PERFORMANCE-BASED ANALYSIS OF A REINFORCED CONCRETE SHEAR
WALL BUILDING

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by

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Shear Wall Building

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ABSTRACT

PERFORMANCE-BASED ANALYSIS OF A REINFORCED CONCRETE SHEAR WALL BUILDING

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In this thesis, a special reinforced concrete shear wall building was designed per ASCE 7-05, and then the performance was investigated using the four analysis procedures outlined in ASCE 41-06. The proposed building was planned as a 6-story office building in San Francisco, CA. The structural system consisted of a two-way flat plate and reinforced concrete columns for gravity loads and slender structural walls for seismic loads. The mathematical building models utilized recommendations from ASCE 41-06 and first-principle mechanics. Moment-curvature analysis and fiber cross-section elements were used in developing the computer models for the nonlinear procedures. The results for the analysis procedures showed that the building met the Basic Safety Objective as defined in ASCE 41-06. The performance levels for the nonlinear procedures showed better building performance than for the linear procedures.

This paper addresses previously found data for similar studies which used steel special moment frames, special concentric braced frames, and buckling restrained braced frames for their primary lateral systems. The results showcase expected seismic performance levels for a commercial office building designed in a high seismicity region with varying structural systems and when using different analysis procedures.

Keywords: reinforced concrete structural walls, shear walls, performance-based analysis, ETABS, Perform-3D, flat plate, two-way slab.

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LIST OF NOMENCLATURE

A_g	= gross area of concrete section (in ²)
A_w	= gross area of concrete web (in ²)
b_{eff}	= slab width used to represent effective two-way slab strip thickness (in)
G_g	= elastic shear modulus of uncracked concrete (ksi)
G_{cr}	= reduced shear stiffness prior to steel yielding (ksi)
G_{eff}	= secant shear stiffness associated with limit state analysis (ksi)
E	= modulus of elasticity (ksi)
f'_c	= compressive strength of concrete (psi)
f_y	= steel yield strength (psi)
H_{eff}	= effective height of applied loading based on first mode shape of building (ft)
I_g	= gross moment of inertia of concrete section (in ⁴)
L_p	= plastic hinge length (in)
M	= moment demand at cross-section (k·ft)
M_a	= actual moment demand corresponding to limit state analysis (k·ft)
M_y	= yield moment associated with bilinear moment-curvature relationship (k·ft)
M_y'	= theoretical moment at which steel yielding occurs (k·ft)
V_u	= nominal shear strength (k)
V_c	= concrete shear strength contribution (k)
V_s	= steel reinforcing shear strength contribution (k)
V_p	= axial load shear strength contribution (k)
γ_{hv}	= shear strain of reinforced concrete section (in/in)

δ_t	= ASCE 41-06 target displacement (in)
ε_{cr}	= cracking tensile strain of concrete (in/in)
ε_{cu}	= ultimate compressive strain of concrete (in/in)
ε_h	= normal strain in horizontal wall reinforcement direction (in/in)
ε_{su}	= ultimate steel strain (in/in)
ε_v	= normal strain in vertical wall reinforcement direction (in/in)
ε_{45}	= strain at 45° to the wall reinforcement, in direction of principal compression strut (in/in)
ρ_v	= ratio of tie reinforcement area to area of gross concrete section (unitless)
ρ_s	= ratio of longitudinal reinforcement area to area of gross concrete section (unitless)
τ	= shear stress of reinforced concrete section (ksi)
ϕ	= curvature at cross-section (/in)
ϕ_y	= bilinearized yield curvature (/in)
ϕ_y'	= theoretical yield curvature (/in)

1.0 INTRODUCTION

The purpose of this project was to use American Society of Civil Engineers (ASCE) Standard 41-06 procedures to evaluate the performance of a reinforced concrete shear wall structure, located in San Francisco, California and designed per ASCE 7-05 equivalent lateral force procedure (ELFP) guidelines. The structural performance level was evaluated with the acceptance criteria and recommendations presented in ASCE 41-06.

This thesis has the potential to assist engineers in choosing a structural system for a desired level of seismic structural performance. Previous research has provided data for the same building plan, first designed with structural steel moment frames (Williams 2009), later designed with special steel concentric braced frames (Adams 2010), and finally designed with buckling restrained braced frames (Burkholder 2010). This paper offers reinforced concrete structural walls as a fourth alternative and provides a brief comparison of the results from the four systems.

In addition to offering comparative information on the performance of the four aforementioned structural systems, this paper addresses several other questions:

- It is generally accepted that a life-safe building is intended by the design recommendations provided in ASCE 7-05 when an importance factor of 1.0 is applied; how does this criteria correspond to the performance objectives outlined in ASCE 41-06 for a reinforced concrete shear wall building?

- Conservative results are anticipated when using the linear procedures outlined in ASCE 41-06 (2); how do the results from the different analytical procedures compare?
- What variance in analysis results might be associated with the extra time and computational expense required in developing a Perform-3D computer model versus an ETABS computer model?

1.1 Performance-Based Assessment

Section 1.4 of ASCE 7-05 and the corresponding commentary section define ASCE's intent to ensure global structural stability, while allowing for local damage (ASCE 7-05). ASCE 7-05 § 1.4 states that buildings should be designed such that the structure is not disproportionately damaged. The section further states that structural elements adjacent to locally damaged areas shall be "capable of resisting those loads without collapse." Although there is not a specific limit state defined for the intent of ASCE 7-05 guidelines, the objective described in Section 1.4 correlates well with what ASCE 41-06 defines as a Life Safety Performance Level, in which a structure may have "damaged components but retains a margin against onset of partial or total collapse" (ASCE 41-06).

Traditionally, the methodologies used to conform with ASCE 7's seismic design objectives are force-based. Design accelerations are taken from an elastic design spectrum and then multiplied by the structural weight for an elastic design force. Because damage is expected in the structure in the event of the design-based earthquake,

significant inelasticity is anticipated in the structure. ASCE 7-05 accounts for the expected inelasticity by dividing the elastic design force by an R-factor, which corresponds with the ductility and overstrength inherent to the chosen lateral force-resisting system.

Structural damage in a building can be related to structural inelastic deformation. The forces formulated by the ASCE 7-05 ELFP correspond to a displacement while still in the elastic range. Thus, a factor similar to the R-factor described earlier, is employed based on the principle of equal displacement, to approximate the maximum inelastic displacement expected under the design earthquake. A check in the structural design is made so that this displacement does not exceed the specified code drift limit. The specified limit is based on committee judgment to limit inelastic strains, to mitigate issues of stability, and to reduce threats against life safety from nonstructural damage (NEHRP 1997).

Recently, developments have been made toward a damage limit state design, in which performance objectives are assigned for the given structure and then evaluated based on the displacements and rotations of structural elements. ASCE 41-06 provides analysis procedures that attempt to more accurately capture inelastic demands than are typically computed using ASCE 7 design methodologies.

In this thesis, a special reinforced concrete shear wall building was designed to meet the life safety performance level that is implied by ASCE 7-05, using ELFP. The

structure was then evaluated using the four analysis procedures in ASCE 41-06, and then the structural performance levels for each analysis procedure were compared.

2.0 BACKGROUND

ASCE 41-06 provides four analytical procedures for evaluating the performance of a structure:

- Linear Static Procedure (LSP)
- Linear Dynamic Procedure (LDP)
- Nonlinear Static Procedure (NSP)
- Nonlinear Dynamic Procedure (NDP)

The procedures vary in complexity, and the required computational cost is generally in accordance with the order as listed. The linear static procedure is the most basic and economical, and the nonlinear dynamic procedure is the most detailed and requires the most computational expense. It is noted that designers typically neglect soil-structure interaction, which would require another added analysis cost.

There are various assumptions used in each of the listed ASCE 41-06 analyses. As the modeled behavior increases in detail, more validity is expected in the analysis. Because buildings are designed to endure material nonlinearities in the event of the design level earthquake, various factors must be employed when using the linear procedures for estimating inelastic seismic deformation demand, thus accuracy in predicting the building internal forces and deformations consequently decreases.

