SENSITIVITY OF SEISMIC RESPONSE OF A 12 STORY
REINFORCED CONCRETE BUILDING TO VARYING
MATERIAL PROPERTIES

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ABSTRACT

Sensitivity of Seismic Response of a 12 Story Reinforced Concrete Building to Varying Material Properties

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The main objective of this investigation is to examine how various material properties, governed by code specification, affect the seismic response of a twelve-story reinforced concrete building. This study incorporates the pushover and response history analysis to examine how varying steel yield strength (F_y), 28 day nominal compressive concrete strength (f'_c), modes, and ground motions may affect the base shear capacity and displacements of a reinforced concrete structure.

Different steel and concrete strengths were found to have minimal impact on the initial stiffness of the structure. However, during the post-yielding phase, higher steel and concrete compressive strengths resulted in larger base shear capacities of up to 22%. The base shear capacity geometric median increased as f'_c or F_y increased, and the base shear capacity dispersion measure decreased as f'_c or F_y increased. Higher mode results were neglected in this study due to non-convergent pushover analyses results.

According to the response history analysis, larger yield and concrete compressive strengths result in lower roof displacement. The difference in roof displacement was less than 12% throughout. This displays the robustness of both analysis methods because material properties have insignificant impact on seismic response. Therefore, acceptable yield and compressive strengths governed by seismic code will result in acceptable building performance.

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NOTATION AND DEFINITION

Notation	Definition
c	Damping coefficient
C_d	Deflection amplification factor
D_n	Peak deformation
F_a	Short-period site coefficient
f_c	28 day nominal compressive concrete strength
f_s	Lateral force
F_v	Long-period site coefficient
F_y/f_y	Yield strength
k	Stiffness
$\mathbf{m}$	Mass matrix
m	Mass
M_n^*	Effective modal mass
R	Response modification coefficient
s_n^*	Force distribution
$\mathbf{s}_n$	Modal inertial force distribution
T_n	Natural vibration period
u	Lateral displacement
$\ddot{u}_g(t)$	Horizontal ground acceleration
u_m	Peak value roof displacement due to n th mode
V_{bn}	Base shear
w	Weight
ζ	Damping ratio
$\mathbf{I}$	Influence vector
Φ_n	n th natural vibration mode
ω_n	Natural vibration frequency
$q_n(t)$	Modal coordinate
$\hat{x}$	Geometric median
Φ	Shape vector
Ω_o	Overstrength factor
δ	Dispersion measure

LEGEND

Script	Description
$*(t)$	respect to time
$_n$	n th mode, nodal
bold	vector or matrix
$\cdot$	derivative
$\ddot{}$	second derivative

ACRONYMS

Acronym	Definition
ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
ATC	Applied Technology Council
CUREe	California Universities for Research in Earthquake Engineering
ELF	Equivalent Lateral Force
FEMA	Federal Emergency Management Agency
MPA	Modal Pushover Analysis
MRS	Modal Response Spectrum
NEHRPP	National Earthquake Hazards Reduction Program
NISEE	National Information Service for Earthquake Engineering
PEER	Pacific Earthquake Engineering Research
RHA	Response History Analysis
SAC	Joint venture between SEAOC ATC and CUREe
SEAOC	Structural Engineers Association of California
SMRF	Special Moment Resisting Frame
SRH	Seismic Response History
UBC	Uniform Building Code
USGS	United States Geological Survey

CHAPTER 1 - INTRODUCTION

1.1 History

Over the past several decades, it has been a difficult task to develop an effective and feasible procedure to analyze and evaluate seismic demands on a structure. There are currently two procedures that can be used to compute seismic demands on a structure: nonlinear dynamic (time history) analysis, or nonlinear static (pushover) analysis. Nonlinear dynamic analysis is computationally intensive, the more rigorous procedure of the two, and is often considered to output "exact" results when modeled properly. Nonlinear static analysis is less computationally intensive because of the many inherent assumptions in the procedure. Engineers are more likely to apply this method before transitioning to nonlinear dynamic analysis due to its simplicity and effectiveness. The nonlinear static procedure is becoming the current standard practice accepted by documents such as FEMA-440 due to its simplicity and efficiency (Deierlein, et al. 2010).

Pushover analysis has gained popularity as a tool for assessment and design verification because of its efficiency and ease of applicability. However, there are limitations to this procedure such as inability to capture higher mode effects (Krawinkler and Seneviratna, 1998) and torsional effects. Current pushover procedures also do not properly account for progressive stiffness degradation, change of modal characteristics, and period elongation (Antoniou et al. 2002). Significant research has been done in attempts to develop a more accurate pushover analysis while retaining its simplicity; an extensive list of literature on this subject is available in FEMA (2005).

1.2 Objective

In the past, variations of the nonlinear static and dynamic analysis have been formulated to investigate how higher modes, P- Δ effects, beam-column connections, and high rise geometry affect building performance (Chopra, 2001; Deierlein, et al., 2010; Antoniou, et al., 2002, Elnashai, 2001; Wilson 2002; Powell 2010; Gupta and Krawinkler, 2000). However, there is a lack of systematic studies to understand the dependence of material properties on building performance. In order to fill this important gap, the primary objective of this research is to develop and analyze the sensitivity of response of a reinforced concrete building to material properties (strength of concrete and reinforcing steel).

The building will be designed in accordance to structural and seismic codes: ACI 318-08, FEMA 450 and IBC 2009. The model will be analyzed with varying steel reinforcement yield strength (F_y), and 28-day nominal concrete crushing strength (f'_c). Material properties will assumed to be 400MPa (~ 60 ksi) for steel and 28MPa (~ 4 ksi) for concrete, F_y and f'_c will change $\pm 25\%$ (21 to 35MPa concrete, 300 to 500 MPa steel) in increments of 5%. The minimum compressive strength of concrete, f'_c , tested is 21MPa because ACI-21.1.4.2 specifies f'_c shall not be less than 3000 psi (20.67MPa). Pushover analyses for modes 1 to 12 and nonlinear dynamic analyses for every pair of ground motions 1-10 will be run for all combinations of F_y and f'_c with the goal of assessing how a building response varies with varying material properties.

1.3 Background

1.3.1 Seismic Design of Reinforced Concrete Special Moment Frames

The model designed for the purpose of this study will use special moment resisting frames (SMRF). SMRFs are required in Seismic Design Categories D, E or F to ensure the building remains ductile under earthquakes as covered in ACI318 Chapter 21 Earthquake Resistant Structures.

SMRFs for reinforced concrete structures have been around in the United States since the 1960's. The use of SMRFs in framing lines of the building was optional for designers and many designers chose not to incorporate such frames in their design. This led to a number of building failures during the 1994 Northridge Earthquake, which then led to stricter frame detailing requirements. The SMRF's goals are to achieve strong-columns/weak-beams to spread inelastic response over many stories, avoid shear failure, and provide details such that yield regions have ductile flexural response (Moehle, et al., 2008). Seismic forces for SMRFs can be determined by equivalent lateral force (ELF) analysis, modal response spectrum (MRS) analysis, and seismic response history (SRH) analysis (ASCE7-05). ELF is the most simple, but can only be used for short period, low-rise buildings.

ACI also requires that the actual longitudinal reinforcement yield strength be at least the specified yield strength, but not much higher because higher forces will transfer to adjacent members which may not have been designed to accommodate yielding. The reinforcements must conform to the American Society for Testing and Materials publication ASTM A706 *Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement*, which specifies that the actual yield strength must not

be more than the specified yield strength by 18000psi (124MPa). The ratio of the specified tensile strength to actual yield strength must be 1.25. ASTM A615 grades 40 and 60 can be used as well as long as the previously stated requirements are met (Moehle, et al. 2008).

1.3.2 Computerized Modeling

Idealized nonlinear models of beam-column elements from simplest to most complex are shown in Figure 1. From left to right, this includes: plastic hinge, nonlinear spring hinge, finite length hinge zone, fiber section, and finite element. Simpler methods such as the plastic hinge, or nonlinear spring hinge modeling (Figure 1a/b) place the inelastic deformations at the nodes of the element. Fiber section models (Figure 1c) uses numerical integration to distribute plasticity through member cross sections along the member length. Finite element (Figure 1d/e) uses small finite elements with many input parameters, and is the most complex of all listed methods (Deierlein, et al., 2010).

Figure 1-Idealized models of beam-column elements from Nonlinear Structural Analysis For Seismic Design (adapted from NEHRP Seismic Design Technical Brief No. 4, 2010)

1.3.3 Foundations and Soil-Structure Interaction

Designers neglect the interaction between the structure and the foundation most of the time; this leads to unrealistic results which are too conservative or unconservative. In most cases, ignoring soil-structure interaction is conservative given the design response spectra and ground motions adequately envelope the kinematic effect of the foundation structure and its effect on site response. Assuming a fixed base type foundation in an analysis model is also permitted according to ASCE 7-12.7.1 (Foundation Modeling) (Deierlein, et al., 2010).

1.3.4 Nonlinear Static Analysis Requirements

Nonlinear static or pushover analysis procedure involves applying an incremental lateral load on a structural model until a defined target displacement is reached; a control point at the roof is usually taken. The load is distributed such that it represents the inertial forces expected during the earthquake ground shaking, and are applied at the nodes where dynamic inertial forces would usually develop. Component stiffness and strengths are modeled to account for degradation due to cyclic loading. Commonly known as a pushover curve of the structure, base shear capacity is typically plotted against displacement. The pushover curve can be simplified by lines that represent elastic, post-yield hardening, and degradation as shown in Figure 2. The seismic demands (e.g., story drifts, plastic rotations) at target displacement are compared with acceptable values. The pushover analysis is suitable for low-rise regular buildings and not for tall, slender, or irregular buildings where multiple vibration modes affect the behavior.

When calculating the seismic demand, gravity loads should be applied before the incremental lateral load. It takes into account the effects of gravity-induced forces and P-

Δ effects on component yielding and post-peak response. The lateral load distribution should reflect the expected inertia forces at the floor levels, which are usually proportional to the floor masses and the modal shape of the fundamental mode.

Figure 2-Idealized static pushover backbone curve (FEMA 2005)

Compared to the nonlinear dynamic analysis, the nonlinear static analysis is limited in its ability to capture dynamic behavior from cyclic loading and degradation. However, it is convenient and reliable if the building is governed by its first mode, symmetrical, rectangular, and less than five stories. (Deierlein, et al., 2010).

1.3.5 Nonlinear Dynamic Analysis Requirements

Nonlinear dynamic analysis generally provides more realistic estimate of structural response to strong ground shaking compared to the nonlinear static analysis. This leads to a more reliable assessment because there are fewer limitations, and it is more accurate than the nonlinear static procedure (Deierlein, et al., 2010).

1.3.5.1 Masses and damping

Masses include the self weight of the building along with the assumptions used to determine seismic masses in ASCE7, or other standards. The masses are usually lumped at the floor levels, and include the half of the weight above and below the centerline of the floor level. Inertial effects are included in both horizontal directions, and about the rotational vertical building axis. Gravity loads should be used to account for effects on force and deformation demands in structural components and large displacement P- Δ effects (Deierlein, et al., 2010).

In an actual building, energy is dissipated through cracking or yielding of structural components, foundation and soil, architectural cladding, partitions and finishes, or through dissipation devices. For analysis modeling, damping is included in the equation of motion through the viscous damping matrix, $\mathbf{c}$, to account for energy dissipation. The damping matrix depends on many factors, and it is difficult to generalize what should be included. Thus the damping matrix for a structure should be determined from its modal damping ratios, which account for all energy-dissipating mechanisms (Chopra, 2006). PEER/ATC 72-1 suggests the critical damping in the first translational mode to be 1% to 10%. The article recommends up to 5% over elastic periods $0.2T$ to $1.5T$ (where T is the fundamental period of the structure) because 10% probably reflects energy dissipation due to both nonlinear hysteretic and inherent damping (Deierlein, et al., 2010).

1.3.5.2 Ground Motions

Input ground motions used should be scaled to closely represent the hazard area of interest. Some items to consider are fault mechanisms, distance to fault, site conditions and earthquake magnitudes. Acceptable ground motions can either be actual recorded ground motions, spectrally matched ground motions, or artificial motions. ASCE 7 also recommends at least seven ground motions to be used for more accurate results (Deierlein, et al., 2010).

1.3.6 Hysteresis and Ductility

Structures need to rely on ductility and hysteretic energy dissipation capacity to avoid collapse when undergoing inelastic deformation during severe earthquakes. Ductility is the property which allows the structure to undergo large plastic deformations without significant loss of strength (Figure 3a). Hysteretic energy is the energy dissipated by inelastic cyclic deformations which is given by the area within the load-deformation curve also called the hysteretic curve (Figure 3b/c). Structures having low hysteretic energy dissipation capacities (Figure 3b), even if the deformations are well below the ultimate deformation, are likely to collapse due to *low-cycle fatigue* effect, which is degradation of strength and stiffness under repeated inelastic cycling. Ensuring that the structure is able to dissipate a large amount of hysteretic energy in each cycle can minimize low-cycle fatigue effect. The area enclosed by the force-deformation loops gives the hysteretic energy. Larger area implies more dissipation of hysteretic energy as shown in Figure 3c. Since earthquake loading produces large deformation as well as low-cycle fatigue, both ductility and energy dissipation capacity are required to resist severe earthquakes (Duggal, 2007).

Figure 3-Ductile/non-ductile behavior and hysteretic energy dissipation capacities (Duggal 2007).

Figure 4 shows various types of hysteretic models. All include the initial stiffness slope which then transitions to the post-yield phase determined from principles of mechanics, experimental data, or influences of cyclic loading and interaction of axial, shear, and flexural effects. The slope going downward and backwards is a result of component strengths deteriorating due to fracture, crushing, local buckling, bond slip, or other phenomena under large inelastic cyclic deformation (Deierlein, et al., 2010).

Figure 4-Types of hysteretic modeling from NEHRP Seismic Design Technical Brief No. 4

1.3.7 Force-Deformation Curve

An important model defined in ASCE 41 shows the idealized force and deformation parameters of nonlinear component models (Figure 5). It is intended to represent the cyclic envelope showing strength degradation due to cyclic loading. Key points are effective yield (B), peak strength (C), residual strength (D) and ultimate deformation (E). It is acceptable to assume a dashed slope from point C to E because the slope from C to D is not very defined (Deierlein, et al., 2010).

Figure 5-Generalized force-deformation curve (PEER/ATC 2010)

1.3.8 P-Δ Effects

P-Δ effects are gravity induced loading acting on deformed structural elements during an earthquake causing increased internal forces measured between building member ends.

Gupta and Krawinkler (2000) found that P-Δ effects can profoundly influence the earthquake response of buildings in their elastic range (Figure 6). In the elastic region, including P-Δ effects minimally reduces the stiffness. However, in the post yield response, P-Δ effects greatly affect the stiffness as shown by the decrease in lateral force resistance. If P-Δ effects were ignored the post yield stiffness would remain positive (Chopra 2006).

Figure 7 shows an idealized base shear capacity versus drift curve of a cantilever structure with and without P- Δ effects. If the gravity load is large, the stiffness reduction (shown by the negative slope) is significant, and contributes to loss of lateral resistance and instability (Deierlein, et al., 2010).

Gravity load-deformation (P- Δ) effect must be modeled in static or dynamic analyses as they can ultimately lead to loss of lateral resistance, ratcheting (a gradual build up of residual deformations under cyclic loading), and dynamic instability. This

means gravity loads of the entire building must be present in the analysis, and appropriate P- Δ analysis techniques should be introduced in the structural model (Wilson 2002; Powell 2010; Deierlein, et al., 2010).

Figure 6-Global Pushover Curves for 20-Story Building Los Angeles (LA) with and without P-Delta (Gupta and Krawinkler 2000).

Figure 7-Force-deformation curve with and without P- Δ effect (PEER/ATC 2010)

CHAPTER 2 - LITERATURE REVIEW

An extensive amount of research has been done in the past decades on the pushover and response history analysis. There have been many challenges faced with creating a pushover analysis that remained simplistic yet achieves similar results to the "exact" time history analysis (Chopra, 2001; Deierlein, et al., 2010; Antoniou, et al., 2002, Elnashai, 2001). This section will highlight important background information as well as findings from articles in this area of research.

2.1 Pushover Analysis

Chopra and Goel (2001) formulated a more accurate pushover analysis procedure in estimating seismic demands on buildings. They wanted to keep the procedure conceptually simple and be similar to other procedures with invariant force distribution during that time period. The standard response spectrum analysis (RSA) for elastic buildings was reformulated to the Modal Pushover Analysis (MPA) beginning with the simplest one-story system, expanding to an elastic multistory building, and finally to an inelastic multistory building.

Through checking their results with the "exact" response history analysis (RHA) and FEMA pushover analysis, it was found that the MPA provided good estimates of floor displacements, story drift, and plastic hinge locations. However, plastic hinge rotations were not as accurate. It was also found that the FEMA-273 force distributions greatly underestimated story drift demands, and the MPA was more accurate.

Antoniou et al. (2002) developed a fully adaptive pushover procedure because they found a need to improve the inelastic static pushover analysis due to its limitations and questionable reliability. Their pushover methodology addressed the need to be adaptive by adjusting the lateral load distribution along the height of the structure by considering current stiffness states and modal properties. The results were compared to pushover and inelastic dynamic analyses, and their results were close to the inelastic dynamic analysis procedure. Their method was based on Elnashai's (2001) proposed adaptive pushover procedure, which was similar to the inelastic dynamic analysis because it accounted for higher mode effects due to being multimodal. Elnashai (2001) also addressed the shortcomings of the static inelastic analysis and proposed a new method that "accounted for spread of inelasticity, geometric nonlinearity, full multi-modal, spectral amplification and period elongation within a framework of fiber modeling of materials."

Goel and Chopra (2004) revised the Modal Pushover Analysis with several improvements. Most notably, $P-\Delta$ effects due to gravity loads were included in the pushover analysis for all modes instead of only the first mode. The importance of the $P-\Delta$ effect was covered by Gupta and Krawinkler (2000) when they did a study on the dynamic P -delta effects for flexible steel structures. It was found that $P-\Delta$ effects can profoundly influence the earthquake response of buildings in their elastic range. Gravity load-deformation ($P-\Delta$) effect must be modeled in static or dynamic analyses as they can ultimately lead to loss of lateral resistance, ratcheting (a gradual build up of residual deformations under cyclic loading), and dynamic instability. This means that the gravity loads of the entire building must be present in the analysis, and appropriate $P-\Delta$ analysis

techniques should be introduced in the structural model (Wilson 2002; Powell 2010; Deierlein, et al., 2010).

2.2 Response History Analysis

The 1994 Northridge earthquakes damaged many steel framed buildings. Engineers reevaluated their approach on seismic design, and realized improvements needed to be made. A group called SAC, which consists of three nonprofit organizations: Structural Engineers Association of California (SEAOC), Applied Technology Council (ATC), and California Universities of Research in Earthquake Engineering (CUREE), was formed in 1994. The group commissioned three firms to design 3-, 9- and 20- story SMRF buildings in accordance to local codes in Los Angeles (ICBO 1994), Seattle (ICBO 1994) and Boston (BOCA 1993). They also assembled 20 ground motion records to represent 2% and 10% probability of exceedance in 50 years (a mean return period of 2475 years and 475 years) (Chopra 2006).

Many nonlinear dynamic analyses use these ground motions for their simulations. The 10% probability of exceedance in 50 years set of ground motions from the Los Angeles area will be used in this analysis as mentioned in Section 3.3.3 of this thesis.

CHAPTER 3 - METHODOLOGY

3.1 Proposed Methodology

In order to analyze the sensitivity response of a reinforced concrete special moment resisting frame several processes must take place:

- Develop the building based on current structural and seismic codes: ACI 318-08, FEMA 450 and IBC 2009.
- Use SAP2000 to model the structure in three dimension. Assign section geometry and properties. Define various load and load cases. Use moments, shears and axial loads to size reinforcements.
- Translate information such that it is readable by Matlab and Opensees. Define nodal locations, section properties, and number of reinforcement.
- Conduct pushover analysis of the structure with various combinations of steel and concrete using a pushover analysis procedure in Matlab and Opensees developed by Dr. Goel.
- Repeat the analysis with a response history analysis for various earthquake ground motions from the SAC project.
- Process and analyze the results.

The procedure in its entirety will be outlined in the sections hereafter in supplement with hand calculations in the Appendix.

3.1.1 Model Description and Assumptions

The initial procedure was to develop a 12 story reinforced concrete structure using ACI318-08 and SAP2000. The 12 story reinforced concrete structure shown in Figure 8 was used as the reference model in this project. It was based on Mwafy's RF-H030 regular frame structure's geometry (Mwafy and Elnashai, 2001), and was designed and detailed in accordance with ACI318-08. The lateral force resisting system was designed as a special moment resisting frame (SMRF) in accordance to ACI/UBC due to weak soil class D. SMRFs have more stringent detailing requirements to ensure ductile behavior at the beam-to-joints connections, and are expected to sustain drift angles of at least 0.04radians. This higher inelastic performance capacity allows the structure to resist strong earthquakes without significant loss in stiffness or strength, and have higher plastic curvature and rotation capacity (Moehle, et al. 2008).

The special reinforced concrete moment frame corresponds to a response modification coefficient (R) of 8, overstrength factor (Ω_o) of 3, and deflection amplification factor (C_d) of 5.5 in accordance to table 4.3-1 Design Coefficients and Factors for Basic Seismic-Force Resisting System in FEMA 450. 28MPa (~4 ksi) cast-in place, normal-weight, non prestressed concrete with rectangular sections; and 400MPa (~60 ksi) steel reinforcements were used in the design of the structure. A fixed base type foundation was used as permitted in ASCE 7-12.7.1 (Foundation Modeling). The building was not actually constructed, but meets seismic code and represents a typical medium-rise building in the Hollywood, California region.

3.1.2 Building Geometry

The designed building is 15m (49.21 ft) x 20m (65.62 ft) in plan, and 36m (118.11 ft) in elevation. The east-west bays are 8m (26.25 ft), 4m (13.12 ft) and 8m (26.25 ft) on center; and the north-south bays are 5m (16.4 ft) each as shown in Figure 8 and Figure 9. East-west, longitudinal, and x-direction will be used synonymously throughout this thesis. Likewise, north-south, transverse, and y-direction will be used to mean the same.

Figure 8-RH30/Proposed Structure Plan View

The 12 story building floors are numbered with respect to the ground level, with the 13th floor being the roof. The 12 floor-to-floor heights, measured from the center-of-beam to center-of-beam, are 3m (13.12 ft) each for a total of 36m (118.11ft) (Figure 9). The flooring system is a solid slab with a thickness of 0.14m (0.46 ft). All beams are 0.35m (1.15 ft) x 0.65m (2.13 ft) in both x and y directions. Interior and exterior columns are 0.7m (2.3 ft) x 0.7m (2.3 ft). The geometric dimensions of the structure are summarized in Table 1 below.

Table 1-Member cross sectional dimensions

Reference Name	Columns (m)	Beams b x h (m)		Slab (m)
		X-direction	Y-direction	
RH30	0.70 x 0.70	0.35 x 0.65	0.35 x 0.65	0.14

Figure 9-RH30/Proposed Structure Section View

3.1.3 Loading

The seismic mass of each floor included the self weight of the column half way above and below the floor, floor slabs, and floor finishes, partitions and fixtures. The contributed weights were calculated to be 0kN on the ground level, 3085kN (694kips) for the first through eleventh floor, and 2755kN (619kips) on the roof level. The weight of the entire structure is 36690kN (8248kips).

Live loads and dead loads (loading from floor finishes and partitions) were both assumed to be 2.0kN/m^2 (42psf) to be consistent with Mwafy's RF-H030's loading. Skin loads were assumed to be 0.48kN/m^2 (10psf). Earthquake loads were determined using the USGS Java Ground Motion Parameter Calculator for zip code 90210 which is further explained in the next section. Loads were factored in accordance to ACI318-08 9.2 Required Strength.

3.1.4 Site Information

The seismic design spectrum used was based on the United States Geological Survey (USGS) zip code 90210 with Site Class D soil type. The spectrum provided by USGS based on these defined parameters is shown in Figure 10. This graph shows that the spectral acceleration which is measured in g (spectral acceleration due to gravity) reaches a maximum peak at 1.2g for short periods and gradually reaches a spectral acceleration of 0.3g at a period of 2.0 seconds. A short-period site coefficient (F_a) of 1, and long-period site coefficient (F_v) of 1.5 were used.

Figure 10-Design Spectrum, S_a vs T , 5% Damping, Zipcode 90210, Site Class D, $F_a=1$, $F_v=1.5$. Source: USGS Java Ground Motion Parameter Calculator. 2003 NEHRP Seismic Design Provisions.

3.1.5 Design Procedure

The building was modeled in SAP2000 by defining geometric nodal point locations in three dimensional space, and connecting the nodes with lines. Frame sections were defined and assigned to the lines to represent beams, slabs and columns. The first story columns were modeled as fixed and secured to the ground. Soil-structure interaction was ignored because in most cases ignoring soil-structure interaction is conservative (Deierlein et al., 2010). The dead and live loads mentioned above were assigned as area loads. An earthquake design spectrum was input as well. Load combinations were defined in accordance to ACI318. Various analyses were run depending on the specific load combination needed to calculate axial, shear and moments for the structure. The

output values for various elements were used to size members, reinforcements and spacing according to ACI 318-08.

The first twelve vibration modes and periods of the building were also determined in OpenSees. Figure 11, Figure 12 and Figure 13 show the first three modes which represent the primary modes in the x-direction, y-direction, and in torsion. Their periods are 1.418, 1.266 and 1.001 seconds, respectively. Every other set of three modes follow the same directional convention. All of the computed 12 mode shapes and periods can be viewed numerically in Table 2, or visually in the Appendix A.

