stereos, etc, smooth surface	Ea. 120.00	0.20	✓	Commercial Office	Acceleration					
E2022.104b	Bookcase, 4 shelves, anchored laterally	Ea. 30.00	0.30	✓	Commercial Office	Acceleration					
E2022.125b	Lateral Filing Cabinet, 4 drawer, anchored laterally	Ea. 30.00	0.30	✓	Commercial Office	Acceleration					
F1012.001	Storage racks designed and installed before 2007, big box retail (12' to 15' tall)	Ea. 8.00	0.30	✓	Commercial Office	Acceleration					

Figure 000: PACT Performance Groups – Non-Directional
Source: PACT 2012

6.3.3 PACT – Analysis Settings and Input of Demand Values

Among the assessment types available in PACT, an “Intensity-Based” analysis was chosen for this study. This assessment type evaluates a building’s response to seismic earthquake intensities, i.e. ground motions scaled to 5% damped response spectrums, as was done in the analysis phase of this study. The “Intensity-Based” assessment differs from the “Scenario-Based” assessment in that the building’s proximity to an actual seismic fault does not need to be taken into account (FEMA P-58 2012). In accordance with the required data needed to perform the analyses, the floor accelerations and interstory drifts found in PERFORM-3D for each seismic event were entered into PACT. An example of how the story drifts were entered in PACT is shown in Figure PPP below. The floor accelerations were input in a similar manner.

The screenshot shows the PACT software interface with the following settings:

- Assessment Type:** Intensity (selected)
- Analysis Type:** Non-Linear (selected)
- Scenario/Intensity Information:** Typical Number of Demand Vectors: 7, Number of Realizations: 200, Non-directional conversion factor: 1.2
- Identify Intensity:** Intensity 1 of 4, Add New Intensity, Delete Intensity
- Intensity ID:** DBE Fixed
- Number of Demand Vectors:** 7, **Modeling Dispersion β_m :** 0.141, **from eq. 5-1**
- For Collapse Only:** $S_a(T)$: 0.1454, Median (g): 0.1454
- Intensity Set:** Direction: Direction 1, Demand Type: Story Drift Ratio

Floor/Story	EQ1	EQ2	EQ3	EQ4	EQ5	EQ6	EQ7
Floor 12-Roof (rad)	0.005967	0.003891	0.004981	0.005953	0.006033	0.004731	0.004399
Floor 11-12 (rad)	0.008978	0.006015	0.007545	0.009086	0.008788	0.007123	0.006318
Floor 10-11 (rad)	0.0112	0.008086	0.009519	0.01163	0.01082	0.00903	0.007679
Floor 9-10 (rad)	0.01324	0.009711	0.01096	0.01281	0.01296	0.01089	0.009809
Floor 8-9 (rad)	0.01488	0.01056	0.01182	0.01264	0.01481	0.01152	0.01081
Floor 7-8 (rad)	0.01584	0.01231	0.01306	0.01221	0.01579	0.01096	0.01136
Floor 6-7 (rad)	0.01647	0.01458	0.0144	0.01103	0.01514	0.009889	0.0125
Floor 5-6 (rad)	0.01905	0.01645	0.01468	0.01122	0.01705	0.008606	0.01471
Floor 4-5 (rad)	0.02186	0.01718	0.01356	0.01301	0.02048	0.01004	0.01683
Floor 3-4 (rad)	0.02379	0.01686	0.01431	0.01444	0.02096	0.01067	0.0178
Floor 2-3 (rad)	0.02512	0.01666	0.01384	0.01407	0.01722	0.01048	0.01577
Floor 1-2 (rad)	0.02097	0.01241	0.0088971	0.009545	0.01082	0.007018	0.01045

Figure PPP: PACT DBE Fixed-Base Story Drift Ratio Input (Direction 1)

Source: PACT 2012

As illustrated in Figure PPP, the nonlinear analysis option was selected in lieu of the simplified (linear) analysis option, which was only recommended when peak drifts were expected to be less than 4 times the drifts at which yield of the structural members were reached (FEMA P-58 2012). This option caused the analysis results to be more accurate. As described in the nonlinear time history analysis procedure, 7 time histories were analyzed, so the same number of demand vectors was input into PACT. For this study, the program default value of 200 realizations was used. PACT used the Monte Carlo process (repeated random sampling) for determining the probability of incurring processes, and 200 realizations were found to be sufficient to allow the probability distribution to be accurate and not change significantly each time the analyses were run (FEMA P-58 2012). A higher number of realizations could have been used if casualties and unsafe placards were also considered in the analysis; however, this option would have also increased computation time (FEMA P-58 2012).

The non-directional conversion factor determined the non-directional response values by multiplying the maximum response values in either direction by the factor. The default value of 1.2 was chosen, since it was deemed to be appropriate for most structures (FEMA P-58 2012). A modeling dispersion factor (β_m) of 0.141 was used for this study. This value was derived from the variables β_c and β_q , which were each selected as 0.10 due to the high quality of construction practice, drawings, and modeling quality as shown in Tables 25 and 26 on page 127.

Table 25: PACT Default Descriptions and Values for β_c

Source: FEMA P-58 2012

Definition and Construction Quality Assurance	β_c
<p><i>Superior Definition</i></p> <p>New Buildings – The building is completely designed and will be constructed with rigorous construction quality assurance, including special inspection, materials testing and structural observation.</p> <p>Existing Buildings – Drawings and specification are available and they are representative of the actual construction or if not, that the actual construction is understood. Extensive material testing confirms the strength of materials.</p>	0.10
<p><i>Average Definition</i></p> <p>New Buildings – The building is defined to the level typical in design development, or construction quality assurance and inspection are anticipated to be of limited quality.</p> <p>Existing Buildings – Documents defining the building's design are available and are confirmed by visual observation. Limited material testing generally confirms material properties.</p>	0.25
<p><i>Limited Definition</i></p> <p>New Buildings – Design is completed to a schematic or similar level of detail.</p> <p>Existing Buildings – Construction Documents are not available and knowledge of the structural system is based on limited field investigation. Material properties are based on typical values for buildings of the type, location and construction age.</p>	0.40

Table 26: PACT Default Descriptions and Values for β_q

Source: FEMA P-58 2012

Model Quality and Completeness	β_q
<p><i>Model Quality</i> – The numerical model is robust over the anticipated range of response. Strength and stiffness deterioration and all likely failure modes are modeled explicitly. Model accuracy is established with data from large-scale component tests through failure.</p> <p><i>Completeness</i> – The mathematical model includes all structural components and nonstructural components in the building that contribute strength and/or stiffness.</p>	0.10
<p><i>Model Quality</i> – The numerical model is robust over the anticipated range of displacement or deformation response. Strength and stiffness deterioration is fairly well represented, though some failure modes are simulated indirectly. Accuracy is established through a combination of judgment and large-scale component tests.</p> <p><i>Completeness</i> – The mathematical model includes most structural components and nonstructural components in the building that contribute significant strength and/or stiffness.</p>	0.25
<p><i>Model Quality</i> – The numerical model is based on cyclic envelope curve models of ASCE-41 or comparable guidelines where strength and stiffness deterioration and failure modes are incorporated indirectly.</p> <p><i>Completeness</i> – The mathematical model includes all structural components in the lateral-force-resisting system.</p>	0.40

The equation used to calculate β_m is shown below:

$$\beta_m = \sqrt{\beta_c + \beta_q} \quad \text{Eq. 11}$$

Where: β_m = Modeling Dispersion Factor
 β_c = Definition and Construction Quality Assurance Factor
 β_q = Model Quality and Completeness Factor

Features that are available in PACT but weren't used for this study include a hazard curve, which is only used in a time-based assessment where the probability of seismic demand levels occurring within a period of time is considered. Also, collapse fragility functions (which assess casualties) and residual drift functions (which gauge a building's ability to be repaired) were not used in the study for simplicity and because the primary purpose of the performance assessment phase was to determine and compare only the damage costs (FEMA P-58 2012).

7.0 RESULTS

This section discusses the results obtained from the analysis phase and performance assessment phase, including floor accelerations, interstory drifts, structural seismic performance levels, structural and nonstructural damage costs, proportions of the damage costs assigned to each component, and total cost savings.

7.1 Analysis Phase Results

The figures in this section illustrate the results obtained from the analysis phase. As can be seen by the results in Figures QQQ-XXX on pages 130-137, implementing base isolation reduced the response of the fixed-base structure by roughly half for nearly all response parameters, directions, and ground motion records. The fixed-base response is shown in blue and the base-isolated response is shown in red in each figure.

The implementation of base isolation was slightly more effective for reducing floor accelerations than for reducing interstory drifts. The interstory drifts in the direction of the braced frames were the most consistent parameters for all ground motions and buildings, due to the greater lateral stiffness of the braces compared to the moment frames. Similar response values were obtained for all 7 ground motions in each series, and the maximum considered earthquake (MCE) seismic demand level response values were greater than those at the design basis earthquake (DBE) seismic demand level by a ratio of 3:2, as was expected.

Figure QQQ: DBE SMF-Direction Maximum Floor Accelerations

Source: self-produced

Figure QQQ above illustrates the results for the DBE level floor accelerations in the x-direction (special moment frame direction). Out of all the ground motions, a maximum acceleration of 0.55 g was reached at the roof of the fixed-base structure, while the base-isolated structure had a maximum value of only 0.29 g.

Figure RRR: DBE SMF-Direction Maximum Interstory Drifts
Source: self-produced

Figure RRR above illustrates the results for the DBE level interstory drifts in the x-direction (special moment frame direction). Out of all the ground motions, a maximum interstory drift of 0.025 was reached at the lower half of the fixed-base structure, while the base-isolated structure had a maximum value of only 0.010.

Figure SSS: DBE SCBF-Direction Maximum Floor Accelerations
Source: self-produced

Figure SSS above illustrates the results for the DBE level floor accelerations in the y-direction (special concentrically braced frame direction). Out of all the ground motions, a maximum floor acceleration value of 0.67 g was reached at the roof of the fixed-base structure, while the base-isolated structure had a maximum value of 0.24 g.

Figure TTT: DBE SCBF-Direction Maximum Interstory Drifts
Source: self-produced

Figure TTT above illustrates the results for the DBE level interstory drifts in the y-direction (special concentrically braced frame direction). Out of all the ground motions, a maximum interstory drift of 0.012 was reached at the upper half of the fixed-base structure, while the base-isolated structure had a maximum value of only 0.006.

Figure UUU: MCE SMF-Direction Maximum Floor Accelerations

Source: self-produced

Figure UUU above illustrates the results for the MCE level floor accelerations in the x-direction (special moment frame direction). Out of all the ground motions, a maximum acceleration of 0.80 g was reached at the roof of the fixed-base structure, while the base-isolated structure had a maximum value of only 0.40 g.

Figure VVV: MCE SMF-Direction Maximum Interstory Drifts

Source: self-produced

Figure VVV above illustrates the results for the MCE level interstory drifts in the x-direction (special moment frame direction). Out of all the ground motions, a maximum interstory drift of 0.036 was reached at the lower half of the fixed-base structure, while the base-isolated structure had a maximum value of only 0.020.

Figure WWW: MCE SCBF-Direction Maximum Floor Accelerations

Source: self-produced

Figure WWW above illustrates the results for the MCE level floor accelerations in the y-direction (SCBF direction). Out of all the ground motions, a maximum floor acceleration value of 0.90 g was reached at the lower half of the fixed-base structure, while the base-isolated structure had a maximum value of only 0.28 g.

Figure XXX: MCE SCBF-Direction Maximum Interstory Drifts

Source: self-produced

Figure XXX above illustrates the results for the MCE level interstory drifts in the y-direction (SCBF direction). Out of all the ground motions, a maximum interstory drift of 0.018 was reached at the upper half of the fixed-base structure, while the base-isolated structure had a maximum value of only 0.011.

In addition to finding the floor accelerations and interstory drifts for each ground motion, plastic hinge deformations were also obtained in PERFORM-3D. These plastic hinge rotations were categorized into structural performance levels, including Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP), as explained in section “5.3.2 Guidelines for Analyzing Plastic Deformation.” These structural performance levels gauged the degree to which implementing base isolation technology ameliorated or prevented damage to the office building at DBE and MCE seismic intensities. The structural performance results are shown in Table 27 and Table 28 on pages 139 and 140.

The literature in ASCE 7 intends to design typical fixed-base structures to LS structural performance levels. As illustrated in the “Fixed-Base: DBE” section of Table 27, the office building reached the LS performance level for all ground motions, as expected. Table 28 shows that at the MCE seismic level, the office building reached LS in its beams and columns for all the ground motions and reached CP in its columns for 2 out of the 7 ground motions. This increase in deformation from DBE to MCE seismicity was expected. By implementing base isolation technology in the office building, seismic performance levels sustained operational standards (below IO) for the majority of the DBE ground motions and only reached IO in columns for 3 out of the 7 DBE ground motions. At MCE seismicity, the implementation of base isolation caused the office building to be improved to a mixture of LS, IO, and operational levels for the various ground motions. Therefore, the structure achieved performance levels that were expected for all analyses.

Table 27: DBE Nonlinear Time History Structural Performance Level Results

Source: self-produced

Fixed-Base: DBE			
Ground Motion	Immediate Occupancy (IO)	Life Safety (LS)	Collapse Prevention (CP)
DBE-182	—	X (b,c)	
DBE-183	—	X (c)	
DBE-721	—	X (c)	
DBE-900	—	X (c)	
DBE-1086	—	X (b, c)	
DBE-1158	—	X (c)	
DBE-1605	—	X (c)	
Base-Isolated: DBE			
Ground Motion	Immediate Occupancy (IO)	Life Safety (LS)	Collapse Prevention (CP)
DBE-182			
DBE-183	X (c)		
DBE-721			
DBE-900			
DBE-1086			
DBE-1158	X (c)		
DBE-1605	X (c)		

Notation: b - beams, c - columns

From Table 27 above, one could also predict that under a service level event (SLE, which is likely to occur within a 50-year period), the base-isolated structure would remain fully elastic and have an operational structural performance level. Conversely, the fixed-base structure would likely incur structural deformations to the IO structural performance level under SLE seismicity.

Table 28: MCE Nonlinear Time History Structural Performance Level Results

Source: self-produced

Fixed-Base: MCE			
Ground Motion	Immediate Occupancy (IO)	Life Safety (LS)	Collapse Prevention (CP)
MCE-182	—	—	X (c)
MCE-183	—	—	X (c)
MCE-721	—	X (b,c)	
MCE-900	—	X (b,c)	
MCE-1086	—	X (b,c)	
MCE-1158	—	X (b,c)	
MCE-1605	—	X (b,c)	
Base-Isolated: MCE			
Ground Motion	Immediate Occupancy (IO)	Life Safety (LS)	Collapse Prevention (CP)
MCE-182	X (b,c)		
MCE-183	—	X (c)	
MCE-721			
MCE-900	—	X (b,c)	
MCE-1086			
MCE-1158	X (b,c)		
MCE-1605	—	X (b,c)	

Notation: b - beams, c - columns

One reason why the base-isolated structure had greater variability in its structural performance levels at MCE, as illustrated in Table 28, is due to the use of a bilinear model for the Triple Friction Pendulum (TFP) isolation bearings in lieu of a tri-linear model. The bilinear model was used because the tri-linear model was not available in both ETABS and PERFORM-3D. The tri-linear behavior of TFP bearings allows for optimal seismic performance at all levels of seismicity, which was explained in section “2.3 Triple Friction Pendulum (TFP) Bearings” of this thesis. Since the tri-linear model was not used, its optimal behavior at MCE was not able to be included in this study.

7.2 Performance Assessment Phase Results

The performance assessment phase predicted the damage costs shown in Figures YYY-BBBB on pages 141-144. Figure YYY illustrates the probability of the fixed-base office building incurring damage costs for DBE level seismic events. The x-axis shows the damage costs in thousands of dollars and the y-axis gives the probability of repair costs not surpassing the given damage costs. Accordingly, the fixed-base office building had a 50% probability (which was used for all buildings and seismic demand levels) of incurring \$15.77 million in damage costs when subjected to DBE level seismic events.

Figure YYY: PACT Damage Cost Results – DBE Fixed-Base
Source: PACT 2012

Figure ZZZ, shown below, gives the probability of damage costs being incurred for the base-isolated office building subjected to DBE level seismic demands. This relationship resulted in the base-isolated building having a 50% probability of incurring \$3.80 million in damage costs when subjected to DBE level seismic events. Base isolation therefore reduced DBE level damage costs by \$11.97 million, equal to a 76% reduction in damage costs.

Figure ZZZ: PACT Damage Cost Results – DBE Base-Isolated

Source: PACT 2012

When subjected to MCE level seismic events, the fixed-base office building had a 50% probability of incurring \$23.27 million in damage costs, as shown in Figure AAAA below.