There are also assumptions employed for the first of the nonlinear procedures, the nonlinear static procedure. The nonlinear static procedure is typically only valid for first mode-dominated structures, because the procedure fails to accurately account for higher

mode dynamic effects. Additionally, the assumed loading diagram for the nonlinear static procedure should change as soon as the building undergoes material inelasticity since the dynamic properties of the structure change. The nonlinear dynamic procedure (analogous to a response history analysis) is generally accepted as providing the best estimates of structural behavior under seismic loading, since the NDP explicitly modifies element stiffness with each time-step iteration. Unfortunately, restrictions on time and engineering cost for typical design purposes inhibit designers from performing a response history analysis on all buildings. Also, more risk is involved in that inputs for hysteretic response and ground acceleration records may lead to inaccurate results if poorly chosen. Additionally, for short, regular buildings, higher modes are generally insignificant and thus the need for a detailed response history analysis may not be warranted.

2.1 Reinforced Concrete Structural Wall Design

In order to undergo local damage, while maintaining global stability, ASCE 7-05 requires structural elements adjacent to damaged regions to have the ability to transfer and resist loads without collapse. One way for this to be accomplished is through capacity-based design and detailing to ensure ductile behavior, in which the structural system is designed to displace past events of material yielding without significant degradation in strength. The design of the reinforced concrete (RC) structural walls in this project is based on a capacity-based approach, and ductile behavior is expected. The following section will serve to address some of design considerations used to ensure ductile response for RC structural walls.

In an effort to guarantee ductility in reinforced concrete (RC) structural wall systems, RC structural walls are typically designed so that flexural hinging forms prior to a more brittle and less predictable shear failure. Plastic hinges of the structural walls are expected to form at the base of the walls, where large deformations are expected specifically at the wall ends. Ductile response is anticipated in the longitudinal steel, in which the boundary element¹ longitudinal steel dissipates energy through yielding.

Without sufficient transverse reinforcement, shear failure can occur, and brittle failure is exhibited. Adequate transverse reinforcement was provided in this project to avoid a shear failure mode, in which case post-yield deformations can cause great loss in capacity. Figure A below is representative of an unfavorable shear failure mechanism in Kobe, Japan:

¹ Boundary element refers to the portion along a structural wall strengthened by longitudinal and transverse reinforcement (ACI 318-08)

Figure A: Concrete Wall Failing in Shear in 1995 Kobe Earthquake

Source: Maffei and Yuen 2007

In an additional effort to ensure ductility in concrete walls, efforts are typically made to prevent concrete crushing. ACI 318-08 requires confinement in boundary zones when structural walls don't have the ability to deform to their maximum displacement without exceeding ultimate concrete compressive strains. Adding confinement allows the concrete to exhibit higher compressive strains without a significant degradation in strength, as illustrated in the following figure:

Figure B: Stress-Strain Relationship for Concrete in Compression

Source: Prestley, Calvi, and Kowalski 2007

The use of confined boundary elements was employed in this project for concrete ductility as well as a preventative measure against rebar buckling. Sliding shear was checked at the wall's base as well, and dowels were added at cold joints to ensure the assumed failure mode.

2.2 Analytical Modeling

Both ETABS and Perform-3D were used for the analytical modeling in this thesis. ETABS was used for the two ASCE 41-06 linear analyses, as well as for the nonlinear static procedure. The nonlinear static procedure was repeated in Perform-3D, and the nonlinear dynamic procedure was performed solely in Perform-3D.

There were various considerations and assumptions put in to the mathematical building models. Some general considerations include rigid diaphragms, which were employed at all floor levels. Lumped masses were placed at the center of geometry for each of the corresponding levels; however, no mass was included at the basement levels. Also, p-delta ($P-\Delta$) effects were included, based on the total lumped mass at each level. Additional modeling parameters specific to the ETABS and Perform models and the different types of analyses are discussed in later sections.

2.2.1 Linear Analytical Modeling: Stiffness Modifiers

The linear static procedure and the linear dynamic procedure were performed using a 3-dimensional ETABS computer model. The ETABS model employed frame elements with effective cracked flexural stiffness and shear stiffness properties for members above grade level; uncracked properties were used for elements below grade. Frame elements were used in the ETABS computer model so that lumped plastic hinges could be assigned in walls when performing the nonlinear static procedure – an accepted modeling method per ASCE41-06.

The linear analyses utilized recommended stiffness properties from ASCE 41-06 Table 6-5, Supplement 1:

Table 1: Effective Stiffness Values
Source: ASCE 41-06 Supplement No.1

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams-nonprestressed	$0.3E_cI_g$	$0.4E_cA_w$	-
Beams-prestressed	E_cI_g	$0.4E_cA_w$	-
Columns with compression due to design gravity loads $\geq 0.5 A_g f'_c$	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns with compression due to design gravity loads $\leq 0.1A_g f'_c$ or with tension	$0.3E_cI_g$	$0.4E_cA_w$	E_cA_g
Beam-column joints	See Section 6.4.2.2.1		E_cA_g
Walls-cracked [†]	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Flat Slabs-nonprestressed	See Section 6.4.4.2	$0.4E_cA_g$	-
Flat Slabs-prestressed	See Section 6.4.4.2	$0.4E_cA_g$	-

Note: It shall be permitted to take I_g for T-beams as twice the value of I_g of the web alone. Otherwise, I_g shall be based on the effective width as defined in Section 6.3.1.3. For columns with axial compression falling between the limits provided, linear interpolation shall be permitted. If interpolation is not performed, the more conservative effective stiffnesses shall be used.

[†] See Section 6.7.2.2

2.2.2 Analytical Modeling: Two-way Flat Plate Effective Width

A two-way flat plate was used for the gravity-carrying structural system in the building design. There was an investigation into the slab contribution to the structural stiffness, as well as the moment transfer and rotation demands on the slab itself. Consequently, the slab was explicitly modeled on drag lines. Various studies were performed in order to capture the appropriate slab stiffness.

ASCE 41-06 recommends using an effective beam width model, an equivalent frame model, or a finite element model for capturing the lateral stiffness of a two-way slab (170). As with the frame elements used for modeling the concrete structural walls in the ETABS analysis, frame elements were used for the flat plate drag elements in order to

add nonlinear moment-rotation hinges and to minimize analysis run time. The equivalent frame method, outlined in Chapter 13 of ACI 318-08, was used to compute an appropriate frame stiffness for a typical slab-to-column frame assembly.

The basis for the equivalent frame method is that the shear and flexure in the slab outside of the column width are transferred to the column through transverse torsional elements running perpendicular to the primary slab span (ASCE 41-06). The rotational stiffness of the torsional element is found by dividing an applied unit moment by the member's corresponding average rotation (Vanderbilt and Corley 1983). The column-to-torsional member assembly is shown in Figure C below:

Figure C: Column-To-Torsional Member Assembly
Source: MacGregor 2009

The rotation at mid-span of the torsional member (θ_A in Figure C) is found by assuming that the applied moment is distributed linearly from a maximum magnitude at the column intersection to zero moment at mid-span:

Figure D: Distribution of Torque Per Unit Length Along Column Center Line

Source: MacGregor 2009

One-third of the computed rotation is then taken as an approximate average rotation over the torsional member's length (MacGregor 2009). With this methodology, an effective beam width of 11'-6" was determined for approximating the column-to-torsional member assembly stiffness, as illustrated in Figure E:

Figure E: Effective Beam Width Determined by Equivalent Frame Method

Source: MacGregor 2009

In terms of slab thickness, with a 24 inch wide column and a 12 inch thick slab, an 11.5 foot slab width would extend 4.75 times the slab thickness, $4.75h$, in either direction outside of the column width.

Further investigation was performed in order to confirm a suitable effective width for the flat plate. A model which used elastic shell elements, 4-node elements containing in-plane and out-of plane stiffness, was created for the typical slab-to-column assembly:

Figure F: Shell Element Model for Typical Slab-to-Column Assembly

Source: Author using ETABS v9.7.3

The shell element model utilized bay widths and member sizes consistent with the building design. A similar model, using only frame elements, was created in an effort to match the shell element model's assembly stiffness, shown below in Figure G. Both models employed gross section properties in an effort to match property assumptions used in the equivalent frame method approach.