Table 2 - Modes and Periods (seconds) determined in OpenSees

Mode	1	2	3	4	5	6	7	8	9	10	11	12
Period	1.418	1.266	1.001	0.448	0.405	0.321	0.244	0.225	0.180	0.159	0.148	0.118

Figure 11-Mode 1 in X-direction with Period = 1.418s

Mode 2: Period = 1.266

Figure 12-Mode 2 in Y-direction with Period = 1.266s

Mode 3: Period = 1.001

Figure 13-Mode 2 in torsion with Period = 1.001s

3.2 Pushover Analysis

Once the building is designed, information such as the geometry and section properties will be inputted into text files that are readable by Matlab and Opensees. For the pushover analysis, we will analyze the structure beginning with 400MPa steel and 28MPa concrete, and vary each parameter by +/- 25% in increments of 5% for each mode shape. Of the twelve mode shapes, only eight will be used; four in the x-direction (modes 1, 4, 7 and 10), and four in the y-direction (modes 2, 5, 8, 11). Modes 3, 6, 9 and 12 will be neglected due to their torsional mode shapes which are not excited by the selected ground motions. The pushover analysis theory and process is presented in the section below.

3.2.1 Pushover Analysis Process

The pushover analysis procedure involves applying an incremental lateral load on the structural model until a defined target displacement is reached. The distributed load represents inertial forces expected during the earthquake ground shaking. It is based on the assumptions that the building's response is based on an equivalent single degree-of-freedom (SDOF) system, controlled by a single mode, and that the shape of the mode remains constant throughout the time history (Krawinkler and Seneviratna, 1998). Although the assumptions are incorrect, studies found that the assumptions predicted maximum seismic response of multi degree-of-freedom structures accurately, provided that the response was dominated by a single mode (Lawson, et al. 1994; Saiidi and Sozen, 1989).

The pushover procedure, described in greater detail (Krawinkler and Seneviratna, 1998), is summarized below:

The formulation of the equivalent SDOF system is not unique, but the basic underlying assumption common to all approaches is that the deflected shape of the MDOF system can be represented by a shape vector $\{\Phi\}$ that remains constant throughout the time history, regardless of the level of deformation. Accepting this assumption and defining the relative displacement vector $\mathbf{X}$ of an MDOF system as $\mathbf{X} = \{\Phi\}x_t$, (x_t = roof displacement), the governing differential equation of an MDOF system can be written as:

$$\mathbf{M}\{\Phi\}\ddot{x}_t + \mathbf{C}\{\Phi\}\dot{x}_t + \mathbf{Q} = -\mathbf{M}\{1\}\ddot{x}_g \quad \text{Equation 1}$$

where $\mathbf{M}$ and $\mathbf{C}$ are the mass and damping matrices, $\mathbf{Q}$ denotes the story force vector, and $\ddot{x}_g$ is the ground acceleration.

If we define the reference SDOF displacement x^* as

$$x^* = [\{\Phi\}^T \mathbf{M} \{\Phi\} / \{\Phi\}^T \mathbf{M} \{1\}] x_t \quad \text{Equation 2}$$

and pre-multiply equation (1) by $\{\Phi\}^T$, and substitute for x_t using equation (2), we obtain the following differential equation for the response of the equivalent SDOF system:

$$\mathbf{M}^* \ddot{x}^* + \mathbf{C}^* \dot{x}^* + \mathbf{Q}^* = -\mathbf{M}^* \ddot{x}_g. \quad \text{Equation 3}$$

$\mathbf{M}^*$, $\mathbf{C}^*$ and $\mathbf{Q}^*$ denote the properties of the equivalent SDOF system and are given by

$$\mathbf{M}^* = \{\Phi\}^T \mathbf{M} \{1\} \quad \text{Equation 4}$$

$$\mathbf{Q}^* = \{\Phi\}^T \mathbf{Q} \quad \text{Equation 5}$$

$$\mathbf{C}^* = \{\Phi\}^T \mathbf{C} \{\Phi\} * [\{\Phi\}^T \mathbf{M} \{\Phi\} / \{\Phi\}^T \mathbf{M} \{1\}] \quad \text{Equation 6}$$

Figure 14-Force-displacement characteristics of MDOF structure and equivalent SDOF system (Krawinkler and Seneviratna, 1998)

Presuming that the shape vector $\{ \Phi \}$ is known, the force deformation characteristics of the equivalent SDOF system (Q^*-x^* relationship, see Figure 14b) can be determined from the results of a nonlinear incremental static analysis of the MDOF structure, which usually produces a base shear (V) - roof displacement (x_t or δ_t) diagram of the type shown in Figure 15.

Figure 15-Static base shear vs roof displacement response of MDOF structure (Krawinkler and Seneviratna, 1998)

In order to identify nominal global strength and displacement quantities, the multilinear $V - \delta_t$ diagram needs to be represented by a bilinear relationship that defines a 'yield' strength, V_y , an effective 'elastic' stiffness, $K_e = V_y / \delta_{t,y}$ and a hardening (or softening) stiffness, $K_s = \alpha K_e$ for the structure. Some judgment may be needed to define these properties. The simplified bilinear base shear-roof displacement response curve is needed to define the properties of the equivalent SDOF system. The yield value of the base shear (V_y) and the corresponding roof displacement ($X_{t,y}$) from Figure 14a are used together with equations (2) and (5) to compute the force - displacement relationship for the equivalent SDOF system as follows:

$$x_y^* = [\{\Phi\}^T \mathbf{M} \{\Phi\} / \{\Phi\}^T \mathbf{M} \{\mathbf{1}\}] x_{t,y} \quad Q_y^* = \{\Phi\}^T Q_y \quad \text{Equation 7}$$

where Q_y is the story force vector at yield, i.e. $V_y = \{\mathbf{1}\}^T Q_y$.

The initial period of the equivalent SDOF system T_{eq} can be computed as

$$T_{eq} = 2\pi [x_y^* M^* / Q_y^*]^{1/2} \quad \text{Equation 8}$$

The strain hardening ratio (α) of the $V - x_t$ relationship of the MDOF structure defines the strain hardening ratio of the equivalent SDOF system.

The roof displacement of the structure, x_t , is related to the equivalent SDOF displacement, x^* , by means of equation (2). Thus, the target displacement can be found if the displacement demand of the equivalent SDOF system can be estimated for the design earthquake. For an elastic SDOF system the displacement demand is given by the spectral displacement. For inelastic systems the SDOF displacement demand needs to be obtained from inelastic spectra. The utilization of inelastic spectral demand information requires the estimation of the ratio of elastic strength demand to yield strength of the equivalent SDOF system, usually referred to as the R-factor.

Since inelastic spectra are usually obtained for unit mass systems, it is convenient to divide equation (3) by M^* to obtain the differential equation of the unit mass equivalent SDOF system:

$$\ddot{x}^* + (C^*/M^*)\dot{x}^* + Q^*/M^* = \ddot{x}_g. \quad \text{Equation 9}$$

Equation (9) describes the response of a unit mass SDOF system with period T_{eq} and yield strength $F_{y,eq}$ given as

$$F_{y,eq} = Q_y^*/M^* \quad \text{Equation 10}$$

If the elastic response spectrum is known, the elastic strength demand of the unit mass equivalent SDOF system can be computed as

$$F_{e,eq} = S_a(T_{eq}) \quad \text{Equation 11}$$

where $S_a(T_{eq})$ is the special ordinate of the elastic acceleration spectrum. The strength reduction factor R can then be obtained from the relationship

$$R = F_{e,eq} / F_{y,eq} = S_a(T_{eq})M^*/Q_y^* \quad \text{Equation 12}$$

3.2.2 Pushover Analysis Codes

The practical application of the nonlinear analysis in earthquake engineering in the United States gained popularity in the 1990's, and focused more on existing building assessing and retrofitting rather than design. Nonlinear dynamic analyses were not as popular because the analysis was more complex. FEMA 273 *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA 1997) and ATC 40 *Seismic Evaluation and Retrofit of Concrete Buildings* (ATC 1996) were the first to publish guidelines on the application of nonlinear analysis, and focused mainly on nonlinear static analysis, or pushover analysis. The codes progressed to ASCE 41 *Seismic Rehabilitation of Existing Buildings* (ASCE 2007), and improvements were made in FEMA 440 *Improvement of Nonlinear Static Seismic Analysis Procedures* (FEMA 2005) and FEMA P440A *Effects of Strength and Stiffness Degradation on Seismic Responses* (FEMA 2009a) (Deierlein, et al., 2010).

3.3 Nonlinear Response History Analysis

Once the pushover analysis is complete, the response history analysis will be run beginning with 400MPa steel and 28MPa concrete for each pair of ten Los Angeles Earthquake Ground Motions. Each steel and concrete parameter will be adjusted by +/- 25% in increments of 5%.

3.3.1 Nonlinear Response History Analysis Process

The equations and following notes governing the nonlinear response history analysis (Chopra, 2006) are as follow:

The differential equation governing the response of an MDF system to earthquake ground motion is :

$$\mathbf{m}\ddot{\mathbf{u}} + \mathbf{c}\dot{\mathbf{u}} + \mathbf{k}\mathbf{u} = \mathbf{p}_{\text{eff}}(t) \quad \text{Equation 13}$$

where

$$\mathbf{p}_{\text{eff}}(t) = -\mathbf{m}\ddot{\mathbf{u}}_g(t) \quad \text{Equation 14}$$

For nonlinear response history analysis, the stiffness term is modified to recognize inelastic behavior of the building. The force-deformation relationship for each structural member undergoing cyclic deformation is now nonlinear and hysteretic. The initial loading curve is nonlinear at larger amplitudes of deformation, and the unloading and reloading curves differ from the initial loading branch. The relationship between lateral forces f_s at the N floor levels and resulting lateral floor displacements $\mathbf{u}$ is no longer single valued but depend on the history of displacements, thus

$$\mathbf{f}_s = \mathbf{f}_s(\mathbf{u}, \dot{\mathbf{u}}) \quad \text{Equation 15}$$

The equation of motion generalized for inelastic systems becomes

$$\mathbf{m}\ddot{\mathbf{u}} + \mathbf{c}\dot{\mathbf{u}} + \mathbf{f}_s(\mathbf{u}, \dot{\mathbf{u}}) = -\mathbf{m}\ddot{\mathbf{u}}_g(t) \quad \text{Equation 16}$$

Equation 15 is computationally demanding because the stiffness matrix has to be calculated at each time instant; from the element tangent stiffness matrices corresponding to the deformation and state of each structural element-whether it is initial loading, unloading or reloading. This is extremely complex for large structures with thousands of structural elements.

The second equation is also demanding for large number of degrees of freedom because the coupled differential equations must be solved simultaneously. There has to be very short time steps to ensure proper convergence, stability and accuracy.

Some factors that must also be considered are P- Δ effects, modeling assumptions, and ground motion characteristics which were mentioned in the literature review.

3.3.2 Nonlinear Response History Analysis Codes

Nonlinear dynamic analysis for design has recently made its way into codes as a more complete assessment of building performance than the nonlinear static analysis. The nonlinear dynamic analysis is included in ATC 58 *Guidelines for Seismic Performance Assessment of Buildings*. It is also used to assess performance of structural systems that do not conform to prescriptive seismic force-resisting system types in ASCE 7 *Minimum Design Loads for Buildings and Other Structures* (ASCE 2010) such as tall buildings in high seismic regions (Deierlein, et al., 2010).

3.3.3 Ground Motions Used

The response history analysis will use the SAC Los Angeles ground motions because the test building was designed for Hollywood, Los Angeles, California. The 20 motions (Table 3) represents 10 records with horizontal components in the fault-normal and fault-parallel orientation. The acceleration time histories include historical and artificial records based on rupture and wave propagation through the soil strata. The records have been altered so their mean response spectrum matches the 1997 NEHRP design spectrum, modified from soil type of S_B - S_C to soil type S_D . According to the National Information Service for Earthquake Engineering (NISEE), a public service project of the Pacific Earthquake Engineering Research (PEER) Center Library created by UC Berkeley, each original historical ground motion pair has been altered in the following way:

- The two horizontal components originally provided were resolved into fault-normal and fault-parallel orientations.
- The records are adjusted in the frequency domain to have characteristics appropriate for NEHRP S_O soil sites.
- The records are amplitude scaled so that the average of the two horizontal spectra matches the target spectrum.
- The individual components were rotated 45 degrees away from the fault-normal fault-parallel orientations

Table 3-SAC 20 Ground Motions for Los Angeles

SAC Name	Record	Earthquake Magnitude	Distance (km)	Scale Factor	Number of Points	DT (sec)	Duration (sec)	PGA (cm/sec ²)
LA01	Imperial Valley, 1940, El Centro	6.9	10	2.01	2674	0.02	39.38	452.03
LA02	Imperial Valley, 1940, El Centro	6.9	10	2.01	2674	0.02	39.38	662.88
LA03	Imperial Valley, 1979, Array #05	6.5	4.1	1.01	3939	0.01	39.38	386.04
LA04	Imperial Valley, 1979, Array #05	6.5	4.1	1.01	3939	0.01	39.38	478.65
LA05	Imperial Valley, 1979, Array #06	6.5	1.2	0.84	3909	0.01	39.08	295.69
LA06	Imperial Valley, 1979, Array #06	6.5	1.2	0.84	3909	0.01	39.08	230.08
LA07	Landers, 1992, Barstow	7.3	36	3.2	4000	0.02	79.98	412.98
LA08	Landers, 1992, Barstow	7.3	36	3.2	4000	0.02	79.98	417.49
LA09	Landers, 1992, Yermo	7.3	25	2.17	4000	0.02	79.98	509.7
LA10	Landers, 1992, Yermo	7.3	25	2.17	4000	0.02	79.98	353.35
LA11	Loma Prieta, 1989, Gilroy	7	12	1.79	2000	0.02	39.98	652.49
LA12	Loma Prieta, 1989, Gilroy	7	12	1.79	2000	0.02	39.98	950.93
LA13	Northridge, 1994, Newhall	6.7	6.7	1.03	3000	0.02	59.98	664.93
LA14	Northridge, 1994, Newhall	6.7	6.7	1.03	3000	0.02	59.98	644.49
LA15	Northridge, 1994, Rinaldi RS	6.7	7.5	0.79	2990	0.005	14.945	523.3
LA16	Northridge, 1994, Rinaldi RS	6.7	7.5	0.79	2990	0.005	14.945	568.58
LA17	Northridge, 1994, Sylmar	6.7	6.4	0.99	3000	0.02	59.98	558.43
LA18	Northridge, 1994, Sylmar	6.7	6.4	0.99	3000	0.02	59.98	801.44
LA19	North Palm Springs, 1986	6	6.7	2.97	3000	0.02	59.98	999.43
LA20	North Palm Springs, 1986	6	6.7	2.97	3000	0.02	59.98	967.61

CHAPTER 4 - RESULTS AND DISCUSSION

4.1 Pushover Results

In order to understand the effects of material properties on initial stiffness and strength of the building, pushover analyses were run on the reinforced concrete model through Matlab in conjunction with OpenSees for each combination of material properties. The material properties that changed were: $f_c=28\text{mpa}$ and $F_y=400\text{mpa}$ (+/- 25% in increments of +/-5%). All translational modes (transverse and lateral) between modes 1 to 12 were used in the pushover analyses. This led to a total of 968 pushover analyses. A target displacement of 2m was used to assess the building's base shear strength in different modes.

Each simulation ran at least four hours until the target displacement was reached, or OpenSees was terminated in the task manager because of non-convergence (lack of meeting the prescribed tolerance at element or structural level) of the pushover analysis. To speed up the process, the analysis was simultaneously run on six computers. The computers included: a custom built Intel i7 2600k desktop overclocked to 4.3ghz with 16gb of ram, an Intel Core2Quad desktop overclocked to 3.4ghz with 8gb of ram, an Intel i5 Alienware m11x laptop overclocked to 1.8ghz with 8gb of ram, an Intel Core2Quad Dell XPS desktop running at 2.6ghz with 4gb of ram, and two Core2Quad desktops in the Civil Engineering graduate lab. A free remote access software, called **TeamViewer**, was installed on each computer. Each computer could be remotely controlled by either

computer, or an internet browser. All of the pushover analyses were completed in less than three months.

4.1.1 Combined Pushover Graphs

In order to condense the 968 individual pushover curves, they were plotted in groups by mode then F_y with varying f'_c , or f'_c with varying F_y . Figure 16 has four sample combined pushover graph in the x-direction for modes 1, 4, 7 and 10; $f'_c=28\text{mpa}$; and F_y ranging from 300 to 500mpa. Figure 17 has four sample combined pushover graph in the y-direction for modes 2, 5, 8 and 11; $F_y=400\text{mpa}$; and f'_c ranging from 21 to 35mpa. The complete set of combined pushover graphs can be found in Appendix B.

Figure 16- Combined Pushover Analysis Graphs (Modes 1, 4, 7 and 10; $f'_c=28\text{MPa}$; and varying F_y).

Increasing F_y does not affect the building's initial stiffness (Figure 16). The difference is noticeable in the post-yield section where base shear capacity increases for higher steel yield strengths for a given displacement. Similarly, higher concrete strengths do not affect the initial stiffness but result in higher base shear capacities at a given displacement (Figure 17). The higher base shear capacity with increasing steel yield

strength and concrete strength is due to higher section (beams and columns) moment capacity as apparent from the moment-curvature relationships presented in Figure 18.

Figure 17- Combined Pushover Analysis Graphs (Modes 2, 5, 8 and 11; $F_y=400\text{MPa}$; and varying f'_c).

While the pushover analysis for fundamental transverse and lateral modes (Figures 16a, 16b, 17a, and 17b) led to expected pushover curves, pushover analysis for

higher modes (Figures 16c, 16d, 17c, and 17d) encountered severe convergence issues. In particular, several pushover analyses did not converge. Therefore, results for only the fundamental transverse and longitudinal modes will be reported in the results section.

Figure 18-M - Φ Curves for various f'_c and F_y values. Data from Xtract.

4.1.2 Response Statistics

The next items investigated were the geometric median, $\hat{x}$, and the dispersion measure, δ , (Equation 17, Figure 19) for each of the combined pushover analysis graphs in Appendix B (Benjamin and Cornell 1970). The geometric median divides 50% of the

data above and below, and enables us to compare median base shear capacities between graphs. The dispersion measure, measures how dispersed the data is, and enables us to compare variation in f'_c or F_y between graphs. The complete set of response statistics may be seen in Appendix D.

$$\bar{x} = \exp \left[\frac{\sum_{i=1}^n \ln x_i}{n} \right]; \quad \delta = \left[\frac{\sum_{i=1}^n (\ln x_i - \ln \bar{x})^2}{n-1} \right]^{1/2}$$

Equation 17

Figure 19-Graphical representation of geometric median and dispersion measure. $f'_c=28\text{mpa}$, mode 2.

It would be unfair to compare base shear capacities when the pushover analysis failed because each analysis did not terminate at the same displacement. An assumption was made that the varying pushover curves in each graph would continue to have the same rising trend. The minimum displacement per mode was determined, and the

corresponding base shear capacities were used. By doing this, a consistent trend across f'_c or F_y can be seen.

The minimum displacement determined varied by mode because they are independent of one another. Higher mode pushover analyses failed at lower displacements, so the response statistics would be skewed if a minimum displacement between modes were used. Minimum displacements used to calculate the response statistics were found to be .54m, and .55m for modes 1 and 2.

4.1.2.1 Geometric Median

Table 4 and Table 5 lists the base shear capacity geometric medians for various F_y and f'_c values. As F_y increases, the base shear capacity geometric median of f'_c increases. Likewise, as f'_c increases, the base shear capacity geometric median of F_y increases. The base shear capacity geometric medians of f'_c encompasses a larger range (4.59 to 6.52x10⁴kN) compared to the base shear capacity geometric medians of F_y (5.31 to 5.79x10⁴kN) for the first mode. This suggests that concrete compressive strength has a greater effect on the maximum geometric median base shear capacity. Graphical representations of Table 4 and Table 5 may be seen in Appendix D.

Table 4-Base Shear Capacity Geometric Median of F_y (10^4 kN)

f_c	mode 1	mode 2
21.0	5.31	3.91
22.4	5.36	3.96
23.8	5.41	4.01
25.2	5.46	4.05
26.6	5.51	4.10
28.0	5.56	4.15
29.4	5.61	4.19
30.8	5.66	4.24
32.2	5.69	4.28
33.6	5.75	4.32
35.0	5.79	4.37

Table 5-Base Shear Capacity Geometric Median of f_c (10^4 kN)

F_y	mode 1	mode 2
300	4.59	3.48
320	4.81	3.63
340	5.01	3.78
360	5.22	3.92
380	5.42	4.05
400	5.61	4.18
420	5.80	4.30
440	5.99	4.43
460	6.17	4.55
480	6.35	4.67
500	6.52	4.78

4.1.2.2 Dispersion Measure

Table 6 and Table 7 lists the base shear capacity dispersion measures for various F_y and f_c values. As F_y increases, the base shear capacity dispersion measure of f_c decreases. Likewise, as f_c increases, the base shear capacity dispersion measure of F_y decreases. The base shear capacity dispersion measures of f_c are a magnitude smaller (0.032 to 0.026×10^4 kN) than F_y (0.119 to 0.112×10^4 kN) for the first mode. This suggests that steel yield strength has greater effect on the base shear capacity dispersion measure. Graphical representations of Table 6 and Table 7 are in Appendix D.

Table 6-Base Shear Capacity Dispersion Measures for F_y (10^4 kN)

f_c	mode 1	mode 2
21.0	0.1194	0.1086
22.4	0.1187	0.1078
23.8	0.1178	0.1070
25.2	0.1171	0.1062
26.6	0.1163	0.1054
28.0	0.1156	0.1046
29.4	0.1145	0.1039
30.8	0.1138	0.1032
32.2	0.1162	0.1027
33.6	0.1128	0.1022
35.0	0.1127	0.1019

Table 7-Base Shear Capacity Dispersion Measures for f_c (10^4 kN)

F_y	mode 1	mode 2
300	0.0323	0.0404
320	0.0321	0.0394
340	0.0298	0.0389
360	0.0300	0.0382
380	0.0292	0.0375
400	0.0285	0.0371
420	0.0279	0.0364
440	0.0273	0.0357
460	0.0267	0.0347
480	0.0262	0.0338
500	0.0257	0.0332

4.1.3 Maximum Base Shear Capacities

To investigate every material combination, maximum base shear capacities for each pushover curve were listed (Table 4 and Table 5). The base shear capacities used were taken at the minimum roof displacement using the values specified in the previous section. It can be seen that yield strength has a greater impact on base shear capacity than concrete compression strength. The maximum base shear capacity of 6.7×10^4 kN occurs for the largest F_y and f_c values. The lowest base shear capacity of 4.4×10^4 kN occurs for the smallest F_y and f_c values.

Table 8- Maximum base shear capacity for various f'_c and F_y (10^4 kN), mode 1

f'_c (MPa)	F_y (MPa)										
	300	320	340	360	380	400	420	440	460	480	500
21.0	4.36	4.57	4.78	4.98	5.18	5.36	5.55	5.74	5.91	6.09	6.26
22.4	4.41	4.62	4.83	5.03	5.23	5.42	5.61	5.79	5.97	6.14	6.32
23.8	4.46	4.67	4.88	5.08	5.28	5.47	5.66	5.84	6.02	6.20	6.37
25.2	4.51	4.72	4.92	5.13	5.33	5.52	5.71	5.89	6.07	6.25	6.42
26.6	4.56	4.76	4.97	5.18	5.38	5.57	5.76	5.94	6.12	6.30	6.47
28.0	4.60	4.81	5.02	5.22	5.42	5.62	5.80	5.99	6.17	6.35	6.52
29.4	4.65	4.86	5.07	5.27	5.47	5.67	5.85	6.04	6.22	6.40	6.57
30.8	4.70	4.91	5.11	5.32	5.52	5.71	5.90	6.09	6.27	6.45	6.62
32.2	4.72	4.95	5.16	5.36	5.56	5.76	5.95	6.14	6.32	6.50	6.67
33.6	4.78	4.99	5.20	5.41	5.61	5.80	5.99	6.18	6.37	6.55	6.72
35.0	4.83	5.04	5.20	5.45	5.66	5.85	6.04	6.23	6.41	6.59	6.77

Table 9- Maximum base shear capacity for various f'_c and F_y (10^4 kN), mode 2

f'_c (MPa)	F_y (MPa)										
	300	320	340	360	380	400	420	440	460	480	500
21.0	3.26	3.41	3.55	3.69	3.82	3.94	4.07	4.19	4.31	4.43	4.54
22.4	3.31	3.46	3.60	3.74	3.87	3.99	4.12	4.24	4.36	4.48	4.59
23.8	3.35	3.50	3.65	3.78	3.92	4.04	4.17	4.29	4.41	4.53	4.64
25.2	3.40	3.55	3.69	3.83	3.96	4.09	4.21	4.34	4.46	4.58	4.69
26.6	3.44	3.59	3.74	3.88	4.01	4.14	4.26	4.39	4.51	4.63	4.74
28.0	3.48	3.64	3.78	3.92	4.06	4.18	4.31	4.43	4.55	4.67	4.79
29.4	3.53	3.68	3.83	3.97	4.10	4.23	4.36	4.48	4.60	4.72	4.84
30.8	3.57	3.72	3.87	4.01	4.15	4.28	4.40	4.53	4.65	4.77	4.88
32.2	3.61	3.76	3.91	4.05	4.19	4.32	4.45	4.57	4.69	4.81	4.93
33.6	3.65	3.81	3.96	4.10	4.23	4.37	4.49	4.62	4.74	4.86	4.98
35.0	3.68	3.84	4.00	4.14	4.28	4.41	4.54	4.67	4.79	4.91	5.02

The frequency distribution of the base shears were plotted in Figure 20 and Figure 21. The first mode had a mean of $5.59 \times 10^4 \text{ kN}$ and a standard deviation of $0.63 \times 10^4 \text{ kN}$. The second mode had a mean of $4.16 \times 10^4 \text{ kN}$ and a standard deviation of $0.43 \times 10^4 \text{ kN}$. Both means were $.02 \times 10^4 \text{ kN}$ higher than the base shear capacity at $f'_c = 28 \text{ MPa}$ and $F_y = 400 \text{ MPa}$. 68% of the base shear capacities fall within $\pm$ one standard deviation from the mean.