Figure AAAA: PACT Damage Cost Results – MCE Fixed-Base

Source: PACT 2012

Implementing base isolation in the office building caused a 50% probability of incurring only \$6.96 million for MCE level seismic demands, as shown in Figure BBBB on page 144. Base isolation therefore reduced damage costs by \$16.31 million at MCE, equal to a 70% reduction in damage costs.

Figure BBBB: PACT Damage Cost Results – MCE Base-Isolated
Source: PACT 2012

The damage cost results for each building and seismic demand level are summarized in the Table 29 below.

Table 29: PACT Damage Cost Results
Source: self-produced

	DBE Damage Costs (\$ Millions)	MCE Damage Costs (\$ Millions)
Fixed-Base:	15.77	23.27
Base-Isolated:	3.80	6.96
Savings:	11.97	16.31

Figures CCCC and DDDD on page 145 show the proportion of damage costs attributed to each building component type (for DBE level and MCE level events).

Figure CCCC: PACT Performance Group Results – DBE

Source: self-produced

Figure DDDD: PACT Performance Group Results – MCE

Source: self-produced

The damage costs for the fixed-base and base-isolated office buildings are compared for DBE level events in Figure CCCC, and MCE levels events in Figure DDDD. The MCE level seismic demands damaged the same component groups that were damaged in the DBE level events, but to a further extent. For MCE level events, there was also additional damage to the Recessed Ceiling Lights component group for the fixed-base office building, and no damage to this component group for the base-isolated building. Base isolation was most effective at reducing damage costs for the Furniture & Accessories component group for both the DBE level and MCE level events. The Fixed Partitions were the biggest contributor to damage costs for the base-isolated buildings at both DBE and MCE seismic demand levels.

7.3 Total Cost-Benefit Analysis of Base Isolation

The cost of implementing base isolation on a project is typically 10% of the total construction cost (KPFF 2012). This proportion takes into account not only the cost of the isolation bearings, but all of the other upfront costs of the isolation system as well, including the costs of the moat, isolation platform, and flexible conduit. By using the proportional cost of the base isolation system and the estimated total construction cost for the office building in this study, the following calculations were performed, and the results were summarized in Table 30 below.

- Total Construction Cost = \$51 Million (\$162 per Square Foot)²⁸
- 10% Base Isolation Cost = \$5.10 Million

Table 30: Total Damage Cost Savings Results

Source: self-produced

	Damage Savings (\$ Millions)	Isolation Cost (\$ Millions)	Total Savings (\$ Millions)
DBE	11.97	5.10	6.87
MCE	16.31	5.10	11.21

As shown in Table 30 above, the implementation of base isolation technology would likely achieve between \$6.87 million to \$11.21 million in total savings for the 12-story steel office building, depending on the intensity of the seismic demand. For a service level event (SLE, which typically has a return period within 50 years), seismic damage would likely be minimal to non-existent for the base-isolated structure, with corresponding minimal to non-existent damage costs. For the fixed-based structure, an event with SLE seismicity could result in moderate damage to the fixed partitions,

²⁸ Refer to section 6.3.1 on page 114 for calculations of the total construction cost

workstations, or desktop electronics. The related total damage costs could range between \$1-\$4 million dollars, which is comparable to the damage costs for the base-isolated office building during a DBE event.

Another cost of the isolation system to take into consideration besides the upfront construction cost is the cost of maintaining the system. Compared to other isolation systems, Triple Friction Pendulum bearings require relatively low maintenance, due to the coated Teflon on the stainless steel sliding surface protecting the surface from corrosion, and aging and temperature effects having minimal impact on the bearing's mechanical properties (Cheng et al. 2008). Maintenance costs for the isolation system were therefore not considered for this study.

Since this thesis involved a comparative study, the same lateral systems (special moment frames and special concentrically braced frames) were used for both the fixed-base and base-isolated structures. However, in base-isolated structures, it is more common to use the less expensive ordinary moment frames (OMF) and ordinary concentrically braced frames (OCBF) due to the isolated superstructure's expectation to remain essentially elastic. Therefore, for the majority of base-isolated projects, the cost of the isolation system is offset partially by using less expensive lateral force resisting systems. These potential savings in construction cost would increase the cost effectiveness of using base isolation technology. However, these savings were not accounted for in this study.

8.0 CONCLUSIONS

This section summarizes the conclusions that were reached as a result of this study.

8.1 Benefits of Base Isolation

The benefits of implementing base isolation in the 12-story steel office building were clearly shown by the results of this study, including

- Reduction of floor accelerations and interstory drifts by more than half
- Improvement of structural seismic performance levels
 - Change from Life Safety (LS) to no or very small plastic deformations (Immediate Occupancy, IO) at design basis earthquake (DBE) seismicity
 - Change from Collapse Prevention (CP) to small or moderate plastic deformations (IO/LS) at maximum considered earthquake (MCE) seismicity
- Savings in damage costs
 - \$11.97 million at DBE
 - \$16.31 million at MCE
- Total savings after taking into account the 10% isolation system cost
 - \$6.87 million at DBE
 - \$11.21 million at MCE

This study demonstrated that base isolation is effective at significantly mitigating and in some cases preventing seismic damage to a 12-story steel office building, including its structural and non-structural components. To a greater extent, the results of this study provide evidence that tall, flexible, and non-critical structures are viable candidates for being implemented with base isolation technology. DBE and MCE events have probable return periods of 475 and 2475 years, so they are not guaranteed to occur during a building's lifespan. However, if a DBE or MCE event did occur during the lifespan of the office building in this study, the results described in this thesis show that the total savings in damage costs would be millions of dollars greater in value than the cost of the isolation system.

It is worthwhile to note that modifying or redesigning the superstructure members (from the original fixed-base structure) was not needed to meet the higher performance criteria for the base-isolated structure. Since the structural members of the base-isolated structure were the same as those used in the fixed-base structure, this study can also be used to gauge the performance enhancements of retrofitting an existing 12-story steel office building with Triple Friction Pendulum (TFP) base isolation bearings. Also, the savings in damage costs reported in this thesis are underestimated due to the number of building components and fragility curves available in PACT at the time of this study being less than the number likely existing for an actual office building.

For a service level event (SLE), the implementation of base isolation would likely minimize or negate seismic damage and related damage costs of the office building,

leaving the structure at a fully operational structural performance level. Conversely, if the fixed-base office building were subjected to SLE seismicity it could incur structural deformations to the Immediate Occupancy structural performance level and have damage costs ranging between roughly \$1 million to \$4 million. These values may be estimated based on the expected performance of the base-isolated structure when subjected to DBE seismicity.

8.2 Lessons Learned from Study

Besides the results that were intended to be investigated in this study, there were also many beneficial lessons that were learned, which are summarized below:

- Lowering the coefficients of friction of the TFP bearings is the most effective way to improve seismic performance (i.e. reduce the superstructure's response values, including floor accelerations and interstory drifts) when implementing base isolation in a tall, flexible building.
- Using TFP bearings with larger radii of curvature (R) leads to a more flexible (smaller lateral stiffness) isolation system and improves seismic performance, although larger bearing sizes are also more expensive.
- Most of the seismic damage occurred in the interior partitions and accessories (including workstations and desktop electronics). They were among the most fragile components in the building and had the largest impact on cost.
- The bearing displacements of the base-isolated structure were sensitive to damping during the nonlinear time history analyses. Therefore, it is important to

use good judgment when assigning damping values in models of base-isolated structures. Since base-isolated structures allow the superstructure to remain essentially elastic, it is wise to use a smaller modal damping ratio for an isolated structure than the ratio used for a fixed-base structure (i.e., use 1% in lieu of 3%).

- When modeling base isolation systems, remember to assign rotationally rigid restraints to the isolation platform, which lies directly above the isolation bearings. The isolation platform must be designed and modeled to resist the large bending moments induced by the bearings during seismic events. Since isolation platforms are typically assigned as rigid diaphragms, and rigid diaphragms are often modeled with only translational restraints, it is easy to forget to include the rotationally rigid restraints for the isolation platform.
- The isolation platform should be designed to prevent uplift of the isolation bearings during a seismic event, especially since uplift of the bearings is more likely to occur for tall and flexible buildings. The use of a concrete mat was effective for preventing uplift of the 12-story steel office building in this study. Uplift of isolation bearings is more prone to occur at bearings located along braced frames than bearings located along moment frames, due to the larger aspect ratios (height-to-width) of braced frames.
- In order to reduce the prevalence of torsion in the structure's base-isolated modes, it was effective to design the outer base isolation bearings that run along the

perimeter of the structure to have a lateral stiffness that was roughly a third greater in value than the lateral stiffness of the inner bearings.

- The nonlinear time history analyses that were performed on the 12-story steel office building resulted in an average maximum bearing displacement of 32 inches for the MCE series of ground motions when using the lower-bound lateral stiffness properties of the isolation system. Therefore, it was important to use bearings with large enough diameters to accommodate this displacement, as well as design the isolation system to have a 3-foot moat that took into account additional displacements due to the effects of torsion. Since the building was analyzed for an assumed San Diego location, the building would likely require a wider moat and possibly a larger bearing diameter size if it were designed in a location with greater seismicity, such as Los Angeles.
- It was more advantageous to use TFP bearings instead of lead-rubber bearings on this project, partly because TFP bearings allow the structure's center of gravity and center of mass to naturally align, since the lateral stiffness of TFP bearings depends on the vertical force acting on each bearing. Also, TFP bearings allow base-isolated structures to have a better seismic performance at SLE than lead-rubber bearings. The difference in seismic performance at SLE is due to the smaller lateral forces that are typically required to activate TFP bearings (initiate sliding), which cause the TFP bearings to exert relatively less force on the superstructure. The tri-linear behavior and sequential activation of TFP bearings

allow for the isolation system to behave in an optimal manner at DBE and MCE levels as well, although a bilinear model of the TFP bearings was used for this study since a tri-linear model was not available in both ETABS and PERFORM-3D. At DBE level bearing displacements, the lateral stiffness of a TFP bearing isolation system is decreased in order to reduce seismic forces exerted on the superstructure; and then at MCE level displacements, the lateral stiffness of the system can be increased in order to reduce excessive displacements.

8.3 Suggestions for Future Research

The following are suggestions for areas related to this study that can be improved upon or explored in more detail in future studies:

- Investigate how to properly model the tri-linear behavior of TFP bearings in programs such as PERFORM-3D. This proper modeling technique would allow the increased lateral stiffness of the bearings that is activated at MCE level displacements to be accounted for, thereby permitting the related improvement in the superstructure's seismic performance at MCE.
- Determine the additional cost savings that may be obtained with a more complete list of building components and fragility curves
- Perform similar studies for different
 - building types
 - lateral systems

- seismic demands/locations
 - isolation systems
- Perform a similar study that includes the effects of vertical ground motions
- Include occupant population models, collapse fragility functions, casualties, hazard curves, residual drift functions, and building down-time for more in-depth analyses within the performance assessment phase
- Perform studies that determine the cost-effectiveness of implementing base isolation on a tall, flexible and non-critical structure for various levels of SLE seismicity. From these studies, find the SLE probable return period at which the break-even point would be reached for the cost of the isolation system and the reduction in the structure's damage costs that would result from implementing the base isolation system.

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10.0 APPENDICES

- A. DETERMINATION OF DESIGN CRITERIA
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A. DETERMINATION OF DESIGN CRITERIA

Determination of Design Criteria

Ch.12, ASCE 7-10

Diaphragm Rigidity (12.3, ASCE 7-10)

- $220/110 = 2:1$ Aspect Ratio $< 3:1$ Aspect Ratio
 - Concrete-filled metal deck
 - No horizontal irregularities (According to Table 12.3-1, ASCE 7-10)
- => Idealize Diaphragms as **RIGID DIAPHRAGMS**

Irregularities (12.3, ASCE 7-10)

- **No Horizontal Irregularities** (According to Table 12.3-1, ASCE 7-10)
- **No Vertical Irregularities** (According to Table 12.3-2, ASCE 7-10)

Redundancy (12.3, ASCE 7-10)

Check condition "a" (12.3.4.2, ASCE 7-10)

- Braced Frames (U2 Direction) (According to Table 12.3-2, ASCE 7-10)
 4 braces - 1 brace = 3 braces (and no extreme torsional irregularity, checked above)
 $\frac{1 \text{ brace lost}}{4 \text{ braces}} = 25\% < 33\%$ ✓ (Table 12.3-3, ASCE 7-10)

=> **$\rho = 1.0$** (U2 Direction)

- Moment Frames (U1 Direction) (According to Table 12.3-2, ASCE 7-10)
 10 M.F. bays - 1 bay = 9 bays (and no extreme torsional irregularity, checked above)
 $\frac{1 \text{ M.F. bay lost}}{10 \text{ M.F. bays}} = 10\% < 33\%$ ✓ (Table 12.3-3, ASCE 7-10)

=> **$\rho = 1.0$** (U1 Direction)

Determination of Design Criteria Continued...**Ch.12, ASCE 7-10**Seismic Load Combos

(12.4.2.3, ASCE 7-10)

- Seismic Load Effect "E" combined w/ other loads (D, L, L_r)
- Structure not subject to flood or atmospheric ice loads

=> Use the following load combos:

$$5.) (1.2 + 0.2 S_{DS})D + \rho Q_E + L + (0.2)S = [1.2 + (0.2)(0.805)]D + (1.0)Q_E + L$$

$$\Rightarrow 5.) \quad (1.361)D + Q_E + L$$

$$7.) (0.9 - 0.2 S_{DS})D + \rho Q_E = [0.9 - (0.2)(0.805)]D + (1.0)Q_E$$

$$\Rightarrow 7.) \quad (0.739)D + Q_E$$

Direction of Loading

(12.5, ASCE 7-10)

- No column or wall forming part of 2 or more intersecting SFRS ✓ (Section 12.5.4, ASCE 7-10)
- No horizontal structural irregularity Type 5 (Nonparallel System Irregularity) ✓ (Section 12.5.3, ASCE 7-10)

=> **Design seismic forces permitted to be applied independently in each of the 2 orthogonal directions**and => **Orthogonal interaction effects can be ignored**Story Drift Limit

(12.2, ASCE 7-10)

(Fixed Structure)

U1 Direction:	Special Moment Frames =>	All other Structures and Risk Category III	=> $\Delta =$	$(0.015)h_{sx}$
U2 Direction:	Special Concentric Braced Frames =>	All other Structures and Risk Category III	=> $\Delta =$	$(0.015)h_{sx}$

Risk Category12 floors with 100+ people => $(12)(100) = 1,200$ people=> **Risk Category III** (Figure C1-1, pg.382, ASCE 7-10)Rigid End ZonesUse Rigid End Zone Factor = **0.5** (Typical for steel frame beam-column connection stiffness)

R.E.O. length automatically determined by ETABS

- Intersecting column depth for beams
- Intersecting beams depth for columns

Determination of Design Criteria Continued...**Seismic Design Manual (AISC 341-05)**Expected Strength / Minimum Strength Ratios

(Table I-6-1, AISC 341-05)

- U1 Direction: Special Moment Frames
- Moment Frame Girders: A992, Grade 50 =>
 - Other WF Shapes, WT Shapes: A992, Grade 50 =>
- U1 Direction: Special Concentric Braced Frames
- Braces (HSS): A500, Grade B =>
 - WF Shapes, WT Shapes: A992, Grade 50 =>
(columns and beams)

Ry = 1.1, Rt = 1.1
Ry = 1.1, Rt = 1.1

Ry = 1.4, Rt = 1.3
Ry = 1.1, Rt = 1.1

Seismic Load Combos with Overstrength Factor

(12.4.3, ASCE 7-10)

- Seismic load effect w/ overstrength factor "Em" combined w/ other loads (D, L, L_r)
- Structure not subject to flood or atmospheric ice loads

(12.4.3.2, ASCE 7-10)

=> Use the following load combos:

$$5.) (1.2 + 0.2 S_{DS})D + \Omega_o Q_E + L + (0.2)S = [1.2 + (0.2)(0.805)]D + \Omega_o Q_E + L$$

$$\Rightarrow 5.) \quad (1.361)D + \Omega_o Q_E + L$$

$$7.) (0.9 - 0.2 S_{DS})D + \Omega_o Q_E = [0.9 - (0.2)(0.805)]D + \Omega_o Q_E$$

$$\Rightarrow 7.) \quad (0.739)D + \Omega_o Q_E$$

U1 Direction (SMF): $\Omega_o = 3$

(Table 12.2-1, ASCE 7-10)

$$5.) \quad (1.361)D + (3)Q_E + L$$

$$7.) \quad (0.739)D + (3)Q_E$$

U2 Direction (SCBF): $\Omega_o = 2$

(Table 12.2-1, ASCE 7-10)

$$5.) \quad (1.361)D + (2)Q_E + L$$

$$7.) \quad (0.739)D + (2)Q_E$$

B. FIXED-BASE RESPONSE SPECTRUM ANALYSIS CALCULATIONS

Modal Response Spectrum Analysis**(FIXED - BASIC INFO)**Comined Modal Mass $\geq$ 90%

(12.9, ASCE 7-10)

Scaling of Respons Spectra**(U1 Direction, SMF)**

$$\begin{aligned} R &= 8 \\ \Omega_o &= 3 \\ C_d &= 5.5 \end{aligned} \quad (\text{Table 12.2-1, ASCE 7-10})$$

$$\text{Risk Category} = \text{III} \Rightarrow I_e = 1.25 \quad (\text{Table 1.5-2, ASCE 7-10})$$

FORCES: Divide by R/I_e (12.9.2, ASCE 7-10)

$$I_e/R = 0.15625$$

DEFLECTIONS: Divide by R/I_e and Multiply by C_d/I_e (12.9.2, ASCE 7-10)

$$(I_e/R)(C_d/I_e) = (C_d/R) = 0.6875$$

(U2 Direction, SCBF)

$$\begin{aligned} R &= 6 \\ \Omega_o &= 2 \\ C_d &= 5 \end{aligned} \quad (\text{Table 12.2-1, ASCE 7-10})$$

$$\text{Risk Category} = \text{III} \Rightarrow I_e = 1.25 \quad (\text{Table 1.5-2, ASCE 7-10})$$

FORCES: Divide by R/I_e (12.9.2, ASCE 7-10)

$$I_e/R = 0.20833$$

DEFLECTIONS: Divide by R/I_e and Multiply by C_d/I_e (12.9.2, ASCE 7-10)

$$(I_e/R)(C_d/I_e) = (C_d/R) = 0.8333$$

Use CQC Method (12.9.3, ASCE 7-10)

Modal Response Spectrum Analysis Continued...