Figure G: Effective Beam Width Frame Element Model

Source: Author using ETABS v9.7.3

In the frame element model, exterior slab lines were modeled with half the width of interior slab lines. An arbitrary load was applied at the top story of each of the models, and resulting drifts were compared. An appropriate effective width that attained an approximately equivalent stiffness for the two models was found to be 80 inches, or 2.33 times the slab depth outside of the column width. Corresponding roof lateral displacements were 4.85 inches and 4.76 inches for the frame model and shell model respectively, a 1.9% difference.

Examining the moment distribution along the column centerline may provide evidence for the narrower effective width found with the finite element model than that found with the equivalent frame methodology, shown in Figure H below:

Figure H: Moment Distribution of Slab Along Column Gridline

Source: Author using Microsoft Excel 2007

The ETABS output for moment distribution along the gridline degrades upon moving away from the column at a much faster rate than the assumed distribution by the equivalent frame method, in which the moment is assumed to drop to zero at mid-way in between column lines. This implies a more flexible torsional member, which may be the case for this particular building due to the thirty-foot spans, considered to be on the upper limits of a flat plate span.

ACI 318-08 and ASCE 41-06 recommend that slab reinforcing resisting flexure transferred to columns should be located within $1.5h$ and $2.5h$ on each side of the column, respectively:

Figure I: Effective Slab Width per ACI 318-08

Source: ACI 318-08

The $2.33h$ determined for an effective beam width in the finite element study falls within these recommended values, and so it was decided that a conservative value of approximately $2h$, or 72 inches, would be selected for the purpose of investigating moment transfer and stiffness contribution from the flat plate in this project. For the linear procedures, a stiffness modifier of 0.25 was then applied to the moment of inertia of the frame element to account for concrete cracking, consistent with ACI 318-08

recommendations. The nonlinear models utilized moment-curvature analysis for a flexural stiffness modifier.

2.2.3 Nonlinear Analytical Modeling: Flexural Behavior of Walls in ETABS

A moment curvature relationship was developed to determine the effective flexural stiffness properties of the frame elements in the nonlinear models. The material properties recommended by Priestley, Calvi and Kowalski in *Displacement-based Seismic Design of Structures* (2007) were used in deriving the moment-curvature relationship.

To determine the effective flexural stiffness of the wall frame elements, the flexural capacity at the base of the structure (where hinging is expected) was used to quantify the moment demand up the height of the building. For the portion of the wall expected to undergo yielding, the moment-curvature relationship was used to approximate a bilinear effective cracked cross-section flexural stiffness, using an equal energy methodology.

To employ equal-energy principles for the bilinear moment-curvature relationship, numerical integration was used to determine the total area under the theoretical moment-curvature response. A secant to the yield moment was drawn to represent the elastic flexural stiffness. Next, the post-yield stiffness line was created such that the final point at ultimate moment capacity met the elastic stiffness line where equal area would be encompassed under the bilinear relationship as with the theoretical

moment-curvature relationship. An illustration of this concept is shown in Figure J below:

Figure J: Bilinear Moment-Curvature Relationship

Source: Author using Autodesk AutoCAD Civil 3D 2011

In Figure J, M_y' denotes the point at which steel yields in the theoretical moment-curvature relationship, and M_y is the yield moment to be used in the bilinear moment-curvature relationship.

Where yielding was not anticipated at the capacity level, but cracking was expected, a secant was drawn to the corresponding moment demand using limit state analysis, M_a . See Figure K below:

Figure K: Effective stiffness properties for yielded and non-yielded frame elements

Source: Author using Microsoft Excel 2007

The moment-curvature relationship was also used in developing the nonlinear hinge properties defined in ETABS and Perform-3D. In ETABS, nonlinear flexural hinge properties must be input as moment-rotation relationships, rather than moment-curvature relationships. Although Perform-3D allows for moment-curvature hinges, modeling consistency was achieved in using moment-rotation relationships for both platforms. The remainder of this section serves to show how the moment-curvature relation was used to formulate a moment-rotation response for computer modeling. It is noted that the degraded portion of the moment-rotation response was consistent with ASCE 41-06 recommendations.

To develop a back bone response for the elements that was consistent with ASCE 41-06, chord rotations were defined, as displayed in Figure L below. In Figure L, θ is

defined as the total elastic and plastic rotation of the member, θ_y is the rotation at yield, Δ is the total elastic and plastic displacement, and Δ_y is the yield displacement:

Figure L: Chord Rotation per ASCE 41-06

Source: ASCE 41-06

To determine Δ and Δ_y , moment-area was employed, in which the following assumptions were used for the curvature distribution over the wall's height. A linear moment-distribution was assumed, in which a point of maximum moment, M_u , is located at the base of the wall; and a point of zero moment is located at the building's resultant force, based on the building's first fundamental mode shape. The bilinear moment-curvature response was then used to approximate the curvature distribution over the wall's height, as shown in Figure M below:

Figure M: Derivation of Plastic Hinge Length

Source: Author using Autodesk AutoCAD Civil 3D 2011

First principles were used to derive the length of the “plastic hinge,” L_p , to be lumped at a distance $L_p/2$ from the point of ultimate moment demand. The plastic hinge represents a region of lumped plasticity, over which there is a constant curvature demand equal to the section’s ultimate curvature capacity, ϕ_u . The hinge length was approximated as half the distance over which yielding was anticipated in the wall so that integration of the bilinear curvature distribution over the height of the wall would yield similar results to that of the theoretical curvature-distribution. A distance equal to 10% of the wall’s length was added to the calculated hinge length to account for tension shift (Preistley, Calvi, and Kowalski 2007).

It should be noted that for the walls in this project, a strain penetration term was not necessary for the calculation of L_p . In this building, there were two basement levels below grade; thus, the region of plasticity was not extended to within the foundation, as is

typically seen with RC structural walls. The moment distribution was assumed to be consistent with Figure N below:

Figure N: Assumed Moment Distribution

Source: Author using Autodesk AutoCAD Civil 3D 2011

The plastic hinges for the walls in this project were located at the centers of regions at which yielding was anticipated.

2.2.4 Nonlinear Analytical Modeling: Shear Stiffness

Table 6-5 in ASCE 41-06 recommends $0.4 \cdot E$ for the shear modulus, shown previously in Table 1. This value is reasonable prior to shear cracking, however after cracking occurs, $0.4 \cdot E$ greatly overestimates a wall's shear stiffness (Powell *Tall Shear Wall Building* 2007). The walls in this project are slender (aspect ratio equal to 2.8), so

shear deformations were not greatly significant; however, material mechanics was used in order to determine a more appropriate shear stiffness than $0.4E$.

For shear stiffness modeling parameters, the shear demand was approximated from the flexural strength of the wall. It should be noted that the shear walls in this project were designed to remain elastic as the walls reached their ultimate flexural strength. Shear stresses were monitored throughout the analyses to ensure that shear demands did not exceed the shear wall capacities. Where shear cracking was anticipated, a secant was taken to the wall's shear stress demand. The shear demands were based on the walls' nominal flexural capacity, in which the height of the applied load was taken as H_{eff} , defined in Figure M. This idea is illustrated in Figure O below:

Figure O: Structural Wall Shear Stress-Strain Plot

Source: Author using Microsoft Excel 2007

Recommendations from *Displacement-Based Seismic Design of Structures* were used for calculating the wall nominal shear strength, in which effects from curvature demand and axial load are included (2007). The nominal shear strength, V_n , is equal to the summation of contributions from the concrete, V_c , the shear reinforcing, V_s , and the axial load, V_p :

$$V_n = V_c + V_s + V_p \quad \text{Eq 1.}$$

In *Accounting for Shear in Seismic Analysis of Concrete Structures*, Adebar and Gerin suggest an equation for determining the shear strain at yielding of the shear reinforcement, based on the quantity of reinforcement and the applied axial stress (2004). The proposed model for shear strain at yield utilizes principles of strain compatibility and

force equilibrium. A strain rosette methodology is employed, where the shear strain, γ_{hv} , can be determined from a strain transformation equation:

$$\gamma_{hv} = \varepsilon_h + \varepsilon_v - 2\varepsilon_{45} \quad \text{Eq 2.}$$

where: ε_h = normal horizontal strain (in/in)

ε_v = normal vertical strain (in/in)

ε_{45} = normal strain at 45° to the reinforcement, in line with the assumed principal compression (in/in)

Assumptions for ε_h , ε_v , and ε_{45} consist of the following: there is a 45° concrete compression strut; there is a linear stress-strain relationship for the vertical reinforcement (this was determined to be a reasonable assumption for the walls in this project when investigating the vertical reinforcement strains near the center of the wall, where shear cracking is expected); and the shear force is large enough to cause tension strains in the vertical reinforcement (Adebar and Gerin, 2004).