Figure 20-Histogram of Base Shear Capacities for Mode 1

Figure 21- Histogram of Base Shear Capacities for Mode 2

4.1.4 Base Shear Capacity Percentage Differences

The base shear capacity percentage difference was also determined for every steel and concrete parameter in comparison to $f'_c=28\text{MPa}$ and $F_y=400\text{MPa}$ (Table 10 and Table 11 for the first and second mode). As f'_c and F_y move away from the nominal values, the percentage difference increase. A 22% difference can be seen at the extreme lower f'_c and F_y values, and a -22.5% difference can be seen at the higher f'_c and F_y values. An interesting diagonal pattern can be seen along $f'_c=26.6$ to 29.4MPa with decreasing F_y values where the percentage difference remains less than 2%. This may suggest that lower concrete compressive strengths with higher yield strength rebar, or higher concrete compressive strength with lower yield strength rebar compensate one another, and have the same effect as nominal concrete and steel.

Table 10-Base shear capacity % difference from $f'_c=28\text{MPa}$ and $F_y=400\text{MPa}$, Mode 1

Fy(MPa)	f _c (MPa)										
	21.0	22.4	23.8	25.2	26.6	28.0	29.4	30.8	32.2	33.6	35.0
300	22.33	18.60	14.91	11.31	7.82	4.50	1.12	-2.15	-5.31	-8.44	-11.48
320	21.46	17.72	14.05	10.44	6.89	3.51	0.18	-3.10	-6.28	-9.40	-12.45
340	20.60	16.83	13.17	9.56	6.02	2.59	-0.71	-3.99	-7.22	-10.35	-13.41
360	19.75	16.00	12.32	8.70	5.17	1.71	-1.60	-4.89	-8.14	-11.27	-14.34
380	18.90	15.17	11.52	7.86	4.28	0.85	-2.47	-5.79	-9.03	-12.17	-15.25
400	18.07	14.36	10.70	7.02	3.46	0.00	-3.34	-6.66	-9.93	-13.06	-16.14
420	17.20	13.47	9.80	6.16	2.61	-0.87	-4.19	-7.52	-10.78	-13.94	-17.03
440	16.40	12.67	9.02	5.31	1.77	-1.69	-5.07	-8.36	-11.63	-14.80	-17.90
460	15.63	11.91	8.13	4.52	0.96	-2.48	-5.91	-9.26	-12.50	-15.69	-18.76
480	14.91	11.13	7.43	3.72	0.13	-3.31	-6.72	-10.11	-13.36	-16.54	-19.63
500	14.07	10.32	7.34	2.92	-0.72	-4.19	-7.56	-10.93	-14.18	-17.38	-20.48

Table 11-Base shear capacity % difference from $f'_c=28\text{MPa}$ and $F_y=400\text{MPa}$, Mode 2

Fy(MPa)	f _c (MPa)										
	21.0	22.4	23.8	25.2	26.6	28.0	29.4	30.8	32.2	33.6	35.0
300	22.03	18.42	15.06	11.81	8.72	5.75	2.81	-0.11	-2.99	-5.83	-8.57
320	20.96	17.34	13.94	10.68	7.55	4.56	1.62	-1.29	-4.19	-7.03	-9.78
340	19.90	16.27	12.84	9.56	6.41	3.39	0.44	-2.47	-5.37	-8.21	-10.97
360	18.85	15.21	11.74	8.46	5.29	2.24	-0.72	-3.65	-6.53	-9.38	-12.14
380	17.80	14.16	10.66	7.37	4.18	1.12	-1.88	-4.80	-7.67	-10.53	-13.29
400	16.75	13.11	9.61	6.29	3.07	0.00	-3.01	-5.95	-8.82	-11.66	-14.44
420	15.73	12.10	8.56	5.22	1.99	-1.12	-4.12	-7.08	-9.94	-12.79	-15.58
440	14.72	11.08	7.52	4.17	0.91	-2.21	-5.22	-8.20	-11.06	-13.90	-16.70
460	13.79	10.06	6.49	3.12	-0.14	-3.29	-6.32	-9.31	-12.17	-15.02	-17.81
480	12.87	9.06	5.47	2.07	-1.17	-4.36	-7.41	-10.40	-13.27	-16.12	-18.92
500	12.01	8.21	4.46	1.04	-2.22	-5.41	-8.46	-11.48	-14.36	-17.21	-20.01

4.2 Response History Analysis Results

4.2.1 Response History Graphs

After each response history analysis was completed, the output containing displacement in each direction, per story, per time step, was recorded. Each response history analyses ran at least three hours. Although there were more simulations than the pushover analysis, the entire RHA took less than two months. This is because the next analysis would automatically continue when the previous analysis failed - eliminating the need to periodically check on the script. Setting three forloops (for ground motion, f'_c and F_y) reduced the number of computer interactions.

Figure 22 is a time history floor displacement graph where the displacement per floor was recorded at each time step. The left side is the longitudinal motion, and the right is the transverse motion. The red dots are the maximum displacement per floor throughout the analysis. These graphs were made for every combination of steel, concrete and earthquake motion. To conserve space in this thesis, only one of the 1210 time history plots are shown. Instead, the data was processed so a general trend could be understood.

Originally, the data from all earthquakes were used in processing the results, but the results were inaccurate. It was determined that several RHAs did not run the entire duration and skewed the results. Appendix E lists the time each analysis elapsed before it failed. Earthquakes 2, 3, 4, 5 and 8 had most of the RHAs run for the entire duration so those earthquakes were used for the results in this section.

Figure 22-Sample RHA Time History - Floor Displacement Plot ($F'_c=30.8$, $F_y=500$, EQ1).

In order to condense the data, median floor displacements were plotted in a similar fashion as the pushover graphs (Figure 23). The entire set of combined RHA graphs can be seen in Appendix B. The presented results indicate that the story displacement decreased with increasing f'_c and F_y . This is due to the fact that higher concrete and steel strengths results in stronger sections.

Figure 23-Sample Response History Analyses graph ($F_y=400$ MPa, various f'_c determined from the average of several earthquake motions, X-direction)

4.2.2 Maximum Roof Displacements

To compare the data sets in its entirety, the maximum roof displacements were recorded. Table 12 and Table 13 shows the roof displacements to varying steel and concrete properties in the x and y direction, respectively. Additionally, graphical

representations were created to visualize the trend of displacement to steel-concrete strength in appendix E.

Table 12-Roof displacement(cm) table for varying f'_c and F_y . X-direction.

Fy(MPa)	f _c (MPa)										
	21.0	22.4	23.8	25.2	26.6	28.0	29.4	30.8	32.2	33.6	35.0
300	12.67	12.50	12.33	12.09	11.95	11.82	11.66	11.50	11.33	11.18	11.12
320	12.65	12.50	12.28	12.05	11.94	11.78	11.63	11.45	11.28	11.19	11.13
340	12.66	12.47	12.25	12.04	11.90	11.77	11.60	11.42	11.25	11.19	11.13
360	12.66	12.46	12.24	12.02	11.89	11.75	11.57	11.41	11.23	11.19	11.13
380	12.63	12.46	12.23	12.01	11.89	11.74	11.56	11.39	11.23	11.19	11.13
400	12.65	12.47	12.22	12.02	11.89	11.72	11.56	11.39	11.23	11.19	11.13
420	12.64	12.46	12.21	12.02	11.88	11.72	11.55	11.38	11.23	11.19	11.13
440	12.64	12.46	12.20	12.02	11.88	11.72	11.55	11.37	11.23	11.19	11.13
460	12.64	12.45	12.20	12.01	11.87	11.71	11.54	11.37	11.23	11.18	11.13
480	12.63	12.45	12.19	12.01	11.87	11.71	11.54	11.36	11.23	11.18	11.13
500	12.63	12.44	12.18	12.01	11.87	11.71	11.54	11.36	11.23	11.18	11.13

Table 13-Roof displacement (cm) table for varying f'_c and F_y . Y-direction.

Fy(MPa)	f _c (MPa)										
	21.0	22.4	23.8	25.2	26.6	28.0	29.4	30.8	32.2	33.6	35.0
300	14.24	13.99	13.76	13.56	13.36	13.18	13.01	12.83	12.67	12.50	12.33
320	14.16	13.90	13.67	13.47	13.27	13.08	12.90	12.69	12.54	12.37	12.20
340	14.07	13.82	13.59	13.37	13.17	12.98	12.79	12.61	12.44	12.27	12.10
360	13.98	13.72	13.49	13.28	13.07	12.88	12.69	12.51	12.34	12.17	12.00
380	13.89	13.63	13.39	13.18	12.98	12.79	12.60	12.43	12.26	12.09	11.91
400	13.80	13.54	13.30	13.09	12.89	12.71	12.52	12.35	12.18	12.01	11.84
420	13.72	13.46	13.22	13.02	12.82	12.63	12.45	12.28	12.12	11.94	11.77
440	13.65	13.39	13.15	12.95	12.75	12.57	12.39	12.22	12.05	11.89	11.71
460	13.59	13.33	13.09	12.89	12.69	12.51	12.33	12.17	12.00	11.84	11.66
480	13.53	13.28	13.04	12.84	12.65	12.46	12.29	12.13	11.96	11.80	11.63
500	13.49	13.23	13.00	12.80	12.61	12.43	12.26	12.09	11.93	11.77	11.60

Roof displacement decreases with higher steel yield and compressive concrete strengths. This is due to the fact that higher yield strength steel increases the strength, so the building displaces less as described in the pushover section. Higher compressive strength concrete also increases the building strength, so relative roof displacement

decreases. Interestingly, f_c has a greater affect on roof displacement with a difference of 1.55cm difference between $f_c=21$ to 35mpa at $F_y=300$ mpa compared to a 0.04 difference between $F_y=300$ to 500mpa at $f_c=21$ mpa (Table 8).

The frequency distribution of the roof displacements were plotted in Figure 24 and Figure 25. The x-direction had a mean of 11.78cm and a standard deviation of 0.50cm. The second mode had a mean of 12.79cm and a standard deviation of 0.65cm. Both means were within 0.1cm of the roof displacement at $f_c=28$ Mpa and $F_y=400$ MPa.

Figure 24-Histogram of roof displacement (cm), x-direction

Figure 25-Histogram of roof displacement (cm), y-direction

4.2.3 Percentage Difference Comparison

Lastly, we observe the percentage difference for various steel and concrete strengths with respect to nominal steel and concrete strength. Table 14 and Table 15 outline the roof displacement percentage differences in the transverse and longitudinal directions, respectively.

Table 14-Roof displacement percent difference (%) from $f'_c=28\text{MPa}$ and $F_y=400\text{MPa}$. X-direction

$F_y(\text{MPa})$	$f'_c (\text{MPa})$										
	21.0	22.4	23.8	25.2	26.6	28.0	29.4	30.8	32.2	33.6	35.0
300	-8.08	-6.64	-5.14	-3.15	-1.97	-0.86	0.07	1.94	3.39	4.64	5.17
320	-7.91	-6.48	-4.76	-2.83	-1.81	-0.51	0.79	2.33	3.82	4.56	5.09
340	-7.98	-6.40	-4.52	-2.69	-1.53	-0.38	1.04	2.59	4.01	4.54	5.05
360	-8.00	-6.26	-4.41	-2.55	-1.45	-0.22	1.32	2.71	4.18	4.55	5.06
380	-7.74	-6.32	-4.34	-2.45	-1.44	-0.10	1.41	2.80	4.19	4.57	5.07
400	-7.93	-6.33	-4.26	-2.50	-1.41	0.00	1.43	2.88	4.20	4.58	5.08
420	-7.85	-6.31	-4.18	-2.51	-1.37	0.05	1.45	2.95	4.20	4.59	5.08
440	-7.83	-6.25	-4.10	-2.49	-1.33	0.08	1.48	3.00	4.21	4.59	5.09
460	-7.80	-6.20	-4.02	-2.45	-1.28	0.09	1.53	3.05	4.21	4.60	5.09
480	-7.77	-6.15	-3.96	-2.43	-1.24	0.12	1.56	3.09	4.22	4.60	5.09
500	-7.74	-6.12	-3.92	-2.42	-1.22	0.13	1.58	3.11	4.22	4.60	5.10

Table 15-Roof displacement percent difference (%) from $f'_c=28\text{MPa}$ and $F_y=400\text{MPa}$. Y-direction

$F_y(\text{MPa})$	$f'_c (\text{MPa})$										
	21.0	22.4	23.8	25.2	26.6	28.0	29.4	30.8	32.2	33.6	35.0
300	-12.05	-10.14	-8.32	-6.69	-5.18	-3.77	-2.38	-1.02	0.30	1.62	2.95
320	-11.41	-9.41	-7.62	-6.00	-4.44	-2.97	-1.52	0.14	1.29	2.65	3.94
340	-10.74	-8.76	-6.92	-5.27	-3.67	-2.18	-0.70	0.77	2.13	3.46	4.80
360	-10.04	-8.00	-6.14	-4.49	-2.90	-1.38	0.12	1.53	2.86	4.20	5.56
380	-9.33	-7.26	-5.38	-3.72	-2.14	-0.65	0.84	2.18	3.50	4.87	6.24
400	-8.64	-6.56	-4.69	-3.05	-1.48	0.00	1.44	2.78	4.11	5.46	6.85
420	-8.00	-5.93	-4.08	-2.44	-0.88	0.58	2.01	3.33	4.65	6.00	7.38
440	-7.43	-5.37	-3.52	-1.90	-0.36	1.10	2.50	3.81	5.12	6.46	7.83
460	-6.93	-4.88	-3.05	-1.44	0.09	1.54	2.93	4.22	5.52	6.84	8.20
480	-6.52	-4.48	-2.66	-1.06	0.47	1.90	3.27	4.55	5.83	7.15	8.50
500	-6.19	-4.16	-2.35	-0.76	0.75	2.17	3.53	4.81	6.08	7.38	8.72

The roof displacement percentage difference increases as the material properties move further from the nominal value. Largest percentage differences can be seen at the extreme material properties with a maximum of -12% and 8.7% in the y-direction.

Concrete strength affects the roof displacement significantly compared to steel. In the x-direction, the largest range f'_c covers is from -8% to 5%, and the largest range F_y covers is from 5.17% to 5.10%.

An interesting observation is that the combination of material properties where the percentage difference below 1% exists. It can be seen as a “slash” diagonal line crossing through nominal values. This suggests that a material property can compensate for another’s increase or decrease in material strength.

CHAPTER 5 - CONCLUSION

This thesis investigated how different steel yield and compressive concrete strengths affect building performance. Pushover analyses were run for eight different modes, and response history analyses were run for ten pairs of ground motions. A brief overview of challenges facing the nonlinear static analysis, and modeling assumptions made for nonlinear static and dynamic analyses were discussed.

Due to non-convergence issues for higher modes, only the first two modes were used in the results section. It was found that increasing yield or compressive strengths do not have an effect on the initial stiffness. Once the pushover curve enters the post-yield range, higher F_y and f'_c values result in higher base shear capacities at a given displacement. Maximum base shear capacities can vary from nominal steel and concrete as much as 22%.

In terms of response statistics, the geometric median increased as f'_c or F_y increased. The dispersion measure decreased as f'_c or F_y increased. f'_c had a greater effect on the base shear capacity and F_y had a greater effect on the dispersion measure.

It was found through the response history analysis that increased steel yielding and concrete compressive strengths reduced the floor displacement of a building. The response history analyses also showed that concrete compressive strength had a greater effect on roof displacement than steel. The percentage difference in roof displacement varied as much as 12% compared to nominal steel and concrete strengths. It was observed that specific combinations of f'_c and F_y result in a roof displacement percentage difference of less than 1%. It can be concluded that larger steel yield and

compressive strengths result in higher base shear capacities and smaller story displacements.

Since all tested values ($f'_c=21$ to 35MPa and $F_y=300\text{MPa}$ to 500MPa) conformed with ACI-21.1.4.2 and ASTM A706, there is no cause for concern when using any of the properties in between. The pushover analyses showed that the base shear capacity for every combination fell within 22% of nominal f'_c and F_y values. The response history analysis showed that the roof displacements for every combination of material fell within 1.5cm (12%) of the nominal f'_c and F_y that was recommended. This displays the robustness of the analysis methods due to the insignificant differences in seismic response for large variations in material properties.

CHAPTER 6 – RECOMMENDATIONS FOR SUBSEQUENT STUDY

Several limitations and numerical issues were observed in this study. Since the pushover analysis was formulated with the assumptions that a structure was governed by the first mode, higher mode pushover curves were inaccurate due to non-convergence. For the response history analysis, many of the ground motions did not run the entire time step. These issues should be addressed in subsequent study.

Other methods may be used to expand the research topic. Instead of using Matlab and OpenSees, other software such as Perform 3D or Abaqus may be used. In Abaqus, implicit and explicit capabilities for seismic modeling may be addressed to determine when either should be used. Steel braced frames, different beam orientations, or lump plasticity models may be explored as well.