(FIXED, U1 Direction, SMF)

$S_1 =$	0.452	[g]	(From "Design Maps" Report)	
$S_{Ds} =$	0.805	[g]	(From "Design Maps" Report)	
$S_{D1} =$	0.466	[g]	(From "Design Maps" Report)	<input type="text"/> = Input Value
$T_L =$	8	[sec.]	(Figure 22-12, ASCE 7-10, From "Design Maps" Report)	
$W =$	25796	[kips]	(Weight of building)	

Calculation of V in U1 Direction (SMF):

$R =$	8	(Table 12.2-1, ASCE 7-10)	
$I_e =$	1.25	(Table 1.5-2, ASCE 7-10)	
$T =$	3.224	[sec.]	(From ETABS modal analysis)
$C_u =$	1.4	(Table 12.8-1, ASCE 7-10)	
LFRS:	SMF	=>	$C_t =$ 0.028 (Table 12.8-2, ASCE 7-10)
			$x =$ 0.800

Determination of T :

$h_n =$	160	[feet]	(height of structure)
$T_a =$	1.624	[sec.]	$T_a = C_t h_n^x$ (Eq. 12.8-7, ASCE 7-10)

$$C_u T_a = 2.273 \text{ [sec.]}$$

$$\Rightarrow T = 3.224 \text{ [sec.]} < 2.273 \text{ [sec.]} = C_u T_a \quad (12.9.4.1, \text{ ASCE 7-10})$$

$$T = C_u T_a = 2.273 \text{ [sec.]}$$

$$T = 2.273 \text{ [sec.]} \leq 8 \text{ [sec.]} = T_L \Rightarrow C_s \leq S_{D1}/[T(R/I_e)] \quad (\text{Eq. 12.8-3, ASCE 7-10})$$

Determination of C_s :

$$C_s = 0.1258 \quad C_s = S_{Ds}/(R/I_e) \quad (\text{Eqn. 12.8-2, ASCE 7-10})$$

$$\Rightarrow C_s = 0.1258 \leq 0.03203 = S_{D1}/[T(R/I_e)] \quad (\text{Eq. 12.8-3, ASCE 7-10})$$

$$C_s = S_{D1}/[T(R/I_e)] = 0.03203$$

$$0.044 S_{Ds} I_e = 0.04428 \geq 0.001 \quad (\text{Eq. 12.8-5, ASCE 7-10})$$

$$\Rightarrow C_s = 0.03203 \geq 0.04428 = 0.044 S_{Ds} I_e \quad (\text{Eq. 12.8-5, ASCE 7-10})$$

$$C_s = 0.04428 \quad (\text{GOVERNS})$$

Determination of V:

$$V = 1142 \text{ [kips]} \quad V = C_s W \quad (\text{Eq. 12.8-1, ASCE 7-10})$$

Scaling of Forces

(12.9.4.1, ASCE 7-10)

Check that $V_t \geq (0.85)(V)$

$$V_t = 526 \text{ [kips]} \quad (\text{From ETABS linear-elastic analysis})$$

$$V_t = 526 \text{ [kips]} \geq 971 \text{ [kips]} = (0.85)(V)$$

Need to multiply forces by $(0.85)(V)/(V_t) =$

$$1.846$$

Scaling of Drifts

(12.9.4.2, ASCE 7-10)

Check that $V_t \geq (0.85)(C_s)(W)$

$$C_s = 0.03531 \quad C_s = 0.5 S_1/(R/I_e) \quad (\text{Eqn. 12.8-6, ASCE 7-10})$$

$$(0.85)(C_s)(W) = 774 \text{ [kips]}$$

$$V_t = 526 \text{ [kips]} \quad (\text{From ETABS linear-elastic analysis})$$

$$V_t = 526 \text{ [kips]} \geq 774 \text{ [kips]} =$$

Need to multiply drifts by $(0.85)(C_s)(W)/(V_t) =$

$$1.472$$

Modal Response Spectrum Analysis Continued...**(FIXED, U2 Direction, SCBF)**

$S_1 =$	0.452	[g]	(From "Design Maps" Report)	
$S_{D1} =$	0.805	[g]	(From "Design Maps" Report)	
$S_{D1} =$	0.466	[g]	(From "Design Maps" Report)	
$T_L =$	8	[sec.]	(Figure 22-12, ASCE 7-10, From "Design Maps" Report)	
$W =$	25796	[kips]	(Weight of building)	

= Input Value

Calculation of V in U1 Direction (SMF):

$R =$	6	(Table 12.2-1, ASCE 7-10)		
$I_e =$	1.25	(Table 1.5-2, ASCE 7-10)		
$T =$	2.723	[sec.]	(From ETABS modal analysis)	
$C_u =$	1.4	(Table 12.8-1, ASCE 7-10)		
LFRS: SCBF (Other Structural System) =>			$C_t =$	0.020 (Table 12.8-2, ASCE 7-10)
			$x =$	0.750
Determination of T:				
$h_n =$	160	[feet]	(height of structure)	
$T_a =$	0.8997	[sec.]	$T_a = C_t h_n^x$ (Eq. 12.8-7, ASCE 7-10)	
$C_u T_a =$	1.260	[sec.]		
$T =$	2.723	[sec.]	<	1.260 [sec.] = $C_u T_a$ (12.9.4.1, ASCE 7-10)
=> $T = C_u T_a =$	1.260	[sec.]		
$T =$	1.260	[sec.]	≤	8 [sec.] = T_L => $C_s \leq S_{D1}/[T(R/I_e)]$ (Eq. 12.8-3, ASCE 7-10)
Determination of C_s:				
$C_s =$	0.1677		$C_s = S_{D2}/(R/I_e)$	(Eqn. 12.8-2, ASCE 7-10)
=> $C_s =$	0.1677	≤	0.07707	= $S_{D1}/[T(R/I_e)]$ (Eq. 12.8-3, ASCE 7-10)
$C_s = S_{D1}/[T(R/I_e)] =$	0.07707			
$0.044 S_{D1} I_e =$	0.04428	≥	0.001	(Eq. 12.8-5, ASCE 7-10)
$C_s =$	0.07707	≥	0.04428	= $0.044 S_{D1} I_e$ (Eq. 12.8-5, ASCE 7-10)
=> $C_s =$	0.07707			
Determination of V:				
$V =$	1988	[kips]	$V = C_s W$	(Eq. 12.8-1, ASCE 7-10)

Scaling of Forces

(12.9.4.1, ASCE 7-10)

Check that $V_t \geq (0.85)(V)$

$V_t =$	999	[kips]	(From ETABS linear-elastic analysis)
$V_t =$	999	[kips]	≥ 1690 [kips] = (0.85)(V)

Need to multiply forces by $(0.85)(V)/(V_t) =$

1.692

Scaling of Drifts

(12.9.4.2, ASCE 7-10)

Check that $V_t \geq (0.85)(C_d)(W)$

$C_d =$	0.04708		$C_d = 0.5 S_d/(R/I_e)$	(Eqn. 12.8-6, ASCE 7-10)
$(0.85)(C_d)(W) =$	1032	[kips]		
$V_t =$	999	[kips]	(From ETABS linear-elastic analysis)	
$V_t =$	999	[kips]	≥ 1032 [kips] =	

Need to multiply drifts by $(0.85)(C_d)(W)/(V_t) =$

1.033

Modal Response Spectrum Analysis Continued...**(FIXED, LOAD COMBOS)****Solve for Load Combo Scale Factors**

= Input Value

To Solve for Forces: (12.9.2 and 12.9.4.1, ASCE 7-10)**SMF (U1 Direction)**

$$I_e/R = 0.1563 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(0.85)(V)/(V_t) = 1.846 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(I_e/R)[(0.85)(V)/(V_t)] = 0.2884$$

F: COMBO 5.)	$(1.361)D + (0.2884)Q_E + L$
F: COMBO 7.)	$(0.739)D + (0.2884)Q_E$

SCBF (U2 Direction)

$$I_e/R = 0.2083 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(0.85)(V)/(V_t) = 1.692 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(I_e/R)[(0.85)(V)/(V_t)] = 0.3524$$

F: COMBO 5.)	$(1.361)D + (0.3524)Q_E + L$
F: COMBO 7.)	$(0.739)D + (0.3524)Q_E$

To Solve for Deflections: (12.9.2 and 12.9.4.2, ASCE 7-10)**SMF (U1 Direction)**

$$(I_e/R)(C_d/I_e) = (C_d/R) = 0.6875 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(0.85)(C_s)(W)/(V_t) = 1.472 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(C_d/R)[(0.85)(C_s)(W)/(V_t)] = 1.0120$$

D: COMBO 5.)	$(1.361)D + (1.0120)Q_E + L$
D: COMBO 7.)	$(0.739)D + (1.0120)Q_E$

SCBF (U2 Direction)

$$(I_e/R)(C_d/I_e) = (C_d/R) = 0.8333 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(0.85)(C_s)(W)/(V_t) = 1.033 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(C_d/R)[(0.85)(C_s)(W)/(V_t)] = 0.8612$$

D: COMBO 5.)	$(1.361)D + (0.8612)Q_E + L$
D: COMBO 7.)	$(0.739)D + (0.8612)Q_E$

To Solve for Gravity Only:

COMBO 1.)	$(1.4)D$
COMBO 2.)	$(1.2)D + (1.6)L + (0.5)L_r$
COMBO 3.)	$(1.2)D + (1.6)L_r + L$

(2.3.2, ASCE 7-10)

C. BASE-ISOLATED RESPONSE SPECTRUM ANALYSIS CALCULATIONS

Modal Response Spectrum Analysis

(ISOLATED - BASIC INFO)

Comined Modal Mass $\geq$ 90%

(12.9, ASCE 7-10)

Scaling of Respons Spectra

(U1 Direction, SMF)

$R = 8$ (Table 12.2-1, ASCE 7-10)

$\Omega_o = 3$

$C_d = 5.5$

$R_I = 2$ (17.5.4.2, ASCE 7-10)(See Determination of Isolated Design Forces)

Risk Category = III $\Rightarrow I_e = 1.25$ (Table 1.5-2, ASCE 7-10)

FORCES: Divide by R/I_e (12.9.2, ASCE 7-10) (Forces reduced by R_I , 17.6.4.2, ASCE 7-10)

$$I_e/R_I = 0.6250$$

DEFLECTIONS: Divide by R/I_e and Multiply by R_I/I_e (12.9.2, ASCE 7-10) (Replace C_d with R_I , 17.6.4.4, ASCE 7-10)

$$(I_e/R_I)(R_I/I_e) = (R_I/R_I) = 1.000$$

(U2 Direction, SCBF)

$R = 6$ (Table 12.2-1, ASCE 7-10)

$\Omega_o = 2$

$C_d = 5$

$R_I = 2$ (17.5.4.2, ASCE 7-10)(See Determination of Isolated Design Forces)

Risk Category = III $\Rightarrow I_e = 1.25$ (Table 1.5-2, ASCE 7-10)

FORCES: Divide by R/I_e (12.9.2, ASCE 7-10) (Forces reduced by R_I , 17.6.4.2, ASCE 7-10)

$$I_e/R_I = 0.6250$$

DEFLECTIONS: Divide by R/I_e and Multiply by R_I/I_e (12.9.2, ASCE 7-10) (Replace C_d with R_I , 17.6.4.4, ASCE 7-10)

$$(I_e/R_I)(R_I/I_e) = (R_I/R_I) = 1.000$$

Use CQC Method (12.9.3, ASCE 7-10)

Modal Response Spectrum Analysis Continued...			(ISOLATED, U1 Direction, SMF)	
$S_1 =$	0.452 [g]	(From "Design Maps" Report)		
$S_{D5} =$	0.805 [g]	(From "Design Maps" Report)		= Input Value
$S_{D1} =$	0.466 [g]	(From "Design Maps" Report)		
$T_L =$	8 [sec.]	(Figure 22-12, ASCE 7-10, From "Design Maps" Report)		
$W =$	25796 [kips]	(Weight of building)		
Calculation of V in U1 Direction (SMF):				
$R =$	8	(Table 12.2-1, ASCE 7-10)		
$I_e =$	1.25	(Table 1.5-2, ASCE 7-10)		
$T =$	5.322 [sec.]	(From ETABS modal analysis)		
$C_u =$	1.4	(Table 12.8-1, ASCE 7-10)		
LFRS:	SMF	=>	$C_t =$ 0.028	(Table 12.8-2, ASCE 7-10)
			$x =$ 0.800	
Determination of T:				
$h_n =$	160 [feet]	(height of structure)		
$T_a =$	1.624 [sec.]	$T_a = C_t h_n^x$ (Eq. 12.8-7, ASCE 7-10)		
$C_u T_a =$	2.273 [sec.]			
$T =$	5.322 [sec.]	<	2.273 [sec.] = $C_u T_a$	(12.9.4.1, ASCE 7-10)
=>	$T = C_u T_a =$	2.273 [sec.]		
$T =$	2.273 [sec.]	$\leq$	8 [sec.] = T_L	=> $C_s \leq S_{D1}/[T(R/I_e)]$ (Eq. 12.8-3, ASCE 7-10)
Determination of C_s:				
$C_s =$	0.1258	$C_s = S_{D5}/(R/I_e)$		(Eqn. 12.8-2, ASCE 7-10)
=>	$C_s =$ 0.1258	$\leq$	0.03203	$= S_{D1}/[T(R/I_e)]$ (Eq. 12.8-3, ASCE 7-10)
	$C_s = S_{D1}/[T(R/I_e)] =$	0.03203		
$0.044 S_{D5} I_e =$	0.04428	$\geq$	0.001	(Eq. 12.8-5, ASCE 7-10)
$C_s =$	0.03203	$\geq$	0.04428	$= 0.044 S_{D5} I_e$ (Eq. 12.8-5, ASCE 7-10)
=>	$C_s =$	0.04428		
Determination of V:				
$V =$	1142 [kips]	$V = C_s W$		(Eq. 12.8-1, ASCE 7-10)
$V_s =$	960 [kips]	(Solved in Friction Pendulum Bearing Design Calculations)(Eqn. 17.5-8, ASCE 7-10)		
Scaling of Forces (12.9.4.1, ASCE 7-10)				
Check that $V_t \geq (0.80)(V_s)$		(17.6.4.2, ASCE 7-10)		
$V_t =$	1297 [kips]	(From ETABS linear-elastic analysis)		
$V_t =$	1297 [kips]	$\geq$	768 [kips]	$= (0.80)(V_s)$
Don't need to multiply forces by $(0.80)(V_s)/(V_t)$; $(V_t)/(V_t) =$		1.000		
Scaling of Drifts (12.9.4.2, ASCE 7-10)				
Check that $V_t \geq (0.85)(C_s)(W)$				
$C_s =$	0.03531	$C_s = 0.5 S_1/(R/I_e)$		(Eqn. 12.8-6, ASCE 7-10)
$(0.85)(C_s)(W) =$	774 [kips]			
$V_t =$	1297 [kips]	(From ETABS linear-elastic analysis)		
$V_t =$	1297 [kips]	$\geq$	774 [kips]	$=$
Don't need to multiply drifts by $(0.85)(C_s)(W)/(V_t)$; $(V_t)/(V_t) =$		1.000		

Modal Response Spectrum Analysis Continued...