It is assumed that $G_g = .4 * E$ up to the point of shear cracking. The shear stress at cracking is taken as the smaller of the following: the theoretical shear stress at cracking using Mohr's circle and f_{cr} as defined in ACI 318-08; the added contributions from V_c and V_p from Eq. 1 above.

Reported cracked shear stiffness is generally in the range of $G_g/20$ and $G_g/10$ (ATC 72). The methodologies described above consistently provided a cracked shear stiffness of about 7 to 8% of G_g , sufficiently within the given range.

2.2.5 Experimental Wall Verification: Bilinear Model

In order to verify the modeling assumptions described which utilized bilinear force-displacement approximations, results were compared against that of an experimental wall with a similar aspect ratio, tested at UCLA (Englekirk 2003).

(a) Experimental Wall RW2 Elevation

(b) Experimental Wall RW2 Section

Figure P: Experimental Wall RW2 Properties

Source: Englekirk 2003

The wall was loaded axially with 96 kips, and then cyclic lateral loading was applied. The experimental results are summarized below:

Figure Q: Experimental Wall RW2 Results

Source: Englekirk 2003

A moment curvature response was formulated based on the principals discussed in Section 2.2.3. The response was then compared to the experimental results and also to the UCLA idealized response. Note that information regarding input for the calculated moment-curvature response is included in the appendix.

Figure R: Comparison of Moment-Curvature Results

Source: Author using Microsoft Excel 2007

The calculated moment-curvature results were consistent with the UCLA idealized moment-curvature response up to where steel yielding occurred. At the point of yielding, there was a greater post-yield stiffness in the calculated moment-curvature response. This discrepancy is likely due to a greater post-yield steel stiffness taken from Priestley, Calvi and Kowalsky's steel stress-strain relationship (Mander, Priestley, and Park 1988) than that used in the UCLA model (Menegotto and Pinto 1973). The difference in the steel stress-strain models is demonstrated qualitatively in Figure S below:

Figure S: Comparison of Stress-Strain Characteristics

Source: (a) Menegotto and Pinto, 1973; (b) Mander, Priestley, and Park 1988

The methodologies previously discussed for flexural wall behavior were used to convert the calculated moment-curvature response to a bilinear force-displacement relationship. Shear deformations were superimposed with the flexural displacements, as explained in Section 2.2.4. Displayed below, Figure T shows good agreement between the calculated force-displacement response and the experimental results. Also included is a force-displacement response that would be achieved in using ASCE 41-06 recommendations:

Figure T: Experimental Versus Calculated Force-Displacement Response

Source: Author using Microsoft Excel 2007

2.2.6 Concrete Column Modeling

Formulations for the plastic hinge lengths of the concrete columns were taken directly from the recommendations of Priestley, Calvi and Kowalsky (2007). The previously outlined approach was not used because for the concrete columns in this project, a reduction in the effective cross-section, due to spalling of the cover concrete, resulted in a negative stiffness immediately after the columns reached their peak flexural capacities. An example of this behavior is demonstrated for the moment-curvature response of a typical column below:

Figure U: Moment-Curvature Response of Typical Column

Source: Author using Microsoft Excel 2007

Strict application of moment-area for the member would imply that the column's ultimate displacement is reached as soon as the critical cross-section reaches its peak moment (Priestley, Calvi and Kowalsky 2007). If this were the case the calculated hinge length would be equal to 0. This is not in accordance with experimental observations, and thus the equation for hinge length outlined in *Displacement-based Seismic Design of Structures* was used for the columns in this project (2007), in which the plastic hinge length is simply a function of the column height and ratio of steel ultimate strength to steel yield strength. Because there was little coupling that occurred with the 30-foot

slabs, the axial demand on columns was reasonably constant, and thus there was no need for a P-M interaction surface.

The component backbone modeling for the concrete columns also had to change so that convergence could be reached in the ETABS and the Perform-3D models. To prevent a backbone with negative stiffness, a conservative backbone was used, in which the nominal capacity was taken at the onset of steel yielding, and then a line was drawn to the peak moment capacity. At this point, the column was assumed to have lost any significant capacity, and the residual strength recommended in ASCE 41-06 Table 6-8 was employed. Because the rotational demand was relatively small for the columns in this project, this ultimate point of rotation was never reached. An example of the backbone behavior is demonstrated in Figure V below:

Figure V: Moment-Curvature Modeling for Typical Column

Source: Author using Microsoft Excel 2007

2.2.7 Perform-3D Analytical Model

Dr. Graham Powell provides recommendations for nonlinear analysis of wall elements in *Performance Base Design Using Nonlinear Analysis* (Powell *Performance Based Design* 2007). Powell explains that an interaction between axial load and moment, along with varying amounts of cracking and yielding, causes shifting of the neutral axis. Additionally, Powell contends, interaction surfaces and plasticity theory do not apply for

reinforced concrete sections (*Performance Based Design* 2007). To account for these phenomena, Powell suggests using fiber cross-section elements.

The Perform-3D model in this thesis will employ fiber cross-sections, consisting of steel and concrete fibers that run vertically up the height of the building. See Figure W below for a graphical representation of typical steel and concrete fibers and their corresponding generalized stress-strain relationships:

(a) Example of Reinforced Concrete Fiber Cross-Section

(b) Stress-Strain Response of Confined Concrete for Fiber Modeling

(c) Stress-Strain Response of Reinforcing Steel for Fiber Modeling

Figure W: Fiber Cross-Section Modeling

Source: (a) Powell *Performance Based Design* 2007; (b) Author using Microsoft Excel 2007; (c) Author using Microsoft Excel 2007

It should be noted that the fiber cross-section dimensions were reduced to account for cover concrete spalling per Powell's recommendations (*Performance Based Design* 2007). Additionally, to reduce modeling complexity, Perform-3D's option for a uniform steel distribution based on the steel reinforcing ratio was selected, rather than lumping the steel in respective groups of bars. An investigation of the moment-curvature response was performed to verify that the response would not deviate from the expected theoretical results significantly; the results are shown in Figure X below:

Figure X: Comparison of Uniform vs. Lumped Steel Distribution

Source: Author using Microsoft Excel 2007

The moment-curvature response for the distributed steel did not vary significantly from that of the lumped steel. The notable differences are that the uniformly distributed steel had a slightly greater yield curvature and a reduced post-yield stiffness. Because the ASCE 41-06 acceptance criteria are based on rotation (independent of actual moment-curvature relationship), the results using uniformly distributed steel are expected to be slightly conservative.

To validate the Perform-3D modeling parameters, the experimental wall described in Section 2.2.5 was modeled with fiber cross-section elements in Perform-3D, and the force-displacement results were compared. The results for the computer model were consistent with the experimental results, as shown in Figure Y below.

Figure Y: Comparison of Experimental Wall and Perform-3D Response

Source: Author using Microsoft Excel 2007

ASCE 7-05 states that one way to achieve ductility is through energy dissipation, often through hysteretic behavior of building components under cyclic loading. Opening and closing of cracks in concrete often results in pinching action of the hysteresis response of a concrete shear wall, which in turn reduces the energy dissipation achieved. In an investigation of whether pinching action was being captured in Perform-3D, a cyclic load case was performed on the fiber cross-section wall model of Wallace's experimental wall. Pinching behavior was displayed, as demonstrated in Figure Z. The results are comparable to the experimental results, previously shown in Figure Q (a).

Figure Z: Hysteretic Behavior of Fiber Cross-Section Element

Source: Author using Microsoft Excel 2007

An additional concern for earthquake demands on structures is the strength degradation that occurs under cyclic loading. *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region* states that hysteresis pinching and strength degradation do not necessarily need to be modeled for time history analysis unless reaching states exceeding Collapse Prevention limits, since limits specified in ASCE 41-06 nonlinear response procedures are selected so that significant degradation should not occur before reaching CP (25). An additional limit to account for cyclic degradation has been included by reducing the ultimate steel strain by 40% (Priestley, Calvi and Kowalsky 2007).