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APPENDICES

Appendix A: Mode Shapes and Periods

Mode 1: Period = 1.418

Figure A1 - Mode 1 Shape and Period

Mode 2: Period = 1.266

Figure A2 - Mode 2 Shape and Period

Mode 3: Period = 1.001

Figure A3 - Mode 3 Shape and Period

Mode 4: Period = 0.448

Figure A4 - Mode 4 Shape and Period

Mode 5: Period = 0.405

Figure A5 - Mode 5 Shape and Period

Mode 6: Period = 0.321

Figure A6 - Mode 6 Shape and Period

Mode 7: Period = 0.244

Figure A7 - Mode 7 Shape and Period

Mode 8: Period = 0.225

Figure A8 - Mode 8 Shape and Period

Mode 9: Period = 0.180

Figure A9 - Mode 9 Shape and Period

Mode 10: Period = 0.159

Figure A10- Mode 10 Shape and Period

Mode 11: Period = 0.148

Figure A11 - Mode 11 Shape and Period

Mode 12: Period = 0.118

Figure A12 - Mode 12 Shape and Period

Appendix B: Combined Pushover Analysis Graphs

Combined Pushover Graphs

f'_c Variation, Mode 1

Fig. B1 - Pushover: $f_c=21.0$ MPa, Mode=1

Fig. B2 - Pushover: $f_c=22.4$ MPa, Mode=1

Fig. B3 - Pushover: $f_c=23.8$ MPa, Mode=1

Fig. B4 - Pushover: $f_c=25.2$ MPa, Mode=1

Fig. B5 - Pushover: $f_c=26.6$ MPa, Mode=1

Fig. B6 - Pushover: $f_c=28.0$ MPa, Mode=1

Fig. B7 - Pushover: $f_c=29.4$ MPa, Mode=1

Fig. B8 - Pushover: $f_c=30.8$ MPa, Mode=1

Fig. B9 - Pushover: $f_c=32.2$ MPa. Mode=1

Fig. B10 - Pushover: $f_c=33.6$ MPa. Mode=1

Fig. B11 - Pushover: $f_c=35.0$ MPa. Mode=1

f'_c Variation, Mode 2

Fig. B12 - Pushover: $f_c=21.0$ MPa, Mode=2

Fig. B13 - Pushover: $f_c=22.4$ MPa, Mode=2

Fig. B14 - Pushover: $f_c=23.8$ MPa, Mode=2

Fig. B15 - Pushover: $f_c=25.2$ MPa, Mode=2

Fig. B16 - Pushover: $f_c=26.6$ MPa, Mode=2

Fig. B17 - Pushover: $f_c=28.0$ MPa, Mode=2

Fig. B18 - Pushover: $f_c=29.4$ MPa, Mode=2

Fig. B19 - Pushover: $f_c=30.8$ MPa, Mode=2

Fig. B20 - Pushover: $f_c=32.2$ MPa, Mode=2

Fig. B21 - Pushover: $f_c=33.6$ MPa, Mode=2

Fig. B22 - Pushover: $f_c=35.0$ MPa, Mode=2

f'c Variation, Mode 4

Fig. B23 - Pushover: $f_c=21.0$ MPa, Mode=4

Fig. B24 - Pushover: $f_c=22.4$ MPa, Mode=4

Fig. B25 - Pushover: $f_c=23.8$ MPa, Mode=4

Fig. B26 - Pushover: $f_c=25.2$ MPa, Mode=4

Fig. B27 - Pushover: $f_c=26.6$ MPa, Mode=4

Fig. B28 - Pushover: $f_c=28.0$ MPa, Mode=4

Fig. B29 - Pushover: $f_c=29.4$ MPa, Mode=4

Fig. B30 - Pushover: $f_c=30.8$ MPa, Mode=4

Fig. B31 - Pushover: $f_c=32.2$ MPa, Mode=4

Fig. B32 - Pushover: $f_c=33.6$ MPa, Mode=4

Fig. B33 - Pushover: $f_c=35.0$ MPa, Mode=4

f'c Variation, Mode 5

Fig. B34 - Pushover: $f_c=21.0$ MPa, Mode=5

Fig. B35 - Pushover: $f_c=22.4$ MPa, Mode=5

Fig. B36 - Pushover: $f_c=23.8$ MPa, Mode=5

Fig. B37 - Pushover: $f_c=25.2$ MPa, Mode=5

Fig. B38 - Pushover: $f_c=26.6$ MPa, Mode=5

Fig. B39 - Pushover: $f_c=28.0$ MPa, Mode=5

Fig. B40 - Pushover: $f_c=29.4$ MPa, Mode=5

Fig. B41 - Pushover: $f_c=30.8$ MPa, Mode=5

Fig. B42 - Pushover: $f_c=32.2$ MPa, Mode=5

Fig. B43 - Pushover: $f_c=33.6$ MPa, Mode=5

Fig. B44 - Pushover: $f_c=35.0$ MPa, Mode=5

f'c Variation, Mode 7

Fig. B45 - Pushover: $f_c=21.0$ MPa, Mode=7

Fig. B46 - Pushover: $f_c=22.4$ MPa, Mode=7

Fig. B47 - Pushover: $f_c=23.8$ MPa, Mode=7

Fig. B48 - Pushover: $f_c=25.2$ MPa, Mode=7

Fig. B49 - Pushover: $f_c=26.6$ MPa, Mode=7

Fig. B50 - Pushover: $f_c=28.0$ MPa, Mode=7

Fig. B51 - Pushover: $f_c=29.4$ MPa, Mode=7

Fig. B52 - Pushover: $f_c=30.8$ MPa, Mode=7

Fig. B53 - Pushover: $f_c=32.2$ MPa, Mode=7

Fig. B54 - Pushover: $f_c=33.6$ MPa, Mode=7

Fig. B55 - Pushover: $f_c=35.0$ MPa, Mode=7

f'c Variation, Mode 8

Fig. B56 - Pushover: $f_c=21.0$ MPa, Mode=8

Fig. B57 - Pushover: $f_c=22.4$ MPa, Mode=8

Fig. B58 - Pushover: $f_c=23.8$ MPa, Mode=8

Fig. B59 - Pushover: $f_c=25.2$ MPa, Mode=8

Fig. B60 - Pushover: $f_c=26.6$ MPa, Mode=8

Fig. B61 - Pushover: $f_c=28.0$ MPa, Mode=8

Fig. B62 - Pushover: $f_c=29.4$ MPa, Mode=8

Fig. B63 - Pushover: $f_c=30.8$ MPa, Mode=8

Fig. B64 - Pushover: $f_c=32.2$ MPa, Mode=8

Fig. B65 - Pushover: $f_c=33.6$ MPa, Mode=8

Fig. B66 - Pushover: $f_c=35.0$ MPa, Mode=8

f_c Variation, Mode 10

Fig. B67 - Pushover: f_c=21.0 MPa, Mode=10

Fig. B68 - Pushover: f_c=22.4 MPa, Mode=10

Fig. B69 - Pushover: f_c=23.8 MPa, Mode=10

Fig. B70 - Pushover: f_c=25.2 MPa, Mode=10

Fig. B71 - Pushover: $f_c=26.6$ MPa, Mode=10

Fig. B72 - Pushover: $f_c=28.0$ MPa, Mode=10

Fig. B73 - Pushover: $f_c=29.4$ MPa, Mode=10

Fig. B74 - Pushover: $f_c=30.8$ MPa, Mode=10

Fig. B75 - Pushover: $f_c=32.2$ MPa, Mode=10

Fig. B76 - Pushover: $f_c=33.6$ MPa, Mode=10

Fig. B77 - Pushover: $f_c=35.0$ MPa, Mode=10

f'_c Variation, Mode 11

Fig. B78 - Pushover: $f'_c=21.0$ MPa, Mode=11

Fig. B79 - Pushover: $f'_c=22.4$ MPa, Mode=11

Fig. B80 - Pushover: $f'_c=23.8$ MPa, Mode=11

Fig. B81 - Pushover: $f'_c=25.2$ MPa, Mode=11

Fig. B82 - Pushover: $f_c=26.6$ MPa, Mode=11

Fig. B83 - Pushover: $f_c=28.0$ MPa, Mode=11

Fig. B84 - Pushover: $f_c=29.4$ MPa, Mode=11

Fig. B85 - Pushover: $f_c=30.8$ MPa, Mode=11

Fig. B86 - Pushover: $f_c=32.2$ MPa, Mode=11

Fig. B87 - Pushover: $f_c=33.6$ MPa, Mode=11

Fig. B88 - Pushover: $f_c=35.0$ MPa, Mode=11

Fy Variation, Mode 1

Fig. B89 - Pushover: Fy=300 MPa, Mode=1

Fig. B90 - Pushover: Fy=320 MPa, Mode=1

Fig. B91 - Pushover: Fy=340 MPa, Mode=1

Fig. B92 - Pushover: Fy=360 MPa, Mode=1

Fig. B93 - Pushover: $F_y=380$ MPa, Mode=1

Fig. B94 - Pushover: $F_y=400$ MPa, Mode=1

Fig. B95 - Pushover: $F_y=420$ MPa, Mode=1

Fig. B96 - Pushover: $F_y=440$ MPa, Mode=1

Fig. B97 - Pushover: $F_y=460$ MPa, Mode=1

Fig. B98 - Pushover: $F_y=480$ MPa, Mode=1

Fig. B99 - Pushover: $F_y=500$ MPa, Mode=1

Fy Variation, Mode 2

Fig. B100 - Pushover: Fy=300 MPa, Mode=2

Fig. B101 - Pushover: Fy=320 MPa, Mode=2

Fig. B102 - Pushover: Fy=340 MPa, Mode=2

Fig. B103 - Pushover: Fy=360 MPa, Mode=2

Fig. B104 - Pushover: $F_y=380$ MPa, Mode=2

Fig. B105 - Pushover: $F_y=400$ MPa, Mode=2

Fig. B106 - Pushover: $F_y=420$ MPa, Mode=2

Fig. B107 - Pushover: $F_y=440$ MPa, Mode=2

Fig. B108 - Pushover: $F_y=460$ MPa, Mode=2

Fig. B109 - Pushover: $F_y=480$ MPa, Mode=2

Fig. B110 - Pushover: $F_y=500$ MPa, Mode=2

Fy Variation, Mode 4

Fig. B111 - Pushover: $F_y=300$ MPa, Mode=4

Fig. B112 - Pushover: $F_y=320$ MPa, Mode=4

Fig. B113 - Pushover: $F_y=340$ MPa, Mode=4

Fig. B114 - Pushover: $F_y=360$ MPa, Mode=4

Fig. B115 - Pushover: $F_y=380$ MPa, Mode=4

Fig. B116 - Pushover: $F_y=400$ MPa, Mode=4

Fig. B117 - Pushover: $F_y=420$ MPa, Mode=4

Fig. B118 - Pushover: $F_y=440$ MPa, Mode=4

Fig. B119 - Pushover: $F_y=460$ MPa, Mode=4

Fig. B120 - Pushover: $F_y=480$ MPa, Mode=4

Fig. B121 - Pushover: $F_y=500$ MPa, Mode=4

Fy Variation, Mode 5

Fig. B122 - Pushover: Fy=300 MPa, Mode=5

Fig. B123 - Pushover: Fy=320 MPa, Mode=5

Fig. B124 - Pushover: Fy=340 MPa, Mode=5

Fig. B125 - Pushover: Fy=360 MPa, Mode=5

Fig. B126 - Pushover: $F_y=380$ MPa, Mode=5

Fig. B127 - Pushover: $F_y=400$ MPa, Mode=5

Fig. B128 - Pushover: $F_y=420$ MPa, Mode=5

Fig. B129 - Pushover: $F_y=440$ MPa, Mode=5

Fig. B130 - Pushover: $F_y=460$ MPa, Mode=5

Fig. B131 - Pushover: $F_y=480$ MPa, Mode=5

Fig. B132 - Pushover: $F_y=500$ MPa, Mode=5

Fy Variation, Mode 7

Fig. B133 - Pushover: Fy=300 MPa, Mode=7

Fig. B134 - Pushover: Fy=320 MPa, Mode=7

Fig. B135 - Pushover: Fy=340 MPa, Mode=7

Fig. B136 - Pushover: Fy=360 MPa, Mode=7

Fig. B137 - Pushover: $F_y=380$ MPa, Mode=7

Fig. B138 - Pushover: $F_y=400$ MPa, Mode=7

Fig. B139 - Pushover: $F_y=420$ MPa, Mode=7

Fig. B140 - Pushover: $F_y=440$ MPa, Mode=7

Fig. B141 - Pushover: $F_y=460$ MPa, Mode=7

Fig. B142 - Pushover: $F_y=480$ MPa, Mode=7

Fig. B143 - Pushover: $F_y=500$ MPa, Mode=7

Fy Variation, Mode 8

Fig. B144 - Pushover: Fy=300 MPa, Mode=8

Fig. B145 - Pushover: Fy=320 MPa, Mode=8

Fig. B146 - Pushover: Fy=340 MPa, Mode=8

Fig. B147 - Pushover: Fy=360 MPa, Mode=8

Fig. B148 - Pushover: $F_y=380$ MPa, Mode=8

Fig. B149 - Pushover: $F_y=400$ MPa, Mode=8

Fig. B150 - Pushover: $F_y=420$ MPa, Mode=8

Fig. B151 - Pushover: $F_y=440$ MPa, Mode=8

Fig. B152 - Pushover: $F_y=460$ MPa, Mode=8

Fig. B153 - Pushover: $F_y=480$ MPa, Mode=8

Fig. B154 - Pushover: $F_y=500$ MPa, Mode=8

Fy Variation, Mode 10

Fig. B155 - Pushover: Fy=300 MPa, Mode=10

Fig. B156 - Pushover: Fy=320 MPa, Mode=10

Fig. B157 - Pushover: Fy=340 MPa, Mode=10

Fig. B158 - Pushover: Fy=360 MPa, Mode=10

Fig. B159 - Pushover: $F_y=380$ MPa, Mode=10

Fig. B160 - Pushover: $F_y=400$ MPa, Mode=10

Fig. B161 - Pushover: $F_y=420$ MPa, Mode=10

Fig. B162 - Pushover: $F_y=440$ MPa, Mode=10

Fig. B163 - Pushover: $F_y=460$ MPa, Mode=10

Fig. B164 - Pushover: $F_y=480$ MPa, Mode=10

Fig. B165 - Pushover: $F_y=500$ MPa, Mode=10

Fy Variation, Mode 11

Fig. B166 - Pushover: Fy=300 MPa, Mode=11

Fig. B167 - Pushover: Fy=320 MPa, Mode=11

Fig. B168 - Pushover: Fy=340 MPa, Mode=11

Fig. B169 - Pushover: Fy=360 MPa, Mode=11

Fig. B170 - Pushover: $F_y=380$ MPa, Mode=11

Fig. B171 - Pushover: $F_y=400$ MPa, Mode=11

Fig. B172 - Pushover: $F_y=420$ MPa, Mode=11

Fig. B173 - Pushover: $F_y=440$ MPa, Mode=11

Fig. B174 - Pushover: $F_y=460$ MPa, Mode=11

Fig. B175 - Pushover: $F_y=480$ MPa, Mode=11

Fig. B176 - Pushover: $F_y=500$ MPa, Mode=11

Appendix C: Combined Response History Analysis Graphs

Combined RHA Graphs

X-Direction, f'_c Variation

Fig. C1 - RHA: $f'_c=21.0$ MPa, X-direction

Fig. C2 - RHA: $f'_c=22.4$ MPa, X-direction

Fig. C3 - RHA: $f'_c=23.8$ MPa, X-direction

Fig. C4 - RHA: $f'_c=25.2$ MPa, X-direction

Fig. C5 - RHA: $f'_c=26.6$ MPa, X-direction

Fig. C6 - RHA: $f'_c=28.0$ MPa, X-direction

Fig. C7 - RHA: $f_c=29.4$ MPa, X-direction

Fig. C8 - RHA: $f_c=30.8$ MPa, X-direction

Fig. C9 - RHA: $f_c=32.2$ MPa, X-direction

Fig. C10 - RHA: $f_c=33.6$ MPa, X-direction

Fig. C11 - RHA: $f_c=35.0$ MPa, X-direction

X-Direction, F_y Variation

Fig. C12 - RHA: $F_y=300$ MPa, X-direction

Fig. C13 - RHA: $F_y=320$ MPa, X-direction

Fig. C14 - RHA: $F_y=340$ MPa, X-direction

Fig. C15 - RHA: $F_y=360$ MPa, X-direction

Fig. C16 - RHA: $F_y=380$ MPa, X-direction

Fig. C17 - RHA: $F_y=400$ MPa, X-direction

Fig. C18 - RHA: $F_y=420$ MPa, X-direction

Fig. C19 - RHA: $F_y=440$ MPa, X-direction

Fig. C20 - RHA: $F_y=460$ MPa, X-direction

Fig. C21 - RHA: $F_y=480$ MPa, X-direction

Fig. C22 - RHA: $F_y=500$ MPa, X-direction

Y-Direction, f'_c Variation

Fig. C23 - RHA: $f'_c=21.0$ MPa, Y-direction

Fig. C24 - RHA: $f'_c=22.4$ MPa, Y-direction

Fig. C25 - RHA: $f'_c=23.8$ MPa, Y-direction

Fig. C26 - RHA: $f'_c=25.2$ MPa, Y-direction

Fig. C27 - RHA: $f'_c=26.6$ MPa, Y-direction

Fig. C28 - RHA: $f'_c=28.0$ MPa, Y-direction

Fig. C29 - RHA: $f_c=29.4$ MPa, Y-direction

Fig. C30 - RHA: $f_c=30.8$ MPa, Y-direction

Fig. C31 - RHA: $f_c=32.2$ MPa, Y-direction

Fig. C32 - RHA: $f_c=33.6$ MPa, Y-direction

Fig. C33 - RHA: $f_c=35.0$ MPa, Y-direction

Y-Direction, F_y Variation

Fig. C34 - RHA: $F_y=300$ MPa, Y-direction

Fig. C35 - RHA: $F_y=320$ MPa, Y-direction

Fig. C36 - RHA: $F_y=340$ MPa, Y-direction

Fig. C37 - RHA: $F_y=360$ MPa, Y-direction

Fig. C38 - RHA: $F_y=380$ MPa, Y-direction

Fig. C39 - RHA: $F_y=400$ MPa, Y-direction

Fig. C40 - RHA: $F_y=420$ MPa, Y-direction

Fig. C41 - RHA: $F_y=440$ MPa, Y-direction

Fig. C42 - RHA: $F_y=460$ MPa, Y-direction

Fig. C43 - RHA: $F_y=480$ MPa, Y-direction

Fig. C44 - RHA: $F_y=500$ MPa, Y-direction

Appendix D: Pushover Results

Pushover Results

Table D1- Base Shear Capacity Geometric Median of F_y (10^4 kN)

f'_c	mode 1	mode 2	mode 4	mode 5	mode 7	mode 8	mode 10	mode 11
21	5.31	3.91	0.48	0.51	5.22	4.71	1.80	1.74
22.4	5.36	3.96	0.49	0.51	5.50	4.43	1.87	1.79
23.8	5.41	4.01	0.49	0.52	5.55	4.67	1.88	1.78
25.2	5.46	4.05	0.50	0.53	5.81	4.70	1.90	1.82
26.6	5.51	4.10	0.50	0.53	5.71	5.07	1.92	1.81
28	5.56	4.15	0.51	0.54	5.56	4.74	1.94	1.84
29.4	5.61	4.19	0.51	0.55	5.53	5.17	1.96	1.87
30.8	5.66	4.24	0.52	0.55	5.15	4.84	1.99	1.89
32.2	5.69	4.28	0.53	0.56	5.47	5.19	2.00	1.91
33.6	5.75	4.32	0.53	0.57	5.52	4.88	2.01	1.91
35	5.79	4.37	0.54	0.57	5.74	4.74	2.04	1.95

Figure D1-Geometric Median of F_y for various modes

Table D2-Base Shear Capacity Geometric Median of f'_c (10^4kN)

Fy	mode 1	mode 2	mode 4	mode 5	mode 7	mode 8	mode 10	mode 11
300	4.59	3.48	0.51	0.54	4.69	3.76	1.93	1.83
320	4.81	3.63	0.51	0.54	5.07	4.21	1.90	1.87
340	5.01	3.78	0.51	0.54	4.87	4.58	1.93	1.84
360	5.22	3.92	0.51	0.54	5.07	4.79	1.94	1.84
380	5.42	4.05	0.51	0.54	5.09	4.72	1.92	1.83
400	5.61	4.18	0.51	0.54	5.47	4.96	1.95	1.85
420	5.80	4.30	0.51	0.54	5.90	4.96	1.95	1.85
440	5.99	4.43	0.51	0.54	6.04	5.28	1.95	1.85
460	6.17	4.55	0.51	0.54	5.91	5.47	1.95	1.85
480	6.35	4.67	0.51	0.54	6.26	5.15	1.95	1.85
500	6.52	4.78	0.51	0.54	6.73	5.51	1.95	1.85

Figure D2-Geometric Median of f'_c for various modes

Table D3- Base Shear Capacity Dispersion measure of Fy (10⁴kN)

f _c	mode 1	mode 2	mode 4	mode 5	mode 7	mode 8	mode 10	mode 11
21	0.1194	0.1086	0.0004	0.0004	0.1611	0.1295	0.0857	0.0092
22.4	0.1187	0.1078	0.0003	0.0003	0.1517	0.2283	0.0009	0.0287
23.8	0.1178	0.1070	0.0002	0.0003	0.1889	0.1688	0.0213	0.0346
25.2	0.1171	0.1062	0.0002	0.0002	0.2131	0.2014	0.0217	0.0255
26.6	0.1163	0.1054	0.0001	0.0002	0.1584	0.1851	0.0146	0.0386
28	0.1156	0.1046	0.0001	0.0001	0.1876	0.1640	0.0217	0.0234
29.4	0.1145	0.1039	0.0001	0.0001	0.2285	0.1081	0.0165	0.0029
30.8	0.1138	0.1032	0.0002	0.0032	0.2036	0.2143	0.0020	0.0045
32.2	0.1162	0.1027	0.0002	0.0001	0.1902	0.1333	0.0201	0.0028
33.6	0.1128	0.1022	0.0003	0.0001	0.1280	0.0977	0.0208	0.0260
35	0.1127	0.1019	0.0004	0.0002	0.1089	0.1623	0.0025	0.0026

Figure D3-Dispersion Measure of Fy for various modes

Table D4- Base Shear Capacity Dispersion Measure for f'_c (10^4 kN)

Fy	mode 1	mode 2	mode 4	mode 5	mode 7	mode 8	mode 10	mode 11
300	0.0323	0.0404	0.0361	0.0396	0.1516	0.1964	0.0323	0.0377
320	0.0321	0.0394	0.0360	0.0395	0.1662	0.1172	0.1055	0.0330
340	0.0298	0.0389	0.0360	0.0394	0.1510	0.1268	0.0462	0.0468
360	0.0300	0.0382	0.0359	0.0394	0.1460	0.1181	0.0423	0.0487
380	0.0292	0.0375	0.0358	0.0393	0.1035	0.1458	0.0364	0.0556
400	0.0285	0.0371	0.0357	0.0393	0.1564	0.1039	0.0324	0.0367
420	0.0279	0.0364	0.0357	0.0392	0.1155	0.1024	0.0324	0.0367
440	0.0273	0.0357	0.0356	0.0400	0.1433	0.1302	0.0323	0.0374
460	0.0267	0.0347	0.0355	0.0391	0.0953	0.1022	0.0322	0.0366
480	0.0262	0.0338	0.0355	0.0391	0.1477	0.1606	0.0322	0.0366
500	0.0257	0.0332	0.0354	0.0390	0.1387	0.1154	0.0321	0.0365

Figure D4-Dispersion Measure of f'_c for various modes

Table D5-Maximum base shear capacity for various f'_c and F_y (10^4 kN), Mode 1

f'_c (MPa)	F_y (MPa)										
	300	320	340	360	380	400	420	440	460	480	500
21	4.36	4.57	4.78	4.98	5.18	5.36	5.55	5.74	5.91	6.09	6.26
22.4	4.41	4.62	4.83	5.03	5.23	5.42	5.61	5.79	5.97	6.14	6.32
23.8	4.46	4.67	4.88	5.08	5.28	5.47	5.66	5.84	6.02	6.20	6.37
25.2	4.51	4.72	4.92	5.13	5.33	5.52	5.71	5.89	6.07	6.25	6.42
26.6	4.56	4.76	4.97	5.18	5.38	5.57	5.76	5.94	6.12	6.30	6.47
28	4.60	4.81	5.02	5.22	5.42	5.62	5.80	5.99	6.17	6.35	6.52
29.4	4.65	4.86	5.07	5.27	5.47	5.67	5.85	6.04	6.22	6.40	6.57
30.8	4.70	4.91	5.11	5.32	5.52	5.71	5.90	6.09	6.27	6.45	6.62
32.2	4.72	4.95	5.16	5.36	5.56	5.76	5.95	6.14	6.32	6.50	6.67
33.6	4.78	4.99	5.20	5.41	5.61	5.80	5.99	6.18	6.37	6.55	6.72
35	4.83	5.04	5.20	5.45	5.66	5.85	6.04	6.23	6.41	6.59	6.77

Figure D5- Base Shear for various f'_c and F_y . Mode 1.

Table D6- Maximum base shear capacity for various f'_c and F_y (10^4 kN), Mode 2

f'_c (MPa)	F_y (MPa)											
	300	320	340	360	380	400	420	440	460	480	500	
21	3.26	3.41	3.55	3.69	3.82	3.94	4.07	4.19	4.31	4.43	4.54	
22.4	3.31	3.46	3.60	3.74	3.87	3.99	4.12	4.24	4.36	4.48	4.59	
23.8	3.35	3.50	3.65	3.78	3.92	4.04	4.17	4.29	4.41	4.53	4.64	
25.2	3.40	3.55	3.69	3.83	3.96	4.09	4.21	4.34	4.46	4.58	4.69	
26.6	3.44	3.59	3.74	3.88	4.01	4.14	4.26	4.39	4.51	4.63	4.74	
28	3.48	3.64	3.78	3.92	4.06	4.18	4.31	4.43	4.55	4.67	4.79	
29.4	3.53	3.68	3.83	3.97	4.10	4.23	4.36	4.48	4.60	4.72	4.84	
30.8	3.57	3.72	3.87	4.01	4.15	4.28	4.40	4.53	4.65	4.77	4.88	
32.2	3.61	3.76	3.91	4.05	4.19	4.32	4.45	4.57	4.69	4.81	4.93	
33.6	3.65	3.81	3.96	4.10	4.23	4.37	4.49	4.62	4.74	4.86	4.98	
35	3.68	3.84	4.00	4.14	4.28	4.41	4.54	4.67	4.79	4.91	5.02

Figure D6-Base shear capacity for various f'_c and F_y . Mode 2.

Table D7-Base shear capacity % difference from $f'_c=28\text{MPa}$ and $F_y=400\text{MPa}$.

Mode 1.

Fy(MPa)	f _c (MPa)											
	21	22.4	23.8	25.2	26.6	28	29.4	30.8	32.2	33.6	35	
300	22.33	18.60	14.91	11.31	7.82	4.50	1.12	-2.15	-5.31	-8.44	-11.48	
320	21.46	17.72	14.05	10.44	6.89	3.51	0.18	-3.10	-6.28	-9.40	-12.45	
340	20.60	16.83	13.17	9.56	6.02	2.59	-0.71	-3.99	-7.22	-10.35	-13.41	
360	19.75	16.00	12.32	8.70	5.17	1.71	-1.60	-4.89	-8.14	-11.27	-14.34	
380	18.90	15.17	11.52	7.86	4.28	0.85	-2.47	-5.79	-9.03	-12.17	-15.25	
400	18.07	14.36	10.70	7.02	3.46	0.00	-3.34	-6.66	-9.93	-13.06	-16.14	
420	17.20	13.47	9.80	6.16	2.61	-0.87	-4.19	-7.52	-10.78	-13.94	-17.03	
440	16.40	12.67	9.02	5.31	1.77	-1.69	-5.07	-8.36	-11.63	-14.80	-17.90	
460	15.63	11.91	8.13	4.52	0.96	-2.48	-5.91	-9.26	-12.50	-15.69	-18.76	
480	14.91	11.13	7.43	3.72	0.13	-3.31	-6.72	-10.11	-13.36	-16.54	-19.63	
500	14.07	10.32	7.34	2.92	-0.72	-4.19	-7.56	-10.93	-14.18	-17.38	-20.48

Figure D7-Base shear capacity % difference from $f'_c=28\text{MPa}$ and $F_y=400\text{MPa}$.

Mode 1.

Table D8-Base shear capacity % difference from $f'_c=28\text{MPa}$ and $F_y=400\text{MPa}$.

Mode 2.

Fy(MPa)	fc (MPa)											
	21	22.4	23.8	25.2	26.6	28	29.4	30.8	32.2	33.6	35	
300	22.03	18.42	15.06	11.81	8.72	5.75	2.81	-0.11	-2.99	-5.83	-8.57	
320	20.96	17.34	13.94	10.68	7.55	4.56	1.62	-1.29	-4.19	-7.03	-9.78	
340	19.90	16.27	12.84	9.56	6.41	3.39	0.44	-2.47	-5.37	-8.21	-10.97	
360	18.85	15.21	11.74	8.46	5.29	2.24	-0.72	-3.65	-6.53	-9.38	-12.14	
380	17.80	14.16	10.66	7.37	4.18	1.12	-1.88	-4.80	-7.67	-10.53	-13.29	
400	16.75	13.11	9.61	6.29	3.07	0.00	-3.01	-5.95	-8.82	-11.66	-14.44	
420	15.73	12.10	8.56	5.22	1.99	-1.12	-4.12	-7.08	-9.94	-12.79	-15.58	
440	14.72	11.08	7.52	4.17	0.91	-2.21	-5.22	-8.20	-11.06	-13.90	-16.70	
460	13.79	10.06	6.49	3.12	-0.14	-3.29	-6.32	-9.31	-12.17	-15.02	-17.81	
480	12.87	9.06	5.47	2.07	-1.17	-4.36	-7.41	-10.40	-13.27	-16.12	-18.92	
500	12.01	8.21	4.46	1.04	-2.22	-5.41	-8.46	-11.48	-14.36	-17.21	-20.01

Figure D8-Base shear capacity % difference from $f'_c=28\text{MPa}$ and $F_y=400\text{MPa}$.

Mode 2.