(ISOLATED, U2 Direction, SCBF)

$S_1 =$	0.452	[g]	(From "Design Maps" Report)	
$S_{D5} =$	0.805	[g]	(From "Design Maps" Report)	= Input Value
$S_{D1} =$	0.466	[g]	(From "Design Maps" Report)	
$T_1 =$	8	[sec.]	(Figure 22-12, ASCE 7-10, From "Design Maps" Report)	
$W =$	25796	[kips]	(Weight of building)	

Calculation of V in U1 Direction (SMF):

$R =$	6	(Table 12.2-1, ASCE 7-10)
$I_e =$	1.25	(Table 1.5-2, ASCE 7-10)

$T =$	5.016	[sec.]	(From ETABS modal analysis)
$C_u =$	1.4		(Table 12.8-1, ASCE 7-10)

LFRS: SCBF (Other Structural System) =>	$C_t =$	0.020	(Table 12.8-2, ASCE 7-10)
	$\alpha =$	0.750	

Determination of T :

$h_n =$	160	[feet]	(height of structure)
$T_a =$	0.8997	[sec.]	$T_a = C_t h_n^x$ (Eq. 12.8-7, ASCE 7-10)

$$C_u T_a = 1.260 \quad [\text{sec.}]$$

$$\Rightarrow T = 5.016 \quad [\text{sec.}] < 1.260 \quad [\text{sec.}] = C_u T_a \quad (12.9.4.1, \text{ASCE 7-10})$$

$$\Rightarrow T = C_u T_a = 1.260 \quad [\text{sec.}]$$

$$T = 1.260 \quad [\text{sec.}] \leq 8 \quad [\text{sec.}] = T_1 \Rightarrow C_s \leq S_{D1}/[T(R/I_e)] \quad (\text{Eq. 12.8-3, ASCE 7-10})$$

Determination of C_s :

$$C_s = 0.1677 \quad C_s = S_{D5}/(R/I_e) \quad (\text{Eqn. 12.8-2, ASCE 7-10})$$

$$\Rightarrow C_s = 0.1677 \leq 0.07707 = S_{D1}/[T(R/I_e)] \quad (\text{Eq. 12.8-3, ASCE 7-10})$$

$$C_s = S_{D1}/[T(R/I_e)] = 0.07707$$

$$0.044 S_{D5} I_e = 0.04428 \geq 0.001 \quad (\text{Eq. 12.8-5, ASCE 7-10})$$

$$\Rightarrow C_s = 0.07707 \geq 0.04428 = 0.044 S_{D5} I_e \quad (\text{Eq. 12.8-5, ASCE 7-10})$$

Determination of V:

$V =$	1988	[kips]	$V = C_s W$ (Eq. 12.8-1, ASCE 7-10)
$V_s =$	960	[kips]	(Solved in Friction Pendulum Bearing Design Calculations)(Eqn. 17.5-8, ASCE 7-10)

Scaling of Forces

(12.9.4.1, ASCE 7-10)

Check that $V_t \geq (0.80)(V_s)$ (17.6.4.2, ASCE 7-10)

$V_t =$	1340	[kips]	(From ETABS linear-elastic analysis)
$V_t =$	1340	[kips]	$\geq 768 \quad [\text{kips}] = (0.80)(V_s)$

Don't need to multiply forces by $(0.80)(V_s)/(V_t)$; $(V_t)/(V_t) = 1.000$ Scaling of Drifts

(12.9.4.2, ASCE 7-10)

Check that $V_t \geq (0.85)(C_s)(W)$

$C_s =$	0.04708		$C_s = 0.5 S_1/(R/I_e)$ (Eqn. 12.8-6, ASCE 7-10)
$(0.85)(C_s)(W) =$	1032	[kips]	
$V_t =$	1340	[kips]	(From ETABS linear-elastic analysis)
$V_t =$	1340	[kips]	$\geq 1032 \quad [\text{kips}] =$

Don't need to multiply drifts by $(0.85)(C_s)(W)/(V_t)$; $(V_t)/(V_t) = 1.000$

Modal Response Spectrum Analysis Continued...
Solve for Load Combo Scale Factors

(ISOLATED, LOAD COMBOS)

= Input Value

To Solve for Forces: (12.9.2 and 12.9.4.1, ASCE 7-10)

SMF (U1 Direction)

$$I_e/R_i = 0.6250 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(V_t)/(V_l) = 1.000 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(I_e/R_i)[(V_t)/(V_l)] = 0.6250$$

F: COMBO 5.)	$(1.361)D + (0.6250)Q_E + L$
F: COMBO 7.)	$(0.739)D + (0.6250)Q_E$

SCBF (U2 Direction)

$$I_e/R_i = 0.625 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(V_t)/(V_l) = 1.000 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(I_e/R_i)[(V_t)/(V_l)] = 0.6250$$

F: COMBO 5.)	$(1.361)D + (0.6250)Q_E + L$
F: COMBO 7.)	$(0.739)D + (0.6250)Q_E$

To Solve for Deflections: (12.9.2 and 12.9.4.2, ASCE 7-10)

SMF (U1 Direction)

$$(I_e/R_i)(R_i/I_e) = (R_i/R_i) = 1.000 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(V_t)/(V_l) = 1.000 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(R_i/R_i)[(V_t)/(V_l)] = 1.000$$

D: COMBO 5.)	$(1.361)D + (1.0000)Q_E + L$
D: COMBO 7.)	$(0.739)D + (1.0000)Q_E$

SCBF (U2 Direction)

$$(I_e/R_i)(R_i/I_e) = (R_i/R_i) = 1.000 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(V_t)/(V_l) = 1.000 \quad (\text{Solved earlier in Modal Response Spectrum Analysis calculations})$$

$$(R_i/R_i)[(V_t)/(V_l)] = 1.000$$

D: COMBO 5.)	$(1.361)D + (1.0000)Q_E + L$
D: COMBO 7.)	$(0.739)D + (1.0000)Q_E$

To Solve for Gravity Only:

COMBO 1.)	$(1.4)D$
COMBO 2.)	$(1.2)D + (1.6)L + (0.5)L_r$
COMBO 3.)	$(1.2)D + (1.6)L_r + L$

(2.3.2, ASCE 7-10)

Modal Response Spectrum Analysis Continued...**(ISOLATED, U1 Direction, SMF)****Section 12.9, ASCE 7-10**

- Determine/check vertical and horizontal distribution of seismic forces

U1 Direction (SMF):

(12.9.5, ASCE 7-10)

$$K = 2 \quad (T = 3.224 \text{ sec.} > 2.5 \text{ sec.})$$

(12.8.3, ASCE 7-10)

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$$

(Eqn. 12.8-12, ASCE 7-10)

$$F_x = C_{vx} V$$

(Eqn. 12.8-11, ASCE 7-10)

$$V_x = \sum F_i$$

(Eqn. 12.8-13, ASCE 7-10)

$$k = 2$$

$$V = 971 \text{ kips}$$

Level	W_i [kips]	h_i [ft]	h_i^k	$w_i h_i^k$	C_{vx}	F_x [kips]	V_x [kips]
12	2249.5	160.00	25600.00	57588045	0.23022	223.5	223.5
11	2140.6	146.67	21512.09	46048068	0.18409	178.7	402.3
10	2140.6	133.33	17776.89	38052622	0.15212	147.7	550.0
9	2140.6	120.00	14400.00	30824165	0.12323	119.7	669.7
8	2140.6	106.67	11378.49	24356418	0.09737	94.5	764.2
7	2140.6	93.33	8710.49	18645385	0.07454	72.4	836.6
6	2140.6	80.00	6400.00	13699629	0.05477	53.2	889.8
5	2140.6	66.67	4444.89	9514582	0.03804	36.9	926.7
4	2140.6	53.33	2844.09	6087963	0.02434	23.6	950.3
3	2140.6	40.00	1600.00	3424907	0.01369	13.3	963.6
2	2140.6	26.67	711.29	1522562	0.00609	5.9	969.5
1	2140.6	13.33	177.69	380355	0.00152	1.5	971.0
Σ	25796			250144700			

Modal Response Spectrum Analysis Continued...**(ISOLATED, U2 Direction, SCBF)****Section 12.9, ASCE 7-10**

- Determine/check vertical and horizontal distribution of seismic forces

U2 Direction (SCBF):

(12.9.5, ASCE 7-10)

$$K = 2 \quad (T = 2.723 \text{ sec.} > 2.5 \text{ sec.})$$

(12.8.3, ASCE 7-10)

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$$

(Eqn. 12.8-12, ASCE 7-10)

$$F_x = C_{vx} V$$

(Eqn. 12.8-11, ASCE 7-10)

$$V_x = \sum F_i$$

(Eqn. 12.8-13, ASCE 7-10)

$$k = 2$$

$$V = 1690 \text{ kips}$$

Level	W_i [kips]	h_i [ft]	h_i^k	$w_i h_i^k$	C_{vx}	F_x [kips]	V_x [kips]
12	2249.5	160.00	25600.00	57588045	0.23022	389.1	389.1
11	2140.6	146.67	21512.09	46048068	0.18409	311.1	700.2
10	2140.6	133.33	17776.89	38052622	0.15212	257.1	957.3
9	2140.6	120.00	14400.00	30824165	0.12323	208.3	1165.5
8	2140.6	106.67	11378.49	24356418	0.09737	164.6	1330.1
7	2140.6	93.33	8710.49	18645385	0.07454	126.0	1456.0
6	2140.6	80.00	6400.00	13699629	0.05477	92.6	1548.6
5	2140.6	66.67	4444.89	9514582	0.03804	64.3	1612.9
4	2140.6	53.33	2844.09	6087963	0.02434	41.1	1654.0
3	2140.6	40.00	1600.00	3424907	0.01369	23.1	1677.1
2	2140.6	26.67	711.29	1522562	0.00609	10.3	1687.4
1	2140.6	13.33	177.69	380355	0.00152	2.6	1690.0
Σ	25796			250144700			

Modal Response Spectrum Analysis Continued...

Calculation of center of rigidity:

(12.9.5, ASCE 7-10)

$$C_{RX} = \frac{(30')(40') + (30')(80') + (30')(140') + (30')(180')}{(30') + (30') + (30') + (30')} = 110'$$

$$C_{RY} = \frac{(5)(30')(0') + (5)(30')(110')}{(5)(30') + (5)(30')} = 55'$$

$$\text{C.O.R.} = \boxed{(110', 55')}$$

Inherent Torsion: (12.8.4.1, ASCE 7-10)C.O.M. = $\boxed{(110', 55')}$ (From MMI Calculation spreadsheets)

=> No eccentricity between C.O.M. and C.O.R. =>

No Inherent Torsional Moment, M_t Accidental Torsion: (12.8.4.2, ASCE 7-10)U1 Direction (SMF): $(0.05)(110') = 5.5'$

$$M_{ta,U1} = \boxed{(V_{x,U1})(5.5')}$$

U2 Direction (SCBF): $(0.05)(220') = 11.0'$

$$M_{ta,U2} = \boxed{(V_{x,U2})(11.0')}$$

(Offset C.O.M. in ETABS model)

Amplification of Accidental Torsion:

- No Type 1a or 1b torsional irregularities (12.8.4.3, ASCE 7-10)
- Amplification of torsion not required for Modal Response Spectrum Analysis (12.9.5, ASCE 7-10)

=> **No Amplification of Accidental Torsion**

Modal Response Spectrum Analysis Continued...Determination of P-Δ Effects: (12.9.6, ASCE 7-10)

$$\theta = \frac{P_X \Delta I_e}{V_X h_{sx} C_d} \leq 0.10 \quad (\text{Eqn. 12.8-16, ASCE 7-10}) \quad (12.8.7, \text{ASCE 7-10})$$

U1 Direction:

Check P-Δ requirement (Example calc. at 1st story)(see next page for all calculations)

$$P_X = W_{\text{BLDG}} = (1)(2249.5 \text{ kips}) = (11)(2140.6 \text{ kips}) = 25,796 \text{ kips}$$

$$\Delta = (0.008683 \text{ rad.})(160 \text{ in.}) = 1.389 \text{ in. (from ETABS linear elastic analysis)}$$

$$V_X = 971.0 \text{ kips}$$

$$h_{sx} = 13.33' \times 12 \text{ in./ft.} = 160 \text{ in.}$$

$$C_d = 5.5$$

$$I_e = 1.25$$

$$\theta = \frac{(25,796 \text{ kips})(1.389 \text{ in.})(1.25)}{(971 \text{ kips})(160 \text{ in.})(5.5)} = 0.0524 \leq 0.10 \quad \checkmark$$

=> **P-Δ Effects don't need to be included in analysis for U1 Direction**U2 Direction:

Check P-Δ requirement (Example calc. at 1st story)(see next page for all calculations)

$$P_X = W_{\text{BLDG}} = (1)(2249.5 \text{ kips}) = (11)(2140.6 \text{ kips}) = 25,796 \text{ kips}$$

$$\Delta = (0.003657 \text{ rad.})(160 \text{ in.}) = 0.585 \text{ in. (from ETABS linear elastic analysis)}$$

$$V_X = 1,690 \text{ kips}$$

$$h_{sx} = 13.33' \times 12 \text{ in./ft.} = 160 \text{ in.}$$

$$C_d = 5$$

$$I_e = 1.25$$

$$\theta = \frac{(25,796 \text{ kips})(0.585 \text{ in.})(1.25)}{(1,690 \text{ kips})(160 \text{ in.})(5)} = 0.0140 \leq 0.10 \quad \checkmark$$

=> **P-Δ Effects don't need to be included in analysis for U2 Direction**Soil Structure Interaction Reduction: (12.9.7, ASCE 7-10)Permitted but not required => **Don't Include Soil Structure Interaction Reduction**

Modal Response Spectrum Analysis Continued...**DETERMINATION OF P-Δ EFFECTS (12.9.6, ASCE 7-10)****U1 Direction (SMF)**

$$I_e = 1.25 \quad \theta = P_x \Delta I_e / (V_x h_{sx} C_d) \leq 0.10 = \theta_{\text{limit}}$$

$$C_d = 5.5 \quad (\text{Eqn. 12.8-16, ASCE 7-10})$$

$$V = 971 \text{ kips}$$

Level	W_i [kips]	P_x [kips]	Δ [rad]	Δ [in]	V_x [kips]	h_{sx} [ft]	h_{sx} [in]	θ	CHECK
12	2249.5	2250	0.004981	0.797	223.5	160.00	1920	0.0009	PASS
11	2140.6	4390	0.007279	1.165	402.3	146.67	1760	0.0016	PASS
10	2140.6	6531	0.009014	1.442	550.0	133.33	1600	0.0024	PASS
9	2140.6	8671	0.010362	1.658	669.7	120.00	1440	0.0034	PASS
8	2140.6	10812	0.011309	1.809	764.2	106.67	1280	0.0045	PASS
7	2140.6	12952	0.012354	1.977	836.6	93.33	1120	0.0062	PASS
6	2140.6	15093	0.013287	2.126	889.8	80.00	960	0.0085	PASS
5	2140.6	17234	0.014072	2.252	926.7	66.67	800	0.0119	PASS
4	2140.6	19374	0.014400	2.304	950.3	53.33	640	0.0167	PASS
3	2140.6	21515	0.014798	2.368	963.6	40.00	480	0.0250	PASS
2	2140.6	23655	0.014096	2.255	969.5	26.67	320	0.0391	PASS
1	2140.6	25796	0.008683	1.389	971.0	13.33	160	0.0524	PASS
Σ	25796								

U2 Direction (SCBF)

$$I_e = 1.25 \quad \theta = P_x \Delta I_e / (V_x h_{sx} C_d) \leq 0.10 = \theta_{\text{limit}}$$

$$C_d = 5 \quad (\text{Eqn. 12.8-16, ASCE 7-10})$$

$$V = 1690 \text{ kips}$$

Level	W_i [kips]	P_x [kips]	Δ [rad]	Δ [in]	V_x [kips]	h_{sx} [ft]	h_{sx} [in]	θ	CHECK
12	2249.5	2250	0.011221	1.795	389.1	160.00	1920	0.0014	PASS
11	2140.6	4390	0.011603	1.856	700.2	146.67	1760	0.0017	PASS
10	2140.6	6531	0.011981	1.917	957.3	133.33	1600	0.0020	PASS
9	2140.6	8671	0.011040	1.766	1165.5	120.00	1440	0.0023	PASS
8	2140.6	10812	0.011010	1.762	1330.1	106.67	1280	0.0028	PASS
7	2140.6	12952	0.009652	1.544	1456.0	93.33	1120	0.0031	PASS
6	2140.6	15093	0.009580	1.533	1548.6	80.00	960	0.0039	PASS
5	2140.6	17234	0.008335	1.334	1612.9	66.67	800	0.0045	PASS
4	2140.6	19374	0.007410	1.186	1654.0	53.33	640	0.0054	PASS
3	2140.6	21515	0.005798	0.928	1677.1	40.00	480	0.0062	PASS
2	2140.6	23655	0.005740	0.918	1687.4	26.67	320	0.0101	PASS
1	2140.6	25796	0.003657	0.585	1690.0	13.33	160	0.0140	PASS
Σ	25796								

D. CALCULATION OF DBE AND MCE RESPONSE SPECTRA (USGS 2012)

"DesignMaps" Summary Report **User-Specified Input**

Report Title Calculation of San Diego Response Spectra
Wed January 18, 2012 19:56:49 UTC

Building Code Reference Document 2010 ASCE 7 Standard
(which makes use of 2008 USGS hazard data)

Site Coordinates 32.87856°N, 117.22578°W

Site Soil Classification Site Class D – "Stiff Soil"

Site Risk Category Risk Category III – "Substantial Hazard"

USGS-Provided Output

$S_S = 1.169 \text{ g}$	$S_{MS} = 1.207 \text{ g}$	$S_{DS} = 0.805 \text{ g}$
$S_1 = 0.452 \text{ g}$	$S_{M1} = 0.699 \text{ g}$	$S_{01} = 0.466 \text{ g}$

For information on how the S_S and S_1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

USGS "DesignMaps" Detailed Report
 2010 ASCE 7 Standard (32.87856°N, 117.22578°W)
 Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1).

From Figure 22-1

$$S_S = 1.169 \text{ g}$$

From Figure 22-2

$$S_1 = 0.452 \text{ g}$$

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
Any profile with more than 10 ft of soil having the characteristics:			
<ul style="list-style-type: none"> • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf 			
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

$$\text{For SI: } 1\text{ft/s} = 0.3048 \text{ m/s } 1\text{lb/ft}^2 = 0.0479 \text{ kN/m}^2$$

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = 3 and $S_S = 1.169$, $F_a = 1.032$

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = 3 and $S_1 = 0.452$, $F_v = 1.548$

Equation (11.4-1): $S_{MS} = F_a S_S = 1.032 \times 1.169 = 1.207 \text{ g}$

Equation (11.4-2): $S_{M1} = F_v S_1 = 1.548 \times 0.452 = 0.699 \text{ g}$

Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4-3): $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.207 = 0.805 \text{ g}$

Equation (11.4-4): $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.699 = 0.466 \text{ g}$

Section 11.4.5 — Design Response Spectrum

From Figure 22-12

$T_L = 8 \text{ seconds}$

Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.

Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7

$$PGA = 0.506$$

Equation (11.8-1):

$$PGA_M = F_{PGA}PGA = 1.000 \times 0.506 = 0.506 \text{ g}$$

Table 11.8-1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	$PGA \leq 0.1$	$PGA = 0.2$	$PGA = 0.3$	$PGA = 0.4$	$PGA \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = 3 and $PGA = 0.506$, $F_{PGA} = 1.000$

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17

$$C_{RS} = 0.874$$

From Figure 22-18

$$C_{RI} = 0.922$$

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = III and $S_{DS} = 0.805$, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = III and $S_{D1} = 0.466$, Seismic Design Category = D

Note: When S_1 is greater than $0.75g$, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category $\equiv$ "the more severe design category in accordance with
Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

E. ROOF UNIT LOAD TAKE-OFF

Dead Load

Material	Weight (psf)				
Dead Load	Steel Decking	Beams	Girders	Columns	Seismic
Conc. o/ stl deck	35.0	35.0	35.0	35.0	35.0
Fireproofing	3.0	3.0	3.0	3.0	3.0
MEP		7.0	7.0	7.0	7.0
Beams		4.0	4.0	4.0	4.0
Acoustical fiber board celing		1.0	1.0	1.0	1.0
Girders			2.7	2.7	2.7
Columns				3.0	3.0
Braced Frames					4.0
Moment Frames					5.3
Partitions					10.0
Mechanical Equipment					8.4
Misc.	1.0	2.0	2.3	3.3	5.6
Total Dead Load:	39.0	52.0	55.0	59.0	89.0

Seismic weight from Mechanical: $(135 \text{ psf})(0.25) = 37.5 \text{ psf}$
 $(37.5 \text{ psf})(6 \times 30' \times 30') / (220' \times 110') = 8.4 \text{ psf}$

Seismic weight from Glass Curtain Wall Exterior: $(15 \text{ psf})(3' + 6.67') = 145.05 \text{ plf}$

Roof Live Load

(Table 4-1, ASCE 7-10)

Occupancy	Weight (psf)
Roof (Ordinary Flat)	20
Mechanical Equipment	150

Roof Live Load Reductions

(Section 4.7, ASCE 7-10)

Moment Frame Beams: $AT = 20' \times 30' = 600 \text{ ft}^2$

$R1 = 0.6, R2 = 1$

$Lr = Lo * R1 * R2$

$Lr = 20 * 0.6 * 1 = 12 \text{ psf}$

Moment Frame Columns: $AT = 20' \times 30' = 600 \text{ ft}^2$

$R1 = 0.6, R2 = 1$

$Lr = Lo * R1 * R2$

$Lr = 20 * 0.6 * 1 = 12 \text{ psf}$

Braced Frame Beams: $AT = 10' \times 30' = 300 \text{ ft}^2$

$R1 = 0.9, R2 = 1$

$Lr = Lo * R1 * R2$

$Lr = 150 * 0.9 * 1 = 135 \text{ psf}$

Braced Frame Columns: $AT = 30' \times 35' = 1050 \text{ ft}^2$

$R1 = 0.6, R2 = 1$

$Lo = (150 \text{ psf})(0.43) + (20 \text{ psf})(0.57) = 76 \text{ psf}$

$Lr = Lo * R1 * R2$

$Lr = 76 * 0.6 * 1 = 46 \text{ psf}$

Gravity Corner Columns: $AT = 20' \times 20' = 400 \text{ ft}^2$

$R1 = 0.8, R2 = 1$

$Lr = Lo * R1 * R2$

$Lr = 20 * 0.8 * 1 = 16 \text{ psf}$

Gravity Side Columns: $AT = 35' \times 20' = 700 \text{ ft}^2$

$R1 = 0.6, R2 = 1$

$Lr = Lo * R1 * R2$

$Lr = 20 * 0.6 * 1 = 12 \text{ psf}$

Total Reduced Live Loads
Moment Frames: 12 psf
Moment Frame Columns: 12 psf
Braced Frame Beams: 135 psf
Braced Frame Columns: 46 psf
Gravity Corner Columns: 16 psf
Gravity Side Columns: 12 psf

F. FLOOR UNIT LOAD TAKE-OFF

Dead Load

Material	Weight (psf)				
Dead Load	Steel Decking	Beams	Girders	Columns	Seismic
Carpet & Pad	3.0	3.0	3.0	3.0	3.0
Conc. o/ stl deck	35.0	35.0	35.0	35.0	35.0
Fireproofing	3.0	3.0	3.0	3.0	3.0
MEP		7.0	7.0	7.0	7.0
Beams		4.0	4.0	4.0	4.0
Acoustical fiber board celing		1.0	1.0	1.0	1.0
Girders			2.7	2.7	2.7
Columns				3.0	3.0
Braced Frames					4.0
Moment Frames					5.3
Partitions					10.0
Misc.	1.0	2.0	2.3	3.3	5.0
Total Dead Load:	42.0	55.0	58.0	62.0	83.0

Seismic weight from Glass Curtain Wall Exterior: (15 psf)(13.33')=200 plf

Live Load

(Table 4-1, ASCE 7-10)

Occupancy	Weight (psf)	Use Percent.	Factored Weight (psf)
Offices	50	0.80	40.0
Corridors	80	0.20	16.0
Total Unreduced Live Load:			56.0

Live Load Reductions

(Section 4.7, ASCE 7-10)

Moment Frame Beams: $AT = 20' \times 30' = 600 \text{ ft}^2$

$KLL = 2$

$L = Lo(0.25 + 15 / (KLL \cdot AT)^{1/2}) + 15 \text{ psf partition load}$

$L = 56 * (0.25 + 15 / (2 * 600)^{1/2}) + 15 \text{ psf} = 38.3 \text{ psf} + 15 \text{ psf} = 53.3 \text{ psf}$

Moment Frame Columns: $AT = 20' \times 30' = 600 \text{ ft}^2$

$KLL = 4$

$L = Lo(0.25 + 15 / (KLL \cdot AT)^{1/2}) + 15 \text{ psf partition load}$

$L = 56 * (0.25 + 15 / (4 * 600)^{1/2}) + 15 \text{ psf} = 31.1 \text{ psf} + 15 \text{ psf} = 46.1 \text{ psf}$

Braced Frame Beams: $AT = 10' \times 30' = 300 \text{ ft}^2$

$KLL = 2$

$L = Lo(0.25 + 15 / (KLL \cdot AT)^{1/2}) + 15 \text{ psf partition load}$

$L = 56 * (0.25 + 15 / (2 * 300)^{1/2}) + 15 \text{ psf} = 48.3 \text{ psf} + 15 \text{ psf} = 63.3 \text{ psf}$

Braced Frame Columns: $AT = 30' \times 35' = 1050 \text{ ft}^2$

$KLL = 4$

$L = Lo(0.25 + 15 / (KLL \cdot AT)^{1/2}) + 15 \text{ psf partition load}$

$L = 56 * (0.25 + 15 / (4 * 1050)^{1/2}) + 15 \text{ psf} = 27.0 \text{ psf} + 15 \text{ psf} = 42.0 \text{ psf}$

Gravity Corner Columns: $AT = 20' \times 20' = 400 \text{ ft}^2$

$KLL = 4$

$L = Lo(0.25 + 15 / (KLL \cdot AT)^{1/2}) + 15 \text{ psf partition load}$

$L = 56 * (0.25 + 15 / (4 * 400)^{1/2}) + 15 \text{ psf} = 35.0 \text{ psf} + 15 \text{ psf} = 50.0 \text{ psf}$

Gravity Side Columns: $AT = 35' \times 20' = 700 \text{ ft}^2$

$KLL = 4$

$L = Lo(0.25 + 15 / (KLL \cdot AT)^{1/2}) + 15 \text{ psf partition load}$

$L = 56 * (0.25 + 15 / (4 * 700)^{1/2}) + 15 \text{ psf} = 30.0 \text{ psf} + 15 \text{ psf} = 45.0 \text{ psf}$

Total Reduced Live Loads
Moment Frame Beams: 53.3 psf
Moment Frame Columns: 46.1 psf
Braced Frame Beams: 63.3 psf
Braced Frame Columns: 42.0 psf
Gravity Corner Columns: 50.0 psf
Gravity Side Columns: 45.0 psf

12 STORY OFFICE BUILDING MMI CALCULATION

Roof

[illegible]

Shape Key

R =	Rectangle
T1 =	Triangle
T2 =	Triangle
T3 =	Triangle
T4 =	Triangle

Center of Mass			
$X_c =$	110.00 ft	=	1320 in
$Y_c =$	55.00 ft	=	660 in
$W =$	2249.5 kip		
$M =$	69.861 kip-sec ² /ft	=	5.822 kip-sec ² /in
$MMI =$	364.209 kip-ft-sec ²	=	4370504 kip-in-sec ²

I. TRIPLE FRICTION PENDULUM BEARING DESIGN CALCULATIONS

UPPER-BOUND (DBE)

$$SD_1 = 0.466 \text{ g}$$

FP-FLEXIBLE (INNER)

$$\mu_2 = 0.0100$$

$$\mu_1 = 0.0200$$

$$R_{eff2} = 33 \text{ in.}$$

$$R_{eff1} = 303 \text{ in.}$$

$$\# \text{ Bearings} = 12$$

$$\text{Assume } D_D = 13.46 \text{ in.}$$

FP-STIFF (OUTER)

$$\mu_2 = 0.0100$$

$$\mu_1 = 0.0300$$

$$R_{eff2} = 33 \text{ in.}$$

$$R_{eff1} = 303 \text{ in.}$$

$$\# \text{ Bearings} = 20$$

FP-FLEXIBLE (INNER)

$$D_y = (\mu_1 - \mu_2)(R_{eff2})$$

$$D_y = (0.02 - 0.01)(33 \text{ in.}) = \boxed{0.33 \text{ in.}}$$

$$F_D = \left(\mu_1 + \frac{D_D - D_y}{R_{eff1}} \right) (W)$$

$$F_D = \left(0.02 + \frac{13.46 - 0.33}{303} \right) (W)$$

$$F_D = \boxed{(0.0633)(W)}$$

FP-STIFF (OUTER)

$$\text{(Eq. 12, Zekioglu et al. 2009)}$$

$$D_y = (0.03 - 0.01)(33 \text{ in.}) = \boxed{0.66 \text{ in.}}$$

$$\text{(Eq. 13, Zekioglu et al. 2009)}$$

$$F_D = \left(0.03 + \frac{13.46 - 0.66}{303} \right) (W)$$

$$F_D = \boxed{(0.0722)(W)}$$

$$\sum F_D = \sum (\# \text{ Bearings})(F_D)(W)$$

$$\text{FPA: } \sum F_D = (12)(0.0633)(913 \text{ K}) = 693.5 \text{ K}$$

$$\text{FPB: } \sum F_D = (2)(0.0722)(889 \text{ K}) = 128.4 \text{ K}$$

$$\text{FPC: } \sum F_D = (8)(\text{ " }) (810 \text{ K}) = 467.9 \text{ K}$$

$$\text{FPD: } \sum F_D = (2)(\text{ " }) (772 \text{ K}) = 111.5 \text{ K}$$

$$\text{FPE: } \sum F_D = (4)(\text{ " }) (1196 \text{ K}) = 345.4 \text{ K}$$

$$\text{FPF: } \sum F_D = (4)(\text{ " }) (598 \text{ K}) = 172.7 \text{ K}$$

$$\sum F_D = 1919.5 \text{ K} \approx \boxed{1920 \text{ K}}$$

$$K_D = \frac{\Sigma F_D}{D_D} = \frac{1920 \text{ K}}{13.46 \text{ in}} = \boxed{142.6 \text{ K/in.}} \quad (\text{Eq. 17.8-4, ASCE 7-10})$$

$$\Sigma W = 27,934 \text{ K} \quad (\text{see pg. 1})$$

$$T_D = 2\pi \sqrt{\frac{\Sigma W}{K_D(g)}} \quad (\text{Eq. 17.5-2, ASCE 7-10})$$

$$T_D = 2\pi \sqrt{\frac{27,934 \text{ K}}{(142.6 \text{ K/in.})(386.4 \text{ in/sec}^2)}} = \boxed{4.47 \text{ sec.}}$$

Determine Area of E_{base} (Bilinear Approximation)

FP-FLEXIBLE (INNER)

FP-STIFF (OUTER)

$$E_D = 4 \left[D_C F_C - \frac{1}{2} D_B F_B - \frac{1}{2} (F_C - F_B)(D_C - D_B) - \frac{1}{2} (F_C - F_B)(D_C - D_B) - \frac{1}{2} F_D D_D - F_D (D_C - D_D) - (F_C - F_B) D_B \right]$$

FP-FLEXIBLE (INNER)

$$E_D = 4 \left[(13.46)(0.0633) - \frac{1}{2} (0.33)(0.02) - \frac{1}{2} (0.0633 - 0.02)(13.46 - 0.33) - \frac{1}{2} (0.0633 - 0.0433)(13.46 - 13.13) - \frac{1}{2} (0.0433)(13.13) - (0.0433)(13.46 - 13.13) - (0.0633 - 0.02)(0.33) \right] = \boxed{(0.993)(W) \text{ Kip-in.}}$$

FP-STIFF (OUTER)

$$E_D = 4 \left[(13.46)(0.0722) - \frac{1}{2} (0.66)(0.03) - \frac{1}{2} (0.0722 - 0.03)(13.46 - 0.66) - \frac{1}{2} (0.0722 - 0.0422)(13.46 - 12.80) - \frac{1}{2} (0.0422)(12.80) - (0.0422)(13.46 - 12.80) - (0.0722 - 0.03)(0.66) \right] = \boxed{(1.424)(W) \text{ Kip-in.}}$$

$$\Sigma E_D = \Sigma (\# \text{ Bearings})(E_D)(W)$$

$$\text{FPA: } \Sigma E_D = (12)(0.993)(913 \text{ K}) = 10,880 \text{ Kip-in.}$$

$$\text{FPB: } \Sigma E_D = (2)(1.424)(889 \text{ K}) = 2532 \text{ Kip-in.}$$

$$\text{FPC: } \Sigma E_D = (8)(\quad)(810 \text{ K}) = 9228 \text{ Kip-in.}$$

$$\text{FPD: } \Sigma E_D = (2)(\quad)(772 \text{ K}) = 2199 \text{ Kip-in.}$$

$$\text{FPE: } \Sigma E_D = (4)(\quad)(1196 \text{ K}) = 6812 \text{ Kip-in.}$$

$$\text{FPF: } \Sigma E_D = (4)(\quad)(598 \text{ K}) = 3406 \text{ Kip-in.}$$

$$\Sigma E_D = 35057 \text{ Kip-in.} \approx \boxed{35060 \text{ Kip-in.}}$$

Viscous Damping of Isolation System

$$\beta_D = \frac{E_D}{2\pi K_D D_D^2} (100) \quad (\text{Eq. 17.8-7, ASCE 7-10})$$

$$\beta_D = \frac{35,060 \text{ kip-in.}}{2\pi(142.6 \text{ k/in.})(13.46 \text{ in.})^2} (100) = \boxed{21.59\%} = 0.2159$$

$$B_D = 2.4(\beta_D)^{0.3} \quad (\text{Eq. 1, Morgan 2011})$$

$$B_D = 2.4(0.2159)^{0.3} = \boxed{1.515}$$

$$D_D' = \frac{S_{D1} T_D}{4\pi^2 B_D} g \quad (\text{Eq. 17.5-1, ASCE 7-10})$$

$$D_D' = \frac{(0.466)(4.47 \text{ sec.})}{4\pi^2 (1.515)} (386.4 \text{ in./sec.}^2) = \boxed{13.46 \text{ in.}} = 13.46 \text{ in.} = D_D \quad \checkmark$$

Design Works!

$$R_2 = 20 \text{ [in.]}$$

$$R_1 = 156 \text{ [in.]}$$

$$h_2 = 3.5 \text{ [in.]}$$

$$h_1 = 4.5 \text{ [in.]}$$

Figure A-AP: Triple-Friction Pendulum Diagram

Source: (Zekioglu et al. 2009)

$$R_{\text{eff}2} = 33 \text{ [in.]} \quad R_{\text{eff}2} = 2(R_2 - h_2) \quad (\text{Assuming } R_2 = R_3, \mu_2 = \mu_3)$$

$$R_{\text{eff}1} = 303 \text{ [in.]} \quad R_{\text{eff}1} = 2(R_1 - h_1) \quad (\text{Assuming } R_1 = R_4, \mu_1 = \mu_4)$$

LOWER-BOUND (MCE)Assume $\pm 10\%$ variation of coefficients of friction

$$S_{M1} = 0.699 g$$

Ratios of Coefficients of Friction

Upper	1.1	(DBE)
Nominal	1	
Lower	0.9	(MCE)

$$\mu_{1-\min} = \mu_{1-\max} \left(\frac{0.9}{1.1} \right)$$

FP-FLEXIBLE (INNER)

$$\mu_2 = 0.0100$$

$$\mu_1 = 0.0164$$

$$R_{eff2} = 33 \text{ in.}$$

$$R_{eff1} = 303 \text{ in.}$$

$$\mu_1 = (0.0200)(0.9/1.1) = 0.0164$$

$$\text{Assume } D_M = 27.58 \text{ in.}$$

$$\# \text{ Bearings} = 12$$

FP-STIFF (OUTER)

$$\mu_2 = 0.0100$$

$$\mu_1 = 0.0245$$

$$R_{eff2} = 33 \text{ in.}$$

$$R_{eff1} = 303 \text{ in.}$$

$$\# \text{ Bearings} = 20$$

$$\mu_1 = (0.0300)(0.9/1.1) = 0.0245$$

FP-FLEXIBLE (INNER)

$$D_y = (\mu_1 - \mu_2)(R_{eff2})$$

$$D_y = (0.0164 - 0.0100)(33 \text{ in.}) = \boxed{0.21 \text{ in.}}$$

$$F_M = (\mu_1 + \frac{D_M - D_y}{R_{eff1}})(W)$$

$$F_M = (0.0164 + \frac{27.58'' - 0.21''}{303''})(W)$$

$$F_M = \boxed{(0.1067)(W)}$$

FP-STIFF (OUTER)

$$(\text{Eq. 12, Zekioglu et al. 2009})$$

$$D_y = (0.0245 - 0.0100)(33 \text{ in.}) = \boxed{0.48 \text{ in.}}$$

$$(\text{Eq. 13, Zekioglu et al. 2009})$$

$$F_M = (0.0245 + \frac{27.58'' - 0.48''}{303''})(W)$$

$$F_M = \boxed{(0.1139)(W)}$$

$$\Sigma F_M = \Sigma (\# \text{ Bearings})(F_M)(W)$$

$$\text{FPA: } \Sigma F_M = (12)(0.1067)(913 \text{ K}) = 1169.0 \text{ K}$$

$$\text{FPB: } \Sigma F_M = (2)(0.1139)(889 \text{ K}) = 202.5 \text{ K}$$

$$\text{FPC: } \Sigma F_M = (8)(\quad)(810 \text{ K}) = 738.1 \text{ K}$$

$$\text{FPD: } \Sigma F_M = (2)(\quad)(472 \text{ K}) = 175.9 \text{ K}$$

$$\text{FPE: } \Sigma F_M = (4)(\quad)(1196 \text{ K}) = 544.9 \text{ K}$$

$$\text{FPF: } \Sigma F_M = (4)(\quad)(598 \text{ K}) = 272.4 \text{ K}$$

$$\Sigma F_M = \boxed{3103 \text{ K}}$$

$$K_M = \frac{\Sigma F_M}{D_M} = \frac{3103 \text{ K}}{27.58''} = \boxed{112.5 \text{ K/in.}} \quad (\text{Eq. 17.8-4, ASCE 7-10})$$

$$\Sigma W = 27,934 \text{ K} \quad (\text{see pg.1})$$

$$T_M = 2\pi \sqrt{\frac{\Sigma W}{(K_M)(g)}}$$

$$(\text{Eq. 17.5-2, ASCE 7-10})$$

$$T_M = 2\pi \sqrt{\frac{27,934 \text{ K}}{(112.5 \text{ K/in.})(386.4 \text{ in./sec}^2)}} = 5.037 \text{ sec.} \approx \boxed{5.04 \text{ sec.}}$$

Determine Area of E-Loop (Bilinear Approximation)

FP-FLEXIBLE (INNER) FP-STIFF (OUTER)

$$E_M = 4[D_C F_C - \frac{1}{2} D_B F_B - \frac{1}{2} (F_C - F_B)(D_C - D_B) - \frac{1}{2} (F_C - F_D)(D_C - D_D) - \frac{1}{2} F_D D_D - F_D (D_C - D_B) - (F_C - F_B) D_B]$$

FP-FLEXIBLE (INNER)

$$E_M = 4 \left[(27.58)(0.1067) - \frac{1}{2}(0.21)(0.0164) - \frac{1}{2}(0.1067 - 0.0164)(27.58 - 0.21) \right. \\ \left. - \frac{1}{2}(0.1067 - 0.09033)(27.58 - 27.37) - \frac{1}{2}(0.09033)(27.37) - (0.09033)(27.58 - 27.37) \right. \\ \left. - (0.1067 - 0.0164)(0.21) \right]$$

$$E_M = (1.718)(W) \text{ Kip-in.}$$

FP-STIFF (OUTER)

$$E_M = 4 \left[(27.58)(0.1139) - \frac{1}{2}(0.48)(0.0245) - \frac{1}{2}(0.1139 - 0.0245)(27.58 - 0.48) - \frac{1}{2}(0.1139 - 0.0894)(27.58 - 27.10) \right. \\ \left. - \frac{1}{2}(0.0894)(27.10) - (0.0894)(27.58 - 27.10) - (0.1139 - 0.0245)(0.48) \right]$$

$$E_M = (2.484)(W) \text{ Kip-in.}$$

$$\Sigma E_M = \Sigma (\# \text{ Bearings})(E_M)(W)$$

$$\text{FPA: } \Sigma E_M = (12)(1.718)(913 \text{ K}) = 18,822 \text{ Kip-in.}$$

$$\text{FPB: } \Sigma E_M = (2)(2.484)(889 \text{ K}) = 4,417 \text{ Kip-in.}$$

$$\text{FPC: } \Sigma E_M = (8)(\quad)(810 \text{ K}) = 16,096 \text{ Kip-in.}$$

$$\text{FPD: } \Sigma E_M = (2)(\quad)(772 \text{ K}) = 3,835 \text{ Kip-in.}$$

$$\text{FPE: } \Sigma E_M = (4)(\quad)(1196 \text{ K}) = 11,883 \text{ Kip-in.}$$

$$\text{FPF: } \Sigma E_M = (4)(\quad)(598 \text{ K}) = 5,941 \text{ Kip-in.}$$

$$\Sigma E_M = 60,994 \text{ Kip-in.} \approx \boxed{61,000 \text{ Kip-in.}}$$

Viscous Damping of Isolation System

$$\beta_M = \frac{E_M}{2\pi K_M D_M^2} (100) \quad (\text{Eq. 17.8-7, ASCE 7-10})$$

$$\beta_M = \frac{61,000 \text{ Kip-in.}}{2\pi(112.5 \text{ K/in.})(27.58 \text{ in.})^2} (100) = \boxed{11.34\%} = 0.1134$$

$$B_M = 2.4(\beta_M)^{0.3} \quad (\text{Eq. 1, Morgan 2011})$$

$$B_M = 2.4(0.1134)^{0.3} = \boxed{1.25}$$

$$D_M' = \frac{S_{M1} T_M}{4\pi^2 B_M} g \quad (\text{Eq. 17.5-1, ASCE 7-10})$$

$$D_M' = \frac{(0.699)(5.04 \text{ sec.})}{4\pi^2(1.25)} (386.4 \text{ in./sec}^2) = \boxed{27.58 \text{ in.}} = 27.58 \text{ in.} = D_M \quad \checkmark$$

Design Works!

Friction Pendulum Isolator Design**DBE (Upper Bound) - Total Isolation System Calculations**Calculation of Damped/Isolated Response Spectrum

Assume lowest isolated period = $(0.8)(T_D)$ (Pg. 417, Zekioglu et al. 2009)

$$T_D = 4.47 \text{ [sec.]}$$

$$(0.8)(T_D) = 3.58 \text{ [sec.]} \quad \boxed{3.50 \text{ [sec.]}} \text{ (Lowest Isolated Period)}$$

Response Spectrum Scale Factor: $B = 4/[5.6 - \ln(100\beta_{eff})]$ (Eq. 1-13, ASCE 41-06)(Eq. 19, Zekioglu et al. 2009)

$$\beta_D = 21.59 \text{ \% Damping} \quad 0.2159$$

$$B = \boxed{1.58} \text{ (Isolated Response Spectrum Scale Factor)}$$

Calculation of Isolated Response Modification Coefficient and Isolated Design Forces

(17.6.3.3, ASCE 7-10) (17.5.4.2, ASCE 7-10)

(Table 12.2-1, ASCE 7-10) $1 \leq R_I \leq 2$

$$U1 \text{ (SMF): } R = 8 \quad R_I = (3/8)(R) = 3.0 \quad R_I = 2$$

$$U2 \text{ (SCBF): } R = 6 \quad R_I = (3/8)(R) = 2.3 \quad R_I = 2$$

$$K_D = 142.68 \text{ [k/in.]}$$

$$D_D = 13.46 \text{ [in.]}$$

$$U1 \text{ (SMF): } V_s = 960 \text{ [kips]} \quad V_s = K_D D_D / R_I \text{ (Eq. 17.5-8, ASCE 7-10)}$$

$$U2 \text{ (SCBF): } V_s = 960 \text{ [kips]} \quad V_s = K_D D_D / R_I \text{ (Eq. 17.5-8, ASCE 7-10)}$$

$$\text{Isolation System: } V_b = 1920 \text{ [kips]} \quad V_b = k_D D_D \text{ (Eq. 17.5-7, ASCE 7-10)}$$

(base-isolated) (fixed-base, from ETABS)

$$V_s = 960 \text{ [kips]} \leq 971 \text{ [kips]} = V$$

$$V_s = 960 \text{ [kips]} \leq 1690 \text{ [kips]} = V$$

Calculation of T_{D-all} (including isolation bearing weight)

$$M_{\text{bearings}} = 0.0216 \times 32 \text{ bearings} = 0.6912$$

$$W_{\text{bearings}} = 8.346 \text{ [kips]} \times 32 \text{ bearings} = 267 \text{ [kips]}$$

$$W_{\text{building}} = 27934 \text{ [kips]}$$

$$W_{All} = 28201 \text{ [kips]}$$

$$T_{D-all} = 4.49 \text{ [sec.]} \quad T_D = 2\pi \sqrt{W/(K_D * g)} \text{ (Eq. 17.5-2, ASCE 7-10)}$$

(Including Isolation Bearing Weight)

$$T_{D-all} = 4.49 \text{ [sec.]} \approx 4.47 \text{ [sec.]} = T_D$$

Friction Pendulum Isolator Design**MCE (Lower Bound) - Total Isolation System Calculations**Calculation of Lower Bound Properties

Ratios of Coefficients of Friction

Upper	1.1	(DBE)
Nominal	1	
Lower	0.9	(MCE)

Assume +/- 10% variation of coefficients of friction

MCE - Inner Bearings

Original Upper Bound (DBE) $\mu_{1-\max} = 0.0200 * W$ [kips]

Target Lower Bound (MCE) $\mu_{1-\min} = 0.0164 * W$ [kips]

$\mu_{1-\min} = \mu_{1-\max} * (0.9/1.1)$

MCE - Outer Bearings

Original Upper Bound (DBE) $\mu_{1-\max} = 0.0300 * W$ [kips]

Target Lower Bound (MCE) $\mu_{1-\min} = 0.0245 * W$ [kips]

$\mu_{1-\min} = \mu_{1-\max} * (0.9/1.1)$

Determination of Gap (Moat) Size, Including Torsion

$D_{M-\max} = 27.58$ in.

$D_{TM} = 30.34$ in.

$D_{TM} = 1.1 * D_{M-\max}$ [in.] (17.5.3.5, ASCE 7-10)

(FP Bearings have negligible torsion)

(Section 3.B.2, Morgan 2011)

From Time History $D_{M-\max} = 31.70$ in.

$D_{TM} = 34.87$ in.

$D_{TM} = 1.1 * D_{M-\max}$ [in.] (17.5.3.5, ASCE 7-10)

(GOVERNS) (FP Bearings have negligible torsion)

(Section 3.B.2, Morgan 2011)

Moat size = 36 in. = 3 ft.

Calculation of lumped bearing mass in PERFORM-3D model

$M_{\text{bearing}} = 0.0216$ [kip-sec.²/in.]

In PERFORM-3D: 32 bearings X $M_{\text{bearing}} = 0.6912$ [kip-sec.²/in.]

$M_{\text{lumped-total}} = 5.54 + 0.6912 = 6.2312$ [kip-sec.²/in.]

Calculation of $T_{M-\text{all}}$ (including isolation bearing weight)

$M_{\text{bearings}} = 0.0216 \times 32 \text{ bearings} = 0.6912$ [kip-sec.²/in.]

$W_{\text{bearings}} = 8.346$ [kips] x 32 bearings = 267 [kips]

$W = (m)(g), g = 386.4$ in/sec.²

$W_{\text{building}} = 27934$ [kips]

$W_{\text{All}} = 28201$ [kips]

$T_{M-\text{all}} = 5.06$ [sec.] $T_M = 2\pi * \text{SQRT}(W/(K_M * g))$ (Eq. 17.5-2, ASCE 7-10)

(Including Isolation Bearing Weight)

$T_{M-\text{all}} = 5.06$ [sec.] ≈ 5.04 [sec.] = T_M

Determination of Bearing Weight/Mass

Standard Bearing Diameters and Weights (Kravchuk et al. 2008)

$$\begin{array}{ll} D_3 = 3 \text{ [ft.]} & W_3 = 2 \text{ [kips]} \\ D_{13} = 13 \text{ [ft.]} & W_{13} = 40 \text{ [kips]} \end{array}$$

Determination of Bearing Diameter

$$\begin{array}{ll} D = 69.74 \text{ [in.]} & D = (2)(D_{TM}) \\ D = 5.81 \text{ [ft.]} & D = (D)(1 \text{ ft} / 12 \text{ in}) \\ D_6 \approx 6 \text{ [ft.]} & \end{array}$$

Determination of Bearing Weight/Mass (by best-fit polynomial curve)

$$\begin{array}{ll} D_6 = 6 \text{ [ft.]} & W_6 = 8.34 \text{ [kips]} \\ g = 386.4 \text{ [in./sec.}^2\text{]} & M_6 = 0.0216 \text{ [kip-sec.}^2\text{/in.]} \quad M = W/g \end{array}$$

Series 1

D [ft.]	W [kips]
0	0
3	2
13	40

Series 2

D [ft.]	W [kips]
0	0.00
3	2.00
6	8.34
13	40.00

$$W = 0.241(D)^2 - 0.0564(D)$$

J. HINGE PROPERTY EXAMPLE CALCULATIONS – SMF COLUMNS

Hinge Property Calculator FRAMES A992 Steel		= input value = reference pre-existing cell	
$F_y = 50$ ksi $R_y = F_y / F_e = 1.1$ $F_u = 65$ ksi $\phi_u = 0.90$ $E = 29000$ ksi	(Table 2-3, AISC 360-05) (Table 1-6.1, Seismic Manual) (Table 2-3, AISC 360-05) (bending & compression)	$F_y = F_y \cdot R_y$ $F_u = F_u \cdot R_y$ $E = E$	(Columns - pure tension) (Eq. 5-8, pg. 121, ASCE 41-06) (Columns - combined axial and bending) (Table 6-1, AISC 360-05) (Columns - X-axis bending) (Eq. 5-4, pg. 120, ASCE 41-06) (Columns - X-axis bending) (Eq. 5-4, pg. 120, ASCE 41-06) (Columns - X-axis Yield rotation) (Eq. 5-2, pg. 120, ASCE 41-06) (Columns - Y-axis Yield rotation) (Eq. 5-2, pg. 120, ASCE 41-06)
$F_y = 55$ ksi $F_u = 70$ ksi	$F_y = F_y \cdot R_y$ $F_u = F_u \cdot R_y$	$P_c = F_y \cdot A$ $P_t = 1 / (p \cdot \phi_u)$ $M_u = 1.18 \cdot Z_u \cdot F_y \cdot (1 - P/P_{pl}) \leq F_y \cdot Z_u$ $M_y = 1.18 \cdot Z_y \cdot F_y \cdot (1 - P/P_{pl}) \leq F_y \cdot Z_y$ $\theta_{y, yield} = (Z_u \cdot F_y \cdot A_u) / (6 \cdot E \cdot I_y) \cdot (1 - P/P_{pl})$ $\theta_{x, yield} = (Z_y \cdot F_y \cdot A_y) / (6 \cdot E \cdot I_x) \cdot (1 - P/P_{pl})$	
Moment Frame Columns W27X217 $A = 64$ in ² $Z_u = 711$ in ³ $Z_y = 154$ in ³ $p = 0.419 \times 10^{-3}$ $I_u = 8910$ in ⁴ $I_y = 704$ in ⁴	length = 13.33 feet =	160 inches W27X194 $A = 57.2$ in ² $Z_u = 631$ in ³ $Z_y = 136$ in ³ $p = 0.47 \times 10^{-3}$ $I_u = 7860$ in ⁴ $I_y = 619$ in ⁴	Assumed P/P_{pl} ratio = 0.19 for moment frame columns W27X178 $A = 52.5$ in ² $Z_u = 570$ in ³ $Z_y = 122$ in ³ $p = 0.515 \times 10^{-3}$ $I_u = 7020$ in ⁴ $I_y = 555$ in ⁴
ASCE-41 Values		ASCE-41 Values	
$P_t = 3520$ kips $P_c = 2652$ kips $M_u = 37377$ Kip-in $M_y = 8096$ Kip-in $\theta_{y, yield} = 0.00327$ rad. $\theta_{x, yield} = 0.00896$ rad.	$\leq$ $\leq$	$P_t = 3146$ kips $P_c = 2364$ kips $M_u = 33171$ Kip-in $M_y = 7149$ Kip-in $\theta_{y, yield} = 0.00329$ rad. $\theta_{x, yield} = 0.00900$ rad.	$P_t = 2888$ kips $P_c = 2157$ kips $M_u = 29964$ Kip-in $M_y = 6413$ Kip-in $\theta_{y, yield} = 0.00333$ rad. $\theta_{x, yield} = 0.00901$ rad.
Shear Strength Section $V_n = 708$ kips $b_f = 14.1$ in. $t_f = 1.50$ in. $A_f = 42.300$ in. ² $V_{ns} = 778.8$ kips $V_{ns} = 1395.9$ kips		Shear Strength Section $V_n = 632$ kips $b_f = 14.0$ in. $t_f = 1.34$ in. $A_f = 37.520$ in. ² $V_{ns} = 695.2$ kips $V_{ns} = 1238.2$ kips	
Strong-Axis Shear Calculation (V_n from Table 3-2, AISC 360-05) ($\phi_v = 1.00$) $V_n = V_n \cdot [F_y / F_u]$ Weak-Axis Shear Calculations (Chapter G [16.1.65], [16.1.68] AISC 360-05) $V_{ns} = 0.6 \cdot F_u \cdot A_f \cdot C_u$ $A_f = 2 \cdot b_f \cdot t_f$ Assume $C_u = 1.0$		Shear Strength Section $V_n = 605$ kips $b_f = 14.1$ in. $t_f = 1.19$ in. $A_f = 33.558$ in. ² $V_{ns} = 665.5$ kips $V_{ns} = 1107.4$ kips	

(TABLE 5-6, pg. 116, ASCE 41-06)

$$P/P_{ye} \text{ ratio} = 0.19 < 0.2$$

(Table 1-1, AISC 360-05)

$$b_f/(2*t_f) = 4.71 \leq 7.01 = 52/\text{SQRT}(F_y E)$$

$$h/(t_w) = 28.70 \leq 40.45 = 300/\text{SQRT}(F_y E)$$

(Use Columns - Flexure - (a), Table 5-6, ASCE 41-06)

W27X217					
BENDING			BENDING		
X-Dir.		$M_x = 37377$ Kip-in	Y-Dir.		$M_y = 8096$ Kip-in
		$\theta_{x-yield} = 0.00327$ rad.			$\theta_{y-yield} = 0.00896$ rad.
IO	1	$\theta_{x-yield} = 0.00327$ rad.	IO	1	$\theta_{y-yield} = 0.00896$ rad.
LS	6	$\theta_{x-yield} = 0.01961$ rad.	LS	6	$\theta_{y-yield} = 0.05377$ rad.
CP	8	$\theta_{x-yield} = 0.02615$ rad.	CP	8	$\theta_{y-yield} = 0.07169$ rad.
a = DU	9	$\theta_{x-yield} = 0.02942$ rad.	a = DU	9	$\theta_{y-yield} = 0.08065$ rad.
b = DX	11	$\theta_{x-yield} = 0.03596$ rad.	b = DX	11	$\theta_{y-yield} = 0.09857$ rad.
c = FR/FU	0.6		c = FR/FU	0.6	
DL	1.01	$\theta_{x-yield} = 0.02971$ rad.	DL	1.01	$\theta_{y-yield} = 0.08146$ rad.
AXIAL			AXIAL		
TENSION			COMP.		
a = DU	5	$\Delta_y = 2.276$ in.	a = DU	1.00	$\Delta_y = 0.343$ in.
b = DX	7	$\Delta_y = 3.186$ in.	b = DX	8.00	$\Delta_y = 2.743$ in.
c = FR/FU	1.0		c = FR/FU	0.50	

$$\text{length} = 240 \text{ inches}$$

$$A = 64 \text{ in}^2$$

$$E = 29000 \text{ ksi}$$

$$DR/DL = 1.01 \text{ (assumed)}$$

$$\theta_{y-yield}/\theta_{x-yield} = 2.741$$

$$\Delta_y = P*L/(A*E)$$

(yield displacement)

(TABLE 5-7, pg. 119, ASCE 41-06)

(See Columns - Axial Comp. Sheet)

$$M_{ultimate} = 37377 \text{ kip-in. (referenced above)}$$

$$E_{\text{strain hardening}}/E_{\text{elastic}} \text{ Ratio} = 0.03 \text{ (assumed)}$$

$$M_{yield}/M_{ultimate} = 0.9993$$

$$M_{yield}/M_{ultimate} = (M_{ultimate} - a * 0.03 * E) / M_{ultimate}$$

(TABLE 5-6, pg. 116, ASCE 41-06)

$$P/P_{ye} \text{ ratio} = 0.19 < 0.2$$

(Table 1-1, AISC 360-05)

$$b_f/(2*t_f) = 5.24 \leq 7.01 = 52/\text{SQRT}(F_y E)$$

$$h/(t_w) = 31.80 \leq 40.45 = 300/\text{SQRT}(F_y E)$$

(Use Columns - Flexure - (a), Table 5-6, ASCE 41-06)

W27X194					
BENDING			BENDING		
X-Dir.		$M_x = 33171$ Kip-in	Y-Dir.		$M_y = 7149$ Kip-in
		$\theta_{x-yield} = 0.00329$ rad.			$\theta_{y-yield} = 0.00900$ rad.
IO	1	$\theta_{x-yield} = 0.00329$ rad.	IO	1	$\theta_{y-yield} = 0.00900$ rad.
LS	6	$\theta_{x-yield} = 0.01973$ rad.	LS	6	$\theta_{y-yield} = 0.05400$ rad.
CP	8	$\theta_{x-yield} = 0.02631$ rad.	CP	8	$\theta_{y-yield} = 0.07200$ rad.
a = DU	9	$\theta_{x-yield} = 0.02960$ rad.	a = DU	9	$\theta_{y-yield} = 0.08100$ rad.
b = DX	11	$\theta_{x-yield} = 0.03618$ rad.	b = DX	11	$\theta_{y-yield} = 0.09901$ rad.
c = FR/FU	0.6		c = FR/FU	0.6	
DL	1.01	$\theta_{x-yield} = 0.02989$ rad.	DL	1.01	$\theta_{y-yield} = 0.08181$ rad.
AXIAL			AXIAL		
TENSION			COMP.		
a = DU	5	$\Delta_y = 2.276$ in.	a = DU	1.00	$\Delta_y = 0.342$ in.
b = DX	7	$\Delta_y = 3.186$ in.	b = DX	8.00	$\Delta_y = 2.736$ in.
c = FR/FU	1.0		c = FR/FU	0.50	

$$\text{length} = 240 \text{ inches}$$

$$A = 57.2 \text{ in}^2$$

$$E = 29000 \text{ ksi}$$

$$DR/DL = 1.01 \text{ (assumed)}$$

$$\theta_{y-yield}/\theta_{x-yield} = 2.737$$

$$\Delta_y = P*L/(A*E)$$

(yield displacement)

(TABLE 5-7, pg. 119, ASCE 41-06)

(See Columns - Axial Comp. Sheet)

$$M_{ultimate} = 33171 \text{ kip-in. (referenced above)}$$

$$E_{\text{strain hardening}}/E_{\text{elastic}} \text{ Ratio} = 0.03 \text{ (assumed)}$$

$$M_{yield}/M_{ultimate} = 0.9992$$

$$M_{yield}/M_{ultimate} = (M_{ultimate} - a * 0.03 * E) / M_{ultimate}$$

K. HINGE PROPERTY EXAMPLE CALCULATIONS – SMF BEAMS

Moment Frame Beams $M_x = F_{ye} \cdot Z_x$ (Beams - X-axis bending)(Eq. 5-3, pg. 120, ASCE 41-06) $\theta_{x,yield} = [Z_x \cdot F_{ye} \cdot L] / [6 \cdot E \cdot I_x]$ length = 30 feet = 360 inches (Beams - X-axis Yield rotation) (Eq. 5-1, pg. 120, ASCE 41-06)			
W24X131	W24X146	W30X108	W30X116
$Z_x = 370 \text{ in}^3$ $I_x = 4020 \text{ in}^4$ $V_n = 444 \text{ kips}$	$Z_x = 418 \text{ in}^3$ $I_x = 4580 \text{ in}^4$ $V_n = 482 \text{ kips}$	$Z_x = 346 \text{ in}^3$ $I_x = 4470 \text{ in}^4$ $V_n = 488 \text{ kips}$	$Z_x = 378 \text{ in}^3$ $I_x = 4930 \text{ in}^4$ $V_n = 509 \text{ kips}$
ASCE-41 Values			
$M_x = 20350 \text{ Kip-in}$ $\theta_{x,yield} = 0.01047 \text{ rad.}$ $V_{rx} = 488.4 \text{ kips}$	$M_x = 22990 \text{ Kip-in}$ $\theta_{x,yield} = 0.01039 \text{ rad.}$ $V_{rx} = 530.2 \text{ kips}$	$M_x = 19030 \text{ Kip-in}$ $\theta_{x,yield} = 0.00881 \text{ rad.}$ $V_{rx} = 536.8 \text{ kips}$	$M_x = 20790 \text{ Kip-in}$ $\theta_{x,yield} = 0.00872 \text{ rad.}$ $V_{rx} = 559.9 \text{ kips}$

Strong-Axis Shear Calculation (V_n from Table 3-2, AISC 360-05)($\phi_v = 1.00$)
 $V_{rx} = V_n \cdot (F_{ye}/F_y)$

(Table 1-1, AISC 360-05) (TABLE 5-6, pg. 116, ASCE 41-06)
 $b_f / (2 \cdot t_f) = 6.17 \leq 7.01 = 52 / \sqrt{F_{ye}}$
 $h / (t_w) = 47.80 \leq 56.36 = 418 / \sqrt{F_{ye}}$
(Use Beams - Flexure - (a), Table 5-6, ASCE 41-06)

BENDING		W30X116	W30X108
X-Dir.	$M_{x,yield}$	20722 Kip-in	19958 Kip-in
	$M_{x,ultimate}$	20790 Kip-in	19958 Kip-in
	$\theta_{x,yield}$	0.00872 rad.	0.00872 rad.
IO	$1 \cdot \theta_{x,yield}$	0.00872 rad.	0.00872 rad.
LS	$6 \cdot \theta_{x,yield}$	0.05235 rad.	0.05235 rad.
CP	$8 \cdot \theta_{x,yield}$	0.06980 rad.	0.06980 rad.
a = DU	$9 \cdot \theta_{x,yield}$	0.07852 rad.	0.07852 rad.
b = DX	$11 \cdot \theta_{x,yield}$	0.09597 rad.	0.09597 rad.
c = FR/FU	0.6		
DL	$1.01 \cdot \theta_{x,yield}$	0.07931 rad.	0.07931 rad.
DR	$1.01 \cdot \theta_{x,yield}$	0.08010 rad.	0.08010 rad.

$E = 29000 \text{ ksi}$

$M_{ultimate} = 20790 \text{ kip-in. (referenced above)}$

$E_{strain\text{Hardening}}/E_{elastic} \text{ Ratio} = 0.03 \text{ (assumed)}$

$M_{yield}/M_{ultimate} = 0.9967$

$M_{yield} = 20722 \text{ kip-in.}$

$M_{yield}/M_{ultimate} = (M_{ultimate} \cdot a \cdot 0.03 \cdot E) / M_{ultimate}$

$M_{yield} = (M_{ultimate} \cdot a \cdot 0.03 \cdot E)$

L. HINGE PROPERTY EXAMPLE CALCULATIONS – SCBF COLUMNS

Braced Frame Columns		
$P_t = F_{ye} \cdot A$ $P_c = 1/(\rho \cdot \phi_c)$ $M_x = 1.18 \cdot Z_x \cdot F_{ye} \cdot (1-P/P_{yc}) \leq F_{ye} \cdot Z_x$ $M_y = 1.18 \cdot Z_y \cdot F_{ye} \cdot (1-P/P_{yc}) \leq F_{ye} \cdot Z_y$ $\theta_{x-yield} = [Z_x \cdot F_{ye} \cdot L] / (6 \cdot E \cdot I_x) \cdot (1-P/P_{yc})$ $\theta_{y-yield} = [Z_y \cdot F_{ye} \cdot L] / (6 \cdot E \cdot I_y) \cdot (1-P/P_{yc})$		
length = 13.33 feet = 160 inches	Assumed P/P_{yc} ratio = 0.45 for braced frame columns	
W18X234	W18X211	W18X192
$A = 68.8 \text{ in}^2$ $Z_x = 549 \text{ in}^3$ $Z_y = 149 \text{ in}^3$ $\rho = 0.416 \times 10^{-3} = 0.000416$ $I_x = 4900 \text{ in}^4$ $I_y = 558 \text{ in}^4$	$A = 62.1 \text{ in}^2$ $Z_x = 490 \text{ in}^3$ $Z_y = 132 \text{ in}^3$ $\rho = 0.464 \times 10^{-3} = 0.000464$ $I_x = 4330 \text{ in}^4$ $I_y = 493 \text{ in}^4$	$A = 56.4 \text{ in}^2$ $Z_x = 442 \text{ in}^3$ $Z_y = 119 \text{ in}^3$ $\rho = 0.514 \times 10^{-3} = 0.000514$ $I_x = 3870 \text{ in}^4$ $I_y = 440 \text{ in}^4$
ASCE 41 Values	ASCE 41 Values	ASCE 41 Values
$P_t = 3784 \text{ kips}$ $P_c = 2671 \text{ kips}$ $M_x = 19597 \text{ Kip-in}$ $M_y = 5319 \text{ kip-in}$ $\theta_{x-yield} = 0.00312 \text{ rad.}$ $\theta_{y-yield} = 0.00743 \text{ rad.}$	$P_t = 3415.5 \text{ kips}$ $P_c = 2395 \text{ kips}$ $M_x = 17491 \text{ Kip-in}$ $M_y = 4712 \text{ kip-in}$ $\theta_{x-yield} = 0.00315 \text{ rad.}$ $\theta_{y-yield} = 0.00745 \text{ rad.}$	$P_t = 3102 \text{ kips}$ $P_c = 2162 \text{ kips}$ $M_x = 15777 \text{ Kip-in}$ $M_y = 4248 \text{ kip-in}$ $\theta_{x-yield} = 0.00318 \text{ rad.}$ $\theta_{y-yield} = 0.00752 \text{ rad.}$
Shear Strength Section	Shear Strength Section	Shear Strength Section
$V_n = 733 \text{ kips}$ $b_f = 11.7 \text{ in.}$ $t_f = 2.11 \text{ in.}$ $A_f = 49.374 \text{ in}^2$ $V_m = 806.3 \text{ kips}$ $V_{mv} = 1629.3 \text{ Kips}$	$V_n = 657 \text{ kips}$ $b_f = 11.6 \text{ in.}$ $t_f = 1.91 \text{ in.}$ $A_f = 44.312 \text{ in}^2$ $V_m = 722.7 \text{ kips}$ $V_{mv} = 1462.3 \text{ Kips}$	$V_n = 586 \text{ kips}$ $b_f = 11.5 \text{ in.}$ $t_f = 1.75 \text{ in.}$ $A_f = 40.250 \text{ in}^2$ $V_m = 644.6 \text{ kips}$ $V_{mv} = 1328.3 \text{ Kips}$
Strong-Axis Shear Calculation (V_n from Table 3-2, AISC 360-05) ($\phi_v = 1.00$) $V_n = V_n \cdot (F_y/F_u)$ Weak-Axis Shear Calculations (Chapter G [16.1-65],[16.1-68] AISC 360-05) $V_m = 0.6 \cdot F_y \cdot A_f \cdot C_v$ $A_f = 2 \cdot b_f \cdot t_f$ Assume $C_v = 1.0$		

(TABLE 5-6, pg. 116, ASCE 41-06)

(Table 1-1, AISC 360-05)

$$0.2 \leq \frac{P}{P_{ye}} = 0.45 \leq 0.5$$

$$b_f/(2t_f) = 2.76 \leq 7.01 = 52/\text{SQRT}(F_y)$$

$$h/(t_w) = 13.80 \leq 35.06 = 260/\text{SQRT}(F_y)$$

(Use Columns - Flexure - (a), Table 5-6, ASCE 41-06)

W18X234			
BENDING X-Dir.	$M_x = 19597$ Kip-in $\theta_{x-yield} = 0.00312$ rad.	BENDING Y-Dir.	$M_y = 5319$ Kip-in $\theta_{y-yield} = 0.00743$ rad.
IO	1 * $\theta_{x-yield} = 0.00312$ rad.	IO	1 * $\theta_{y-yield} = 0.00743$ rad.
LS	6 * $\theta_{x-yield} = 0.01870$ rad.	LS	6 * $\theta_{y-yield} = 0.04457$ rad.
CP	8 * $\theta_{x-yield} = 0.02493$ rad.	CP	8 * $\theta_{y-yield} = 0.05942$ rad.
a = DU	9 * $\theta_{x-yield} = 0.02805$ rad.	a = DU	9 * $\theta_{y-yield} = 0.06685$ rad.
b = DX	11 * $\theta_{x-yield} = 0.03428$ rad.	b = DX	11 * $\theta_{y-yield} = 0.08170$ rad.
c = FR/FU	0.6	c = FR/FU	0.6
DL	1.01 * DU = 0.02833 rad.	DL	1.01 * DU = 0.06752 rad.
AXIAL TENSION	Pt = 3784 kips $\Delta_{y-tens} = 0.455$ in.	AXIAL COMP.	Pc = 2671 kips $\Delta_{y-comp} = 0.321$ in.
a = DU	5 * $\Delta_y = 2.276$ in.	a = DU	1.00 * $\Delta_y = 0.321$ in.
b = DX	7 * $\Delta_y = 3.186$ in.	b = DX	8.00 * $\Delta_y = 2.570$ in.
c = FR/FU	1.0	c = FR/FU	0.50

(TABLE 5-7, pg. 119, ASCE 41-06)

(See Columns - Axial Comp. Sheet)

$$M_{ultimate} = 19597 \text{ kip-in. (referenced above)}$$

$$E_{strain \text{ Hardening}}/E_{elastic} \text{ Ratio} = 0.03 \text{ (assumed)}$$

$$M_{yield}/M_{ultimate} = 0.9988$$

$$M_{yield}/M_{ultimate} = (M_{ultimate} \cdot a \cdot 0.03 \cdot E)/M_{ultimate}$$

$$\text{length} = 240 \text{ inches}$$

$$A = 68.8 \text{ in}^2$$

$$E = 29000 \text{ ksi}$$

$$DR/DL = 1.01 \text{ (assumed)}$$

$$\theta_{y-yield}/\theta_{x-yield} = 2.383$$

$$\Delta_y = P \cdot L / (A \cdot E)$$

(yield displacement)

M. HINGE PROPERTY EXAMPLE CALCULATIONS – SCBF BEAMS

Braced Frame Beams

$$M_x = F_{ye} * Z_x \quad (\text{Beams - X-axis bending})(\text{Eq. 5-3, pg. 120, ASCE 41-06})$$

$$\theta_{x\text{-yield}} = (Z_x * F_{ye} * L) / (6 * E * I_x) \quad (\text{Beams - X-axis Yield rotation})(\text{Eq. 5-1, pg. 120, ASCE 41-06})$$

$$\text{length} = 30 \text{ feet} = 360 \text{ inches}$$

W16X45	W16X31
$Z_x = 82.3 \text{ in}^3$ $I_x = 586 \text{ in}^4$ $V_n = 167 \text{ kips}$	$Z_x = 54 \text{ in}^3$ $I_x = 375 \text{ in}^4$ $V_n = 131 \text{ kips}$
ASCE-41 Values	ASCE-41 Values
$M_x = 4527 \text{ Kip-in}$ $\theta_{x\text{-yield}} = 0.01598 \text{ rad.}$ $V_{nx} = 183.7 \text{ kips}$	$M_x = 2970 \text{ Kip-in}$ $\theta_{x\text{-yield}} = 0.01639 \text{ rad.}$ $V_{nx} = 144.1 \text{ kips}$

Strong-Axis Shear Calculation (V_n from Table 3-2, AISC 360-05)($\phi_v = 1.00$)

$$V_{nx} = V_n * (F_{ye} / F_y)$$

(Table 1-1, AISC 360-05)

(TABLE 5-6, pg. 116, ASCE 41-06)

$$b_f / (2 * t_f) = 6.23 \leq 7.01 = 52 / \text{SQRT}(F_{ye})$$

$$h / (t_w) = 41.10 \leq 56.36 = 418 / \text{SQRT}(F_{ye})$$

(Use Beams - Flexure - (a) , Table 5-6, ASCE 41-06)

W16X45			.096 * Mu
BENDING			
X-Dir.			
		$M_{x\text{-yield}} = 4401 \text{ Kip-in}$	$M_{x\text{-yield-min}} = 4345 \text{ Kip-in}$
		$M_{x\text{-ultimate}} = 4527 \text{ Kip-in}$	
		$\theta_{x\text{-yield}} = 0.01598 \text{ rad.}$	
IO	1	$*\theta_{x\text{-yield}} = 0.01598 \text{ rad.}$	
LS	6	$*\theta_{x\text{-yield}} = 0.09589 \text{ rad.}$	
CP	8	$*\theta_{x\text{-yield}} = 0.12785 \text{ rad.}$	
a = DU	9	$*\theta_{x\text{-yield}} = 0.14383 \text{ rad.}$	
b = DX	11	$*\theta_{x\text{-yield}} = 0.17580 \text{ rad.}$	
c = FR/FU	0.6		
DL	1.01	$*DU = 0.14527 \text{ rad.}$	
DR	1.02	$*DL = 0.14818 \text{ rad.}$	

$$E = 29000 \text{ ksi}$$

$$M_{\text{ultimate}} = 4527 \text{ kip-in. (referenced above)}$$

$$E_{\text{Strain Hardening}} / E_{\text{Elastic}} \text{ Ratio} = 0.03 \text{ (assumed)}$$

$$M_{\text{yield}} / M_{\text{ultimate}} = 0.9724 \quad M_{\text{yield}} / M_{\text{ultimate}} = (M_{\text{ultimate}} - a * 0.03 * E) / M_{\text{ultimate}}$$

$$M_{\text{yield}} = 4401 \text{ kip-in.} \quad M_{\text{yield}} = (M_{\text{ultimate}} - a * 0.03 * E)$$

N. HINGE PROPERTY EXAMPLE CALCULATIONS – SCBF BRACES

BRACES (HSS)

A500 Gr. B Steel

$$F_y = 46 \text{ ksi}$$

(Table 2-3, AISC 360-05)

$$R_y = F_y / F_y = 1.4$$

(Table I-6.1, Seismic Manual)

$$F_u = 58 \text{ ksi}$$

(Table 2-3, AISC 360-05)

$$\phi_b = \phi_c = 0.90$$

(bending & compression)

$$F_{yw} = 64.4 \text{ ksi}$$

$$F_{yw} = F_y \cdot R_y$$

$$P_t = F_{yw} \cdot A$$

(pure tension)(Eq. 5-8, pg. 121, ASCE 41-06)

$$P_c = \phi_c \cdot P_u / \phi_c$$

(pure compression)(Table 4-4, AISC 360-05)

$$M_x = F_{ye} \cdot Z_x$$

(X-axis bending)(Eq. 5-4, pg. 120, ASCE 41-06) (No axial loads on braces)

$$M_y = F_{ye} \cdot Z_y$$

(Y-axis bending)(Eq. 5-4, pg. 120, ASCE 41-06) (No axial loads on braces)

$$\theta_{yield} = (Z \cdot F_{ye} \cdot L) / (6 \cdot E \cdot I) \cdot (1 - P/P_y)$$

(Yield rotation) (Eq. 5-2, pg. 120, ASCE 41-06)

$$\text{length} = 20.00 \text{ feet} = 240 \text{ inches}$$

Assumed P/P_{ye} ratio = 0 for braces

$$\text{HSS7X7X1/2}$$

$$\text{HSS8X8X5/8}$$

$$\text{ASCE-41 Values}$$

$$A = 11.6 \text{ in}^2$$

$$Z_x = 44.7 \text{ in}^3$$

$$Z_y = 27.9 \text{ in}^3$$

$$\phi \cdot P_c = 275$$

$$I_x = 80.5 \text{ in}^4$$

$$\text{ASCE-41 Values}$$

$$P_t = 747 \text{ kips}$$

$$P_c = 306 \text{ kips}$$

$$M_x = 1797 \text{ Kip-in}$$

$$M_y = 1797 \text{ kip-in}$$

$$\theta_{yield} = 0.02629 \text{ rad.}$$

$$d = 7 \text{ in.}$$

$$t_w = 0.465 \text{ in.}$$

$$h = 4.21 \text{ in.}$$

$$A_w = 1.958 \text{ in}^2$$

$$V_n = 75.6 \text{ Kips}$$

$$P_t = 1056 \text{ kips}$$

$$P_c = 488 \text{ kips}$$

$$M_x = 2879 \text{ Kip-in}$$

$$M_y = 2879 \text{ kip-in}$$

$$\theta_{yield} = 0.02323 \text{ rad.}$$

$$d = 8 \text{ in.}$$

$$t_w = 0.581 \text{ in.}$$

$$h = 4.514 \text{ in.}$$

$$A_w = 2.623 \text{ in}^2$$

$$V_n = 101.3 \text{ Kips}$$

Shear Calculations (Chapter G [16.1-65], AISC 360-05)

$$h = d \cdot 2 / (3 \cdot t_w)$$

$$A_w = h \cdot t_w$$

$$\text{Assume } C_y = 1.0$$

$$V_n = 0.6 F_{yw} A_w \cdot C_y$$

(Shear strength)

$$P/P_{ye} \text{ ratio} = 0 < 0.2$$

$$b/t = 12.10$$

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$$b/t = 12.10$$

O. ETABS AND PERFORM-3D PERIOD CHECK CALCULATIONS

U1 Direction (SMF) Fundamental Period Check

$k_w = 1.838$
 $V = 971$ kips

Level	W_i [kips]	h_i [ft]	h_i^3	w_i/h_i^3	C_{ux}	F_x [kips]	f_i	V_i [kips]
12	2249.5	160.00	11250.52	25308412	0.21931	212.9	212.9	212.9
11	2140.6	146.67	9588.16	20524100	0.17785	172.7	385.6	385.6
10	2140.6	133.33	8046.69	17224486	0.14926	144.9	530.6	530.6
9	2140.6	120.00	6630.33	14192665	0.12298	119.4	650.0	650.0
8	2140.6	106.67	5340.01	11430640	0.09905	96.2	746.2	746.2
7	2140.6	93.33	4177.33	8941861	0.07748	75.2	821.4	821.4
6	2140.6	80.00	3146.87	6736095	0.05837	56.7	878.1	878.1
5	2140.6	66.67	2251.04	4818513	0.04175	40.5	918.6	918.6
4	2140.6	53.33	1493.39	3196705	0.02770	26.9	945.5	945.5
3	2140.6	40.00	880.21	1884148	0.01633	15.9	961.4	961.4
2	2140.6	26.67	417.86	894456	0.00775	7.5	968.9	968.9
1	2140.6	13.33	116.80	250015	0.00217	2.1	971.0	971.0
Σ	25796		115402098					

Level	δi (UX)	$f_i \delta_i^2$	$f_i \delta_i$	$w_i \delta_i^2$
12	7.04	10558.47	1499.46	111537.97
11	6.81	8015.79	1176.54	99358.63
10	6.46	6047.51	936.19	89320.99
9	5.99	4284.30	715.28	76796.07
8	5.42	2827.35	521.47	62926.25
7	4.79	1724.81	360.24	49072.38
6	4.10	950.85	232.15	35911.00
5	3.36	457.83	136.24	24171.90
4	2.60	181.39	69.85	14435.75
3	1.84	53.55	29.14	7230.57
2	1.09	8.91	8.19	2534.82
1	0.41	0.35	0.85	353.02
Σ		35111.10	5685.60	573649.33

STORY DISPLACEMENTS

Level	δi (UX)	$f_i \delta_i$	$w_i \delta_i^2$
12	5.983	1274.06	80524.93
11	5.783	998.67	71587.17
10	5.489	795.51	64493.40
9	5.087	607.48	55392.67
8	4.621	444.44	45708.90
7	4.082	307.12	35667.68
6	3.484	197.47	25982.75
5	2.868	116.28	17607.07
4	2.215	59.58	10502.10
3	1.569	24.87	5269.56
2	0.9294	6.99	1848.99
1	0.3446	0.72	254.19
Σ		4833.19	414839.42

$g = 386.4$

$$\text{SQRT}[(\Sigma w_i \delta_i^2) / (g \Sigma f_i \delta_i)] = 0.4713$$

$$T = 3.21 \text{ secs} \quad T = 2\pi * \text{SQRT}[(\Sigma w_i \delta_i^2) / (g \Sigma f_i \delta_i)]$$

ASCE 7-5, Section 15.4.4

$$T = 2.96 \text{ secs} \quad T = 2\pi * \text{SQRT}[(\Sigma w_i \delta_i^2) / (g \Sigma f_i \delta_i)]$$

ASCE 7-5, Section 15.4.4