The shear wall elements in Perform-3D are four-node wall elements which only have translational degrees of freedom. Because there is no rotational degree of freedom, frame elements which connect to the wall must continue past the exterior corner wall node to at least the adjacent interior wall node so that moment is transferred to the frame elements. Consequently, the slab elements in this project (modeled as frame elements as discussed in Section 2.2.2) were embedded continuously through the wall elements. An investigation was performed on the additional stiffness contribution this modeling parameter had on the building, and the change in results was negligible.

In a cantilever shear wall, higher strains are concentrated near the base of the wall, where curvature demands are the highest; thus, refining a the mesh toward the base of the wall and then measuring the strain over the fiber-cross section element can result in very high reported strains. However, Powell demonstrates in his video seminar that placing a strain gauge over a constant height, independent of the number of meshed elements, reports stable results (*Performance Based Design* 2007). For this reason, strains and corresponding wall rotations should be based on the anticipated hinge length, rather than individual element lengths. In this project, a four-node rotation gauge was placed that spanned from the calculated upper and lower bounds of the hinge length calculated in previous discussions, based on moment-curvature analysis.

Figure AA: Rotation Gage Assignment in Perform-3D

Source: Author using Perform-3D V4.0

ATC 72 shows that having nonlinear fiber elements modeled in the hinge region and elastic elements modeled above the hinge region can result in artificially high shear and moment demands in upper levels. This phenomenon occurs due to higher mode effects on the peak strains along the wall, as shown in the Figure BB below (4-44):

(a) Comparison of shear force distribution over height for fiber-hinge and fiber-all models

(b) Comparison of moment distribution over height for fiber-hinge and fiber-all models

Figure BB: Force-distribution for Fiber-hinge and Fiber-all Models

Source: ATC 72

In order to combat the issue of artificially high force demands, nonlinear fiber cross-sections were continued up the height of the building, wherever cracking was anticipated. The only walls modeled as uncracked elastic fiber cross-section elements were the perimeter basement-level walls which are surrounded by soil and are anticipated

to remain elastic. Some cracking may be anticipated due to out-of-plane wall loading, however such influence is beyond the scope of this project.

3.0 Building Design

The proposed reinforced concrete shear wall building is located in San Francisco, California. Code requirements from ASCE 7-05 and ACI 318-08 were used for the structural design.

The building plan is similar to that used in FEMA 439 B. The six-story structure is 83 feet tall above grade and includes two basement levels below. The typical story height is 12.5 feet, with a bottom story of 20.5 feet. The columns are located at 30-foot grid lines in each direction. The building spans a total of 150 feet in the north-south direction and 180 feet in the east-west direction. A typical elevation and floor plan are shown in Figure CC below:

Figure CC: Proposed Building Plans

Source: (a) FEMA 439B; (b) Author using Autodesk Revit Structure 2011

The first story of the building has a 20 psf glass curtain wall, and the stories above have 75 psf precast concrete panels. Weight for mechanical units and their housing at the roof level were also accounted for in the total building weight. Floor design loads included a reducible live load of 100 psf.

The building has an occupancy category of II, per Table 1-1 in ASCE 7-05, corresponding to an importance factor of 1.0. The seismic design criteria for the shear wall structure and its site are summarized in Table 2 below:

Table 2: Table 2: Seismic Design Criteria

Site Class:	D
$S_S =$	1.5
$S_{DS} =$	1
$S_1 =$	0.65
$S_{D1} =$	0.65
$R =$	6

An ETABS model was created to finalize the lateral design of the walls and flat plate reinforcing on wall-lines. The Equivalent Lateral Force Procedure, in ASCE 7-05, was used for the vertical distribution of forces.

In the ETABS design model, only walls were modeled in order to verify sufficient strength and stiffness. Rigid diaphragms, along with lumped point masses, were assigned at each level. A cracked flexural stiffness modifier of 0.35 was applied to the walls above grade, in compliance with ACI 318-08. Walls below grade were modeled as

uncracked. The ETABS model, displaying rigid diaphragm assignments, is shown below:

Figure DD: ETABS Model for Finalizing Code Design

Source: Author using ETABS Nolinear v9.7.3

A combination of studies were performed to validate the ETABS computer model. The stiffness was checked by hand and with a simplified RISA-2D frame model.

The period was also checked with Raleigh's equation (ASCE 41-06), using forces and corresponding deflections from the RISA-2D model. The stiffness, first mode period, assembled point masses, and mode shapes all compared well against calculated or anticipated results.

The ETABS model used for design had a period slightly greater than one second in both orthogonal directions. The corresponding base shear was 5580 kips, as shown in the design model summary, Table 3:

Table 3: ETABS Design Model Summary:

Building Period, T	1.12 s
Total Building Weight, W	39,700 k
Seismic Response Coefficient, C_s	0.14 g
Base Shear, V	5580 k
First Mode Mass Participation	78%

The resulting building design is summarized in the figures that follow:

Figure FF: Column Longitudinal Reinforcing
Source: Author using Autodesk AutoCAD Civil 3D 2011

Figure GG: Wall Vertical Reinforcing and Boundary Element Detailing

Source: Author using Autodesk AutoCAD Civil 3D 2011

For shear reinforcing of the columns, four #5 ties at six inches on center each way were used at the lower level columns, and three #4 bars were used at five-inch spacing in each direction above the fourth floor. For the walls, three curtains of #4 bars were placed at five inches on center for shear reinforcing throughout.

The 12-inch concrete slab reinforcing consisted of #9 bars spaced at 12 inches on top and #6 bars at 18 inches on bottom throughout column strips; the slab had #6 bars at 18 inches on center, top and bottom, in middle strips. At collector lines, three rows of #9 bars were spaced at 6 inches on center longitudinally, and #5 rebar were placed at six inches on center transversely for confinement within the effective slab width of 72 inches.

4.0 ASCE 41-06 Analyses: Analysis and Results

Since displacements are more directly related to a building's damage than are forces, the ASCE 41-06 procedures make an attempt to capture internal forces and rotational demands associated with the maximum structural displacement anticipated in the design-level earthquake. ASCE 41-06 provides recommendations for performance objectives for earthquakes with various probabilities of exceedance. A summary for the different rehabilitation objectives is shown in Table C1-1 in ASCE 41-06, Table 2 below:

Table 4: ASCE 41-06 Rehabilitation Objectives

Source: ASCE 41-06

		Target Building Performance Levels			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (~ 10%/50 year)	i	j	k	l
	BSE-2 (~ 2%/50 year)	m	n	o	p

¹Each cell in the above matrix represents a discrete Rehabilitation Objective.

²The Rehabilitation Objectives in the matrix above may be used to represent the three specific Rehabilitation Objectives defined in Sections 1.4.1, 1.4.2, and 1.4.3, as follows:

Basic Safety Objective (BSO)	k and p
Enhanced Objectives	k and m, n, or o p and i or j k and p and a, b, e, or f m, n, or o alone
Limited Objectives	k alone p alone c, d, g, h, or l alone

The performance objective typically considered acceptable in the U.S. is the Basic Safety Objective (BSO), which achieves a Life Safety Performance Level (3-C) at the BSE-1 Earthquake Hazard Level and a Collapse Prevention Performance Level (5-E) at the BSE-2 Earthquake Hazard Level (ASCE 41-06, 8). The BSE-2 Earthquake Hazard Level is associated with the maximum considered earthquake (MCE), and has a 2% probability of exceedance in 50 years. The BSE-1 Earthquake Hazard Level is consistent with the design-based earthquake in ASCE 7-05 and is 2/3 of the MCE, or has a 10% probability of exceedance in 50 years.

There is typically significant material inelasticity anticipated in response to the design-based earthquake, and consequently, the internal forces associated with the target displacement are larger than would actually develop when using an elastic model. It is for this reason that the linear procedures in ASCE 41-06 employ factors like the R-factor in ASCE 7-05 to compare member demand-to-capacity ratios (DCR's), called "*m*-factors" for deformation-controlled elements and "*j*-factors" for force-controlled elements. These factors are dependent on the nonlinear deformation capacity of the elements (ASCE 41-06). Primary components (defined as components relied upon to resist seismic forces) are considered deformation-controlled if ductile behavior is exhibited such that $d > 2g$ for a Type 1 curve and $e > 2g$ for a Type 2 curve, illustrated in Figure HH below (i.e. ductility of 2 or greater); note that e , g , and f are defined in ASCE 41-06 as parameters used to measure the deformation capacity in component load-deformation curves. Furthermore, secondary components are identified as deformation-

controlled for any member exhibiting a Type 1 curve and when $f > 2g$ in a Type 2 curve. All elements modeled in this project were considered deformation-controlled elements, since their behavior is consistent with the criteria discussed above.