Appendix E: RHA Results

RHA Results

Time to failure (seconds)

Table E1-	eq1	Imperial Valley, 1940, El Centro					53.46s	duration				
Fy(Mpa)	Fc(Mpa)	21	22.4	23.8	25.2	26.6	28	29.4	30.8	32.2	33.6	35
300	2.2	2.2	2.56	2.74	53.46	2.28	2.22	53.46	53.46	12.48	2.18	
320	2.2	2.3	2.28	53.46	2.28	53.46	2.26	2.18	2.18	2.2	53.46	
340	2.2	2.52	53.46	53.46	53.46	53.46	2.18	2.18	2.18	2.18	2.18	
360	53.46	53.46	53.46	53.46	53.46	53.46	53.46	53.46	53.46	2.18	2.18	
380	2.92	53.46	3.1	53.46	53.46	53.46	53.46	3.06	53.46	53.46	53.46	
400	2.9	53.46	2.54	6.88	3.3	53.46	53.46	2.46	5.08	53.46	53.46	
420	3.28	4.02	3.08	2.46	2.46	2.46	2.46	53.46	3.26	2.66	3.26	
440	53.46	53.46	2.88	53.46	53.46	53.46	53.46	53.46	2.84	53.46	53.46	
460	3.74	53.46	53.46	53.46	12.44	3.28	53.46	53.46	53.46	53.46	53.46	
480	53.46	53.46	53.46	53.46	53.46	12.8	53.46	6.82	53.46	53.46	53.46	
500	53.46	53.46	53.46	53.46	53.46	53.46	33.34	53.46	53.46	53.46	53.46	

Table E2-	eq2	Imperial Valley, 1979, Array #05					39.38s	duration				
Fy(Mpa)	Fc(Mpa)	21	22.4	23.8	25.2	26.6	28	29.4	30.8	32.2	33.6	35
300	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38
320	32.7	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38
340	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38
360	39.38	39.38	15.36	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38
380	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38
400	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38
420	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38
440	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38
460	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38
480	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38
500	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38	39.38

Table E3-	eq3	Imperial Valley, 1979, Array #06					39.08s	duration				
Fy(Mpa)	Fc(Mpa)	21	22.4	23.8	25.2	26.6	28	29.4	30.8	32.2	33.6	35
300	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08
320	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08
340	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08
360	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08
380	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	38.95	39.08	
400	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	
420	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	
440	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	
460	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	
480	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	
500	39.08	39.08	35.65	39.08	39.08	39.08	39.08	39.08	39.08	39.08	39.08	

Table E4-	eq4	Landers, 1992, Barstow					79.98s	duration				
Fy(Mpa)	Fc(Mpa)	21	22.4	23.8	25.2	26.6	28	29.4	30.8	32.2	33.6	35
300	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	60	79.98	79.98	79.98
320	79.98	79.98	79.98	79.98	16.94	79.98	79.98	79.98	79.98	79.98	79.98	79.98
340	79.98	16.56	79.98	79.98	79.98	79.98	79.98	79.98	16.88	79.98	79.98	79.98
360	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98
380	79.98	79.98	79.98	16.9	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98
400	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98
420	79.98	79.98	18	79.98	79.98	16.88	79.98	79.98	79.98	79.98	79.98	79.98
440	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98
460	79.98	79.98	17.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98
480	79.98	17.6	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98	79.98
500	79.98	79.98	79.98	79.98	79.98	17.9	79.98	79.98	79.98	79.98	79.98	79.98

Table E5-	eq5	Landers, 1992, Yermo	79.98s	duration
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[illegible]

Table E6-	eq6	Loma Prieta, 1989, Gilroy	39.98s	duration
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[illegible]

Table E7-	eq7	Northridge, 1994, Newhall	59.98s	duration
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Fy(Mpa)	Fc(Mpa)										
	21	22.4	23.8	25.2	26.6	28	29.4	30.8	32.2	33.6	35
300	5.72	4.86	5.5	4.86	5.52	5.7	5.68	7.46	5.66	5.46	6.36
320	6.2	4.86	5.52	5.5	4.58	5.26	5.84	5.66	5.28	59.98	4.9
340	4.54	5.5	5.68	6	5.66	6.12	5.66	5.5	5.68	5.72	59.98
360	5.52	5.5	59.98	6.8	59.98	5.96	5.68	7.08	59.98	59.98	5.68
380	6.18	6.08	5.94	6.42	59.98	7.7	6.26	6.44	59.98	6.08	59.98
400	59.98	6.48	5.94	5.66	4.8	5.68	5.72	5.82	6.26	6.26	5.7
420	7.48	5.72	6.26	5.68	6.26	6.5	6.26	6.52	5.66	6.16	6.2
440	5.66	59.98	6.56	5.5	5.68	5.5	6.2	6.2	59.98	6.1	9.4
460	59.98	6.58	6.52	5.5	5.5	6.54	7.2	59.98	8.02	6.5	6.32
480	6.26	6.58	6.54	6.52	6.5	6.52	7.4	10.04	6.46	8.14	7.96
500	6.46	59.98	6.54	6.52	8	6.24	5.62	7.1	8.18	59.98	7.74

Table E8-	eq8	Northridge, 1994, Rinaldi RS	14.95s	duration
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[illegible]

Table E9-	eq9	Northridge, 1994, Sylmar				59.98s	duration					
Fy(Mpa)	f _c (Mpa)	21	22.4	23.8	25.2	26.6	28	29.4	30.8	32.2	33.6	35
300	4.42	4.52	4.4	15.64	5.22	59.98	4.6	3.9	3.9	4.36	4.02	
320	0.52	59.98	4.7	4.46	5.06	4.74	4.34	4.34	3.98	4.18	4.2	
340	3.92	4.24	7.74	4.2	3.96	4.7	4.72	4.38	4.12	4.12	4.62	
360	59.98	59.98	4.58	4.56	59.98	5.22	59.98	59.98	4.4	59.98	5.16	
380	4.54	4.24	59.98	5.04	13.06	59.98	59.98	59.98	4.38	5.2	5.12	
400	59.98	35.34	59.98	59.98	59.98	59.98	59.98	4	4.16	4.16	59.98	
420	59.98	59.98	59.98	59.98	4.36	59.98	59.98	59.98	59.98	59.98	59.98	
440	5.56	59.98	59.98	59.98	4.72	5.46	4.42	5.16	5.14	4.2	5.1	
460	59.98	5.54	59.98	59.98	4.4	5.16	59.98	59.98	59.98	59.98	59.98	
480	59.98	59.98	59.98	59.98	59.98	5.16	59.98	5.8	59.98	59.98	59.98	
500	6.18	59.98	59.98	59.98	59.98	59.98	59.98	59.98	59.98	59.98	5.46	

Table E10-	eq10	North Palm Springs, 1986				59.98s	duration					
Fy(Mpa)	f'c(Mpa)	21	22.4	23.8	25.2	26.6	28	29.4	30.8	32.2	33.6	35
300	59.98	59.98	59.98	59.98	59.98	59.98	6.36	4.18	5.24	3.96	3.7	3.2
320	3.52	4.24	6.7	3.28	59.98	3.28	3.06	6.66	3.06	3.06	3.06	3.3
340	3.7	5.98	2.8	59.98	3.88	4.76	2.9	2.9	3.22	59.98	59.98	59.98
360	4.46	6.38	6.38	59.98	3.86	3.86	3.32	3.24	3.06	3.06	7.58	7.58
380	4.94	9.18	3.86	3.86	3.86	3.3	3.22	3.24	3.24	5.7	5.04	5.04
400	3.2	3.86	4.18	3.1	3.08	3.08	3.06	3.68	3.68	4.74	3.42	3.42
420	59.98	4.18	4.16	4.2	59.98	59.98	3.94	3.44	59.98	59.98	4.42	4.42
440	4.22	4.18	59.98	59.98	59.98	3.08	6.68	5.42	3.78	3.04	59.98	59.98
460	9.54	7.38	4.14	7.66	4.76	4.12	3.44	5.74	4.1	4.42	9.08	9.08
480	59.98	5.12	4.5	59.98	6.94	59.98	3.42	59.98	59.98	3.06	59.98	59.98
500	6.72	9.4	4.44	59.98	59.98	59.98	5.42	4.12	3.7	59.98	7.9	7.9

Table E11-Roof Displacement (cm), X-direction, EQ1

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	15.06	15.15	15.25	15.29	15.34	15.38	15.41	15.42	15.43	15.44	15.27
320	15.25	15.39	15.42	15.48	15.52	15.56	15.58	15.31	15.37	15.59	15.58
340	15.37	15.49	15.55	15.61	15.66	15.69	15.41	15.48	15.54	15.60	15.64
360	15.52	15.54	15.63	15.69	15.74	15.78	15.80	15.81	15.81	15.71	15.76
380	15.55	15.59	15.67	15.74	15.79	15.83	15.85	15.86	15.86	15.85	15.83
400	15.57	15.62	15.70	15.77	15.82	15.85	15.88	15.89	15.89	15.88	15.87
420	15.58	15.64	15.72	15.79	15.84	15.87	15.90	15.91	15.90	15.90	15.90
440	15.59	15.65	15.74	15.80	15.85	15.89	15.91	15.92	15.92	15.91	15.91
460	15.60	15.67	15.75	15.82	15.86	15.90	15.92	15.93	15.93	15.91	15.93
480	15.61	15.67	15.76	15.82	15.87	15.91	15.93	15.94	15.93	15.92	15.94
500	15.61	15.68	15.76	15.83	15.88	15.91	15.93	15.94	15.94	15.93	15.94

Figure E1-Roof Displacement (cm), X-direction, EQ1

Table E12-Roof Displacement (cm), X-direction, EQ2

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	9.09	9.47	9.79	10.05	10.27	10.44	10.58	10.68	10.73	10.78	10.79
320	9.10	9.47	9.80	10.05	10.28	10.44	10.58	10.68	10.74	10.79	10.79
340	9.10	9.48	9.80	10.05	10.28	10.44	10.58	10.68	10.74	10.79	10.79
360	9.10	9.48	9.80	10.06	10.28	10.45	10.58	10.68	10.74	10.79	10.79
380	9.11	9.48	9.80	10.06	10.28	10.45	10.58	10.68	10.74	10.79	10.79
400	9.11	9.49	9.80	10.06	10.28	10.45	10.58	10.68	10.74	10.79	10.79
420	9.11	9.49	9.81	10.06	10.28	10.45	10.58	10.68	10.74	10.79	10.79
440	9.11	9.49	9.81	10.06	10.29	10.45	10.59	10.68	10.74	10.79	10.79
460	9.12	9.49	9.81	10.06	10.29	10.45	10.59	10.68	10.74	10.79	10.79
480	9.12	9.50	9.81	10.07	10.29	10.45	10.59	10.68	10.74	10.79	10.79
500	9.12	9.50	9.81	10.07	10.29	10.45	10.59	10.68	10.74	10.79	10.79

Figure E2-Roof Displacement (cm), X-direction, EQ2

Table E13-Roof Displacement (cm), X-direction, EQ3

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	8.71	8.69	8.67	8.66	8.64	8.62	8.60	8.58	8.55	8.53	8.49
320	8.71	8.69	8.67	8.66	8.64	8.62	8.60	8.58	8.55	8.53	8.49
340	8.71	8.69	8.67	8.66	8.64	8.62	8.60	8.58	8.55	8.53	8.49
360	8.70	8.69	8.67	8.66	8.64	8.62	8.60	8.58	8.55	8.53	8.49
380	8.70	8.69	8.67	8.66	8.64	8.62	8.60	8.58	8.55	8.53	8.50
400	8.70	8.69	8.67	8.66	8.64	8.62	8.60	8.58	8.55	8.53	8.50
420	8.70	8.69	8.67	8.66	8.64	8.62	8.60	8.58	8.55	8.53	8.50
440	8.70	8.69	8.67	8.66	8.64	8.62	8.60	8.58	8.55	8.53	8.49
460	8.70	8.69	8.67	8.66	8.64	8.62	8.60	8.58	8.55	8.53	8.49
480	8.70	8.69	8.67	8.66	8.64	8.62	8.60	8.58	8.55	8.53	8.49
500	8.70	8.69	8.67	8.66	8.64	8.62	8.60	8.58	8.55	8.53	8.49

Figure E3-Roof Displacement (cm), X-direction, EQ3

Table E14-Roof Displacement (cm), X-direction, EQ4

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	13.40	12.97	12.52	11.99	11.86	11.82	11.73	11.63	11.56	11.47	11.36
320	13.36	12.94	12.44	11.92	11.88	11.84	11.75	11.63	11.61	11.51	11.40
340	13.33	12.89	12.36	11.91	11.89	11.85	11.76	11.69	11.63	11.53	11.43
360	13.31	12.85	12.31	11.91	11.88	11.85	11.76	11.70	11.63	11.53	11.44
380	13.29	12.82	12.27	11.90	11.88	11.84	11.75	11.69	11.62	11.53	11.44
400	13.28	12.80	12.24	11.90	11.88	11.84	11.75	11.69	11.62	11.52	11.43
420	13.27	12.79	12.23	11.90	11.88	11.84	11.75	11.69	11.62	11.52	11.43
440	13.26	12.78	12.21	11.89	11.88	11.84	11.74	11.69	11.62	11.52	11.43
460	13.25	12.77	12.20	11.89	11.88	11.84	11.74	11.69	11.62	11.52	11.43
480	13.25	12.76	12.19	11.89	11.87	11.83	11.74	11.69	11.62	11.52	11.43
500	13.24	12.75	12.18	11.89	11.87	11.83	11.74	11.69	11.62	11.52	11.43

Figure E4-Roof Displacement (cm), X-direction, EQ4

Table E15-Roof Displacement (cm), X-direction, EQ5

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	19.48	18.88	18.32	17.67	17.05	16.42	14.30	15.09	14.46	13.93	13.83
320	19.45	16.67	18.22	17.59	16.94	16.22	15.59	14.91	14.21	13.93	13.82
340	19.50	18.84	18.18	17.53	16.81	16.15	15.46	14.73	14.10	13.92	13.81
360	19.53	18.81	18.18	17.47	16.77	16.08	15.34	14.67	14.02	13.92	13.80
380	19.42	18.86	18.19	17.43	16.77	16.03	15.30	14.62	14.02	13.91	13.80
400	19.52	18.88	18.17	17.45	16.75	15.98	15.29	14.59	14.02	13.91	13.79
420	19.49	18.88	18.15	17.46	16.74	15.96	15.29	14.56	14.02	13.91	13.79
440	19.49	18.86	18.12	17.45	16.72	14.36	15.27	14.54	14.01	13.91	13.79
460	19.48	18.85	18.10	17.43	16.69	15.94	15.25	14.51	14.01	13.91	13.79
480	19.47	18.83	18.08	17.42	16.68	15.93	15.24	14.49	14.01	13.91	13.79
500	19.46	18.82	18.06	17.41	16.67	15.93	15.23	14.48	14.01	13.91	13.79

Figure E5-Roof Displacement (cm), X-direction, EQ5

Table E16-Roof Displacement (cm), X-direction, EQ6

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	10.51	10.28	10.05	9.84	9.63	9.43	9.24	9.06	8.89	8.73	8.57
320	10.51	10.27	10.05	9.84	9.63	9.43	9.24	10.15	10.29	8.73	8.57
340	10.51	10.27	10.05	9.83	9.63	9.43	9.24	9.06	10.29	10.50	8.57
360	10.69	10.27	10.05	9.83	9.63	9.43	10.23	9.06	8.89	8.73	8.57
380	10.50	10.27	10.05	9.83	9.63	9.43	9.24	9.06	8.89	8.72	8.57
400	10.50	10.53	10.04	9.83	9.63	9.43	9.24	9.06	8.89	8.72	8.57
420	10.70	10.27	10.42	9.83	10.31	10.30	10.22	10.27	10.32	10.51	8.57
440	10.50	10.26	10.43	10.36	10.32	10.32	10.33	10.29	10.32	10.51	10.71
460	10.72	10.57	10.04	10.37	10.34	10.34	10.34	10.31	10.32	10.51	10.71
480	10.73	10.57	10.45	10.39	10.35	10.35	10.36	10.32	10.32	10.51	10.71
500	10.73	10.58	10.46	10.40	10.36	10.36	10.37	10.33	10.33	10.51	10.71

Figure E6-Roof Displacement (cm), X-direction, EQ6

Table E17-Roof Displacement (cm), X-direction, EQ7

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	17.10	9.30	9.29	9.28	9.27	17.63	17.34	18.03	16.82	9.46	18.41
320	17.00	9.30	9.30	9.28	8.71	9.28	17.80	16.74	9.28	18.22	9.25
340	5.55	9.30	16.75	17.29	16.29	17.57	16.62	9.28	17.72	18.22	18.37
360	9.30	9.31	17.31	17.26	17.40	17.56	17.39	17.88	18.04	18.20	18.09
380	16.88	16.99	17.11	17.25	17.87	17.84	17.71	17.87	18.03	18.18	18.32
400	18.36	16.99	17.11	16.24	9.28	17.26	17.70	17.86	18.02	18.17	18.31
420	18.80	17.00	17.12	16.97	17.40	17.55	17.71	17.86	17.14	18.15	18.30
440	15.97	19.20	17.14	9.28	17.16	9.28	17.72	17.87	18.98	18.15	18.74
460	19.50	17.03	17.15	9.28	9.28	17.58	19.49	19.44	19.35	18.16	18.28
480	16.95	17.05	17.17	17.30	17.44	17.59	19.84	19.77	18.02	19.52	19.38
500	16.96	20.08	17.18	17.31	20.09	17.60	14.17	20.06	19.95	19.78	19.72

Figure E7-Roof Displacement (cm), X-direction, EQ7

Table E18-Roof Displacement (cm), X-direction, EQ8

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	15.62	15.51	15.39	15.28	15.16	15.03	14.91	14.78	14.65	14.51	14.38
320	15.51	15.40	15.29	15.18	15.06	14.94	14.81	14.69	14.56	14.42	14.29
340	15.79	15.59	15.40	15.21	15.05	14.86	14.74	14.61	14.48	14.35	14.21
360	16.07	15.87	15.69	15.49	15.29	15.11	14.92	14.72	14.54	14.36	14.17
380	16.33	16.12	15.91	15.70	15.10	15.30	15.10	14.90	14.71	14.52	14.33
400	15.29	16.31	16.09	15.89	15.68	15.47	15.28	15.07	14.87	14.67	14.48
420	16.70	16.48	16.27	16.05	15.84	15.63	15.42	15.22	15.00	14.80	14.60
440	16.55	16.63	16.40	16.18	15.96	15.75	15.54	15.33	15.12	14.91	14.70
460	16.99	16.76	16.52	16.30	16.08	15.86	15.65	15.43	15.22	14.99	14.79
480	17.10	16.86	16.62	16.40	16.17	15.95	15.73	15.51	15.29	15.06	14.86
500	17.17	16.94	16.70	16.47	16.24	16.02	15.79	15.57	15.34	15.12	14.91

Figure E8-Roof Displacement (cm), X-direction, EQ8

Table E19-Roof Displacement (cm), X-direction, EQ9

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	12.64	12.63	12.64	12.99	12.66	13.04	12.65	4.23	4.31	12.58	5.07
320	0.10	12.66	12.65	12.64	12.67	12.67	12.66	12.64	5.03	8.94	10.57
340	4.43	12.09	12.90	9.81	4.89	12.69	12.68	12.66	5.03	5.05	12.55
360	12.67	12.66	12.69	12.70	13.05	12.71	12.92	12.78	12.65	12.59	12.57
380	12.69	12.13	12.98	12.72	13.08	13.03	12.91	12.78	12.67	12.63	12.59
400	12.70	12.76	13.06	13.12	13.16	13.03	12.90	5.01	7.04	7.19	12.61
420	12.72	12.78	13.09	13.15	12.76	13.02	12.90	12.77	12.70	12.66	12.62
440	12.73	12.78	13.10	13.17	12.77	12.77	12.76	12.74	12.71	10.61	12.63
460	12.74	12.73	13.10	13.17	12.78	12.78	12.89	12.77	12.72	12.68	12.64
480	12.74	12.79	13.10	13.17	13.19	12.79	12.89	12.75	12.72	12.69	12.65
500	12.75	12.79	13.10	13.17	13.19	13.02	12.89	12.76	12.73	12.69	12.65

Figure E9-Roof Displacement (cm), X-direction, EQ9

Table E20-Roof Displacement (cm), X-direction, EQ10

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	13.40	13.01	13.04	13.15	13.29	13.36	12.62	12.64	12.65	12.65	12.63
320	12.20	12.30	13.19	12.46	13.49	12.56	12.60	13.46	12.62	12.62	12.59
340	12.20	12.29	12.37	13.76	12.50	12.55	12.58	12.60	12.60	13.59	13.48
360	12.19	13.18	13.33	14.01	12.50	12.54	12.57	12.59	12.59	12.58	13.58
380	12.19	13.99	12.36	12.43	12.49	12.53	12.56	12.58	12.58	12.57	12.54
400	12.19	12.28	12.36	12.43	12.49	12.53	12.56	12.57	12.57	12.56	12.53
420	14.07	12.28	12.36	12.43	14.42	14.44	12.55	12.57	14.12	13.92	12.53
440	12.18	12.27	14.44	14.48	14.48	12.52	13.81	12.57	12.56	12.55	13.83
460	14.09	14.26	12.35	14.51	12.48	12.52	12.55	12.56	12.56	12.55	13.92
480	14.10	12.27	12.35	14.53	14.54	14.55	12.55	14.34	14.22	12.55	13.98
500	13.30	14.29	12.35	14.54	14.55	14.57	12.55	12.56	12.56	14.06	14.00

Figure E10-Roof Displacement (cm), X-direction, EQ10

Table E21-Roof Displacement (cm), Y-direction, EQ1

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	11.81	11.81	18.13	18.06	18.06	17.97	14.74	18.04	18.08	18.07	11.87
320	12.13	18.42	18.33	18.41	18.43	18.41	18.01	11.81	11.83	13.58	18.46
340	12.70	18.71	18.73	18.75	18.76	18.76	11.79	11.81	11.82	11.83	11.84
360	19.00	19.01	19.02	19.03	19.03	19.03	19.02	19.01	19.00	11.85	12.01
380	19.21	19.23	19.24	19.24	19.23	19.22	19.22	19.20	19.19	19.18	19.16
400	19.38	19.39	19.40	19.39	19.38	19.36	19.35	19.34	19.33	19.35	19.36
420	19.50	19.51	19.52	19.51	19.49	19.48	19.47	19.49	19.51	19.52	19.53
440	19.56	19.58	19.58	19.58	19.57	19.59	19.62	19.64	19.65	19.66	19.67
460	19.59	19.61	19.62	19.62	19.65	19.69	19.72	19.74	19.76	19.78	19.78
480	19.60	19.62	19.63	19.65	19.70	19.74	19.78	19.81	19.83	19.84	19.85
500	19.59	19.61	19.62	19.67	19.72	19.77	19.81	19.84	19.87	19.88	19.89

Figure E11-Roof Displacement (cm), Y-direction, EQ1

Table E22-Roof Displacement (cm), Y-direction, EQ2

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	9.57	9.36	9.13	8.90	8.64	8.39	8.13	7.87	7.63	7.38	7.13
320	9.63	9.41	9.15	8.91	8.64	8.39	8.13	7.87	7.62	7.37	7.13
340	9.65	9.42	9.16	8.91	8.64	8.39	8.13	7.87	7.62	7.37	7.12
360	9.65	9.42	9.16	8.91	8.64	8.39	8.13	7.86	7.62	7.37	7.12
380	9.65	9.42	9.16	8.91	8.64	8.39	8.12	7.86	7.62	7.37	7.12
400	9.65	9.42	9.16	8.91	8.64	8.39	8.12	7.86	7.62	7.37	7.12
420	9.65	9.42	9.16	8.91	8.64	8.39	8.12	7.86	7.62	7.37	7.12
440	9.65	9.42	9.16	8.91	8.64	8.39	8.12	7.86	7.62	7.37	7.12
460	9.65	9.42	9.15	8.91	8.64	8.38	8.12	7.86	7.62	7.37	7.12
480	9.64	9.41	9.15	8.91	8.63	8.38	8.12	7.86	7.62	7.37	7.12
500	9.64	9.41	9.15	8.91	8.63	8.38	8.12	7.86	7.62	7.37	7.12

Figure E12-Roof Displacement (cm), Y-direction, EQ2

Table E23-Roof Displacement (cm), Y-direction, EQ3

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	7.33	7.04	6.77	6.52	6.30	6.10	5.92	5.76	5.62	5.51	5.42
320	7.32	7.04	6.77	6.52	6.30	6.10	5.92	5.76	5.62	5.51	5.42
340	7.32	7.04	6.77	6.52	6.30	6.10	5.92	5.76	5.63	5.51	5.42
360	7.32	7.04	6.77	6.52	6.30	6.10	5.92	5.76	5.63	5.51	5.42
380	7.32	7.04	6.77	6.52	6.30	6.10	5.92	5.76	5.63	5.51	5.42
400	7.32	7.04	6.77	6.52	6.30	6.10	5.92	5.76	5.63	5.51	5.42
420	7.32	7.04	6.77	6.52	6.30	6.10	5.92	5.76	5.63	5.51	5.42
440	7.32	7.03	6.76	6.52	6.30	6.10	5.92	5.76	5.63	5.51	5.42
460	7.32	7.03	6.76	6.52	6.30	6.10	5.92	5.76	5.63	5.51	5.42
480	7.31	7.03	6.76	6.52	6.30	6.09	5.92	5.76	5.63	5.51	5.42
500	7.31	7.03	6.76	6.52	6.30	6.09	5.91	5.76	5.63	5.51	5.42

Figure E13-Roof Displacement (cm), Y-direction, EQ3

Table E24-Roof Displacement (cm), Y-direction, EQ4

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	11.17	11.07	11.01	10.97	10.96	10.95	10.91	10.83	10.73	10.58	10.39
320	11.45	11.31	11.26	11.24	11.21	11.17	11.09	10.83	10.84	10.67	10.50
340	11.69	11.56	11.48	11.43	11.39	11.33	11.24	11.10	10.97	10.82	10.62
360	11.88	11.71	11.61	11.55	11.50	11.44	11.33	11.22	11.09	10.92	10.69
380	11.99	11.79	11.68	11.61	11.56	11.49	11.38	11.30	11.16	10.97	10.73
400	12.03	11.82	11.70	11.63	11.58	11.51	11.41	11.33	11.19	11.00	10.74
420	12.05	11.83	11.71	11.64	11.59	11.51	11.42	11.34	11.20	11.00	10.74
440	12.05	11.84	11.71	11.65	11.59	11.51	11.42	11.34	11.20	11.00	10.73
460	12.06	11.84	11.72	11.65	11.59	11.51	11.43	11.34	11.20	11.00	10.73
480	12.06	11.84	11.72	11.65	11.59	11.51	11.43	11.34	11.20	10.99	10.73
500	12.06	11.84	11.72	11.65	11.59	11.51	11.43	11.34	11.20	10.99	10.72

Figure E14-Roof Displacement (cm), Y-direction, EQ4

Table E25-Roof Displacement (cm), Y-direction, EQ5

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	11.00	10.53	10.03	9.49	9.07	8.92	7.58	8.61	8.44	8.27	7.26
320	11.05	7.69	10.01	9.42	9.22	8.06	8.93	8.78	8.61	8.44	7.37
340	11.10	10.59	9.97	9.43	7.73	8.17	9.07	8.91	8.75	8.56	8.39
360	9.34	10.58	9.94	9.53	7.73	9.32	9.17	9.01	8.85	7.37	8.47
380	7.59	10.56	9.91	7.75	9.49	9.38	9.24	9.06	8.91	8.71	8.51
400	11.16	10.54	9.87	9.64	9.53	9.42	9.27	8.33	7.36	8.74	8.53
420	11.16	10.53	9.86	9.67	9.56	9.44	9.29	9.12	8.95	8.75	8.53
440	11.17	10.52	9.84	7.75	9.57	7.67	9.31	9.13	8.95	8.76	8.54
460	11.17	10.52	9.84	9.70	9.59	9.46	9.31	9.14	8.96	8.76	7.57
480	11.17	10.52	9.84	9.71	9.59	9.47	9.32	9.14	8.96	8.76	8.54
500	11.17	10.52	9.83	9.71	9.60	9.47	9.32	9.14	8.96	8.77	8.54

Figure E15-Roof Displacement (cm), Y-direction, EQ5

Table E26-Roof Displacement (cm), Y-direction, EQ6

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	10.87	10.95	11.05	11.16	11.27	11.37	11.47	11.57	11.67	11.76	11.85
320	11.22	12.77	11.43	12.14	11.65	11.76	11.87	11.96	12.05	12.15	12.23
340	11.48	11.61	11.74	11.86	11.98	12.09	12.19	12.28	12.48	12.54	12.54
360	13.43	12.12	11.97	12.09	12.21	12.33	12.91	12.54	12.64	12.73	12.81
380	12.59	12.14	12.15	12.27	12.40	12.52	12.64	12.74	12.84	12.93	13.02
400	11.94	13.47	12.27	12.43	12.56	12.70	12.81	12.92	13.02	13.11	13.19
420	13.74	12.18	14.09	12.50	14.28	14.29	14.24	14.24	14.21	14.22	13.31
440	13.94	12.22	14.52	14.66	14.71	14.67	14.64	14.62	14.60	14.62	14.59
460	14.63	14.82	14.90	15.03	15.05	14.96	14.97	14.92	14.96	14.95	14.96
480	14.97	15.12	15.23	15.31	15.29	15.22	15.21	15.26	15.27	15.23	15.27
500	15.18	15.34	15.44	15.51	15.47	15.41	15.49	15.54	15.52	15.51	15.52

Figure E16-Roof Displacement (cm), Y-direction, EQ6

Table E27-Roof Displacement (cm), Y-direction, EQ7

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	21.49	10.56	21.92	10.32	22.15	22.21	22.22	24.79	22.32	22.47	25.21
320	24.19	10.56	22.60	22.66	10.21	15.23	24.79	22.76	18.58	25.03	9.71
340	10.69	23.02	23.13	24.68	23.24	24.81	23.15	23.13	23.11	23.02	25.23
360	23.41	23.51	24.79	24.80	24.86	24.91	23.48	25.15	25.17	25.18	23.04
380	25.01	25.08	25.04	25.07	25.15	25.19	25.23	25.24	25.26	25.25	25.24
400	25.37	25.37	25.38	24.15	10.35	23.98	23.85	25.38	25.35	25.37	23.15
420	25.86	24.33	25.72	24.30	25.62	25.60	25.58	25.60	23.55	25.53	25.40
440	24.42	26.24	26.11	24.34	24.21	24.02	25.91	25.81	25.69	25.59	25.50
460	26.88	26.70	26.60	24.32	24.15	26.32	26.16	25.99	25.88	25.81	25.72
480	27.46	27.27	27.12	26.98	26.85	26.69	26.50	26.38	26.28	26.16	26.04
500	28.09	27.88	27.65	27.44	27.25	27.12	23.79	26.81	26.65	26.47	26.28

Figure E17-Roof Displacement (cm), Y-direction, EQ7

Table E28-Roof Displacement (cm), Y-direction, EQ8

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	26.99	26.63	26.27	25.91	25.56	25.23	24.89	24.57	24.25	23.93	23.61
320	26.42	26.04	25.67	25.31	24.96	24.62	24.29	23.97	23.64	23.33	23.01
340	25.85	25.47	25.11	24.75	24.40	24.06	23.73	23.40	23.09	22.77	22.46
360	25.29	24.92	24.56	24.21	23.87	23.53	23.21	22.89	22.57	22.27	21.96
380	24.79	24.42	24.07	23.72	23.38	23.05	22.73	22.42	22.11	21.81	21.51
400	24.34	23.98	23.62	23.28	22.95	22.63	22.32	22.01	21.71	21.42	21.13
420	23.93	23.57	23.23	22.89	22.57	22.26	21.95	21.65	21.37	21.08	20.80
440	23.56	23.21	22.87	22.55	22.24	21.93	21.64	21.35	21.07	20.79	20.52
460	23.24	22.90	22.57	22.25	21.95	21.65	21.36	21.09	20.82	20.55	20.29
480	22.98	22.64	22.32	22.01	21.71	21.42	21.15	20.88	20.61	20.36	20.10
500	22.78	22.44	22.13	21.82	21.53	21.25	20.98	20.71	20.46	20.21	19.96

Figure E18-Roof Displacement (cm), Y-direction, EQ8

Table E29-Roof Displacement (cm), Y-direction, EQ9

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	20.34	20.32	20.25	20.24	20.25	20.18	20.12	10.79	10.63	19.88	19.79
320	0.13	20.79	20.75	20.68	20.57	20.49	20.41	20.33	18.66	20.17	20.09
340	12.15	21.24	21.12	21.02	17.03	20.83	20.73	20.62	20.51	20.40	20.29
360	21.72	21.58	21.47	21.35	21.22	21.09	20.96	20.83	20.70	20.57	20.43
380	22.03	21.88	21.73	21.57	21.42	21.26	21.11	20.96	20.81	20.66	20.51
400	22.27	22.09	21.91	21.74	21.56	21.39	21.22	20.92	20.89	20.73	20.58
420	22.46	22.26	22.06	21.86	21.67	21.48	21.30	21.12	20.94	20.79	20.66
440	22.60	22.38	22.16	21.95	21.74	21.53	21.33	21.14	20.98	20.85	20.71
460	22.68	22.44	22.21	21.98	21.76	21.55	21.34	21.16	21.01	20.87	20.73
480	22.73	22.48	22.23	21.99	21.76	21.54	21.32	21.17	21.02	20.87	20.73
500	22.74	22.48	22.23	21.99	21.75	21.53	21.32	21.17	21.02	20.87	20.73

Figure E19-Roof Displacement (cm), Y-direction, EQ9

Table E30-Roof Displacement (cm), Y-direction, EQ10

Fy(Mpa)	f'c (Mpa)										
	21.00	22.40	23.80	25.20	26.60	28.00	29.40	30.80	32.20	33.60	35.00
300	16.13	15.86	15.64	15.48	15.36	15.26	15.19	15.14	15.12	15.10	15.11
320	15.95	15.71	15.51	15.35	15.23	15.13	15.06	15.01	14.98	14.97	14.97
340	15.84	15.60	15.41	15.25	15.13	15.03	14.96	14.91	14.87	14.86	14.85
360	15.76	15.52	15.33	15.18	15.05	14.95	14.88	14.83	14.80	14.79	14.80
380	15.70	15.47	15.28	15.12	15.00	14.92	14.85	14.81	14.78	14.77	14.77
400	15.67	15.45	15.26	15.11	15.00	14.91	14.84	14.80	14.77	14.76	14.76
420	15.67	15.44	15.26	15.11	14.99	14.90	14.84	14.79	14.77	14.75	14.75
440	15.67	15.44	15.26	15.11	14.99	14.90	14.84	14.79	14.76	14.75	14.75
460	15.67	15.44	15.26	15.11	14.99	14.90	14.84	14.79	14.76	14.75	14.75
480	15.67	15.45	15.26	15.11	15.00	14.91	14.84	14.80	14.77	14.75	14.75
500	15.67	15.45	15.26	15.12	15.00	14.91	14.84	14.80	14.77	14.76	14.75

Figure E20-Roof Displacement (cm), Y-direction, EQ10

Roof Displacement

Table E31-Roof displacement(cm) table for varying f'_c and F_y . X-direction.

Fy(MPa)	f'c (MPa)										
	21	22.4	23.8	25.2	26.6	28	29.4	30.8	32.2	33.6	35
300	12.67	12.50	12.33	12.09	11.95	11.82	11.66	11.50	11.33	11.18	11.12
320	12.65	12.50	12.28	12.05	11.94	11.78	11.63	11.45	11.28	11.19	11.13
340	12.66	12.47	12.25	12.04	11.90	11.77	11.60	11.42	11.25	11.19	11.13
360	12.66	12.46	12.24	12.02	11.89	11.75	11.57	11.41	11.23	11.19	11.13
380	12.63	12.46	12.23	12.01	11.89	11.74	11.56	11.39	11.23	11.19	11.13
400	12.65	12.47	12.22	12.02	11.89	11.72	11.56	11.39	11.23	11.19	11.13
420	12.64	12.46	12.21	12.02	11.88	11.72	11.55	11.38	11.23	11.19	11.13
440	12.64	12.46	12.20	12.02	11.88	11.72	11.55	11.37	11.23	11.19	11.13
460	12.64	12.45	12.20	12.01	11.87	11.71	11.54	11.37	11.23	11.18	11.13
480	12.63	12.45	12.19	12.01	11.87	11.71	11.54	11.36	11.23	11.18	11.13
500	12.63	12.44	12.18	12.01	11.87	11.71	11.54	11.36	11.23	11.18	11.13

Figure E21-RHA roof displacement to varying f'_c and F_y . X-direction.

Table E32-Roof displacement (cm) table for varying f'c and Fy. Y-direction.

Fy(MPa)	f'c (MPa)										
	21	22.4	23.8	25.2	26.6	28	29.4	30.8	32.2	33.6	35
300	14.24	13.99	13.76	13.56	13.36	13.18	13.01	12.83	12.67	12.50	12.33
320	14.16	13.90	13.67	13.47	13.27	13.08	12.90	12.69	12.54	12.37	12.20
340	14.07	13.82	13.59	13.37	13.17	12.98	12.79	12.61	12.44	12.27	12.10
360	13.98	13.72	13.49	13.28	13.07	12.88	12.69	12.51	12.34	12.17	12.00
380	13.89	13.63	13.39	13.18	12.98	12.79	12.60	12.43	12.26	12.09	11.91
400	13.80	13.54	13.30	13.09	12.89	12.71	12.52	12.35	12.18	12.01	11.84
420	13.72	13.46	13.22	13.02	12.82	12.63	12.45	12.28	12.12	11.94	11.77
440	13.65	13.39	13.15	12.95	12.75	12.57	12.39	12.22	12.05	11.89	11.71
460	13.59	13.33	13.09	12.89	12.69	12.51	12.33	12.17	12.00	11.84	11.66
480	13.53	13.28	13.04	12.84	12.65	12.46	12.29	12.13	11.96	11.80	11.63
500	13.49	13.23	13.00	12.80	12.61	12.43	12.26	12.09	11.93	11.77	11.60

Figure E22-RHA roof displacement to varying f'c and Fy. Y-direction.

Roof Displacement Percentage Difference

Table E33-Roof displacement percent difference (%) from $f'_c=28\text{MPa}$ and $F_y=400\text{MPa}$. X-direction

$F_y(\text{MPa})$	f'_c (MPa)	21	22.4	23.8	25.2	26.6	28	29.4	30.8	32.2	33.6	35
300		-8.08	-6.64	-5.14	-3.15	-1.97	-0.86	0.07	1.94	3.39	4.64	5.17
320		-7.91	-6.48	-4.76	-2.83	-1.81	-0.51	0.79	2.33	3.82	4.56	5.09
340		-7.98	-6.40	-4.52	-2.69	-1.53	-0.38	1.04	2.59	4.01	4.54	5.05
360		-8.00	-6.26	-4.41	-2.55	-1.45	-0.22	1.32	2.71	4.18	4.55	5.06
380		-7.74	-6.32	-4.34	-2.45	-1.44	-0.10	1.41	2.80	4.19	4.57	5.07
400		-7.93	-6.33	-4.26	-2.50	-1.41	0.00	1.43	2.88	4.20	4.58	5.08
420		-7.85	-6.31	-4.18	-2.51	-1.37	0.05	1.45	2.95	4.20	4.59	5.08
440		-7.83	-6.25	-4.10	-2.49	-1.33	0.08	1.48	3.00	4.21	4.59	5.09
460		-7.80	-6.20	-4.02	-2.45	-1.28	0.09	1.53	3.05	4.21	4.60	5.09
480		-7.77	-6.15	-3.96	-2.43	-1.24	0.12	1.56	3.09	4.22	4.60	5.09
500		-7.74	-6.12	-3.92	-2.42	-1.22	0.13	1.58	3.11	4.22	4.60	5.10

Figure 21-Roof displacement % Difference from $f'_c=28\text{MPa}$ & $F_y=400\text{MPa}$. X-direction.

Table E34-Roof displacement percent difference (%) from $f'_c=28\text{MPa}$ and $F_y=400\text{MPa}$. Y-direction

Fy(MPa)	f'c (MPa)										
	21	22.4	23.8	25.2	26.6	28	29.4	30.8	32.2	33.6	35
300	-12.05	-10.14	-8.32	-6.69	-5.18	-3.77	-2.38	-1.02	0.30	1.62	2.95
320	-11.41	-9.41	-7.62	-6.00	-4.44	-2.97	-1.52	0.14	1.29	2.65	3.94
340	-10.74	-8.76	-6.92	-5.27	-3.67	-2.18	-0.70	0.77	2.13	3.46	4.80
360	-10.04	-8.00	-6.14	-4.49	-2.90	-1.38	0.12	1.53	2.86	4.20	5.56
380	-9.33	-7.26	-5.38	-3.72	-2.14	-0.65	0.84	2.18	3.50	4.87	6.24
400	-8.64	-6.56	-4.69	-3.05	-1.48	0.00	1.44	2.78	4.11	5.46	6.85
420	-8.00	-5.93	-4.08	-2.44	-0.88	0.58	2.01	3.33	4.65	6.00	7.38
440	-7.43	-5.37	-3.52	-1.90	-0.36	1.10	2.50	3.81	5.12	6.46	7.83
460	-6.93	-4.88	-3.05	-1.44	0.09	1.54	2.93	4.22	5.52	6.84	8.20
480	-6.52	-4.48	-2.66	-1.06	0.47	1.90	3.27	4.55	5.83	7.15	8.50
500	-6.19	-4.16	-2.35	-0.76	0.75	2.17	3.53	4.81	6.08	7.38	8.72

Figure 22-Roof Displacement % Difference from $f'_c=28\text{MPa}$ & $F_y=400\text{MPa}$. Y-direction.

Appendix F: Reinforced Concrete Calculations

Reinforced Concrete Calculations

Design Beam/Slab for Moment - 1st Floor

Slab type A	
Top Mu- -594.2109 kN-m	Bottom Mu+ 643.1101 kN-m
Mu_{beam} -378.8094488 kN-m	Mu_{beam} 409.9826888 kN-m
Mu_{col} -31.09243081 kN-m	Mu_{col} 33.65110988 kN-m
Mu_{mid} -59.42109 kN-m	Mu_{mid} 64.31101 kN-m
Asmin1 716.1166882 mm2	Asmin1 716.1166882 mm2
Asmin2 757.8666667 mm2	Asmin2 757.8666667 mm2
As Mu_{beam} 1648.121365 mm2	As Mu_{beam} 1794.652461 mm2
a 0.083098556 m	a 0.090486679 m
Goal seek -4.99E-06	Goal seek -4.5191E-05
Is As Mu-beam > As min?	Is As Mu-beam > As min?
YES	YES
Use Asbeam 1648.121365 mm2	Use Asbeam 1794.652461 mm2
3 29s	3 29s

Slab type B	
Top Mu- -460.9781 kN-m	Bottom Mu+ 481.2421 kN-m
Mu_{beam} -293.8735388 kN-m	Mu_{beam} 306.7918388 kN-m
Mu_{col} -24.12094709 kN-m	Mu_{col} 25.18127267 kN-m
Mu_{mid} -46.09781 kN-m	Mu_{mid} 48.12421 kN-m
Asmin1 716.1166882 mm2	Asmin1 716.1166882 mm2
Asmin2 757.8666667 mm2	Asmin2 757.8666667 mm2
As Mu_{beam} 1258.244531 mm2	As Mu_{beam} 1316.695255 mm2
a 0.063440901 m	a 0.066387996 m
Goal seek 0.000128516	Goal seek 8.94718E-05
Is As Mu-beam > As min?	Is As Mu-beam > As min?
YES	YES
Use Asbeam 1258.244531 mm2	Use Asbeam 1316.695255 mm2
3 25s	3 25s

Slab type C	
Top Mu- -940.1021 kN-m	Bottom Mu+ 994.017 kN-m
Mu_{beam} -599.3150888 kN-m	Mu_{beam} 633.6858375 kN-m
Mu_{col} -49.19138895 kN-m	Mu_{col} 52.01251744 kN-m
Mu_{mid} -94.01021 kN-m	Mu_{mid} 99.4017 kN-m
Asmin1 716.1166882 mm2	Asmin1 716.1166882 mm2
Asmin2 757.8666667 mm2	Asmin2 757.8666667 mm2
As Mu_{beam} 2729.941175 mm2	As Mu_{beam} 2909.131179 mm2
a 0.137644093 m	a 0.146678883 m
Goal seek -1.35514E-06	Goal seek -6.2786E-07
Is As Mu-beam > As min?	Is As Mu-beam > As min?
YES	YES
Use Asbeam 2729.941175 mm2	Use Asbeam 2909.131179 mm2
6 25s	6 25s

Slab type D	
Top Mu- -1624.7484 kN-m	Bottom Mu+ 1644.2737 kN-m
Mu_{beam} -1035.777105 kN-m	Mu_{beam} 1048.224484 kN-m
Mu_{col} -85.01590465 kN-m	Mu_{col} 86.03757733 kN-m
Mu_{mid} -162.47484 kN-m	Mu_{mid} 164.42737 kN-m
Asmin1 716.1166882 mm2	Asmin1 716.1166882 mm2
Asmin2 757.8666667 mm2	Asmin2 757.8666667 mm2
As Mu_{beam} 5314.193971 mm2	As Mu_{beam} 5400.971813 mm2
a 0.267942553 m	a 0.272317907 m
Goal seek -6.82745E-06	Goal seek -8.95209E-06
Is As Mu-beam > As min?	Is As Mu-beam > As min?
YES	YES
Use Asbeam 5314.193971 mm2	Use Asbeam 5400.971813 mm2
6 36s	6 36s

Slab type E	
Top Mu- -10.3323 kN-m	Bottom Mu+ 37.1834 kN-m
Mu_{beam} -6.58684125 kN-m	Mu_{beam} 23.7044175 kN-m
Mu_{col} -0.540643605 kN-m	Mu_{col} 1.945643023 kN-m
Mu_{mid} -1.03323 kN-m	Mu_{mid} 3.71834 kN-m
Asmin1 716.1166882 mm2	Asmin1 716.1166882 mm2
Asmin2 757.8666667 mm2	Asmin2 757.8666667 mm2
As Mu_{beam} 26.85285195 mm2	As Mu_{beam} 96.90078988 mm2
a 0.001353925 m	a 0.004885754 m
Goal seek -2.71034E-05	Goal seek -2.3744E-05
Is As Mu-beam > As min?	Is As Mu-beam > As min?
NO	NO
Use Asbeam 757.8666667 mm2	Use Asbeam 757.8666667 mm2
2 22s	2 22s

Slab type F	
Top Mu- -14.7379 kN-m	Bottom Mu+ 38.5753 kN-m
Mu_{beam} -9.39541125 kN-m	Mu_{beam} 24.59175375 kN-m
Mu_{col} -0.771169186 kN-m	Mu_{col} 2.018475 kN-m
Mu_{mid} -1.47379 kN-m	Mu_{mid} 3.85753 kN-m
Asmin1 716.1166882 mm2	Asmin1 716.1166882 mm2
Asmin2 757.8666667 mm2	Asmin2 757.8666667 mm2
As Mu_{beam} 38.31978978 mm2	As Mu_{beam} 100.5423809 mm2
a 0.00193209 m	a 0.005069364 m
Goal seek -2.59022E-05	Goal seek -2.33964E-05
Is As Mu-beam > As min?	Is As Mu-beam > As min?
NO	NO
Use Asbeam 757.8666667 mm2	Use Asbeam 757.8666667 mm2
2 22s	2 22s

Slab type G	
Top Mu- -6.1698 kN-m	Bottom Mu+ 19.5328 kN-m
Mu_{beam} -3.9332475 kN-m	Mu_{beam} 12.45216 kN-m
Mu_{col} -0.322838372 kN-m	Mu_{col} 1.022065116 kN-m
Mu_{mid} -0.61698 kN-m	Mu_{mid} 1.95328 kN-m
Asmin1 716.1166882 mm2	Asmin1 716.1166882 mm2
Asmin2 757.8666667 mm2	Asmin2 757.8666667 mm2
As Mu_{beam} 16.02804538 mm2	As Mu_{beam} 50.81162976 mm2
a 0.000808137 m	a 0.002561931 m
Goal seek -2.82801E-05	Goal seek -2.853E-05
Is As Mu-beam > As min?	Is As Mu-beam > As min?
NO	NO
Use Asbeam 757.8666667 mm2	Use Asbeam 757.8666667 mm2
2 22s	2 22s

Slab type H	
Top Mu- -7.6071 kN-m	Bottom Mu+ 15.8334 kN-m
Mu_{beam} -4.84952625 kN-m	Mu_{beam} 10.0937925 kN-m
Mu_{col} -0.39804593 kN-m	Mu_{col} 0.82849186 kN-m
Mu_{mid} -0.76071 kN-m	Mu_{mid} 1.58334 kN-m
Asmin1 716.1166882 mm2	Asmin1 716.1166882 mm2
Asmin2 757.8666667 mm2	Asmin2 757.8666667 mm2
As Mu_{beam} 19.76479224 mm2	As Mu_{beam} 41.17274073 mm2
a 0.000996544 m	a 0.002075937 m
Goal seek -2.78691E-05	Goal seek -2.96272E-05
Is As Mu-beam > As min?	Is As Mu-beam > As min?
NO	NO
Use Asbeam 757.8666667 mm2	Use Asbeam 757.8666667 mm2
2 22s	2 22s

Design Beam/Slab for Moment - 4th Floor

Slab type A			
Top		Bottom	
Mu- -314.3822 kN-m		Mu+ 343.0222 kN-m	
Mu ⁻ _{beam} -200.4186525 kN-m		Mu ⁺ _{beam} 218.6766525 kN-m	
Mu ⁻ _{coln} -16.4502314 kN-m		Mu ⁺ _{coln} 17.94883605 kN-m	
Mu ⁻ _{mid} -31.43822 kN-m		Mu ⁺ _{mid} 34.30222 kN-m	
Asmin1 716.1166882 mm2		Asmin1 716.1166882 mm2	
Asmin2 757.8666667 mm2		Asmin2 757.8666667 mm2	
As Mu ⁻ beam 843.841405 mm2		As Mu ⁺ beam 923.6733555 mm2	
a 0.042546625 m		a 0.046571766 m	
Goal seek -3.05176E-07		Goal seek -2.0446E-07	
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
YES		YES	
Use Asbeam 843.841405 mm2		Use Asbeam 923.6733555 mm2	
2 25s		2 25s	

Slab type B			
Top		Bottom	
Mu- -432.1956 kN-m		Mu+ 458.9207 kN-m	
Mu ⁻ _{beam} -275.524695 kN-m		Mu ⁺ _{beam} 292.5619463 kN-m	
Mu ⁻ _{coln} -22.61488605 kN-m		Mu ⁺ _{coln} 24.01329244 kN-m	
Mu ⁻ _{mid} -43.21956 kN-m		Mu ⁺ _{mid} 45.89207 kN-m	
Asmin1 716.1166882 mm2		Asmin1 716.1166882 mm2	
Asmin2 757.8666667 mm2		Asmin2 757.8666667 mm2	
As Mu ⁻ beam 5314.193971 mm2		As Mu ⁺ beam 1252.326712 mm2	
a 0.267942553 m		a 0.063142523 m	
Goal seek 760.2524032		Goal seek 0.000187063	
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
YES		YES	
Use Asbeam 5314.193971 mm2		Use Asbeam 1252.326712 mm2	
7 32s		3 25s	

Slab type C			
Top		Bottom	
Mu- -516.4053 kN-m		Mu+ 566.9448 kN-m	
Mu ⁻ _{beam} -329.2083788 kN-m		Mu ⁺ _{beam} 361.42731 kN-m	
Mu ⁻ _{coln} -27.02120756 kN-m		Mu ⁺ _{coln} 29.66571628 kN-m	
Mu ⁻ _{mid} -51.64053 kN-m		Mu ⁺ _{mid} 56.69448 kN-m	
Asmin1 716.1166882 mm2		Asmin1 716.1166882 mm2	
Asmin2 757.8666667 mm2		Asmin2 757.8666667 mm2	
As Mu ⁻ beam 1418.829261 mm2		As Mu ⁺ beam 1567.239738 mm2	
a 0.07153761 m		a 0.079020491 m	
Goal seek 9.12377E-06		Goal seek 9.22196E-07	
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
YES		YES	
Use Asbeam 1418.829261 mm2		Use Asbeam 1567.239738 mm2	
3 25s		3 29s	

Slab type D			
Top		Bottom	
Mu- -822.3593 kN-m		Mu+ 841.9456 kN-m	
Mu ⁻ _{beam} -524.2540538 kN-m		Mu ⁺ _{beam} 536.74032 kN-m	
Mu ⁻ _{coln} -43.03042849 kN-m		Mu ⁺ _{coln} 44.05529302 kN-m	
Mu ⁻ _{mid} -82.23593 kN-m		Mu ⁺ _{mid} 84.19456 kN-m	
Asmin1 716.1166882 mm2		Asmin1 716.1166882 mm2	
Asmin2 757.8666667 mm2		Asmin2 757.8666667 mm2	
As Mu ⁻ beam 2349.205744 mm2		As Mu ⁺ beam 2411.580527 mm2	
a 0.118447348 m		a 0.121592295 m	
Goal seek -1.15476E-05		Goal seek -1.21498E-05	
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
YES		YES	
Use Asbeam 2349.205744 mm2		Use Asbeam 2411.580527 mm2	
4 29s		4 29s	

Slab type E			
Top		Bottom	
Mu- -42.6996 kN-m		Mu+ 82.6965 kN-m	
Mu ⁻ _{beam} -27.220995 kN-m		Mu ⁺ _{beam} 52.71901875 kN-m	
Mu ⁻ _{coln} -2.234281395 kN-m		Mu ⁺ _{coln} 4.327142442 kN-m	
Mu ⁻ _{mid} -4.26996 kN-m		Mu ⁺ _{mid} 8.26965 kN-m	
Asmin1 716.1166882 mm2		Asmin1 716.1166882 mm2	
Asmin2 757.8666667 mm2		Asmin2 757.8666667 mm2	
As Mu ⁻ beam 111.3387865 mm2		As Mu ⁺ beam 216.5180354 mm2	
a 0.00561372 m		a 0.010916876 m	
Goal seek -1.92586E-05		Goal seek -1.4364E-05	
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbeam 757.8666667 mm2		Use Asbeam 757.8666667 mm2	
2 22s		2 22s	

Slab type F			
Top		Bottom	
Mu- -53.7735 kN-m		Mu+ 83.0587 kN-m	
Mu ⁻ _{beam} -34.28060625 kN-m		Mu ⁺ _{beam} 52.94992125 kN-m	
Mu ⁻ _{coln} -2.813729651 kN-m		Mu ⁺ _{coln} 4.346094767 kN-m	
Mu ⁻ _{mid} -5.37735 kN-m		Mu ⁺ _{mid} 8.30587 kN-m	
Asmin1 716.1166882 mm2		Asmin1 716.1166882 mm2	
Asmin2 757.8666667 mm2		Asmin2 757.8666667 mm2	
As Mu ⁻ beam 140.3727552 mm2		As Mu ⁺ beam 217.4744978 mm2	
a 0.007077618 m		a 0.010965101 m	
Goal seek -1.70511E-05		Goal seek -1.43038E-05	
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbeam 757.8666667 mm2		Use Asbeam 757.8666667 mm2	
2 22s		2 22s	

Slab type G			
Top		Bottom	
Mu- -6.7372 kN-m		Mu+ 29.9345 kN-m	
Mu ⁻ _{beam} -4.294965 kN-m		Mu ⁺ _{beam} 19.08324375 kN-m	
Mu ⁻ _{coln} -0.352527907 kN-m		Mu ⁺ _{coln} 1.566340116 kN-m	
Mu ⁻ _{mid} -0.67372 kN-m		Mu ⁺ _{mid} 2.99345 kN-m	
Asmin1 716.1166882 mm2		Asmin1 716.1166882 mm2	
Asmin2 757.8666667 mm2		Asmin2 757.8666667 mm2	
As Mu ⁻ beam 17.5030637 mm2		As Mu ⁺ beam 77.9524248 mm2	
a 0.000882507 m		a 0.003930374 m	
Goal seek -2.81172E-05		Goal seek -2.5622E-05	
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbeam 757.8666667 mm2		Use Asbeam 757.8666667 mm2	
2 22s		2 22s	

Slab type H			
Top		Bottom	
Mu- -8.7701 kN-m		Mu+ 17.2859 kN-m	
Mu ⁻ _{beam} -5.59093875 kN-m		Mu ⁺ _{beam} 11.01976125 kN-m	
Mu ⁻ _{coln} -0.458900581 kN-m		Mu ⁺ _{coln} 0.904494767 kN-m	
Mu ⁻ _{mid} -0.87701 kN-m		Mu ⁺ _{mid} 1.72859 kN-m	
Asmin1 716.1166882 mm2		Asmin1 716.1166882 mm2	
Asmin2 757.8666667 mm2		Asmin2 757.8666667 mm2	
As Mu ⁻ beam 22.78919826 mm2		As Mu ⁺ beam 44.95640682 mm2	
a 0.001149035 m		a 0.00226671 m	
Goal seek -2.75402E-05		Goal seek -2.91922E-05	
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbeam 757.8666667 mm2		Use Asbeam 757.8666667 mm2	
2 22s		2 22s	

Design Beam/Slab for Moment - 7th Floor

Slab type A			
Top		Bottom	
Mu-	-235.413 kN-m	Mu+	251.6391 kN-m
Mu _{beam}	-150.076 kN-m	Mu _{beam}	160.4199263 kN-m
Mu _{colm}	-12.3181 kN-m	Mu _{colm}	13.16716221 kN-m
Mu _{mid}	-23.5413 kN-m	Mu _{mid}	25.16391 kN-m
Asmin1	716.1167 mm2	Asmin1	716.1166882 mm2
Asmin2	757.8667 mm2	Asmin2	757.8666667 mm2
As Mu _{beam}	626.4129 mm2	As Mu _{beam}	670.7732109 mm2
a	0.031584 m	a	0.033820498 m
Goal seek	-1.4E-06	Goal seek	-1.3273E-06
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbea	757.8667 mm2	Use Asbea	757.8666667 mm2
2 22s		2 22s	

Slab type B			
Top		Bottom	
Mu-	-364.308 kN-m	Mu+	395.2171 kN-m
Mu _{beam}	-232.246 kN-m	Mu _{beam}	251.9509 kN-m
Mu _{colm}	-19.0626 kN-m	Mu _{colm}	20.67996 kN-m
Mu _{mid}	-36.4308 kN-m	Mu _{mid}	39.52171 kN-m
Asmin1	716.1167 mm2	Asmin1	716.1167 mm2
Asmin2	757.8667 mm2	Asmin2	757.8667 mm2
As Mu _{beam}	983.3526 mm2	As Mu _{beam}	1070.554 mm2
a	0.049581 m	a	0.053978 m
Goal seek	-8.8E-08	Goal seek	0.00084
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
YES		YES	
Use Asbea	983.3526 mm2	Use Asbea	1070.554 mm2
2 22s		3 22s	

Slab type C			
Top		Bottom	
Mu-	-401.887 kN-m	Mu+	460.9187 kN-m
Mu _{beam}	-256.203 kN-m	Mu _{beam}	293.8356713 kN-m
Mu _{colm}	-21.029 kN-m	Mu _{colm}	24.11783895 kN-m
Mu _{mid}	-40.1887 kN-m	Mu _{mid}	46.09187 kN-m
Asmin1	716.1167 mm2	Asmin1	716.1166882 mm2
Asmin2	757.8667 mm2	Asmin2	757.8666667 mm2
As Mu _{beam}	1089.453 mm2	As Mu _{beam}	1258.073841 mm2
a	0.05493 m	a	0.063432294 m
Goal seek	0.000594	Goal seek	0.000176212
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
YES		YES	
Use Asbea	1089.453 mm2	Use Asbea	1258.073841 mm2
3 22s		4 22s	

Slab type D			
Top		Bottom	
Mu-	-579.992 kN-m	Mu+	600.4731 kN-m
Mu _{beam}	-369.745 kN-m	Mu _{beam}	382.8016 kN-m
Mu _{colm}	-30.3484 kN-m	Mu _{colm}	31.4201 kN-m
Mu _{mid}	-57.9992 kN-m	Mu _{mid}	60.04731 kN-m
Asmin1	716.1167 mm2	Asmin1	716.1167 mm2
Asmin2	757.8667 mm2	Asmin2	757.8667 mm2
As Mu _{beam}	1605.869 mm2	As Mu _{beam}	1666.78 mm2
a	0.080968 m	a	0.084039 m
Goal seek	-1.1E-06	Goal seek	-4.3E-06
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
YES		YES	
Use Asbea	1605.869 mm2	Use Asbea	1666.78 mm2
5 22s		5 22s	

Slab type E			
Top		Bottom	
Mu-	-58.8184 kN-m	Mu+	102.5354 kN-m
Mu _{beam}	-37.4967 kN-m	Mu _{beam}	65.3663175 kN-m
Mu _{colm}	-3.07771 kN-m	Mu _{colm}	5.365224419 kN-m
Mu _{mid}	-5.88184 kN-m	Mu _{mid}	10.25354 kN-m
Asmin1	716.1167 mm2	Asmin1	716.1166882 mm2
Asmin2	757.8667 mm2	Asmin2	757.8666667 mm2
As Mu _{beam}	153.6216 mm2	As Mu _{beam}	269.0134626 mm2
a	0.007746 m	a	0.013563704 m
Goal seek	-1.6E-05	Goal seek	-1.1371E-05
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbea	757.8667 mm2	Use Asbea	757.8666667 mm2
2 22s		2 22s	

Slab type F			
Top		Bottom	
Mu-	-70.857 kN-m	Mu+	101.6301 kN-m
Mu _{beam}	-45.1713 kN-m	Mu _{beam}	64.78919 kN-m
Mu _{colm}	-3.70763 kN-m	Mu _{colm}	5.317854 kN-m
Mu _{mid}	-7.0857 kN-m	Mu _{mid}	10.16301 kN-m
Asmin1	716.1167 mm2	Asmin1	716.1167 mm2
Asmin2	757.8667 mm2	Asmin2	757.8667 mm2
As Mu _{beam}	185.2931 mm2	As Mu _{beam}	266.6132 mm2
a	0.009343 m	a	0.013443 m
Goal seek	-1.4E-05	Goal seek	-1.1E-05
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbea	757.8667 mm2	Use Asbea	757.8667 mm2
2 22s		2 22s	

Slab type G			
Top		Bottom	
Mu-	-5.9706 kN-m	Mu+	34.6103 kN-m
Mu _{beam}	-3.80626 kN-m	Mu _{beam}	22.06406625 kN-m
Mu _{colm}	-0.31242 kN-m	Mu _{colm}	1.81100407 kN-m
Mu _{mid}	-0.59706 kN-m	Mu _{mid}	3.46103 kN-m
Asmin1	716.1167 mm2	Asmin1	716.1166882 mm2
Asmin2	757.8667 mm2	Asmin2	757.8666667 mm2
As Mu _{beam}	15.51024 mm2	As Mu _{beam}	90.17158581 mm2
a	0.000782 m	a	0.004546467 m
Goal seek	-2.8E-05	Goal seek	-2.4397E-05
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbea	757.8667 mm2	Use Asbea	757.8666667 mm2
2 22s		2 22s	

Slab type H			
Top		Bottom	
Mu-	-8.8559 kN-m	Mu+	18.3263 kN-m
Mu _{beam}	-5.64564 kN-m	Mu _{beam}	11.68302 kN-m
Mu _{colm}	-0.46339 kN-m	Mu _{colm}	0.958934 kN-m
Mu _{mid}	-0.88559 kN-m	Mu _{mid}	1.83263 kN-m
Asmin1	716.1167 mm2	Asmin1	716.1167 mm2
Asmin2	757.8667 mm2	Asmin2	757.8667 mm2
As Mu _{beam}	23.01235 mm2	As Mu _{beam}	47.66727 mm2
a	0.00116 m	a	0.002403 m
Goal seek	-2.8E-05	Goal seek	-2.9E-05
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbea	757.8667 mm2	Use Asbea	757.8667 mm2
2 22s		2 22s	

Design Beam/Slab for Moment - 10th Floor

Slab type A			
Top		Bottom	
Mu-	-120.071 kN-m	Mu+	158.5649 kN-m
Mu ^{beam}	-76.5452625 kN-m	Mu ^{beam}	101.0851238 kN-m
Mu ^{coln}	-6.282784884 kN-m	Mu ^{coln}	8.297000581 kN-m
Mu ^{mid}	-12.0071 kN-m	Mu ^{mid}	15.85649 kN-m
Asmin1	716.1166882 mm2	Asmin1	716.1166882 mm2
Asmin2	757.8666667 mm2	Asmin2	757.8666667 mm2
As Mu ^{beam}	315.5967224 mm2	As Mu ^{beam}	418.4661073 mm2
a	0.01591244 m	a	0.021099131 m
Goal seek	-7.75221E-06	Goal seek	-5.5546E-06
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbear	757.8666667 mm2	Use Asbear	757.8666667 mm2
2 22s		2 22s	

Slab type B			
Top		Bottom	
Mu-	-236.667 kN-m	Mu+	269.8291 kN-m
Mu ^{beam}	-150.8752125 kN-m	Mu ^{beam}	172.0160513 kN-m
Mu ^{coln}	-12.38373837 kN-m	Mu ^{coln}	14.11896453 kN-m
Mu ^{mid}	-23.6667 kN-m	Mu ^{mid}	26.98291 kN-m
Asmin1	716.1166882 mm2	Asmin1	716.1166882 mm2
Asmin2	757.8666667 mm2	Asmin2	757.8666667 mm2
As Mu ^{beam}	629.8359646 mm2	As Mu ^{beam}	720.6943381 mm2
a	0.031756435 m	a	0.03633753 m
Goal seek	-1.39144E-06	Goal seek	-9.57317E-07
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbear	757.8666667 mm2	Use Asbear	757.8666667 mm2
2 22s		2 22s	

Slab type C			
Top		Bottom	
Mu-	-225.4719 kN-m	Mu+	291.6813 kN-m
Mu ^{beam}	-143.7383363 kN-m	Mu ^{beam}	185.9468288 kN-m
Mu ^{coln}	-11.79794826 kN-m	Mu ^{coln}	15.2623936 kN-m
Mu ^{mid}	-22.54719 kN-m	Mu ^{mid}	29.16813 kN-m
Asmin1	716.1166882 mm2	Asmin1	716.1166882 mm2
Asmin2	757.8666667 mm2	Asmin2	757.8666667 mm2
As Mu ^{beam}	599.3150642 mm2	As Mu ^{beam}	780.938373 mm2
a	0.030217566 m	a	0.039375044 m
Goal seek	-1.68086E-06	Goal seek	-6.297E-07
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		YES	
Use Asbear	757.8666667 mm2	Use Asbear	780.938373 mm2
2 22s		2 22s	

Slab type D			
Top		Bottom	
Mu-	-263.7457 kN-m	Mu+	284.7973 kN-m
Mu ^{beam}	-168.1378838 kN-m	Mu ^{beam}	181.5582788 kN-m
Mu ^{coln}	-13.80064709 kN-m	Mu ^{coln}	14.9021843 kN-m
Mu ^{mid}	-26.37457 kN-m	Mu ^{mid}	28.47973 kN-m
Asmin1	716.1166882 mm2	Asmin1	716.1166882 mm2
Asmin2	757.8666667 mm2	Asmin2	757.8666667 mm2
As Mu ^{beam}	703.9761197 mm2	As Mu ^{beam}	761.9276629 mm2
a	0.035494594 m	a	0.038416521 m
Goal seek	-8.57162E-07	Goal seek	-7.20931E-07
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		YES	
Use Asbear	757.8666667 mm2	Use Asbear	761.9276629 mm2
2 22s		2 22s	

Slab type E			
Top		Bottom	
Mu-	-63.9804 kN-m	Mu+	108.7767 kN-m
Mu ^{beam}	-40.787505 kN-m	Mu ^{beam}	69.34514625 kN-m
Mu ^{coln}	-3.347811628 kN-m	Mu ^{coln}	5.69180407 kN-m
Mu ^{mid}	-6.39804 kN-m	Mu ^{mid}	10.87767 kN-m
Asmin1	716.1166882 mm2	Asmin1	716.1166882 mm2
Asmin2	757.8666667 mm2	Asmin2	757.8666667 mm2
As Mu ^{beam}	167.1922973 mm2	As Mu ^{beam}	285.573709 mm2
a	0.008429864 m	a	0.014398674 m
Goal seek	-1.52058E-05	Goal seek	-1.0543E-05
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbear	757.8666667 mm2	Use Asbear	757.8666667 mm2
2 22s		2 22s	

Slab type F			
Top		Bottom	
Mu-	-72.2778 kN-m	Mu+	104.2067 kN-m
Mu ^{beam}	-46.0770975 kN-m	Mu ^{beam}	66.43177125 kN-m
Mu ^{coln}	-3.781977907 kN-m	Mu ^{coln}	5.452676163 kN-m
Mu ^{mid}	-7.22778 kN-m	Mu ^{mid}	10.42067 kN-m
Asmin1	716.1166882 mm2	Asmin1	716.1166882 mm2
Asmin2	757.8666667 mm2	Asmin2	757.8666667 mm2
As Mu ^{beam}	189.0362088 mm2	As Mu ^{beam}	273.4458458 mm2
a	0.009531237 m	a	0.013787186 m
Goal seek	-1.38301E-05	Goal seek	-1.1144E-05
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbear	757.8666667 mm2	Use Asbear	757.8666667 mm2
2 22s		2 22s	

Slab type G			
Top		Bottom	
Mu-	-4.5868 kN-m	Mu+	36.0433 kN-m
Mu ^{beam}	-2.924085 kN-m	Mu ^{beam}	22.97760375 kN-m
Mu ^{coln}	-0.240006977 kN-m	Mu ^{coln}	1.885986628 kN-m
Mu ^{mid}	-0.45868 kN-m	Mu ^{mid}	3.60433 kN-m
Asmin1	716.1166882 mm2	Asmin1	716.1166882 mm2
Asmin2	757.8666667 mm2	Asmin2	757.8666667 mm2
As Mu ^{beam}	11.91375744 mm2	As Mu ^{beam}	93.91874894 mm2
a	0.000600694 m	a	0.004735399 m
Goal seek	-2.87385E-05	Goal seek	-2.4031E-05
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbear	757.8666667 mm2	Use Asbear	757.8666667 mm2
2 22s		2 22s	

Slab type H			
Top		Bottom	
Mu-	-8.58 kN-m	Mu+	18.5128 kN-m
Mu ^{beam}	-5.46975 kN-m	Mu ^{beam}	11.80191 kN-m
Mu ^{coln}	-0.448953488 kN-m	Mu ^{coln}	0.968693023 kN-m
Mu ^{mid}	-0.858 kN-m	Mu ^{mid}	1.85128 kN-m
Asmin1	716.1166882 mm2	Asmin1	716.1166882 mm2
Asmin2	757.8666667 mm2	Asmin2	757.8666667 mm2
As Mu ^{beam}	22.29479058 mm2	As Mu ^{beam}	48.15326882 mm2
a	0.001124107 m	a	0.002427896 m
Goal seek	-2.75937E-05	Goal seek	-2.88289E-05
Is As Mu-beam > As min?		Is As Mu-beam > As min?	
NO		NO	
Use Asbear	757.8666667 mm2	Use Asbear	757.8666667 mm2
2 22s		2 22s	

Design Beam/Slab for Shear - 1st Floor

Slab type A	
element	
V(1.0EQ)max	128.541 kN
V(1.2D+1.0L)max	234.722 kN
B	0.8
Vbeam	316.3186 kN
Vslab	46.9444 kN
<u>Check for Beam</u>	
Is Ve>0.5Vbeam?	
No	
ϕV_c	120.1123 kN
As	94.14244 mm ²
Use	1#13@ .1m o/c starting 50mm
from face of support over a length 1.3m	
<u>Check for Slab</u>	
ϕV_{slab}	$\geq$ Vslab ?
225.5802764	46.9444
YES	

Slab type B	
element	
V(1.0EQ)m:	234.722 kN
V(1.2D+1.0	12.865 kN
B	0.8
Vbeam	245.014 kN
Vslab	2.573 kN
<u>Check for Beam</u>	
Is Ve>0.5Vbeam?	
YES, concrete has cracked	
ϕV_c	0 kN
As	72.92083333 mm ²
Use	1#13@ .1m o/c starting 50mm
from face of support over a length 1.3m	
<u>Check for Slab</u>	
ϕV_{slab}	$\geq$ Vslab ?
225.5803	2.573
YES	

Slab type C	
element	
V(1.0EQ)m:	26.472 kN
V(1.2D+1.0	234.722 kN
B	0.8
Vbeam	214.2496 kN
Vslab	46.9444 kN
<u>Check for Beam</u>	
Is Ve>0.5Vbeam?	
No	
ϕV_c	120.1122521 kN
As	63.7647619 mm ²
Use	1#10@ .1m o/c starting 50mm
from face of support over a length 1.3m	
<u>Check for Slab</u>	
ϕV_{slab}	$\geq$ Vslab ?
225.5803	46.9444
YES	

Slab type D	
element	
V(1.0EQ)max	12.865 kN
V(1.2D+1.0L)max	816.543 kN
B	0.8
Vbeam	666.0994 kN
Vslab	163.3086 kN
<u>Check for Beam</u>	
Is Ve>0.5Vbeam?	
No	
ϕV_c	120.1123 kN
As	198.2439 mm ²
Use	1#16@ .1m o/c starting 50mm
from face of support over a length 1.3m	
<u>Check for Slab</u>	
ϕV_{slab}	$\geq$ Vslab ?
225.5802764	163.3086
YES	

Slab type E	
element	
V(1.0EQ)m:	20.813 kN
V(1.2D+1.0	3.913 kN
B	0.8
Vbeam	23.9434 kN
Vslab	0.7826 kN
<u>Check for Beam</u>	
Is Ve>0.5Vbeam?	
YES, concrete has cracked	
ϕV_c	0 kN
As	7.126011905 mm ²
Use	1#10@ .1m o/c starting 50mm
from face of support over a length 1.3m	
<u>Check for Slab</u>	
ϕV_{slab}	$\geq$ Vslab ?
225.5803	0.7826
YES	

Slab type F	
element	
V(1.0EQ)m:	17.172 kN
V(1.2D+1.0	8.944 kN
B	0.8
Vbeam	24.3272 kN
Vslab	1.7888 kN
<u>Check for Beam</u>	
Is Ve>0.5Vbeam?	
YES, concrete has cracked	
ϕV_c	0 kN
As	7.240238095 mm ²
Use	1#10@ .1m o/c starting 50mm
from face of support over a length 1.3m	
<u>Check for Slab</u>	
ϕV_{slab}	$\geq$ Vslab ?
225.5803	1.7888
YES	

Slab type G	
element	
V(1.0EQ)max	16.082 kN
V(1.2D+1.0L)max	4.09E-06 kN
B	0.8
Vbeam	16.082 kN
Vslab	8.17E-07 kN
<u>Check for Beam</u>	
Is Ve>0.5Vbeam?	
YES, concrete has cracked	
ϕV_c	0 kN
As	4.78631 mm ²
Use	1#10@ .1m o/c starting 50mm
from face of support over a length 1.3m	
<u>Check for Slab</u>	
ϕV_{slab}	$\geq$ Vslab ?
225.5802764	8.17E-07
YES	

Slab type H	
element	
V(1.0EQ)m:	16.082 kN
V(1.2D+1.0	0.000006496 kN
B	0.8
Vbeam	16.0820052 kN
Vslab	1.2992E-06 kN
<u>Check for Beam</u>	
Is Ve>0.5Vbeam?	
YES, concrete has cracked	
ϕV_c	0 kN
As	4.78631107 mm ²
Use	1#10@ .1m o/c starting 50mm
from face of support over a length 1.3m	
<u>Check for Slab</u>	
ϕV_{slab}	$\geq$ Vslab ?
225.5803	1.3E-06
YES	

Design Beam/Slab for Shear - 4th Floor

Slab type A		
element		
V(1.0EQ)max	71.11 kN	
V(1.2D+1.0L)max	221.486 kN	
B	0.8	
Vbeam	248.2988 kN	
Vslab	44.2972 kN	
Check for Beam		
Is Ve>0.5Vbeam?		
No		
ϕV_c	120.1122521 kN	
As	73.89845238 mm2	
Use	1#13@ .1m o/c starting 50mm	
from face of support over a length 1.3m		
Check for Slab		
ϕV_{slab}	$\geq$	Vslab ?
225.5802764		44.2972
YES		

Slab type B		
element		
V(1.0EQ)m:	221.486 kN	
V(1.2D+1.0	12.865 kN	
B	0.8	
Vbeam	231.778 kN	
Vslab	2.573 kN	
Check for Beam		
Is Ve>0.5Vbeam?		
YES, concrete has cracked		
ϕV_c	0 kN	
As	68.98154762 mm2	
Use	1#10@ .1m o/c starting 50mm	
from face of support over a length 1.3m		
Check for Slab		
ϕV_{slab}	$\geq$	Vslab ?
225.5803		2.573
YES		

Slab type C		
element		
V(1.0EQ)m:	28.679 kN	
V(1.2D+1.0	221.486 kN	
B	0.8	
Vbeam	205.8678 kN	
Vslab	44.2972 kN	
Check for Beam		
Is Ve>0.5Vbeam?		
No		
ϕV_c	120.1122521 kN	
As	61.27017857 mm2	
Use	1#10@ .1m o/c starting 50mm	
from face of support over a length 1.3m		
Check for Slab		
ϕV_{slab}	$\geq$	Vslab ?
225.5803		44.2972
YES		

Slab type D		
element		
V(1.0EQ)max	12.865 kN	
V(1.2D+1.0L)max	415.32 kN	
B	0.8	
Vbeam	345.121 kN	
Vslab	83.064 kN	
Check for Beam		
Is Ve>0.5Vbeam?		
No		
ϕV_c	120.1122521 kN	
As	102.7145833 mm2	
Use	1#13@ .1m o/c starting 50mm	
from face of support over a length 1.3m		
Check for Slab		
ϕV_{slab}	$\geq$	Vslab ?
225.5802764		83.064
YES		

Slab type E		
element		
V(1.0EQ)m:	30.701 kN	
V(1.2D+1.0	10.46 kN	
B	0.8	
Vbeam	39.069 kN	
Vslab	2.092 kN	
Check for Beam		
Is Ve>0.5Vbeam?		
YES, concrete has cracked		
ϕV_c	0 kN	
As	11.62767857 mm2	
Use	1#10@ .1m o/c starting 50mm	
from face of support over a length 1.3m		
Check for Slab		
ϕV_{slab}	$\geq$	Vslab ?
225.5803		2.092
YES		

Slab type F		
element		
V(1.0EQ)m:	20.188 kN	
V(1.2D+1.0	23.261 kN	
B	0.8	
Vbeam	38.7968 kN	
Vslab	4.6522 kN	
Check for Beam		
Is Ve>0.5Vbeam?		
YES, concrete has cracked		
ϕV_c	0 kN	
As	11.54666667 mm2	
Use	1#10@ .1m o/c starting 50mm	
from face of support over a length 1.3m		
Check for Slab		
ϕV_{slab}	$\geq$	Vslab ?
225.5803		4.6522
YES		

Slab type G		
element		
V(1.0EQ)max	16.082 kN	
V(1.2D+1.0L)max	0.000006025 kN	
B	0.8	
Vbeam	16.08200482 kN	
Vslab	0.000001205 kN	
Check for Beam		
Is Ve>0.5Vbeam?		
YES, concrete has cracked		
ϕV_c	0 kN	
As	4.786310958 mm2	
Use	1#10@ .1m o/c starting 50mm	
from face of support over a length 1.3m		
Check for Slab		
ϕV_{slab}	$\geq$	Vslab ?
225.5802764		1.21E-06
YES		

Slab type H		
element		
V(1.0EQ)m:	16.082 kN	
V(1.2D+1.0	0.0000148 kN	
B	0.8	
Vbeam	16.08201184 kN	
Vslab	0.00000296 kN	
Check for Beam		
Is Ve>0.5Vbeam?		
YES, concrete has cracked		
ϕV_c	0 kN	
As	4.786313048 mm2	
Use	1#10@ .1m o/c starting 50mm	
from face of support over a length 1.3m		
Check for Slab		
ϕV_{slab}	$\geq$	Vslab ?
225.5803		2.96E-06
YES		

Design Beam/Slab for Shear - 7th Floor

Slab type A			
element			
V(1.0EQ)max	51.976 kN		
V(1.2D+1.0L)max	188.297 kN		
B	0.8		
Vbeam	202.6136 kN		
Vslab	37.6594 kN		
<u>Check for Beam</u>			
Is Ve>0.5Vbeam?			
No			
φVc	120.1123 kN		
As	60.30167 mm2		
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m		
<u>Check for Slab</u>			
φVslab	≥	Vslab	?
225.5802764		37.6594	
YES			

Slab type B			
element			
V(1.0EQ)mi	188.297 kN		
V(1.2D+1.0	12.865 kN		
B	0.8		
Vbeam	198.589 kN		
Vslab	2.573 kN		
<u>Check for Beam</u>			
Is Ve>0.5Vbeam?			
YES, concrete has cracked			
φVc	0 kN		
As	59.10386905 mm2		
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m		
<u>Check for Slab</u>			
φVslab	≥	Vslab	?
225.5803		2.573	
YES			

Slab type C			
element			
V(1.0EQ)m;	30.479	kN	
V(1.2D+1.0	188.297	kN	
B	0.8		
Vbeam	181.1166	kN	
Vslab	37.6594	kN	
<u>Check for Beam</u>			
Is $V_e > 0.5V_{beam}$?			
No			
ϕV_c	120.1122521	kN	
As	53.90375	mm ²	
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m		
<u>Check for Slab</u>			
ϕV_{slab}	$\geq$	Vslab	?
225.5803		37.6594	
YES			

Slab type D			
element			
V(1.0EQ)max	12.865 kN		
V(1.2D+1.0L)max	294.296 kN		
B	0.8		
Vbeam	248.3018 kN		
Vslab	58.8592 kN		
<u>Check for Beam</u>			
Is $V_e > 0.5V_{beam}$?			
No			
ϕV_c	120.1123 kN		
As	73.89935 mm ²		
Use	1#13@ .1m o/c starting 50mm from face of support over a length 1.3m		
<u>Check for Slab</u>			
ϕV_{slab}	$\geq$	Vslab	?
225.5802764		58.8592	
YES			

Slab type E	
element	
V(1.0EQ)mi	36.117 kN
V(1.2D+1.0	12.236 kN
B	0.8
Vbeam	45.9058 kN
Vslab	2.4472 kN
<u>Check for Beam</u>	
Is Ve>0.5Vbeam?	
YES, concrete has cracked	
φVc	0 kN
As	13.66244048 mm2
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m
<u>Check for Slab</u>	
φVslab	≥ Vslab ?
225.5803	2.4472
YES	

Slab type F			
element			
V(1.0EQ)mi	23 kN		
V(1.2D+1.0	27.579 kN		
B	0.8		
Vbeam	45.0632 kN		
Vslab	5.5158 kN		
<u>Check for Beam</u>			
Is Ve>0.5Vbeam?			
YES, concrete has cracked			
φVc	0 kN		
As	13.41166667	mm2	
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m		
<u>Check for Slab</u>			
φVslab	≥	Vslab	?
225.5803		5.5158	
YES			

Slab type G			
element			
V(1.0EQ)max	16.082 kN		
V(1.2D+1.0L)max	6.83E-06 kN		
B	0.8		
Vbeam	16.08201 kN		
Vslab	1.37E-06 kN		
<u>Check for Beam</u>			
Is Ve>0.5Vbeam?			
YES, concrete has cracked			
φVc	0 kN		
As	4.786311 mm2		
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m		
<u>Check for Slab</u>			
φVslab	≥	Vslab	?
225.5802764		1.37E-06	
YES			

Slab type H			
element			
V(1.0EQ)mi	16.082 kN		
V(1.2D+1.0	0.00001833 kN		
B	0.8		
Vbeam	16.08201466 kN		
Vslab	0.000003666 kN		
<u>Check for Beam</u>			
Is Ve>0.5Vbeam?			
YES, concrete has cracked			
φVc	0 kN		
As	4.786313888 mm2		
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m		
<u>Check for Slab</u>			
φVslab	≥	Vslab	?
225.5803		3.67E-06	
YES			

Design Beam/Slab for Shear - 10th Floor

Slab type A			
element			
V(1.0EQ)max	25.9 kN		
V(1.2D+1.0L)max	124.882 kN		
B	0.8		
Vbeam	125.8056 kN		
Vslab	24.9764 kN		
<u>Check for Beam</u>			
Is $V_e > 0.5V_{beam}$?			
No			
ϕV_c	120.1123 kN		
As	37.44214 mm ²		
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m		
<u>Check for Slab</u>			
ϕV_{slab}	$\geq$	Vslab	?
225.5802764		24.9764	
YES			

Slab type B			
element			
V(1.0EQ)mi	124.882 kN		
V(1.2D+1.0	12.865 kN		
B	0.8		
Vbeam	135.174 kN		
Vslab	2.573 kN		
<u>Check for Beam</u>			
Is $V_e > 0.5V_{beam}$?			
YES, concrete has cracked			
ϕV_c	0 kN		
As	40.23035714	mm2	
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m		
<u>Check for Slab</u>			
ϕV_{slab}	$\geq$	Vslab	?
225.5803		2.573	
YES			

Slab type C			
element			
V(1.0EQ)mi	31.534 kN		
V(1.2D+1.0	124.882 kN		
B	0.8		
Vbeam	131.4396 kN		
Vslab	24.9764 kN		
<u>Check for Beam</u>			
Is $V_e > 0.5V_{beam}$?			
No			
ϕV_c	120.1122521 kN		
As	39.11892857	mm2	
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m		
<u>Check for Slab</u>			
ϕV_{slab}	$\geq$	Vslab	?
225.5803		24.9764	
YES			

Slab type D			
element			
V(1.0EQ)max	12.865 kN		
V(1.2D+1.0L)max	136.278 kN		
B	0.8		
Vbeam	121.8874 kN		
Vslab	27.2556 kN		
<u>Check for Beam</u>			
Is $V_e > 0.5V_{beam}$?			
No			
ϕV_c	120.1123 kN		
As	36.27601 mm ²		
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m		
<u>Check for Slab</u>			
ϕV_{slab}	$\geq$	Vslab	?
225.5802764		27.2556	
YES			

Slab type E	
element	
V(1.0EQ)mi:	38.898 kN
V(1.2D+1.0	11.735 kN
B	0.8
Vbeam	48.286 kN
Vslab	2.347 kN
<u>Check for Beam</u>	
Is $V_e > 0.5V_{beam}$?	
YES, concrete has cracked	
ϕV_c	0 kN
As	14.37083333 mm ²
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m
<u>Check for Slab</u>	
$\phi V_{slab} \geq$	Vslab ?
225.5803	2.347
YES	

Slab type F			
element			
V(1.0EQ)mi	24.715 kN		
V(1.2D+1.0	26.663 kN		
B			
	0.8		
Vbeam	46.0454 kN		
Vslab	5.3326 kN		
<u>Check for Beam</u>			
Is $V_e > 0.5V_{beam}$?			
YES, concrete has cracked			
ϕV_c	0 kN		
As	13.7039881	mm ²	
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m		
<u>Check for Slab</u>			
ϕV_{slab}	$\geq$	Vslab	?
225.5803		5.3326	
YES			

Slab type G			
element			
V(1.0EQ)max	16.082 kN		
V(1.2D+1.0L)max	5.69E-06 kN		
B	0.8		
Vbeam	16.082 kN		
Vslab	1.14E-06 kN		
<u>Check for Beam</u>			
Is $V_e > 0.5V_{beam}$?			
YES, concrete has cracked			
ϕV_c	0 kN		
As	4.786311 mm ²		
Use	1#10@ .1m o/c starting 50mm from face of support over a length 1.3m		
<u>Check for Slab</u>			
ϕV_{slab}	$\geq$	Vslab	?
225.5802764		1.14E-06	
YES			

Slab type H			
element			
V(1.0EQ)mi:	16.082 kN		
V(1.2D+1.0	0.00001612 kN		
B	0.8		
Vbeam	16.0820129 kN		
Vslab	0.000003224 kN		
<u>Check for Beam</u>			
Is $V_e > 0.5V_{beam}$?			
YES, concrete has cracked			
ϕV_c	0 kN		
As	4.786313362	mm2	
Use	1#10@ .1m o/c starting 50mm		
from face of support over a length 1.3m			
<u>Check for Slab</u>			
ϕV_{slab}	$\geq$	Vslab	?
225.5803		3.22E-06	
YES			

Column Design (Axial Only)

Design Column for Axial Only - 1st Story

Pu 6508.789 kN
>Pu(bend+axial)? NO
Use rebar from bend+axial calcs
Ag 0.49

Goal seek 0
φPn 6508.789 kN
Use 3#32s
Is $\phi P_n \geq P_u$?
YES

Design Column for Axial Only - 7st Story

Pu 2852.778 kN
>Pu(bend+axial)? NO
Use rebar from bend+axial calcs

Design Column for Axial Only - 4th Story

Pu 4493.302 kN
>Pu(bend+axial)? NO
Use rebar from bend+axial calcs

Design Column for Axial Only - 10th Story

Pu 1535.89 kN
>Pu(bend+axial)? NO
Use rebar from bend+axial calcs

Design Column for Bending+Axial - 1st Story

Circle	
Pu	10861.023 kN
Mu	1843.07535 kN-m
	24#29s
Design Column for Shear Load	
Vu	1233.497 kN
Pu	
1.4P _D	5456.71 kN
1.2P _D +1.6P _L	6508.789 kN
1.2P _D +1.0P _E +1.0P _L	10861.023 kN
0.9P _D +1.0P _E	8546.972 kN
Mpr	1900 kN-m
Ve	1266.66667 kN
Vmax	1266.66667 kN
Is Pu>Ag*f _c /20?	
YES, Compute φVc	
φVc	493.801056 kN
Vmax>φVc?	
YES, compute φVs	
As	420.076065 mm ²
Asmin1	384.929032 mm ²
Asmin2	420.36 mm ²
Use As	420.36 mm ²
Place # 25 over distance lo at each end of the column	
lo	.1m
Place # 25 over s	
s	.1m

X	
Pu	5138.167 kN
Mu	1517.745 kN-m
	24#25s
Design Column for Shear Load	
Vu	755.093 kN
Pu	
1.4P _D	2869.411 kN
1.2P _D +1.6P _L	3114.24 kN
1.2P _D +1.0P _L	5138.167 kN
0.9P _D +1.0P _L	4114.077 kN
Mpr	1900 kN-m
Ve	1266.667 kN
Vmax	1266.667 kN
Is Pu>Ag*f _c /20?	
YES, Compute φVc	
φVc	233.7732 kN
Vmax>φVc?	
YES, compute φVs	
As	561.4091 mm ²
Asmin1	384.929 mm ²
Asmin2	420.36 mm ²
Use As	561.4091 mm ²
Place # 29 over distance lo at each end of the column	
lo	.1m
Place # 25 over s	
s	.1m

Cloud	
Pu	3025.528 kN
Mu	1852.349 kN-m
	24#29s
Design Column for Shear Load	
Vu	1234.899 kN
Pu	
1.4P _D	2164.591 kN
1.2P _D +1.6P _L	2198.298 kN
1.2P _D +1.0P _L	3025.528 kN
0.9P _D +1.0P _L	2347.354 kN
Mpr	1900 kN-m
Ve	1266.667 kN
Vmax	1266.667 kN
Is Pu>Ag*f _c /20?	
YES, Compute φVc	
φVc	137.7818 kN
Vmax>φVc?	
YES, compute φVs	
As	613.5834 mm ²
Asmin1	384.929 mm ²
Asmin2	420.36 mm ²
Use As	613.5834 mm ²
Place # 29 over distance lo at each end of the column	
lo	.1m
Place # 25 over s	
s	.1m

Sq	
Pu	3665.835 kN
Mu	1436.966 kN-m
	24#25s
Design Column for Shear Load	
Vu	718.483 kN
Pu	
1.4P _D	1665.521 kN
1.2P _D +1.6P _L	1478.302 kN
1.2P _D +1.0P _L	3665.835 kN
0.9P _D +1.0P _L	3277.242 kN
Mpr	1500 kN-m
Ve	1000 kN
Vmax	1000 kN
Is Pu>Ag*f _c /20?	
YES, Compute φVc	
φVc	166.8752 kN
Vmax>φVc?	
YES, compute φVs	
As	452.8287 mm ²
Asmin1	384.929 mm ²
Asmin2	420.36 mm ²
Use As	452.8287 mm ²
Place # 25 over distance lo at each end of the column	
lo	.1m
Place # 25 over s	
s	.1m

Design Column for Bending+Axial - 4th Story

Circle		X	
Pu	6684.299 kN	Pu	3967.067 kN
Mu	1406.995 kN-m	Mu	984.0304 kN-m
	24#25s		12#16s
Design Column for Shear Load		Design Column for Shear Load	
Vu	1440.572 kN	Vu	574.651 kN
Pu		Pu	
1.4P ₀	3796.388 kN	1.4P ₀	2332.802 kN
1.2P ₀ +1.6P _L	4493.302 kN	1.2P ₀ +1.6P	2572.507 kN
1.2P ₀ +1.0P _E +1.0P _L	6684.299 kN	1.2P ₀ +1.0P	3967.067 kN
0.9P ₀ +1.0P _E	5096.253 kN	0.9P ₀ +1.0P	3109.079 kN
Mpr	1500 kN-m	Mpr	1250 kN-m
Ve	1000 kN	Ve	833.3333 kN
Vmax	1440.572 kN	Vmax	833.3333 kN
Is Pu>Ag*F _C /20?		Is Pu>Ag*F _C /20?	
YES, Compute ϕV_c		YES, Compute ϕV_c	
ϕV_c	304.0244 kN	ϕV_c	180.5622 kN
Vmax> ϕV_c ?		Vmax> ϕV_c ?	
YES, compute ϕV_s		YES, compute ϕV_s	
As	617.7483 mm ²	As	354.801 mm ²
Asmin1	384.929 mm ²	Asmin1	384.929 mm ²
Asmin2	420.36 mm ²	Asmin2	420.36 mm ²
Use As	617.7483 mm ²	Use As	420.36 mm ²
Place # 29 over distance lo at each end of the column		Place # 25 over distance lo at each end of the column	
lo	.1m	lo	.1m
Place # 25 over s		Place # 25 over s	
s	.1m	s	.1m

Cloud		X	
Pu	1823.737 kN	Pu	1823.737 kN
Mu	1455.931 kN-m	Mu	1455.931 kN-m
	24#25s		12#16s
Design Column for Shear Load		Design Column for Shear Load	
Vu	1452.733 kN	Vu	1452.733 kN
Pu		Pu	
1.4P ₀	1724.701 kN	1.4P ₀	1724.701 kN
1.2P ₀ +1.6P	1780.036 kN	1.2P ₀ +1.6P	1780.036 kN
1.2P ₀ +1.0P	1823.737 kN	1.2P ₀ +1.0P	1823.737 kN
0.9P ₀ +1.0P	1265.583 kN	0.9P ₀ +1.0P	1265.583 kN
Mpr	1500 kN-m	Mpr	1500 kN-m
Ve	1000 kN	Ve	1000 kN
Vmax	1452.733 kN	Vmax	1452.733 kN
Is Pu>Ag*F _C /20?		Is Pu>Ag*F _C /20?	
YES, Compute ϕV_c		YES, Compute ϕV_c	
ϕV_c	83.17634 kN	ϕV_c	83.17634 kN
Vmax> ϕV_c ?		Vmax> ϕV_c ?	
YES, compute ϕV_s		YES, compute ϕV_s	
As	744.3959 mm ²	As	744.3959 mm ²
Asmin1	384.929 mm ²	Asmin1	384.929 mm ²
Asmin2	420.36 mm ²	Asmin2	420.36 mm ²
Use As	744.3959 mm ²	Use As	744.3959 mm ²
Place # 32 over distance lo at each end of the column		Place # 25 over distance lo at each end of the column	
lo	.1m	lo	.1m
Place # 25 over s		Place # 25 over s	
s	.1m	s	.1m

Sq		X	
Pu	2164.407 kN	Pu	2164.407 kN
Mu	766.0559 kN-m	Mu	766.0559 kN-m
	12#16s		12#16s
Design Column for Shear Load		Design Column for Shear Load	
Vu	496.367 kN	Vu	496.367 kN
Pu		Pu	
1.4P ₀	1263.283 kN	1.4P ₀	1263.283 kN
1.2P ₀ +1.6P	1128.876 kN	1.2P ₀ +1.6P	1128.876 kN
1.2P ₀ +1.0P	2164.407 kN	1.2P ₀ +1.0P	2164.407 kN
0.9P ₀ +1.0P	1864.915 kN	0.9P ₀ +1.0P	1864.915 kN
Mpr	750 kN-m	Mpr	750 kN-m
Ve	500 kN	Ve	500 kN
Vmax	500 kN	Vmax	500 kN
Is Pu>Ag*F _C /20?		Is Pu>Ag*F _C /20?	
YES, Compute ϕV_c		YES, Compute ϕV_c	
ϕV_c	98.65527 kN	ϕV_c	98.65527 kN
Vmax> ϕV_c ?		Vmax> ϕV_c ?	
YES, compute ϕV_s		YES, compute ϕV_s	
As	218.1431 mm ²	As	218.1431 mm ²
Asmin1	384.929 mm ²	Asmin1	384.929 mm ²
Asmin2	420.36 mm ²	Asmin2	420.36 mm ²
Use As	420.36 mm ²	Use As	420.36 mm ²
Place # 25 over distance lo at each end of the		Place # 25 over distance lo at each end of the	
lo	.1m	lo	.1m
Place # 25 over s		Place # 25 over s	
s	.1m	s	.1m

Design Column for Bending+Axial - 7th Story

Circle	
Pu	3743.584 kN
Mu	1617.582 kN-m
	18#25s
Design Column for Shear Load	
Vu	1121.632 kN
Pu	
1.4P _D	2421.674 kN
1.2P _D +1.6P _L	2852.778 kN
1.2P _D +1.0P _E +1.0P _L	3743.584 kN
0.9P _D +1.0P _E	2738.993 kN
Mpr	1750 kN-m
Ve	1166.667 kN
Vmax	1166.667 kN
Is Pu>Ag*f _c /20?	
YES, Compute φVc	
φVc	170.4079 kN
Vmax>φVc?	
YES, compute φVs	
As	541.4971 mm ²
Asmin1	384.929 mm ²
Asmin2	420.36 mm ²
Use As	541.4971 mm ²
Place # 29 over distance lo at each end of the column	
lo	.1m
Place # 25 over s	
s	.1m

X	
Pu	2606.441 kN
Mu	803.4895 kN-m
	12#16s
Design Column for Shear Load	
Vu	413.229 kN
Pu	
1.4P _D	1619.016 kN
1.2P _D +1.6P	1798.444 kN
1.2P _D +1.0P	2606.441 kN
0.9P _D +1.0P	2002.811 kN
Mpr	850 kN-m
Ve	566.6667 kN
Vmax	566.6667 kN
Is Pu>Ag*f _c /20?	
YES, Compute φVc	
φVc	118.7398 kN
Vmax>φVc?	
YES, compute φVs	
As	243.4619 mm ²
Asmin1	384.929 mm ²
Asmin2	420.36 mm ²
Use As	420.36 mm ²
Place # 25 over distance lo at each end of the column	
lo	.1m
Place # 25 over s	
s	.1m

Cloud	
Pu	1334.552 kN
Mu	1720.336 kN-m
	24#25s
Design Column for Shear Load	
Vu	1139.424 kN
Pu	
1.4P _D	1189.03 kN
1.2P _D +1.6P	1237.206 kN
1.2P _D +1.0P	1334.552 kN
0.9P _D +1.0P	943.487 kN
Mpr	1500 kN-m
Ve	1000 kN
Vmax	1139.424 kN
Is Pu>Ag*f _c /20?	
YES, Compute φVc	
φVc	60.94937 kN
Vmax>φVc?	
YES, compute φVs	
As	586.1839 mm ²
Asmin1	384.929 mm ²
Asmin2	420.36 mm ²
Use As	586.1839 mm ²
Place # 29 over distance lo at each end of the column	
lo	.1m
Place # 25 over s	
s	.1m

Sq	
Pu	1160.848 kN
Mu	605.0497 kN-m
	12#16s
Design Column for Shear Load	
Vu	381.725 kN
Pu	
1.4P _D	848.396 kN
1.2P _D +1.6P	761.386 kN
1.2P _D +1.0P	1160.848 kN
0.9P _D +1.0P	957.681 kN
Mpr	610 kN-m
Ve	406.6667 kN
Vmax	406.6667 kN
Is Pu>Ag*f _c /20?	
YES, Compute φVc	
φVc	53.05683 kN
Vmax>φVc?	
YES, compute φVs	
As	192.1977 mm ²
Asmin1	384.929 mm ²
Asmin2	420.36 mm ²
Use As	420.36 mm ²
Place # 25 over distance lo at each end of the	
lo	.1m
Place # 25 over s	
s	.1m

Design Column for Bending+Axial - 10th Story

Circle	
Pu	1535.89 kN
Mu	942.6582 kN-m
	18#2s
Design Column for Shear Load	
Vu	674.427 kN
Pu	
1.4P _D	1207.856 kN
1.2P _D +1.6P _L	1423.316 kN
1.2P _D +1.0P _E +1.0P _L	1535.89 kN
0.9P _D +1.0P _E	1034.557 kN
Mpr	1500 kN-m
Ve	1000 kN
Vmax	1000 kN
Is Pu>Ag*f _c /20?	
YES, Compute ϕV_c	
ϕV_c	70.09751 kN
Vmax > ϕV_c ?	
YES, compute ϕV_s	
As	505.4304 mm ²
Asmin1	384.929 mm ²
Asmin2	420.36 mm ²
Use As	505.4304 mm ²
Place # 25 over distance lo at each end of the column	
lo	.1m
Place # 25 over s	
s	.1m

X	
Pu	1206.273 kN
Mu	537.0189 kN-m
	12#16s
Design Column for Shear Load	
Vu	204.045 kN
Pu	
1.4P _D	809.763 kN
1.2P _D +1.6P	899.229 kN
1.2P _D +1.0P	1206.273 kN
0.9P _D +1.0P	904.536 kN
Mpr	550 kN-m
Ve	366.6667 kN
Vmax	366.6667 kN
Is Pu>Ag*f _c /20?	
YES, Compute ϕV_c	
ϕV_c	55.1208 kN
Vmax > ϕV_c ?	
YES, compute ϕV_s	
As	169.3347 mm ²
Asmin1	384.929 mm ²
Asmin2	420.36 mm ²
Use As	420.36 mm ²
Place # 25 over distance lo at each end of the column	
lo	.1m
Place # 25 over s	
s	.1m

Cloud	
Pu	790.164 kN
Mu	1061.546 kN-m
	18#25s
Design Column for Shear Load	
Vu	695.351 kN
Pu	
1.4P _D	598.194 kN
1.2P _D +1.6P	622.831 kN
1.2P _D +1.0P	790.164 kN
0.9P _D +1.0P	593.172 kN
Mpr	1150 kN-m
Ve	766.6667 kN
Vmax	766.6667 kN
Is Pu>Ag*f _c /20?	
YES, Compute ϕV_c	
ϕV_c	36.21416 kN
Vmax > ϕV_c ?	
YES, compute ϕV_s	
As	397.0232 mm ²
Asmin1	384.929 mm ²
Asmin2	420.36 mm ²
Use As	420.36 mm ²
Place # 25 over distance lo at each end of the column	
lo	.1m
Place # 25 over s	
s	.1m

Sq	
Pu	451.52 kN
Mu	389.5361 kN-m
	12#16s
Design Column for Shear Load	
Vu	226.422 kN
Pu	
1.4P _D	423.245 kN
1.2P _D +1.6P	379.532 kN
1.2P _D +1.0P	451.52 kN
0.9P _D +1.0P	350.355 kN
Mpr	500 kN-m
Ve	333.3333 kN
Vmax	333.3333 kN
Is Pu>Ag*f _c /20?	
NO, $\phi V_c=0$	
ϕV_c	0 kN
Vmax > ϕV_c ?	
YES, compute ϕV_s	
As	181.1768 mm ²
Asmin1	384.929 mm ²
Asmin2	420.36 mm ²
Use As	420.36 mm ²
Place # 25 over distance lo at each end of the	
lo	.1m
Place # 25 over s	
s	.1m