Notes:

1. Only secondary component actions permitted between points 2 and 4;
2. The force, Q , after point 3 diminishes to approximately zero.

Figure HH: Component Force-Deformation Curves

Source: ASCE 41-06 Supplement 1

Because the nonlinear material properties are explicitly modeled in the nonlinear procedures, the acceptance criteria for the nonlinear procedures are dependent on the ductility demands of the elements. To provide consistency between results from the linear and nonlinear procedures, rotational demands from the nonlinear analyses and the corresponding element acceptance criteria have been divided by the element yield rotations, derived from the bilinearized moment-curvature relationships discussed in Section 2.2.3. This is the approach used in developing m-factors from experimental values, outlined in ASCE 41-06 § 2.8.3, and it allows results from the nonlinear

procedures to be directly compared to the demand-to-capacity ratios in the linear procedures.

4.1 Linear Static Procedure

The ETABS model for the LSP had the same parameters as in the model used for design, except that the stiffness modifiers were based on Table 1 shown previously, instead of ACI 318-08 recommendations. Also, the slab strips and column lines on drag lines were modeled to investigate moment demands in collector elements. For connectivity between the frame elements which represented the shear walls and the slab, there was a rigid frame element added. There was no significant contribution to the global building stiffness upon adding these elements. The ETABS model used for the linear procedures is shown in Figure II below, and the secondary elements that were investigated are identified in Figure JJ:

Figure II: ETABS Model used for Linear Procedures
Source: Author using ETABS Nonlinear v9.7.3

Figure JJ: Plan View of Secondary Elements Investigated in ASCE 41-06 Procedures
 Source: Author using ETABS Nonlinear v9.7.3

The ETABS model first mode and second mode periods were 1.00 and 1.01 seconds in the x and y-directions, respectively, slightly shorter than the 1.12 seconds each way in the model used for design. The minimal difference in period in each orthogonal direction results from the same wall properties in each way and demonstrates that the stiffness contribution from the added slab elements in the east-west direction is negligible. The shorter period can be attributed to the 0.5-modifier on the wall's flexural

stiffness (per ASCE 41-06 Table 6-5), as opposed to the 0.35-modifier used for design (per ACI 318-08). The first mode mass participation was 78% for each direction, which did not vary significantly from the design model.

The Pseudo Lateral Force was determined in accordance with ASCE 41-06, § 3.3.1, and the resulting spectral accelerations and base shears are summarized for the two Earthquake Hazard Levels, BSE-1 and BSE-2, in Table 5 below:

Table 5: Linear Static Procedure Design Values

Design Value	BSE-1	BSE-2
Building Period, T	1.0 s	1.0 s
Spectral Acceleration, S_a	0.642 g	0.963 g
Pseudo Lateral Force, V	25490 k	38240 k

Elements receiving greater force, due to the required 5% eccentric loading, were used for determination of performance levels. Although some of the secondary elements have been modeled, as discussed above, the shear wall performance was considered the sole indicator for building performance in this project. Material strengths were determined in accordance with Table 6-4 in ASCE 41-06. The shear demand, for use of Table 6-20 in the ASCE 41-06 Supplement 1, corresponded to the nominal flexural strength and an applied load at half the building height, per § 6.7.2.4.1 (ASCE 41-06). Figure KK below illustrates the demand to capacity ratios, DCR's, for the building in relation to the specified acceptance criteria:

Figure KK: Building Results for LSP
Source: Author using Microsoft Excel 2007

As demonstrated in Figure KK, the walls performed within the Life Safety criteria for the BSE-1 level earthquake and within the Collapse Prevention criteria for the BSE-2 level earthquake. The secondary elements all performed within the Immediate Occupancy region for both hazard levels. Because the walls performed within Life Safety and Collapse Prevention regions for BSE-1 and BSE-2, respectively, the building satisfied the Basic Safety Objective, as defined in ASCE 41-06.

4.2 Linear Dynamic Procedure

The same ETABS model shown in the LSP section was used for the Linear Dynamic Procedure. In an effort to remain consistent with previous studies, 5% modal damping was employed. The damped response spectrum formulated from § 1.6 in ASCE 41-06 is displayed below:

Figure LL: ASCE 41-06 Response Spectrum

Source: Author using Microsoft Excel 2007

The results for the LDP are shown in Figure MM below:

Figure MM: Building Results for LDP
Source: Author using Microsoft Excel 2007

The building satisfied the Basic Safety Objective again for the Linear Dynamic Procedure, in which the shear walls performed within the Life Safety criteria and Collapse Prevention criteria for the BSE-1 and BSE-2 hazard levels, respectively.

4.3 Nonlinear Static Procedure

4.3.1 ETABS

The ETABS model used for the NSP was similar to the linear models, however the nonlinear modeling parameters discussed in Section 2.2 were employed. In the analysis, the building was pushed to 150% of the target displacement in either direction to investigate the failure modes and strength degradation under extreme loading conditions (ASCE 41-06). The fundamental first mode shape was used for the lateral loading diagram.

In order to perform an NSP without having to perform an LDP, ASCE 41-06 § 2.4.2.1 requires that response spectrum analyses be performed two ways: using a sufficient number of modes for 90% mass participation and using only first mode participation. If the shear demands for the analysis which includes higher modes exceeds 130% of the shear demands with only the first mode considered, an LDP analysis must also be performed to supplement the NSP. For this project, the analysis which included all 24 modes corresponded to a base shear of 9270 kips; the first mode-only analysis resulted in a base shear of 8390 kips, an exceedence of about ten percent. There were no shear demands that resulted in 130% exceedence, and therefore the NSP was considered acceptable without a supplementary LDP analysis.

With the cracked section properties developed as discussed in Section 2.2, the period of the ETABS model lengthened to 1.18 seconds in either direction. The first mode mass participation was reduced to 76%.

ASCE 41-06 § 3.3.3.2 requires the evaluator use an effective first mode period to determine the target displacement. The effective period is dependent on the ratio of the initial building stiffness to the effective building stiffness, determined from an idealized bilinear force-displacement relationship of the building. Because the force-displacement relationship in ETABS was virtually bilinear to begin with, the effective period was the same as the elastic fundamental period with cracked section properties. The resulting target displacement was 18.0 inches. The force-displacement relationship for the building in ETABS is provided below:

Figure NN: NSP Building Force-Displacement Relationship (ETABS)

Source: Author using Microsoft Excel 2007

As previously discussed, the acceptance criteria for the nonlinear procedures are based on the rotational demands. The linear procedures, however, used m-factors to translate moment demands to acceptable demand-to-capacity ratios (DCR). In order to normalize the ASCE 41-06 analysis results, the rotations from the nonlinear procedure were converted into DCR's. Upon dividing the element rotational demand and the corresponding acceptance criteria by the element yield rotation, a DCR comparable to

those determined in the linear procedures was calculated. The element DCR results for the NSP in ETABS are shown in below:

Figure OO: Building Results for NSP in ETABS

Source: Author using Microsoft Excel 2007

For the NSP in ETABS, the walls performed within the Immediate Occupancy region and Life Safety ranges for the BSE-1 and BSE-2 hazard levels respectively. Note that the rotational DCR for the y-direction wall almost exactly matched up with the limit for Life Safety, however it fell just under the DCR limit of 4.03. The columns and slabs all performed within the Immediate Occupancy regions.

4.3.1 Perform-3D

A Perform-3D model was created for the second iteration of the NSP. The hinge assignments for the slab and column frame elements were the same as in the ETABS

model. As previously discussed, however, the wall elements utilized fiber cross-section elements, or what Perform-3D labels as “shear wall elements.”

The resulting Perform-3D model is shown in below:

Figure PP: Perform-3D Model

Source: Author using Perform-3D V4.0

The first mode period for the Perform-3D model was 0.83 seconds in either direction. The significantly shorter period is due to the Perform-3D model using gross,

uncracked section properties in the modal analysis. Cracking is monitored explicitly at each step in the nonlinear analysis.

As discussed earlier, in order to calculate a target displacement, an effective first mode period must be determined. In the ETABS model, the effective period was equal to the elastic period. However, in Perform, the force-displacement relationship is not bilinear as in ETABS. The methodologies described in ASCE 41-06 § 3.3.3.2 for developing an idealized force-displacement curve were followed.

Formulating the idealized bilinear force-displacement relationship was an iterative process. Perform-3D has a post-processor tool that significantly aids this procedure, which is similar to the method used for formulating a bilinear moment-curvature relationship described in previous sections. First, a trial target displacement and corresponding base shear are established. Next, a secant stiffness is determined which corresponds to a base shear equal to 60% of the effective yield strength of the structure, in which the areas above and below the actual force-displacement relationship are balanced. The bilinear force-displacement formulation for the building in Perform-3D is demonstrated in Figure QQ:

Figure QQ: NSP Building Force-Displacement Relationship (ETABS)

Source: Author using Perform-3D

The resulting effective first mode period in Perform-3D was 1.20 seconds in each direction, not far from the 1.18 seconds determined from the ETABS nonlinear model. The first mode period again resulted in a target displacement of 18.0 inches. A comparison between the force-displacement responses in ETABS and Perform-3D is shown below:

Figure RR: Comparison of ETABS and Perform-3D Force-Displacement Response

Source: Author using Microsoft Excel 2007

There was good agreement between the response from the two analysis models. The Perform-3D model, which utilized fiber cross-section elements, had many events of nonlinearity while the ETABS lumped hinge model was bilinear. This behavior was expected since the fiber cross-section element has a different stiffness with each iteration in displacement, while the frame element is linear until yielding occurs. Since the concrete walls in this project all had the same properties, hinging occurred

simultaneously in all four in-plane loaded walls in a given direction, and there was correspondingly only one change of structural stiffness. It appears as though there is an approximately equal amount of area under each of the force-displacement relationships, satisfying the desired equal-energy principle discussed previously.

The first mode mass participation for the Perform-3D model was 82%, slightly greater than the 79% in the linear ETABS model and the 76% in the nonlinear ETABS model. There is an apparent trend in that the models exhibiting greater wall stiffness correspond to greater first mode mass participation, for which there is likely a lower tendency for the whiplashing effect that occurs in higher modes. Since the Perform-3D model uses uncracked section properties for wall stiffness in the modal calculation, it has the highest stiffness and the highest resulting first mode mass participation.

The building element results in relation to the ASCE 41-06 acceptance criteria is displayed in Figure SS on the following pages:

Figure SS: Building Results for NSP in Perform-3D

Source: Author using Microsoft Excel 2007

For the NSP in Perform-3D the results for the overall building performance (based on shear wall behavior) was within the Immediate Occupancy range for BSE-1, and within the Life Safety region for the BSE-2 hazard level. The results for the columns and slabs were all within the Immediate Occupancy zone.

4.4 Nonlinear Dynamic Procedure

The final analysis procedure performed was the NDP in Perform-3D. The modeling parameters were the same as for the NSP Perform-3D model. In an effort to maintain parity with the previous studies, a modal damping ratio of 5% and a Rayleigh damping ratio of 0.2% at the fundamental period were used.

A suite of seven ground motions corresponding to the building site was used, consistent with those used for the previous studies. The ground motions were scaled to the effective first mode period determined in the Perform NSP analysis. The resulting building results are summarized in the following figure:

Figure TT: Building Results for NDP
Source: Author using Microsoft Excel 2007

The wall results for the NDP were within the Immediate Occupancy zone for the BSE-1 hazard level. The walls fell within the Life Safety limit at the BSE-2 level. All other elements satisfied the Immediate Occupancy criteria at each of the earthquake hazard levels.

4.5 Comparison of Analysis Results

The analysis results were consistent with what was anticipated: the linear procedures produced conservative results when compared against the nonlinear procedures. For the BSE-1 Earthquake Hazard Level, the linear procedures resulted in Life Safe performance, while the nonlinear procedures fell within the Immediate Occupancy range. Averaging the DCR's and comparing to the respective limits for Life Safety resulted in a 144% utilization of demand to capacity for the linear procedures and 81% for the nonlinear procedures.

There was also better performance shown for the nonlinear procedures at the BSE-2 level. The linear procedures were within the Collapse Prevention range, and the nonlinear procedures were within the Life Safety region. The percent utilization of demand to capacity for the Collapse Prevention limit at the BSE-2 hazard level was an average of 94% for the nonlinear procedures and 120% for the linear procedures. A visual comparison of the results for the two earthquake hazard levels is displayed below:

Figure UU: Comparison of Analysis Results
 Source: Author using Microsoft Excel 2007

4.5 Comparison of Different Structural Systems

The study performed in this project was also performed for steel special moment frames, SMF (Williams 2009), special concentric braced frames, SCBF (Adams 2010), and buckling restrained braced frames, BRBF (Burkholder 2010). The results from the procedures were generally consistent with one another when comparing overall building performance. For all systems excluding the special concentric braced frame, the nonlinear procedures produced better building performance than the linear procedures. The special concentric braced frame produced worse performance levels for the nonlinear procedures than it did for the linear procedures, possibly due to the nonductile buckling behavior of the compression braces, corresponding to a rapid loss of strength and stiffness.

The results for the different structural systems and their related performance levels are summarized Table 6 in and Table 7 below:

Table 6: Comparison of Different Systems, BSE-1 Hazard Level

Structural System	Linear Procedures	Nonlinear Procedures
SMF	LS	IO
SCBF	LS	CP
BRBF	LS	IO
RC Shear Wall	LS	IO

Table 7: Comparison of Different Systems, BSE-2 Hazard Level

Structural System	Linear Procedures	Nonlinear Procedures
SMF	CP	LS
SCBF	CP	CP
BRBF	CP	LS
RC Shear Wall	CP	LS

5.0 Conclusions

In this thesis, a reinforced concrete shear wall building was designed per ASCE 7-05 Equivalent Lateral Force Procedure and then analyzed with the procedures laid out in ASCE 41-06. The intent of the project was to investigate the performance level of an Occupancy Category II concrete wall building designed using current design codes and to explore how the performance predicted by the ASCE 41-06 analysis procedures compare against each other.

All of the ASCE 41-06 analyses resulted in building performance levels which met at least the Basic Safety Objective, satisfying Life Safety at the 474-year return period earthquake and Collapse Prevention at the 2475-year return period earthquake. The linear procedures showed performance levels that matched these criteria, while the nonlinear procedures showed better performance.

Because less conservative assumptions are used in the procedures that require more complex modeling, better performance was anticipated when progressing through each of the analyses in the following order: LSP, LDP, NSP, and NDP. The results for this project fell in line with this expectation and may be informative in that additional computational expense has the potential for reducing unnecessary costs for building owners on seismic improvements.

The LSP showed the most conservative results and deviated the least from what would be expected when designing with the ELFP. The LDP showed less severe rotational demands than the LSP. This result was consistent when comparing the static

and dynamic procedures using both linear and nonlinear computer models. The dynamic procedures likely impose lower DCR's because the static procedure target displacements are developed directly from the first mode period, while the dynamic procedures use CQC to combine the different modal contributions. Since by definition, the first mode has the largest modal height, a resultant load formulated using CQC would likely result in a resultant located lower in height versus that from solely first-mode response. The corresponding flexural and rotational demands are thus anticipated to be lower. It is also noted that in the linear static procedure, higher mode effects are accounted for by raising the height to the power, k , artificially increasing the resultant load height.

The NSP and NDP both had lower resulting relative DCR's than the LDP. Additionally, the nonlinear procedures resulted in better performance than the criteria specified for the Basic Safety Objective.

The NDP showed the best building performance. The better performance in the NDP when compared to the NSP is likely due to the inability to accurately capture higher mode effects in the NSP. In the NSP, the maximum building displacement at a given BSE hazard level earthquake is determined from the first mode period and its corresponding response acceleration. Modification factors are employed to capture the effects of higher modes, to achieve representative hysteresis behavior, and to account for whether the building period is within the region for building periods that typically match up with the equal-displacement methodology. Meanwhile, these factors are directly accounted for in the NDP, in which building response is calculated for each input

acceleration and corresponding time step. Because this behavior is directly captured in the NDP, modeling parameters are more critical than for any of the other procedures.

The guidelines that ASCE 41-06 provides for developing m-factors for the linear procedures utilize a reduction factor of 0.75 for the capacities of elements. The reduction factor, which is not applied on the nonlinear procedures, implies that explicit modeling of nonlinear material behavior should result in better prediction of building behavior, and correspondingly less conservative performance criteria. The acceptance criteria used in this project were from ASCE 41-06 criteria for flexurally dominated walls. The language and figures used in ASCE 41-06 specifically describe flexurally dominated walls with rigid bases, where the plasticity is assumed to terminate at the wall foundation, shown below in Figure VV:

Figure VV: ASCE 41-06 Figure for "Plastic Hinge Rotation in Shear Wall where Flexure Dominates Inelastic Response"

Source: ASCE 41-06

Because the walls in this project extended below grade level into a two-story basement, greater plastic rotations and displacements would be expected than for a cantilever wall with a rigid base, as shown in Figure VV. Because of this feature, the displacement-ductility is larger for the walls in this project than would be characteristic of a typical cantilever wall with the same properties. Although the nonlinear procedures showed better performance than the BSO, even better performance levels may have been achieved if the acceptance criteria were specifically developed from the experimental behavior of walls without a rigid base restraint.

The ETABS and Perform-3D models had similar results. The effective period calculated from the Perform-3D model was in good agreement with the ETABS model that utilized cracked section properties from moment-curvature analysis, only 1.7% longer. The NSP in ETABS showed slightly better performance than the NSP in Perform-3D. Although the complexity in the two models thwarted expectations for exactly the same results, a property that could have affected the data output is the assignment of the rotation gage in Perform-3D. As discussed in Section 2.2.7, the rotation gage assignment for this study was based on the plastic hinge length anticipated at the wall's ultimate displacement capacity. In reality, the extent of hinging was constantly changing with respect to building response at each given displacement interval, and thus the rotational gages should theoretically change size in correlation to the changing extent of plasticity. Because maximum rotations were of the greatest interest in this project, the maximum anticipated extent of hinging was decided to be a reasonable estimate for an appropriate rotation gage length. Note that this calculated hinge length assumed a single degree of freedom cantilever wall and that the extent of hinging in the Perform-3D model included various additional variables that could affect the rotational demand at the base (e.g. the extent of cracking up the height of the wall, various applied loads rather than one effective resultant load, multi-iterative wall stiffness rather than bilinear). Despite these inconsistencies, there was only a 10% difference in calculated rotation at the BSE-1 level and a 5% difference at the BSE-2 level.

The results from this thesis are in agreement with the results from the previous studies, excluding the SCBF study. The SMF, BRBF, and concrete shear wall systems displayed better building performance for the linear procedures when compared against the nonlinear procedures, satisfying performance that met the BSO for the linear procedures and performance that exceeded the BSO for the nonlinear procedures. The SCBF exhibited a contrary trend, in which the nonlinear procedures achieved more critical performance levels than in the linear procedures. The behavior may exemplify the inherent lack of ductility in SCBF's due to the nonductile strength and stiffness degradation that occurs upon buckling in the compression braces.

There is potential for future work to expand the findings of this study. An example is the inclusion of soil springs to investigate soil-structure interaction. Buildings that contain structural irregularities could also be investigated, as well as structures with more complex geometric configurations. Additionally, buildings designed with other structural systems and I or T-shaped walls could be explored.

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6.0 Appendix

The following properties were used for the calculated moment-curvature response shown above:

- $\rho_s = 1.13\%$
- $\rho_v = 0.21\%$
- $\varepsilon_{cu} = 0.95\%$
- $\varepsilon_{su} = 0.072\%$
- $f_y = 60,000$ psi
- $f'_c = 4000$ psi

ETABS Modeling

Displaced Shape at Target Displacement (NSP), $\delta_t = 18.0''$:

Typical Wall Hinge Properties:

Frame Hinge Property Data for WALLBASEX - M3

Edit

Point	Moment/SF	Rotation/SF
E-	-0.75	-20
D-	-0.75	-17
C-	-1.26	-14.3
B-	-1	0
A	0	0
B	1	0
C	1.26	14.3
D	0.75	17
E	0.75	20

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative

☐ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative

Life Safety Positive Negative

Collapse Prevention Positive Negative

Typical Column Hinge Properties:

Frame Hinge Property Data for LOWERCOL - M3

Edit

Point	Moment/SF	Rotation/SF
E-	-0.2	-2
D-	-0.2	-0.8
C-	-0.96	-0.7
B-	-1	0
A	0	0
B	1	0
C	0.96	0.7
D	0.2	0.8
E	0.2	2

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative

☐ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative

Life Safety Positive Negative

Collapse Prevention Positive Negative

OK
Cancel

Perform Modeling

Deflected Shape at Target Displacement (NSP), $\delta_t = 18.0''$:

Confined Concrete:

COMPONENT PROPERTIES

Inelastic
Elastic
Cross Sects.

Materials
Strength Sects
Compound

Type: Inelastic Concrete Material

New Choose type and name to edit an existing material.

Name: 5.2 ksi Confined Concrete

Purge
Rename
Text for filter.
Filter

Length Unit: in
 Force Unit: kip

Status: Saved.

Graph
Save
Save As
Delete

Shape of Relationship
☐ E-P-P
☒ Trilinear

Tension Strength
☐ Yes
☒ No

Strength Loss
☒ Yes ☐ No

Strain Capacities
☐ Yes ☒ No

Upper/Lower Bounds
☐ Yes ☒ No

Cyclic Degradation
☒ None
☐ YULRX
☐ YX+3

Import Components
Export Components

☒ Selected components of this type.

☐ All components of all types.

Import ...

The graph shows a trilinear stress-strain relationship. The vertical axis is Force (F) and the horizontal axis is Strain (D). Key points on the curve are labeled: FU (Ultimate Force), FY (Yield Force), FR (Residual Force), DU (Strain at Yield), DL (Strain at Ultimate), DR (Strain at Residual), and DX (Strain at Failure). The initial slope is labeled K0 and the peak slope is labeled KH.

Cyclic Degradation
Upper/Lower Bounds

Basic Relationship
Strength Loss
Strain Capacities

F = stress. D = strain.

Positive

Tension Stresses
 FY:
 FU:

Compression Stresses
 FY: 3.96
 FU: 7.25

Tension Strains
 DU:
 DX:

Compression Strains
 DU: .0034
 DX: .04

Stiffness, K0
 Modulus, E: 4327
 KH/K0 Pos =
 KH/K0 Neg = 0.306

Paste
Copy
Clear

Steel:

Lower Interior Column Hinge:

Earthquake Load Case:

LOAD CASES

Load Case Type: **Dynamic Earthquake**

New | Select name to edit an existing load case.

Status: **New load case. Not yet saved.**

Save | Save As | Delete | UnChange

Load Case Name: **X-direction: BSE2EQ1**

Add/Review/Delete Earthquakes

Control Information for Dynamic Analysis

Total Time (sec): **40.945** | Time Step (sec): **.01** | Limit State to Stop Analysis: Type: **Drift**

Max Events in any Step (analysis stops if exceeded): **200** | Name: **NSP X Drift Limit State**

Save results every **1** time steps (default = every step) | Reference Drift: **X Roof to Grade**

This affects time history plots. Usage ratios are still calculated every step. | This is used only for "thumbnail" plots of the response.

Earthquake Direction in Plan

Angle from structure H1 axis to earthquake Q1 axis (degrees): **0**

Q2 H2 Q1 angle H1

Q1 Earthquake

Group: **BSE2** | Name: **BSE2EQ1**

Peak Acceln (g): **0.7636** | Duration (sec): **40.91** | Acceln Scale Factor: **1.11** | Time Scale Factor: **1.0**

Q2 Earthquake

Group: **NONE** | Name:

Peak Acceln (g): | Duration (sec): | Acceln Scale Factor: **1.0** | Time Scale Factor: **1.0**

V Earthquake (usually not applied)

Group: **NONE** | Name:

Peak Acceln (g): | Duration (sec): | Acceln Scale Factor: **1.0** | Time Scale Factor: **1.0**

First Mode Displaced Shape: