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The Prediction of Stead-State Vibration Due to Rhythmic Activity in a Multi-Story Structure

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ABSTRACT

The goal of this thesis is to add to the body of literature on predicting vibrational behavior in multistory buildings. To achieve this goal, the following research question is proposed: Can a multistory ETABS model predict the possibility of whole building vibration issues due to human induced excitation? The focus of this thesis was developing a modeling procedure that is capable of producing a model that can capture the multi-story response that was seen in a case study building. The ten-story case study building had a combination of dance studio spaces and office spaces on various floors. Vibrations caused by people dancing on one floor resulted in occupants on another floor in an office space complaining about annoying vibrations.

This thesis starts by going over the current literature available for evaluation building vibrations and modeling approaches to use for predicting human induced vibrations. From there, a model was created and evaluated to see the dynamic response of the structure. It was found that it is possible to get a multi-story vibrational response out of a model in ETABS from human induced loading that represents the in-situ behavior. In the end, the amplitude of the floor accelerations obtained from the model did closely match the in-situ values. From the results obtained through the model created and the sensitivity analysis performed in this thesis, valuable insight was gained as to how certain assumptions effect the modal properties of an ETABS model when analyzing floor vibrations.

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CHAPTER 1- INTRODUCTION

In the field of structural engineering, building serviceability is the focus on the overall usability, rather than strength, of the building based on its performance. Serviceability limit states, such as deflection, vibration, and deformation, can impact the layout and sizing of structural members compared to what is required for strength purposes. It is important to note that serviceability guidelines in the United States have tended to be vaguer with respect to requirements to ensure a serviceable structure, particularly for multi-story buildings exhibiting whole building modes vulnerable to rhythmic excitation. The goal of this thesis is to add to the body of literature on predicting vibrational behavior in multistory buildings. To achieve this goal, the following research question is proposed: Can a multistory ETABS model predict the possibility of excessive whole building vibration issues due to human induced excitation? To answer this question, data collected from a 10-story dance studio building was compared to similar metrics extracted from an ETABS model based on modeling recommendations found in the literature.

Among the current codes and design references, there are standards and guidelines to use when it comes analyzing a building structure's design for vibrational issues. The leading reference for vibration serviceability in steel frames buildings in the United States is Design Guide 11: Vibrations of Steel-Framed Structural Systems Due to Human Activity (Murray, T.M., et al. 2016.). The Design Guide is a publication from the American Institute of Steel Construction (AISC) that focuses on how human activity impacts vibrations of steel structures. This document provides a great deal of guidance for floor systems but is mostly silent on predicting vibration levels when whole building modes are excited by rhythmic excitation.

Column shortening and non-structural elements can have a big impact on the vibrational response of a structure due to the addition of mass, stiffness and damping they bring to a building. When looking at vibrational transmission between floors, the inclusion of nonstructural partitions can have a large impact because they can allow up to 65% more transmission between floors (Devin, Fanning, Pavic, 2016). The large effect that partitions can have on vibrations traveling throughout the building should be considered when designing the structural system. Before providing details of this study and answering the research question, the next section of this thesis provides an examination of the pertinent literature.

CHAPTER 2- LITERATURE REVIEW

This section will go over the background research that was done in preparation for conducting the investigation and writing this thesis. The coursework taken up to this point has only very briefly covered the topic of structural dynamics and thus a deeper dive into the literature pertaining to vibrational behavior of buildings was needed.

2.1 Structural Dynamics Overview

When looking at the dynamic response of floors and buildings as a whole, it is important to understand the variables that effect the characteristics of motion; that is, what can affect the vibrations of an object. Vibration is a time dependent condition when looking at human induced vibration since the loads very with time which causes displacement to also be time dependent. This thesis will focus on vibrations due to rhythmic activity where it is people (groups) that are causing the vibrations that disturb other inhabitants. The criterion used for assessment due to rhythmic activity comes from evaluation of steady-state amplitudes due to harmonics of excitation (Murray, T.M., et al. 2016.). Continuous structures have an infinite number of modes shapes and associated natural frequencies but is typically only the fundamental mode shape and frequency that are of interest for vibrational analysis (Hanagan, 2021).

2.1.1 Single Degree of Freedom Systems

It has been shown in the literature that the vibrational behavior of a floor system can be represented by an equivalent single degree of freedom (SDOF) system subjected to transient and harmonic excitation. This section describes the variables and behavior of a SDOF system.

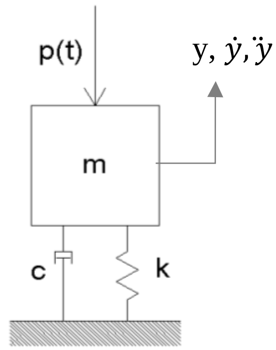

Figure 1:SDOF Model

In a single degree of freedom system there is a mass (m), a spring with stiffness (k), a time variable force (p(t)), and a linear dashpot (c). These four factors affect the motion of the system and are shown in Figure 1. A single degree of freedom system is good to use when looking at modeling the behavior of single objects, such as joists or beams within a floor and how they behave dynamically (Naiem, 1991). Equation 2.1.1.1 is the equation of motion for a single degree of freedom system (Clough and Penzien, 2003).

$$p(t) = m\ddot{y} + c\dot{y} + ky \qquad \text{Eq.2.1.1.1}$$

Equation 2.1.1.1 shows the relationship with how mass (m) effects the acceleration ($\ddot{y}$) of the system, the damping (c) effects the velocity of the system ($\dot{y}$), and stiffness (k) effects the displacement (y) of the system.

2.1.2 Resonance

Resonance is an important aspect to understand when evaluating floor vibrations because floors that are excited with a periodic force that has a harmonic component at a natural frequency of the floor will lead to resonance buildup (Allen, 1990). Because resonance with rhythmic excitations is likely to cause objectionable levels of vibration, Design Guide 11 recommends floors be designed with first natural frequency above the excitation frequency expected (Murray, T.M., et al. 2016). Figure 2 from Design Guide 11 (Murray, T.M., et al. 2016.), depicts the relationship between natural frequency and forcing frequency. When the force frequency and natural frequency are the same value, it causes a drastic amplification in the vibrational response of the floor leading to larger accelerations and displacements.

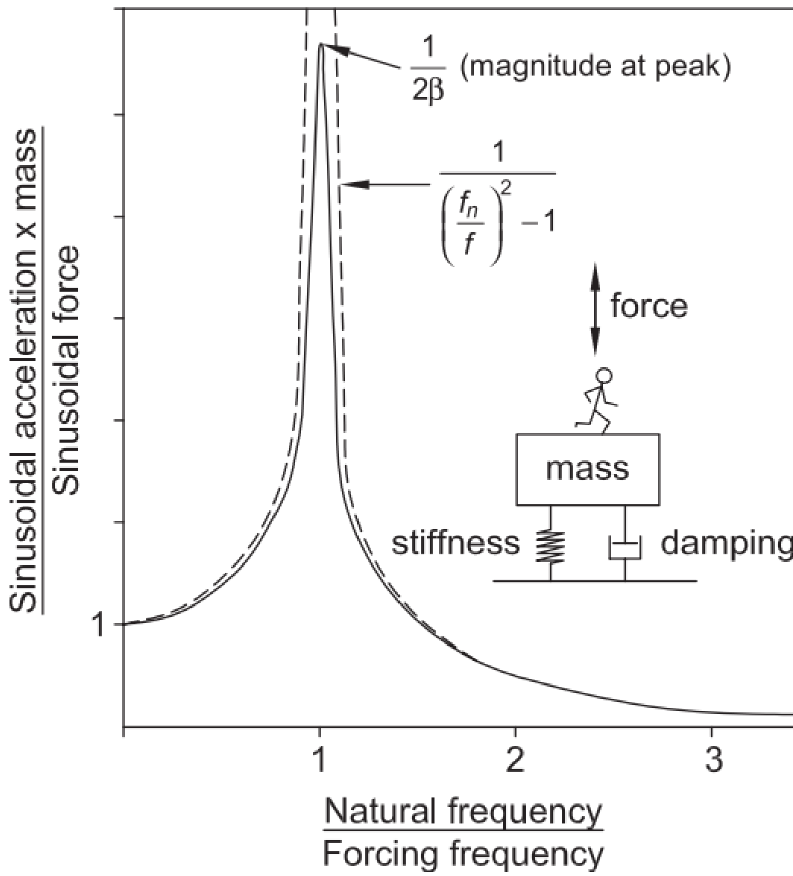

Figure 2: Steady-State Response of Mass-Spring-Damper System to Sinusoidal Force (Murray, T.M., et al. 2016)

The system at resonance only becomes bounded in its response by the damping present in the system, represented as the damping ratio, β. The damping ratio in structures is typically anywhere from 1% to 5% and is dependent on nonstructural features (Murray, T.M., et al. 2016.).

2.2 Perception of Floor Vibrations

When it comes to examining floor vibrations, it is important to look at what are the acceptable levels that a floor can vibrate and move without occupant discomfort or disturbance. Human perception of floor vibrations has been shown to be dependent on frequency, amplitude, and duration, (Lenzen, 1966). When looking at frequency, floors that have a frequency between four and eight hertz are of particular concern (Murry, Allen, Ungar et al, 2016). The reasoning behind the 4-8 Hz range is that human internal organs have a natural frequency in that range (Murry, 1991) so the sensitivity to vibration is this range is more acute. When a floor vibrates at a frequency that is the same as the natural frequency internal organs, it causes resonance buildup, leading to a higher perceived annoyance by humans. Figure 3, taken from AISC Design Guide 11 (Murray, et. al 2016), shows how the acceptable peak acceleration for different occupant spaces are lowest within the range most susceptible to human perception.

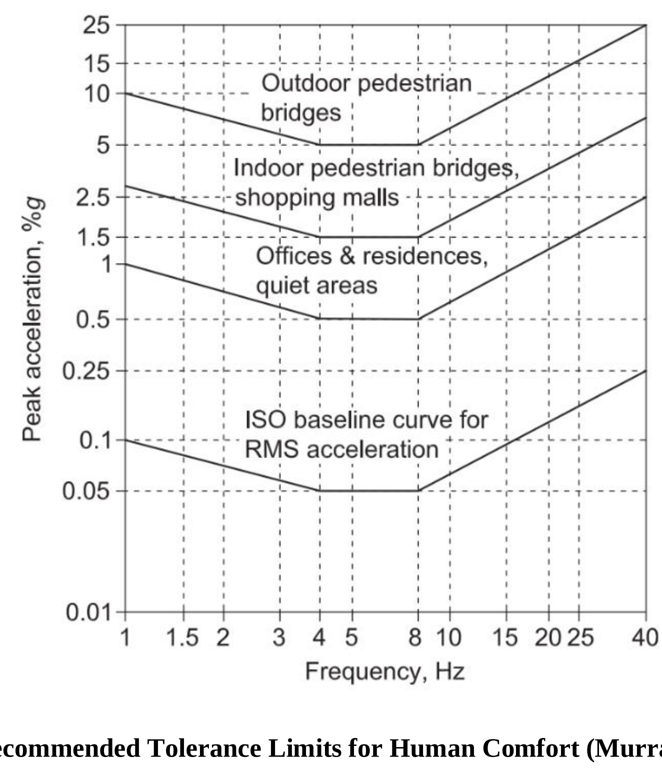

Figure 3: Recommended Tolerance Limits for Human Comfort (Murray et. al 2016)

People’s perception to vibrations are also dependent on the activity they are performing leading to different types of occupancies having different tolerance limits as seen in Figure 3. For example, spaces where occupants are seated and sedentary, like office spaces, the acceptable peak acceleration is lower than for spaces where people are moving about, like pedestrian bridges and ballrooms. In terms of an amplitude for deflections that are distinctly perceivable to inhabitants, as small as 0.006” to 0.018” can be problematic (Lenzen, 1966). The limits set out in the design guides are set to avoid vibrations that can be classified as annoyance vibrations which leads occupants to complain about vibrational issues. It is impractical in most structures to avoid any perceptible vibrations given the extremely high stiffness needed; rather vibrations should be controlled in their amplitude and duration.

2.2.1 Human Perception to Transient and Steady-State Vibrations

Transient vibration can be defined as a rapid build-up to a peak followed by a damped decay; such behaver comes from an impact from a heavy object onto the floor system (Naiem, 1991). Steady-state vibrations are characterized as vibrations that are long in duration and are the harmonic response of the structure (Clough and Penzien, 2003). The longer length of time a floor is oscillating from vibrations, the more perceptible those vibrations are to inhabitants. Reiher and Meister (1931) were to first to study the human perception to vibrations by subjecting a group of people to steady-state vibrations and having them rate the experience. From their work, Lenzen in 1966 did further studies and found that humans are much less sensitive to transient vibrations than they are to steady-state vibrations. Murry then further expanded upon the data collected by Reiher and Meister along with Lenzen’s findings to refine the vibrational criteria resulting in Figure 3.

2.3 Rhythmic Activity Excitation

When it comes to designing a building's structure, special consideration is needed to avoid problematic floor vibrations due to rhythmic excitation. When occupants are performing rhythmic activity, it tends to cause steady-state excitation with dominant harmonics in the range of 1.5-6 Hz; many medium to long span structures have a natural frequency that falls into this range (Allen, Rainer, Pernica, 1985). Since stiffness is what will ultimately mitigate problematic vibrational issues in more commercial and active spaces, it is recommended to have a first natural frequency of the floor system to be above eight hertz to avoid resonance with the first and second harmonics of the activity (Murry, 1991).

In typical situations, occupants of a structure will provide some level of damping to the structure and in doing so will help mitigate some vibrational issues. In more active forms of rhythmic activity, like jumping and aerobics, the third harmonic of the activity frequency can be problematic and because of that reason, it is recommended that the first natural frequency of the floor be above nine or ten hertz (Murry, 1991). Having a floor system with a first natural frequency that high can be rather expensive and it is recommended to have a design with as deep of members as possible and a slab as light as possible to best achieve the needed high natural frequency (Murry, 1991). The more active the inhabitants are in the building, the higher the recommended minimum first natural frequency should be to avoid problematic floor vibrations.

2.3.1 Jumping Excitation

When looking at how large groups of people effect the behavior of a floor, it is important to understand how the inhabitants are interacting with the building. For example, the effect that people have on a floor is dependent on what activity they are preforming. Humans provide a large amount of damping to a floor when they are present (Lenzen, 1966). However, this is only the case if they are in constant contact with the floor and are not performing a group rhythmic activity, like jumping and dancing, where there is no constant contact with the structure and therefore their bodies are unable to provide a significant increase in damping of the floor. When it comes to looking at more active forms of rhythmic activity, such as aerobic exercise where the occupants are not in constant contact with the floor, special consideration needs to take place. The force function that comes from dance-type loads where jumping is involved is best expressed by a Fourier series where the dynamic load factors are a function of the dance frequency (Ellis and Ji, 1994). The force equation for evaluation of jumping rhythmic activity is shown in Equation 2.3.1.1 (Ji and Ellis, 1994).

$$F(t) = G\left(1 + \sum_{n=1}^{\infty} r_n \sin\left(\frac{2n\pi}{T_p}t + \phi_n\right)\right) \qquad \text{Eq. 2.3.1.1}$$

G: weight of the dancers per unit area (load density of the crown)

r_n: the normalized n^{th} Fourier coefficient of harmonic activity

t: period

T_p: period of dynamic load

ϕ_n: phase angle

For Equation 2.3.1.1, Equations 2.3.1.2, 2.3.1.3, 2.3.1.4, 2.3.1.5, and 2.3.1.6 are needed; the equations are as follows:

$$\phi_n = \tan^{-1}\frac{a_n}{b_n} \qquad \text{Eq. 2.3.1.2}$$

$$a_n = 0.5\left[\frac{\cos(2n\alpha-1)\pi-1}{2n\alpha-1} - \frac{\cos(2n\alpha+1)\pi-1}{2n\alpha+1}\right] \qquad \text{Eq. 2.3.1.3}$$

$$b_n = 0.5\left[\frac{\sin(2n\alpha-1)\pi}{2n\alpha-1} - \frac{\sin(2n\alpha+1)\pi}{2n\alpha+1}\right] \qquad \text{Eq. 2.3.1.4}$$

$$r_n = \sqrt{a_n^2 + b_n^2} \qquad \text{Eq. 2.3.1.5}$$

$$\alpha = \frac{t_p}{T_p} \qquad \text{Eq. 2.3.1.6}$$

t_p: contact duration

Table 1 includes values for the coefficients found in Equation 2.3.1.1. In the table, α is the contact ratio represented by t_p/T_p and n represents the mode.

Table 1: Fourier Coefficients and Phase Lags for Various Contact Ratios (Ji and Ellis 1994; Ellis 1997)

		***n* = 1**	***n* = 2**	***n* = 3**	***n* = 4**	***n* = 5**	***n* = 6**
α = 2/3	r_n	1.28571	0.16364	0.13333	0.03643	0.02302	0.03175
	ϕ_n	-π/6	π/6	-π/2	-π/6	π/6	-π/2
α = 1/2	r_n	1.57080	0.66667	0.00000	0.13333	0.00000	0.05714
	ϕ_n	0	-π/2	0	-π/2	0	-π/2
α = 1/3	r_n	1.80000	1.28571	0.66667	0.16364	0.09890	0.01333
	ϕ_n	π/6	-π/6	-π/2	π/6	-π/6	-π/2
α = 1/4	r_n	1.88562	1.57080	1.13137	0.66667	0.26937	0.00000
	ϕ_n	π/4	0	−π/4	-π/2	π/4	0

While there are many different types of music, each with a wide range of beats and frequencies, the majority of music has a beat frequency in a range of 1.5-3.5 Hz, because humans are incapable of dancing at frequencies greater than 3.5 Hz (Ellis and Ji, 1994). Interestingly, the average beat frequency has increased about 0.12 Hz per decade for the past four decades (Farwell-Greiner, 2003). The natural frequency and damping of a system for dynamic crowds that are not constantly in contact with the floor are not affected and remain the same as if unoccupied (Farwell-Greiner, 2003).

2.3.2 Excitation Without Leaving the Floor

When looking at the loading function of occupants dancing but without leaving the floor surface, it is best expressed as a sinusoidal load at the dance frequency (Naiem, 1991). A crowd that is stationary acts as a spring-mass-damper system on the structure and therefor adds stiffness to the structure which alters the vibrational response of the floor (Farwell-Greiner, 2003).

2.4 Predicting Vibrations

There are two common ways for predicting the vibrational response of a floor that is to be constructed. The first method is using hand calculations, which is good for simpler bay geometries and less complicated scenarios. The second method is using finite model analysis programs, such as ETABS, which is better for when floor geometry becomes a lot more complex and irregular. How ETABS goes about predicating the vibrational response of structures is discussed further in Section 2.6 of this thesis.

2.4.1 Design Guide 11 Hand Calculations for Natural Frequency

The first step in understanding how a floor system will behave within a building is to look at the natural frequency of the floor. To calculate the natural frequency of the whole system, first you must calculate the first natural frequency of the individual members that make up the floor. When looking at a simply supported beam under a uniform mass the calculation of its natural frequency is shown in Equation 2.4.1.1 (Murry, Allen, Ungar et al, 2016).

$$f_n = \frac{\pi}{2}\sqrt{\left(\frac{gE_sI_t}{wL^4}\right)} \qquad \text{Eq. 2.4.1.1}$$

E_s: modulus of elasticity of steel = 29,000 (ksi)

I_t: transformed moment of inertia of the beam (in^4)

L: beam span (in)

g: acceleration of gravity = 386 (in/s^2)

w: uniformly distributed weight per unit length (actual, not design, dead and live loads) supported by the member (kip/in)

This equation can be rewritten and simplified to that of Equation 2.4.1.2. Where the delta in that equation is equal to the delta in Equation 2.4.1.3 and all other terms are the same as defined above for Equation 2.4.1.1 (Murry, Allen, Ungar et al, 2016).

$$f_n = 0.18\sqrt{\frac{g}{\Delta}} \qquad \text{Eq. 2.4.1.2}$$

$$\Delta = \frac{5wL^4}{384E_sI_t} \qquad \text{Eq. 2.4.1.3}$$

Finding the natural frequency of joists and girders is done through the same equation with slight alterations being how the deflection is calculated based on loading conditions and end supports. For members that have a point load at midspan rather than a distributed load, such as girders in some cases, the design guide suggests multiplying Equation 2.4.1.2 by 4/p to account for the difference in frequency that is to be expected (Murry, Allen, Ungar et al, 2016). Once the frequencies of the different elements are found, they can be combined using Dunkerley relationship shown in Equation 2.4.1.4 to find the natural frequency of the entire floor.

$$\frac{1}{f_n{}^2} = \frac{1}{f_g{}^2} + \frac{1}{f_b{}^2} \qquad \text{Eq. 2.4.1.4}$$

f_n: natural frequency of the system

f_g: natural frequency of the girder

f_b: natural frequency of the beam (or joist depending on the elements present)

The Dunkerley relationship can be rewritten so that it is easier to calculate the overall natural frequency using the deflection of the individual elements. Equation 2.4.1.5 should be used when evaluating tall structures because the deflection of columns due to axial shortening effect the value of the natural frequency enough that their deformation needs to be accounted for (Murry, Allen, Ungar et al, 2016).

$$f_n = 0.18\sqrt{\frac{g}{\Delta_b+\Delta_g+\Delta_c}} \qquad \text{Eq. 2.4.1.5}$$

D_b: beam or joist and girder midspan deflection due to the weight supported (in)

D_g: girder midspan deflection due to the weight supported (in)

D_c: axial shortening of the column or wall due to the weight supported (in)

The way that the Design Guide 11 (Murray, et. al 2016) goes about accounting for non-structural elements is adding a value to the damping of the structure based on the type of non-structural elements that would be present within the building once built. In a typical building, about 3-3.5% of critical damping will come from a hung ceiling and mechanical ductwork that runs throughout the building (Murry, 1991). It has been shown that not only are the given values for damping overestimates, but also that the effect that non-structural elements have on stiffening and adding mass cannot be ignored when looking at a structure's vibrational response (Sladki, M. J. 1999).

2.4.1.1 Calculation of Transformed Moment of Inertia

An important thing to note in all these equations is that the calculation of member deflections is based on the transformed moment of inertia of elements. When calculating the natural frequency of a floor for vibrational analysis, it is important to transform the moment of inertia being used for the structural members. Because slabs or decks that are even just resting on top of supporting members, such as joists, beams, or girders, have enough friction between the two members that they do not move independently of one another under human induced loading (Murry, 1991). Non-composite members behave compositely under low amplitude vibrations such as those caused by human activity. The shear forces between the slab, whether it be concrete on metal deck or just a concrete slab, and the steel members are small enough that the frictional force between the two elements is sufficient to transfer the force between the two elements (Murry, 1991). The composite action present in the elements cause them to be stiffer which in turn effects the natural frequency of the elements. An important thing to note when calculating the transformed moment of inertia is that the modulus of elasticity for concrete should be taken as 1.35 times the static modulus of elasticity because concrete behaves stiffer under dynamic loading (Murray, T.M., et al. 2016). When concrete cracks significantly, its dynamic properties are notably different when compared to when the concrete is uncracked, and this difference needs to be taken into account during analysis (Reynolds, Pavic, and Waldron, 1998). Another important thing to note is that the effective width of the slab should be taken as the spacing between adjacent members but no more than 0.4 times the member length. For spandrel elements, that changes to half the span and no more than 0.2 times the member length plus projection of slab edge from beam centerline (Murray, T.M., et al. 2016).

2.4.1.2 Loading to Consider

When deciding what loads to consider when analyzing the floor system for the vibration response, it is important not to overestimate the loads that will be present as lightly loaded floors are more susceptible to vibrational issues. Since vibration is a serviceability issue, only service level loading should be used. It is suggested to use 100% of the structure self-weight, recommended superimposed dead loads, and 10-25% of the design live loads when deciding what loading is present for vibrational analysis purposes (Murry, 1991). It should be noted that 0 psf is recommended to be used as live loading in assembly areas, such as schools, churches, malls, and pedestrian bridges during vibrational analysis since it is when they are least occupied that vibrational complaints are most likely to occur (Murry, Allen, Ungar et al, 2016). Table 2 is from Design Guide 11 shows the recommended live loads to include in the weight for the natural frequency calculation. Only loads that are expected to be present in day-to-day use of the building should be used. Design Guide 11 gives the recommendation that a 4 psf superimposed dead load be used for consideration of normal mechanical and ceiling instillations and that value should be adjusted based on engineering judgment on the expected loads of those systems (Murry, Allen, Ungar et al, 2016).

Table 2: Live Load Table from Design Guide 11

Table 3-1. Recommended Superimposed Live Loads for Walking Vibration Analyses	
Occupancy	**Recommended Live Load, psf**
Paper office	11
Electronic office	6–8
Residence	6
Assembly area	0
Shopping mall	0

In total, the 'w', uniformly distributed weight per unit length supported by the member, should be taken to be the value of the self-weight of the member plus the recommended superimposed dead load plus the applicable live load based on occupancy.

2.4.2 Hand Calculation of Peak Acceleration

An important consideration when analyzing vibrations for human perception is the peak acceleration. The peak acceleration when looking at floor vibrations is often expressed in terms of its relationship to the acceleration of gravity. Equation 2.4.2.1 can be used to estimate the acceleration of a floor due to rhythmic activity as a fraction of gravity based on a steady-state acceleration response (Allen, 1990).

$$\frac{a}{g} = \frac{\alpha \frac{W_p}{W} \sin 2\pi f t}{\sqrt{\left[\left(\frac{f_o}{f}\right)^2 - 1\right]^2 + \left[2\beta \frac{f_o}{f}\right]^2}} \qquad \text{Eq. 2.4.2.1}$$

$\frac{a}{g}$: acceleration as a fraction of the acceleration due to gravity

f: forcing frequency

f_o: natural frequency of the spring-mass system

b: damping ratio

W: mass weight

W_p: weight of the person

α: dynamic load factor

2.5 Vibrational Tests

To validate the results achieved through hand calculations and finite model analysis, in-situ tests must be performed to find the actual values and properties of structures once they are built. By having predictions for built structures from ETABS and comparing it to values obtained from in-situ tests, the values can be compared to see if they match. If the values are close in range and reasonable for prediction natural frequency and peak acceleration, two important factors for vibrational analysis, it gives validity to the ETABS model. No matter what type of in-situ testing is being performed, testing should be done when ambient noise is at a minimum to ensure that quality data is collected (Raebel and Hanagan, 2001). Additionally, the driving force for any in-situ testing setup should be placed away from nodal lines, which can be found in finite element model analysis (Raebel and Hanagan, 2001).

2.5.1 Heel Drop Test

A heel drop test is meant to excite and measure the dominant natural frequencies of the structure in a quick and portable way. The impulse is performed by a who supports their weight on their toes with heels raised a set number of inches off the floor and then drops their weight onto the ground through their heels (Murray, 1991). From data collected from this type of test, a frequency response function (FRF) can be computed and is useful to show the sharp peaks at the floor’s natural frequencies (Murray, T.M., et al. 2016.). A key characteristic of this test is that it is performed by a single person in one location at a time to gather data from that point and this is repeated for multiple points along the floor to get a whole floor response to avoid nodal lines.

2.5.2 Walking Test

In a walking test, a person walks at a set pace that is determined to be one that excites the floors natural modes so that acceleration levels are measured (Murray, T.M., et al. 2016.). An issue with this type of testing is that it cannot be the sole type of test performed since additional testing is needed beforehand to establish what the walking pace should be to cause the greatest excitation in the floor. The benefit of this test is that it simulates the type of disturbance typical in floor vibrations and is able to evaluate the acceleration of the floors. This type of testing is important to understand but is not utilized in this thesis.

2.5.3 Shaker Test

A shaker test is the most expensive and laborious of all tests to perform because an expensive and heavy piece of equipment is needed to perform this type of testing. Specifically, an electrodynamic shaker is used to provide the excitation to the floor system for analysis. It is important to avoid an excitation level that is too high as it can cause the structure to produce effects that cause skewed results (Raebel and Hanagan, 2001). Conversely, an excitation level that is too low should also be avoided because it will produce results that will be indistinguishable from ambient noise (Raebel and Hanagan, 2001). The benefit of a shaker test over others is that it allows for higher quality FRFs to be measured (Davis, 2008).

2.6 Finite Element Analysis

Given the complexity of doing vibrational analysis calculations by hand and the limitations on the equations, there is a greater need of finite model analysis software to perform more complex vibrational analysis. Using programs such as ETABS have the potential to provide dynamic behavior predictions of more complex building systems. The construction of the finite element model and its accuracy is very important for achieving viable results for vibrational analysis because the smallest differences can make a big difference in values given by the model leading to no vibrational issues being predicted but with the possibility of issues once built. Many structural designers are well versed in using finite model analysis software for strength analysis of their designs to ensure their design will be capable of carrying the loads the building will experience. However, very few have the knowledge to perform a vibration serviceability analysis in such programs because special considerations need to be taken into account. When looking at the connectivity of beams and girders, typical connections (even shear connections) should be modeled with the moment restrained (Sladki, 1999). The magnitude of force that comes from human induced walking is not large enough to overcome the friction in the connections of members and this leads to their behavior more closely matching a fixed restraint. When it comes to loading the structure, a time dependent loading function is needed to perform a time history analysis. A time history analysis allows a designer to see how the structure behaves through the various stages of human interaction with the structure, from footfall to loss of contact with the structure. When comparing vibrational analysis results between in-situ tests and finite element analysis, often the analysis model created will have a higher stiffness of members (Raebel and Hanagan, 2001).

When it comes to what to include in a finite element model and what not to include, it is important to consider what contributes to a structure's mass, damping, and stiffness under normal service conditions. When modeling structures where there are light weight nonstructural partitions, most sources (Murray, et. al 2016; Ad Hoc Committee, 1986; Raebel and Hanagan, 2001; Davis, 2008) and standards recommend conservatively excluding them within vibration analysis models. However, it has been shown that finite element analysis models that have been created without modeling the partitions give slightly different results to that of in-situ tests performed on those structures where the partitions are present (Miskovic, Pavic, and Reynolds, 2009). Partitions should not be relied upon to meet the vibrational requirements of a floor. However, if the point of the finite element model is to match in-situ conditions, partitions can't be neglected. While the building might be built with a certain layout of interior partitions, that layout can change or be totally removed at any point during the structure's lifespan leading to a potentially drastic change in the building's vibrational response (Pavic and Petrovic, 2011). When including nonstructural partitions, the peak accelerations can decrease by about 17% when compared to the peak response of a model without partitions and the natural frequency increases by roughly 17% (Devin, Fanning, and Pavic, 2016). Boundary conditions need to be accurately represented in a model so that the results obtained closely match that of what the real values would be (Kreidel, 2014).

An important aspect to note is that when looking at analytical results, some programs and equations will give the peak spectral limit while the codes and standards are often interested in the root-mean-squares (RMS) values for accelerations or velocities. To convert the peak spectral limit to an RMS value, you multiple the peak value by 0.7071 for a purely sinusoidal response (Davis and Liu, 2018).

2.6.1 Two-Dimensional Finite Element Floor Model

A two-dimensional finite element model can be created to analyze the vibrational responses of a single floor level or multiple floor levels. In this type of model, all elements are placed on the same 2D plane at each level and adjustments through member property modifiers are made to element properties to account for composite action (Kreidel, 2014).

2.6.2 Three-Dimensional Finite Element Floor Model

A three-dimensional finite element model can be created to either look at the vibration response of a single floor or of an entire structure's vibrational response for multiple floors and as a structure as a whole. An important distinction between a 2D and 3D model is that in the 3D model, the insertion point of members can be altered to get the correct composite action needed for an accurate model. The default insertion point for elements in ETABS is at the centroid of the element, but that can be altered to various locations on the members so that objects are placed at the correct elevation relative to one another. For example, frame elements can be inserted in the 3D model with the insertion point being the top of flange and then shell elements inserted with the insertion point being bottom of shell so that they are placed one on top of the other rather than in the same plane. Another significant difference between a 2D and 3D model is that in the 3D model, both in and out of plane forces can be considered for the area elements versus just out of plane forces in the 2D model (Kreidel, 2014). When accounting for the thickness of shell (area) elements that are to be modeled, they should be taken as the thickness of the topping slab and not included the thickness of concrete in the ribs of the decking if present (Sladki, 1999).

2.6.3 ETABS Calculation of Natural Frequency

ETABS performs a modal analysis of a structure to get the dynamic properties, such as the natural frequency of a structure and modal mass. This process is also referred to as the mode-superposition method and is a linear dynamic-response procedure (CSI Knowledge Base, 2021). The procedure evaluates and superimposes free-vibration mode shapes to characterize displacement patterns that occur throughout a structure and its elements. The mode shapes are normalized and show the displacement pattern of the structure based on the number of degrees of freedom the structure requested. Figure 4 shows how the total displacement, v, is based on the mode-shape matrix, ϕ, and the coordinate vector, Y (CSI Knowledge Base, 2021).

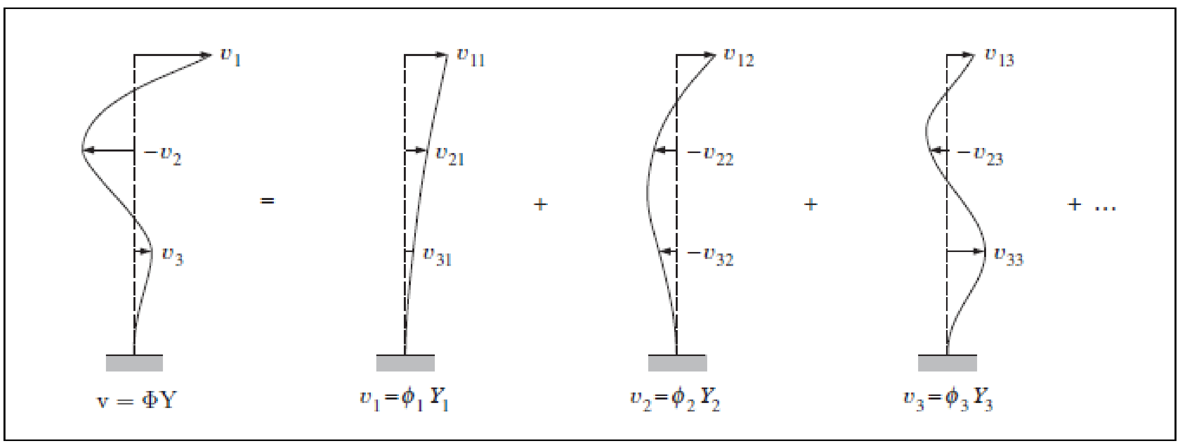

Figure 4: Resultant displacement and modal components (CSI Knowledge Base)

The following set of equations come from ETABS' parent company's website where they lay out the process and equations that the program goes through to get the values for the model analysis that end up giving the natural frequency of the structure. The mode shapes, ϕ_n, which comes from the N x N mode-shape matrix, and their frequencies, w_n, eigen vectors and eigenvalues are obtained through Equation 2.6.3.1. It should be noted that the modal damping ratios, ξ_n, are typically assumed and come from empirical data.

$$[k - \omega^2 m]\hat{v} = 0 \qquad \text{Eq. 2.6.3.1}$$

The N couped equations of motion are given by Equation 2.6.3.2.

$$m\ddot{v}(t) + c\dot{v}(t) + kv(t) = p(t) \qquad \text{Eq. 2.6.3.2}$$

Equation 2.6.3.3 gives the transformation of N to uncoupled differential equations. Equations 2.6.3.4 and 2.6.3.5 accompany Equation 2.6.3.3 to show what certain terms equal. Y_n represents the modal amplitude expressed in the time domain by Duhamel's Integral which is given in Equation 2.6.3.6.

$$\ddot{Y}_n + 2\xi_n\omega_n\dot{Y}_n(t) + \omega_n{}^2Y_n(t) = \frac{P_n(t)}{M_n} \qquad \text{where n=1, 2,…, N} \qquad \text{Eq.2.6.3.3}$$

$$M_n = \phi_n^T m\phi_n \qquad \text{Eq. 2.6.3.4}$$

$$P_n(t) = \phi_n^T p(t) \qquad \text{Eq. 2.6.3.5}$$

$$Y_n(t) = \frac{1}{M_n\omega_n}\int_0^t P_n(\tau)e^{-\xi_n\omega_n(t-\tau)}\sin\omega_{Dn}(t-\tau)d\tau \qquad \text{Eq. 2.6.3.6}$$

From Equations 2.6.3.1 through 2.6.3.6, their solutions give the relationship for total displacement of the structure and its elements given through Equation 2.6.3.7.

$$v = \Phi Y \qquad \text{Eq. 2.4.3.7}$$

All of these equations and expressions are determined in the background of ETABS as it tries to solve for the dynamic response of the finite element model created. This goes to show the power that finite element programs, such as ETABS, gives designers to help aid them in the design of structure because of their ability to perform a far greater magnitude of calculations to get results of a structure.

2.6.4 ETAB Calculation of Peak Acceleration

Determining peak acceleration in ETABS is accomplished through a seismic response spectrum analysis or modal time-history analysis. The modal time-history analysis is most appropriate for structural vibrational analysis that result from human activity. The steps to set up the time history analysis are shown below and come from CSI Knowledge Base website:

1. *Define a load case for each simulated footfall position.*

2. *For each load case, apply a point load at the footfall location. It may be best to assign a unit load, then adjust magnitude when defining the scale factor.*

3. *Define a single time-history function to represent the footfall impulse. If unit loading is applied, magnitude may be set in the function definition. To consider multiple loading scenario, additional footfall functions may be defined.*

4. *Go to Define>Mass Source and uncheck "Include Lateral Mass Only".*

5. *Define a time-history analysis case using either of the following two methods:*

    - ***Modal time history based on Eigen modes**, in which modal time-history analysis proceeds according to an Eigen formulation. A sufficient number of modes should be captured for analysis.*
    - ***Modal time history based on Ritz modes**, which should be better suited for modal time-history analysis because of its condensed formulation. However, each of the footfall loads will need to be used as a starting load vector, therefore a mode will be needed for each load case (100 footfall locations will require 100 modes).*

6. *Each footfall load case must then be added to the Load Assignments section. Each load case requires an impulse function, a scale factor, and an arrival time, which defines when load is applied. Finally, ensure that the Number of Output Time Steps and the Output Time Step Size cover the duration of the time history.*

Once those steps have been followed, the model can be executed, and the dynamic results reviewed to extract the peak acceleration of the system resulting from the force function input.

CHAPTER 3.0 MOTIVATION FOR RESEARCH

The current literature and research on vibrational analysis for structures is rather limited both in scope and quantity when compared to strength analysis. There is very little guidance out there for how to deal with vibrations that are induced by human activity that cause multi-story vibrational issues. **The goal of this thesis is to add to the body of literature on predicting vibrational behavior in multistory buildings. To achieve this goal, the following research question is proposed: Can a multistory ETABS model predict the possibility of whole building vibration issues due to human induced excitation?** A case study structure will be used for the evaluation of multi-story steady-state vibrations. There are measured results of the case study building's vibrational response that will be used to compare to the results obtained from the finite element model created.

CHAPTER 4.0 OVERVIEW OF CASE STUDY

The case study building that will be investigated is a ten-story steel framed building experiencing objectionable levels of whole building vibration felt by office occupants on the 10th floor. The primary function of the building is to serve as a rehearsal space for musical theater productions. Studio spaces exist on the 3rd, 4th, 6th, 7th, and 9th floors. The 5th, 8th, and 10th floors are used primarily for office space. Extensive partitions exist on the 8th floor. The source of the vibration problems on the 10th floor seemed to be dancing excitation in the studio spaces. The tenth floor has a mode of vibration of 4 Hz and an associated damping of 1.7% estimated. From testing performed, it was found that the vibrational issues were a result of a resonance phenomenon due to rhythmic excitation in the studio spaces and therefore the addition of damping to the building could greatly help the vibrational issues. Three rounds of data collection were performed on the building over the course of roughly a year. The first round of data collected was used to assess the vibrational response of the structure. Accelerometers were set up on multiple floors in multiple locations to record the excitation performed by 8 dancers marching to the beat of a metronome. The metronome was set to varying beats per minute (bpm) to evaluate the acceleration of the floor at the data collection points. From that data, the RMS values were found, and the data was evaluated in the frequency domain to show that the floor had a resonant frequency around 4 hertz from the peak in accelerations around that frequency. Additionally, the data showed that there were in fact objectionable vibrations on the tenth floor due to activity performed on the ninth floor. The second round of data collection was focused on seeing the acceleration responses of the floors due to a group of people jumping at a set bpm that caused a resonance response in the building. In addition to looking at human induced excitation, a shaker was used to evaluate more closely the resonance response of the building. The second round of testing better illustrated how the vibrations were being transmitted throughout the structure from floor to floor. The final round of data collected was used to evaluate the effectiveness of a tuned mass damper (TMD) that was installed on the tenth floor to address the vibrational issues and help mitigate them. It was found that the TMD was able to reduce the acceleration levels due to resonant rhythmic excitation by about 65%. Appendix A includes the floor plans that were available for the case study building. No additional information about the building's structure was available outside of the information provided in Appendix A.

4.1 Main Structural Elements of the Building

The building is steel framed with concrete on metal decking as the main structural system. The main lateral system of the building is steel braced frames. There are also concrete and masonry shear walls in the building that also contribute to the lateral stiffness of the building. The foundation system is composed of column and pier footings that bear on bedrock with concrete foundation walls. Many of the floor beams and girders have shear studs attached, creating composite beams with composite metal decking.

4.2 Uses and Loading of Structure

The primary function of the building is to serve as a performing arts complex. The building includes rehearsal studios, administrative offices, and a black box theater. There are a total of 14 studios in the building, each being large open spaces devoid of any obstructing columns. In the dance studios, there are sprung floors and Marley Dance floors in various studios which absorb shock and provide a softer surface for the performance in the space. These types of floors add additional loading to the floors that needs to be considered in the analysis of the building. The dance studios are located on the fourth, sixth, seventh, and ninth floors. There is also a black box theater on the second level of the building that can hold up to 199 people. This black box theater is used for small performances and exhibitions within the building. All of the remaining spaces of the building primarily serve as office space for the building occupants and for visitors. Some photos of the spaces within the building were found and it seems like the spaces are minimally furnished, particularly in the studio spaces. Based on all of the information found and provided, it was assumed that the building was lightly loaded with the exception of having additional loading in the dance studios for the special flooring in those spaces.

CHAPTER 5.0 METHODOLOGY

This section of the thesis will go over the process followed to create the model used for analysis. The finite element modeling software used in this thesis is ETABS, a product of Computers and Structures, Inc. Selecting from software programs commonly utilized in professional practice, ETABS was chosen for the programs ability to perform multi-story analysis accurately and being able to perform time history response analysis. Figure 5 depicts a 3D isometric view of the building that was created within ETABS and shows the structure elements that were included within the analysis model. It should be noted that Sections 5.2 and 5.3 of this thesis goes through how a vibration analysis model should and was constructed with Section 5.4 discussing a sensitivity analysis.

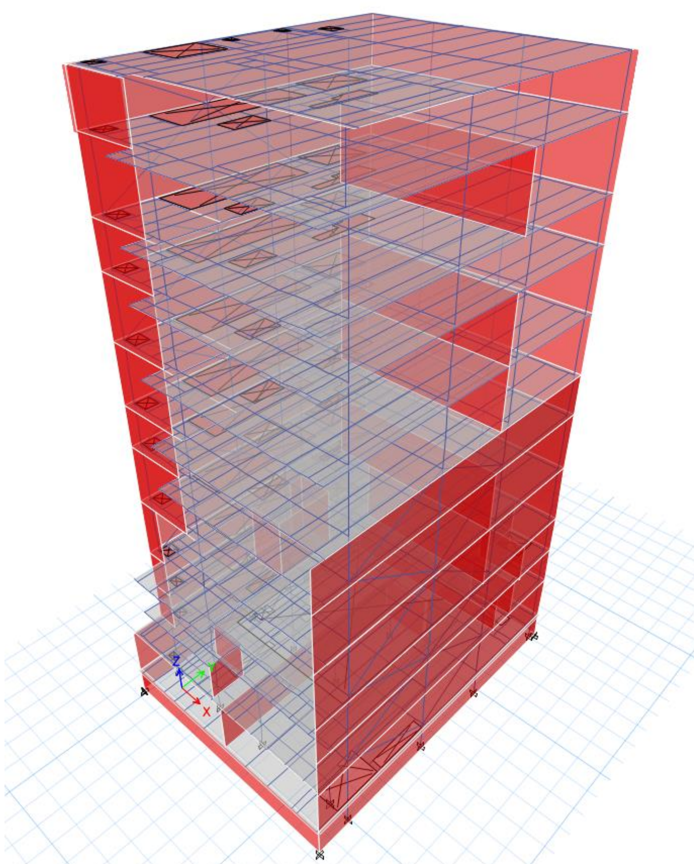

Figure 5: 3D Isometric View of ETABS Model

5.1 Case study Models

Going into constructing the final model for the case study building, there were still some lingering questions that pertained to modeling assumptions. The literature is helpful as a baseline for certain modeling assumptions that should be made but there are some gaps. For example, there is very little guidance that uses ETABS as the FEM program for analysis. While most FEM software are relatively similar, they do differ slightly in their settings and capability. This led to two simple model studies being created so that the effects of certain assumptions could be checked within ETABS by comparing the results to hand calculations.

5.1.1 Discretization of elements

The first case study model created was used to look at the discretization of frame elements. As discussed in Section 2.6.3 of this thesis, ETABS does a modal analysis by evaluating defined points within a model. To analyze the impact the number of joints within a model has on the determination of natural frequency, two beams spanning between two pinned supports connected by a slab was created in ETABS. From a modal analysis of the model, it was found that beams modeled as one frame element between the two points resulted in a natural frequency of the system that was unexpected based on the value obtained through a simple hand calculation. This was due to the beams within the model and their associated panel modes not being considered in the modal analysis. The model was then re-executed with the beams being modeled in as two frame elements on either side of the slab, thus creating a joint at the midspan of each frame element. This created a joint at midspan of the beams in the model, where the modal amplitudes were the largest and the mode shape was expected to be at a maximum. When this was done, the modal analysis produced a result for the natural frequency of the model that matched the value predicted through hand calculations. The model was them re-executed a few more times, dividing the frame elements into smaller sections to create more joints along the beam. When doing this, there was minimal impact on the modal properties of the system leading there to be a negligible change in the natural frequency of the system.

Through this simple study, it was found that for ETABS to correctly calculate the natural frequency of a system, it needs to have defined points at locations where the maximum modal amplitudes of the elements are expected to occur. Without joints along the clear spans of frame elements, their panel modes are not considered in the determination of the natural frequency of the system. Given the relationship between deflection and natural frequency that can be seen in Equation 2.4.1.5, if the max deflection of certain elements is not being captured in the modal analysis of the system by ETABS, it will produce an inaccurate value for the natural frequency of the system. Adding additional joints along the length of the frame elements is not needed for accurate modal responses as long as there is a joint at the midspan of each frame elements' clear spans. Discretizing frame elements so that there is a joint at mid-span captures the fundamental mode shape of the member. With this insight, it was known that the final model created for the case study building must have a joint defined at midspan of every frame element in the building.

5.1.2 Concrete's Modulus of Elasticity

The second case study created was used to investigate how ETABS does or does not account for concrete's increased stiffness under dynamic loads compared to static loads. From the guidance of the literature, the elasticity of concrete should be increased by 1.35 times the original value to account for the fact that concrete behaves stiffer under dynamic behavior. It was not clear from the ETABS documentation whether the increase would be automatic in a dynamic analysis. To analyze if ETABS automatically accounts for this dynamic characteristic of concrete, a model was created of a simply supported concrete beam. The beam was defined as 12" wide by 24" deep and spanned 10'. The beam was modeled twice with one version having the default modulus of elasticity for f'c=3 ksi concrete and the second having the modulus of elasticity being increased by 1.35 that original value. The two models produced a natural frequency of the beam of 30.85 Hz and 35.85 Hz for the default 'E' and increased 'E' respectively. When the natural frequency of the beam was calculated by hand with an increased modulus of elasticity, it was found that the beam should have a natural frequency of 29.39 Hz. From these results, it is clear that ETABS does in fact account for the increased stiffness of concrete under dynamic loads. Therefore, the modulus of elasticity of concrete in ETABS for vibration analysis models should not be increased. This is very important to note when modeling in ETABS because all other literature does manually increase the modulus of elasticity of concrete. However, those sources used other FEM software, particularly SAP, and they were also older versions of the program. While those programs do not automatically account for this stiffening increase, ETABS version 2019 does. If a vibrational analysis model is being created within ETABS that is a newer version of the program, the modulus of elasticity of concrete should be kept at the default value.

5.2 Modeling Process

The approach to the creation of the analytical model starting with reviewing the literature to see what modeling considerations needed to be implemented to get accurate results for vibrational analysis along with the case study models discussed in Section 5.1 of this thesis. Multiple sources were used to see how previous researchers and practicing engineers approached the creation of a FEM suitable for vibration analysis and how different assumptions made in the modeling process effected the results. Given the difference in magnitude of the forces being evaluated in vibrational analysis models when compared to strength analysis models, the connectivity and composite action of members plays a large role in the dynamic behavior of a structure. The following sections will go over the steps taken to create the ETABS model for the case study building studied in this thesis. A 3D approach was used in the modeling of the building to create the composite action of the floor elements; refer to Section 2.6.2 for more details on 3D FEM modeling of floor beams and girders. It should be noted that Sections 5.2.1 through 5.2.7 covers the parameters for the final ETABS model. The following sections are presented in the order of which the model should be created.

5.2.1 Grid and Story Setup

The creation of the model started with the development of the grid systems so that all of the structural elements could be drawn in and placed in the correct locations. From the provided construction documents, a column schedule was included and was used for reference in defining the story elevations. In conjunction, the drawing sheets for each floor was used to slightly adjust the story elevations so that the grids were at the elevation of top of steel at every floor. The top of steel was used as reference for story elevations because of the 3D modeling approach that was used in this thesis. Appendix B includes an image of the stories defined in the case study ETABS model.

When it came to developing a grid system for modeling floor elements, multiple grids had to be developed. The location of beams and girders was inconsistent from floor to floor, so with the use of the AutoCAD files of the building provided, typical grid systems were developed.

The AutoCAD files were edited to remove information not relevant to the model developed for this thesis and the files were used to identify the location of floor elements. Eight different grid systems were developed to capture all of the different geometry present in the case study building. Appendix B includes an image of the grids created and defined in the case study ETABS model.

5.2.2 Material Definition

There are three main structural materials present within the building, concrete, steel, and masonry. Given the limited information available on the case study building, the compressive strength of the concrete used in the building had to be estimated and was assumed to be 3,000 psi. This is consistent with minimum f'c specified by the Steel Deck Institute (ANSI-SDI-C-2017). A sensitivity analysis on the compressive strength of concrete is included in Section 5.4.3 of this thesis. The next material property that was defined was the masonry block present within the structure. Again, given the limited information present on the structure, assumptions had to be made on what type of properties the masonry had in the case study building. Masonry with a compressive strength of 2,000 psi was used. The last material that was defined for the case study structure was steel. Given that wide flange beams were used in the structure, A992 grade 50 steel was used to define the material of the steel members. Images of the various material properties is included in Appendix C.

5.2.3 Section Definitions

The next step for the creation of the ETABS model was defining the sections of the different structural elements present in the case study building. Preloaded frame sections were used for the various steel shapes that made up the structural elements of the case study building. The information about the sections came from AISC Manual 15th edition and are preloaded into the ETABS software. All the different sections present in the building were written down so that all the different shapes could be imported at once into the model so that when it came to the construction of the model by drawing the elements in, it was more streamline of a process. The next step was defining the slab sections that were present in the model. Even though the floors were constructed out of concrete on metal decking in the case study building, flat slab sections were defined for the floor slabs. This decision comes from guidance from the literature and accounts for the fact that the concrete that is present in the ribs does not significantly contribute to the dynamic properties of the floor. Slab sections were defined to be as thick as the thickness of the concrete that is present above the ribs; meaning only the concrete topping used. The properties of an example slab section are included in Appendix D. A sensitivity analysis on the use of ribbed slab sections or flat slab sections is included in Section 5.4.1 of this thesis.

5.2.4 Drawing of Elements

With the material properties and element sections defined, the next step in the creation of the analysis model was to start drawing in the structural elements. The first step was drawing in the columns on all levels, which was done by referencing the column schedule that was provided.

The floor beams and girders were then drawn into place floor by floor. The model was created from the cellar up and only floor elements from the ground floor up were considered. The first floor modeled was the ground floor. The ETABS model was open on one screen and the AutoCAD files were open on another so that as a frame element was drawn into the ETABS model, it was marked on the AutoCAD file as being modeled by changing the color of the element. Appendix E includes images of the AutoCAD files that were used. This process ensured that all relevant elements were included into the ETABS model, and that the location of each frame element was correctly placed. To ensure the correct placement of all elements, the grid systems created in ETABS were imported into the AutoCAD files so the gridline numbering could be referenced for the extent of all elements. The floor frame elements were modeled as continuous so that moment from one element was transferred to another, as recommended in the literature. Additionally, once the floor frame elements were drawn into the model, they were divided. This was done by going to the *edit* tab, selecting *edit frames*, *divide frames*, and then selecting from the three options which one to apply. The three options available when performing this command can be seen in Figure 6.

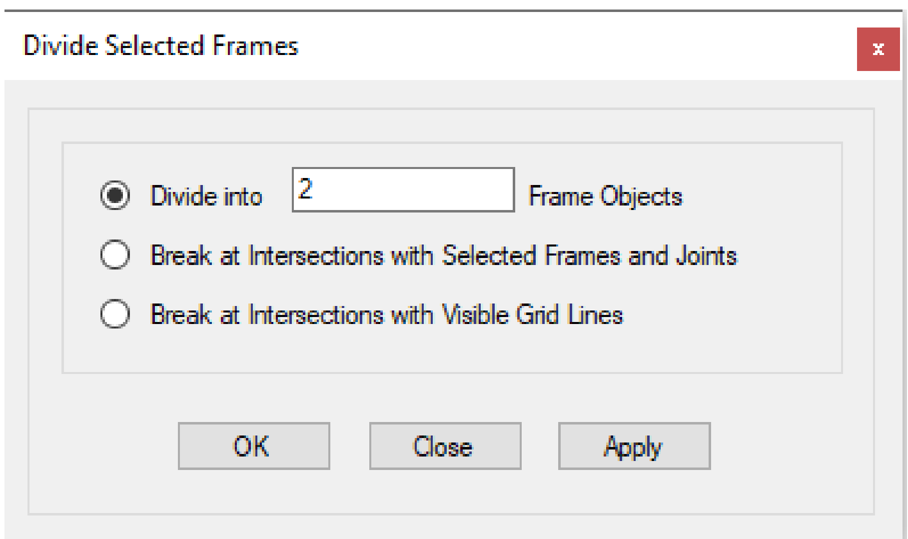

Figure 6: Divide Frames Dialog Box

All floor frame elements were selected on the all of the floors and then using the *break at intersection with selected frames and joints* option was chosen so that elements were broken apart to only span from one element to the next. Then, the *divide into 'n' Frame objects* command was selected so that frame elements were split into two frames, thus creating a point at mid span of every element. By completing these two steps in this order, it ensured that there was a joint created at the mid-span of every frame element. This was done because ETABS performs a modal analysis by looking at joints within the model to calculate the dynamic properties of the model. Without the discretization of the frame elements, the panel modes of the beams and girders cannot be captured as discussed in Section 5.1.1 of this thesis. Once an entire floor was modeled with all of the frame elements being drawn in, the model was executed to ensure that the elements were correctly connected to one another and that no errors occurred. This process was repeated for all of the floors until the final floor, the roof level, was modeled in ETABS.

With the floor frame elements drawn, along with the columns, the lateral bracing present in the building was modeled into ETABS. Again, the AutoCAD files were used as reference for drawing in the lateral elements. Given the limited information provided on the building, assumptions had to be made as to the direction of the diagonal bracing. Once the lateral bracing was modeled into ETABS, all modeling of frame elements into the ETABS model was complete.

The next step was to model the floor slabs. Using the shell sections previously defined, the floors were modeled into ETABS, starting on the ground level. Given that multiple floors had varying slab sections present in different areas of the floor, caution had to be taken to ensure that the correct sections were used in the correct areas. The first slab modeled in was the ground floor and then the floors above were modeled; going floor by floor until all floor slabs were modeled in place. The final elements that had to be included in the ETABS model were the concrete and masonry walls that are present in the building. The same process as with the frame elements and floor shells were followed to create the vertical shell elements.

Once all elements were drawn into the model, the model was re-executed. This was done to ensure that all elements were properly connected to one another. This step is critical to ensure that members are properly connected to one another and no abnormalities exist within the model. Looking at the deflected shape of the building under dead loading and also from the modal analysis assists in this process to see how members are reacting to one another. Appendix F shows elements and their locations on the 9th and 10th floor present in the final ETABS model.

5.2.5 Element Assignments

The next step in the modeling process was adjusting the assignments to the various structural elements. The first step was selecting all of the floor frame elements by using the *select* tool in ETABS and choosing *beams*. Then, going to the *assign* tab, selecting *frames,* and then *insertion point* so that the location of all floor frames could be adjusted. The default insertion point of frame elements is at the centroid of the element, which leads to inaccurate locations of frame elements in reference to one another in terms of how the elements are actually located in the built structure. The insertion point of all of the beams was changed to top center of element so that the top flange of all of the beams lined up with one another. Figure 7 shows the insertion point window, and it is important to note that when adjusting the insertion point of frames, the box at the bottom for transforming frame stiffness for offsets was unchecked. By adjusting the insertion point, composite behavior between the slab and frame elements can be achieved. A sensitivity analysis of the effect of leaving this bottom box either checked or unchecked is included in Section 5.4.6 of this thesis.

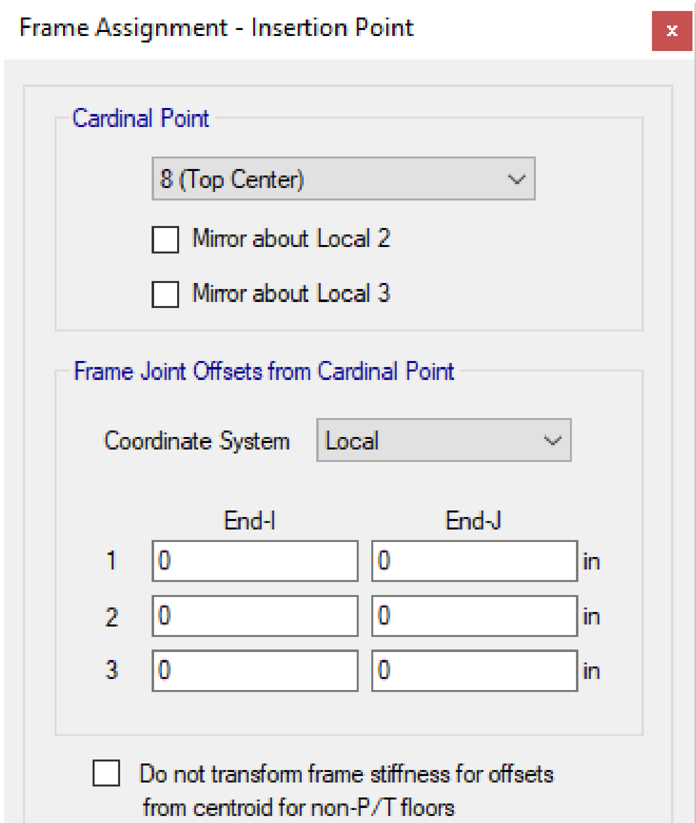

Figure 7: Insertion Point of Frame Elements

The floor frame elements were selected again using the same selection process so that property modifiers could be added. The shear area for all of the floor frame elements was set to zero, following the guidance of the literature (Kreidel, 2014). For spandrel floor frame elements, property modifiers were also used to increase the moment of inertia of those members to 100 times their original values. This was done to account for the exterior cladding that restricts the displacement of these elements.

All floor slabs were then selected using the *select* tool again so that the insertion point of the slabs could be adjusted. Similar to how frame elements default to being inserted at the centroid of the object, shell elements are the same. The insertion point of the shell elements was adjusted so that all floor slab elements were inserted into the model using the bottom of the elements as the reference point. When doing this, the bottom box for transforming the element stiffness was again left unchecked. All the shell elements had auto edge constraints assigned so that all elements framing into the vertical shells were joined and constrained along the walls instead of just at shell corners.

The next step in the construction of the analysis model was to assign the base restraint conditions. Elements that were at the bottom of the cellar level were restrained to act as fixed supports; this included the exterior concrete walls and columns in the building.

5.2.6 Mass Source

When it comes to vibrational analysis, the mass of the structure is critical in the determination of the dynamic response of the structure. To ensure that all mass was included in the ETABS model that contributed to the dynamic properties of the structure, additional mass was applied to the floor slabs. An additional mass of 0.435 lb-s^2/ft^3 and 0.311 lb-s^2/ft^3 was applied to all floor slabs on the office floors and dance floors respectively to account for additional live and dead loading on the structure for the dynamic analysis. Table 3 shows how these values were achieved through engineering judgment and guidance from Design Guide 11 ((Murray, T.M., et al. 2016).

Table 3: Additional Loading for the Floors

Office Spaces/ Other		Dance Studios	
Weight (psf)	Category	Weight (psf)	Category
2	Decking	2	Decking
4	MEP	4	MEP
8	Office	4	Flooring
14	Total	10	Total
Additional Mass		Additional Mass	
0.4347826 lb-s2/ft3		0.3105590 lb-s2/ft3	

The additional mass that was added came from the various materials that were not directly included in the model, but for which their masses were important to include for the dynamic behavior of the structure. The decking category accounts for the steel decking of the slabs that was not included in the shell elements of the model. The MEP is to account for the various utilities that run through the building. The office loading is to account for the live loading of the floors for where there was office furniture present that added to the mass of the floor. The flooring is to account for the special raised floors that were present in the studio spaces.

To ensure that the self-weight of materials was included in the calculation of mass of the building, the *mass source* command was utilized under the *define* tab and modified. Figure 8 shows the *mass source data* dialog box in ETABS with what boxes were checked to ensure that mass was calculated from the self-weight of all members and any additional mass that was added to the model.

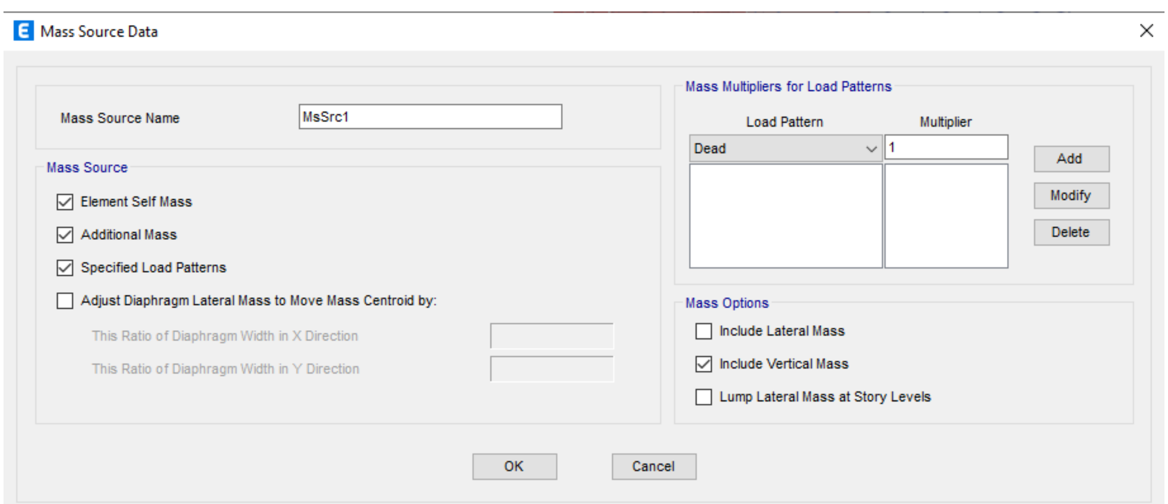

Figure 8: Mass Source Data Dialog Box Settings

Once all of the steps in Sections 5.2.1 through 5.2.6 were completed, the model was executed to evaluate the natural frequency of the 10^{th} floor. It was found that the model produced a natural frequency of 4.18 Hz. While this is slightly larger than the in-situ result for the natural frequency of the 10^{th} floor, 4 Hz, it was within reason. Many factors affect the prediction of natural frequency in ETABS so being able to get a model that produces an identical value for natural frequency to in-situ testing is unrealistic. Rather, being able to get within an acceptable range for the value of the natural frequency is what should be strived for.

5.2.7 Dynamic Loading Applied

With the natural frequency of the 10^{th} floor within the ETABS model found, the dynamic force could then be applied within the model. It is important that the natural frequency of the floor of interest is known within the ETABS model. The natural frequency needs to be known for developing a dynamic force function that will excite the floor to create a resonance response of the floor. When it came to applying a dynamic force in the model for the jumping force that was analyzed in the in-situ testing, a time history load case was used. To create a dynamic load case, first a time history function had to be created. Using Eq. 2.3.1.1, a force function was calculated and used to define the dynamic load case. The parameters of the force function were such that the weight of the group was estimated to be 980 lbs. and had a had a contact ratio (α) of ½. The calculated function can be seen in Figure 9 where the time increment of the function was chosen to be such that it causes a resonance response in the 10^{th} floor.

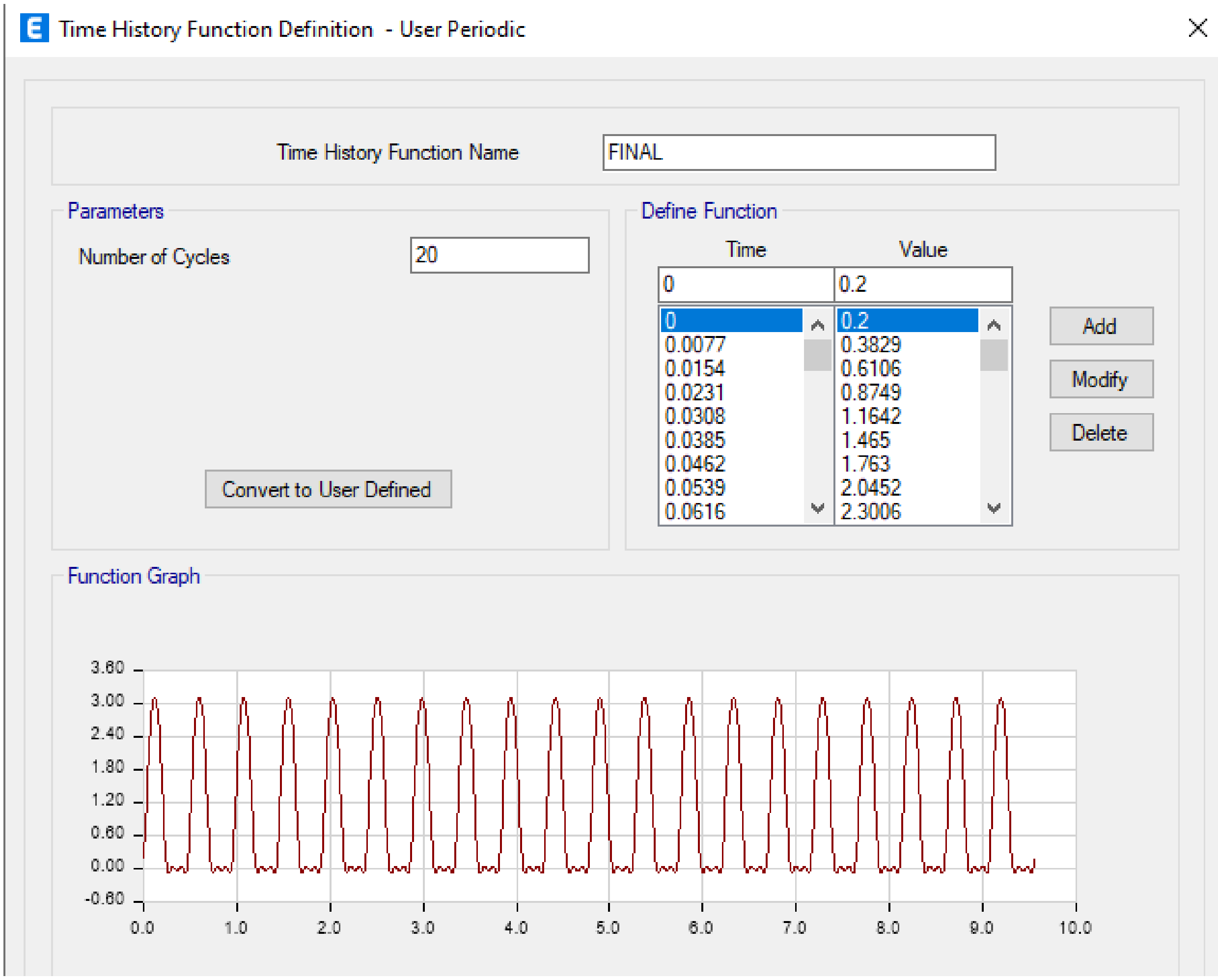

Figure 9: Dynamic Force Function of Group Jumping

Once the force function was created, a load pattern was then defined. A unique load pattern, labeled JUMPING, was created. With those two items created within the model, the next step was using them to set up a load case. A load case was created where it was a time history type where the loading applied came from the load pattern of JUMPING with the time function FINAL applied to it. The output time step size was set to be 0.0077 for the load case. This was done to capture the peak response of the floor and have the force function time window line up with the load case. This value was important in the creation for both the load case and the time function. It was obtained by taking the natural frequency of the dominant vertical mode at the 10^{th} floor, 4.18 Hz, and dividing it by 2 then taking the inverse of that. This produced a value of

.4784 seconds which is the period of the force function created. From there, that value was divided by 62 to divide the period up into time increments for analysis. The number of output time steps for the load case was set to be 1500 so that there was a large enough time window for resonance build up to be evaluated. Figure 10 shows the properties of the load case that was created.

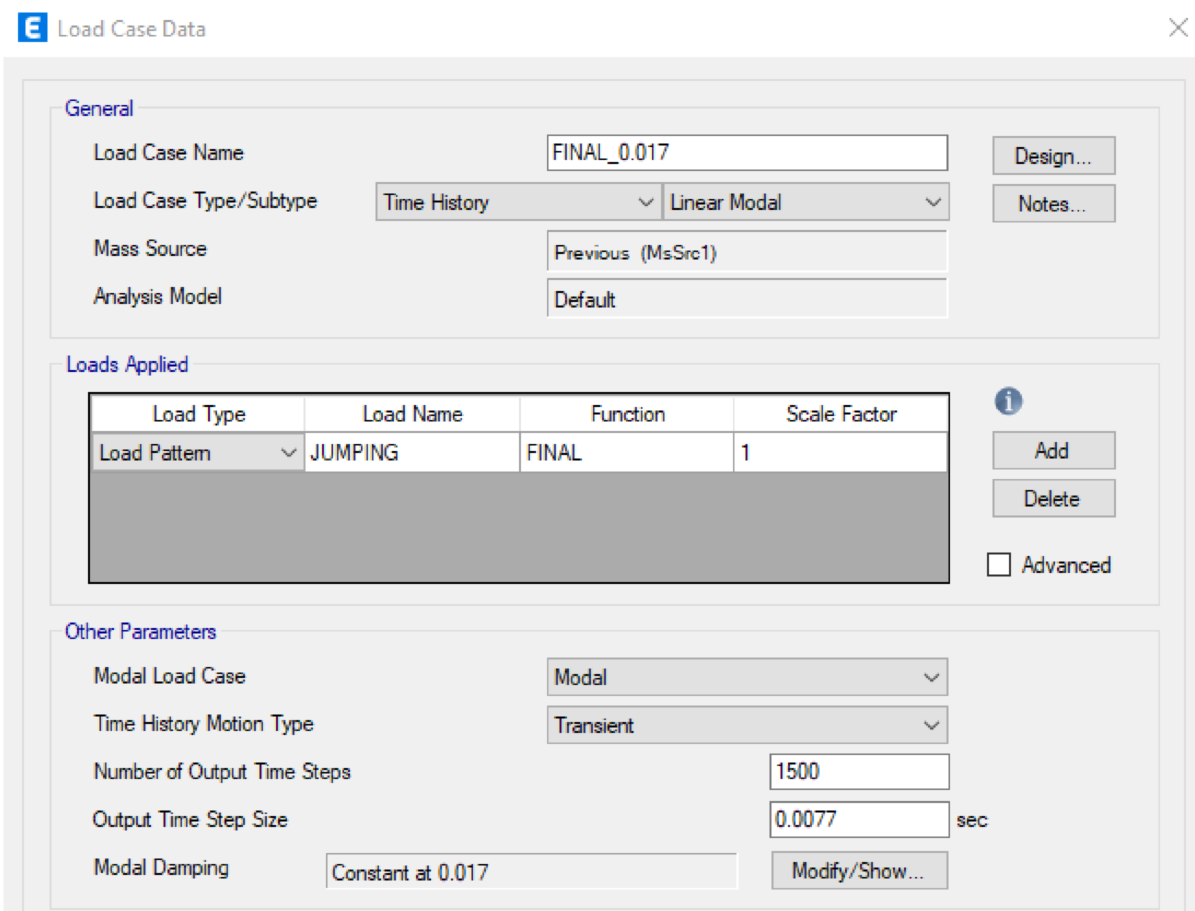

Figure 10: Dynamic Load Case Properties

With the dynamic load case fully developed, the next step was applying this force. The location on the 9th floor where the group of people were jumping was selected and assigned a unit point load. The value of the point load was adjusted to account for the estimated weight of the group. By doing this, it created a location for where the dynamic load case would be applied in the model that matched the location from the in-situ testing.

5.3 Assumptions Made in The ETABS Model

Given the limited construction document information that was available for the case study building evaluated in this thesis, multiple assumptions had to be made about the building and its structure. Some of the assumptions that had to be made delt with the material properties of structural elements while other assumptions delt with how to best model the building in ETABS to get accurate results.

Once all of the elements were drawn into the model, assumptions were made on the properties of certain elements and applied to those various elements. Auto meshing was used on all shell elements so that a properly refined mesh was established allowing for accurate results for shell element behavior once the model was executed. The meshing of floor slab elements is discussed further through a sensitivity analysis in Section 5.4.5 of this report. Cracking was applied to the floor slabs by adjusting the property modifiers m11 and m22 from 1.0 to 0.5. This was done to account for cracking of the concrete that occurs from temperature changes and shrinkage and is discussed further in Section 5.4.4 of this report. Figure 11 shows how these property modifiers were applied to the floor slabs. Additionally, Appendix D includes images of typical model elements and what assignments they have applied to them.

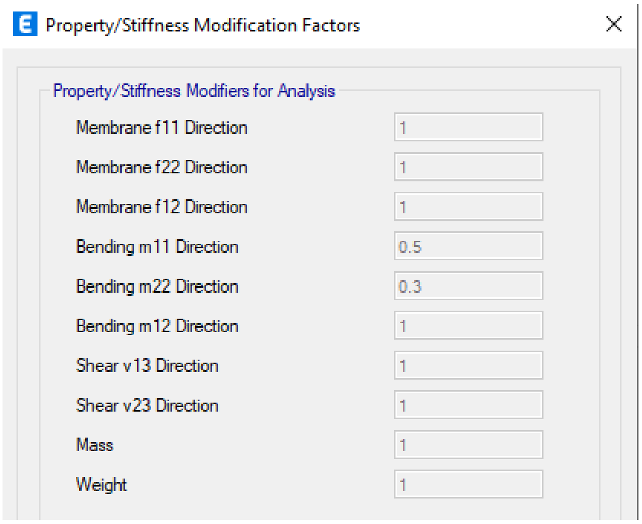

Figure 11: Property Modifiers Applied to Floor Slabs

5.4 Sensitivity Analysis for Various Input Variables

Given the complexity of the model that had to be developed for this thesis and the variability of the parameters in a modal analysis in ETABS, a sensitivity study was performed. A multitude of iterations, with slight modifications being made to certain aspects of the model, were performed to see what impact those small changes had on the behavior of the model. It was helpful making these alterations to the model piece by piece to see how certain adjustments made effected the results of the model analysis. All data given in this section of the thesis is based on the results of the final model versus the results from changing the parameter discussed in each section. The data is presented this way to give a better understanding of how a certain iteration caused the model results to deviate from the final model results. The remainder of this section will go over the various iterations made during the sensitivity analysis.

5.4.1 Floor Area Elements

In the final model created, the floor slabs were defined as flat slabs with the thickness of them being equal to the topping thickness of the concrete from the concrete on metal deck floors. Different types of shell elements for the floor slabs were evaluated to see how they effected the results of the ETABS model analysis. The slabs were modeled as deck sections and as ribbed slab sections.

5.4.1.1 Deck Sections

For this sensitivity analysis, the floor slabs were modeled as deck sections. When this was done many errors in the model analysis were apparent. The first being that composite action was not being achieved between the floor slabs and beams. This was indicated by excessive deflections of the floor slabs under self-loading. Upon further investigation, it was found that the deck sections did not have the correct stiffness along with various other properties being inaccurately represented in the model compared to recommendations in the literature and experimental observations. Figure 12 shows the deflections exhibited from the use of the deck sections under self-loading only with the max deflection on the 10th floor being -10675158”. This was the area that was deforming the most but many other areas in the model had excessive deflections that were unrealistic to what they should have been under that type of loading.

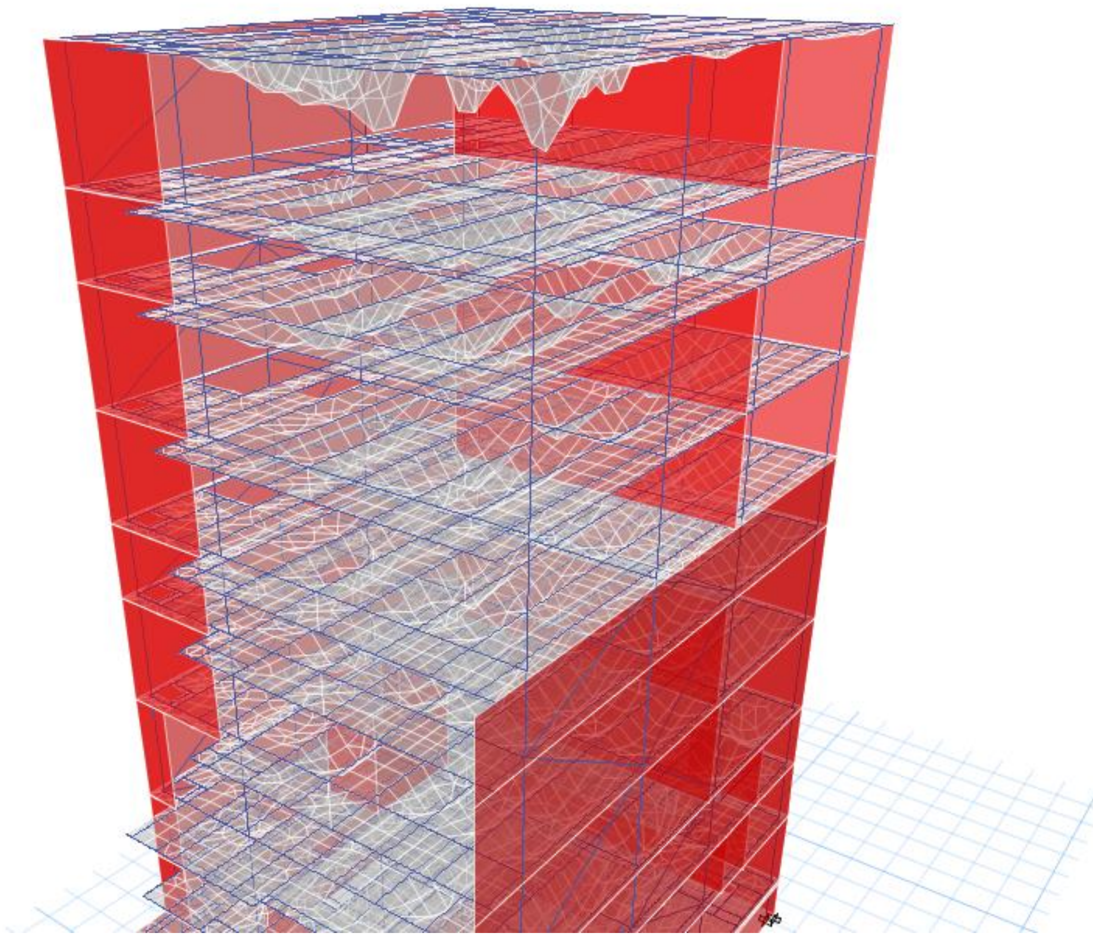

Figure 12: Deflections of the Floors Using Deck Sections

With the large magnitude of the deflections that were exhibited relative to the beams, it was clear that using deck sections to model the floor shell elements was not a viable option.

Therefore, deck sections should not be used when creating a vibrational analysis model in ETABS even if the floor slabs are concrete on metal deck in the real building.

5.4.1.2 Ribbed Slab Sections

Given that the case study building did have concrete on metal decking as the floor slabs, ribbed slab sections were investigated as an option for modeling the floor slabs within the ETABS model. The slab sections were defined as ribbed with the dimensions of the ribs coming from Vulcraft's Steel Roof & Floor Deck catalog for 2VLI composite deck. 2VLI composite deck was chosen because the floor plans provided indicated the decking had two-inch-deep ribs. Figure 13 shows an example slab section definition for the ribbed slab sections.

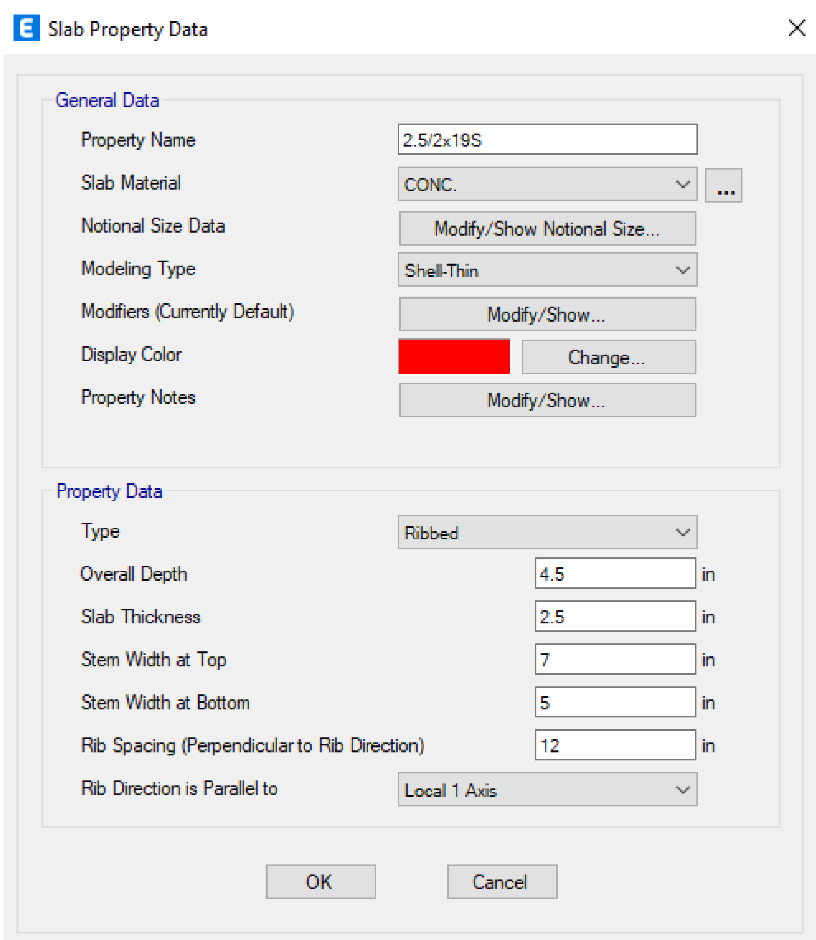

Figure 13: Ribbed Slab Section Properties

All slab sections have near identical properties to the one shown in Figure 13, with the differing factor being the overall depth and slab thickness defined for the various sections that were called out in the plans of the case study structure. With all the floor slabs being defined as ribbed slabs, the model was executed and produced a natural frequency of the 10th floor of 4.83 Hz. When the floor slabs were modeled as flat slabs, it produced a natural frequency of the 10th floor of 4.18 Hz. From these results it can be seen that by modeling the floor slabs as ribbed slab sections, it overestimates the stiffness of the floor slabs. To more accurately model floor slabs in ETABS, flat slabs should be used even if the actual floors in the building being evaluated have concrete on metal decking floors. This assumption is supported in other literature where it is recommended to model all floors slabs to be flat slabs with the thickness being set to being only the thickness of concrete that is located above ribs (Kreidel, 2014). The concrete that is located within the ribs of the decking do not provide stiffness to the composite sections that run perpendicular to the ribs.

5.4.2 Floor Diaphragms

Another sensitivity analysis included the effect of applying a rigid floor diaphragm to all levels in the model had on the analysis results. To see the effect rigid diaphragms have on a vibrational analysis in ETABS, all of the floors in the case study model were selected and had rigid diaphragms applied to them. When floors were modeled with rigid diaphragms, the 10th floor had a natural frequency of 4.51Hz. This value is higher than when no rigid diaphragms are applied. When no diaphragms are applied in the model, the natural frequency of the 10th floor was 4.18 Hz. These results indicate that by applying rigid diaphragms to the model, it creates an unrealistic stiffening effect. In conclusion, from the results from this sensitivity analysis, it is recommended that floor diaphragms not be applied in vibrational analysis models focusing on dominate vertical floor modes.

5.4.3 Concrete Compressive Strength

The compressive strength of concrete used for the floor slabs was 3 ksi in the final model. This is consistent with minimum f'c specified by the Steel Deck Institute (ANSI-SDI-C-2017). When the model was analyzed with 3 ksi concrete for the floor slabs, the natural frequency of the 10^{th} floor was to 4.18Hz. The model was then re-executed with the concrete compressive strength being altered to be 4 ksi. With an f'c= 4 ksi, it produced a natural frequency of the 10^{th} floor of 4.21 Hz. While this is a very small difference, it does have an impact on the dynamic response of the structure, particularly when looking at the acceleration of the floor under dynamic loading at a set frequency which can cause resonance buildup. Therefore, it is important to know what compressive strength concrete will be, or was, used in the building being evaluated.

5.4.4 Cracking of Floor Slabs

Given the fact that the building had been operational for a while before the in-situ testing was conducted, it is reasonable to assume that some cracking of the concrete on the floors was present at the time the data was collected. Cracking of concrete floor slabs is typical from shrinkage of the concrete during curing. Therefore, cracking was applied to the floor slabs in the final model. For flat plates or slabs, it is recommended to apply a property modifier of 0.25 to account for cracking. Since the concrete was supported by metal decking, this value was increased to account for the likelihood that the concrete on the floors exhibited less cracking than a flat slab would. For the slab sections, the properties modifiers applied are shown in Figure 11. With the use of the 0.5 for the property modifier, it assumes that the concrete is 50% cracked and therefor does not have all of the stiffness it would if it were to be uncracked. When the model was executed with the cracked concrete sections, the 10^{th} floor had a natural frequency of 4.18 Hz. The model was also analyzed with no cracking applied to any of the floor slabs. When this was done, it produced a natural frequency of the 10^{th} floor of 4.22 Hz. With no cracking applied to the floor slabs, the natural frequency was slightly higher than if cracking was applied. Further research is needed into whether or not to apply cracking within a FEM analysis for vibrations and to what extent the floor slabs should be considered cracked. However, for the purpose of evaluating whether or not a floor while have problematic vibrations, it is more conservative to assume that the floor slabs are cracked in the FEM analysis of the building.

5.4.5 Meshing of Floor Slabs

Another sensitivity analysis performed was adjusting how the floor shell elements were meshed. The final model constructed had default auto meshing applied to all floor slabs. When this was done, it produced a floor meshing that can be seen in Figure 14.

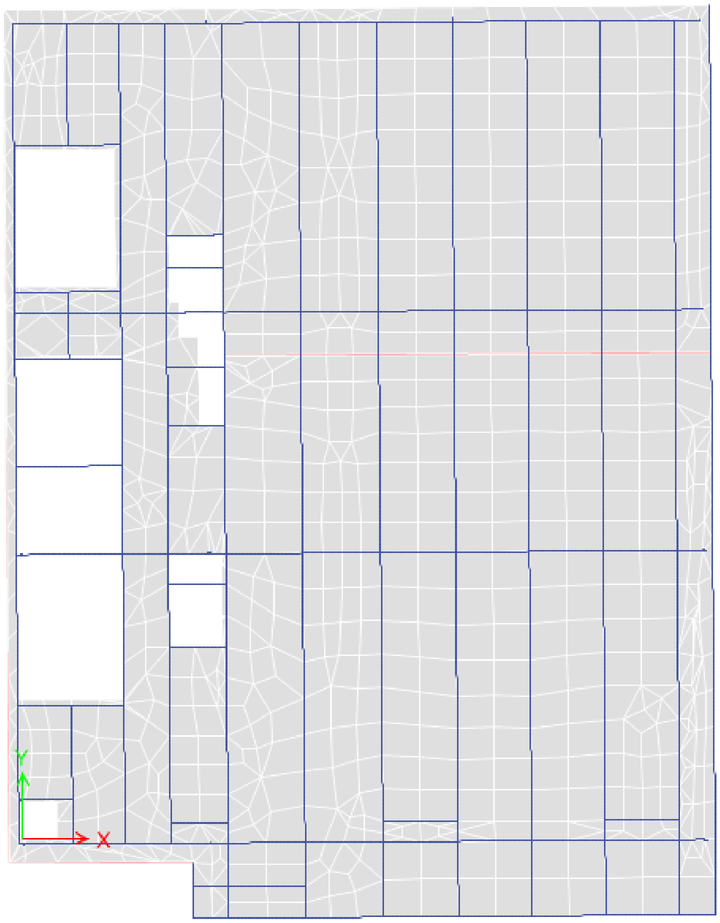

Figure 14: Messing of 10th Floor Slab Shell Element

When the model was executed with this type of meshing, it resulted in a natural frequency of the 10th floor being 4.18 Hz. To refine the floor meshing, instead of using default floor meshing, auto cookie cut floor meshing was set up with max element size set to 12”. This was applied to all floor shell elements on both the 9th and 10th floor. When the model was re-executed with this meshing, it led to a natural frequency of the 10th floor being 4.19 Hz. It should be noted that the max element size of 48” and 24” was also tried. A maximum mesh size of 48” proved to be too large and when ran at 24”, it provided a nearly identical natural frequency to the 12” maximum mesh size. With using auto cookie cut floor meshing, it increased the natural frequency of the floor. While this is a small difference, it does show that how meshing is considered in the ETABS model is important when it comes to modal analysis. Based on the results for this case study structure, it is recommended to use auto meshing of floor shell elements for vibrational analysis in ETABS. The default auto meshing option in ETABS provided a mesh size that is refined enough to properly analyze the modal properties of floors for vibrational analysis.

5.4.6 Transformed Moment of Inertia While Adjusting Floor Element Insertion Points

In the final model created, the insertion point of frame elements was adjusted to be *8 (Top Center)* with a box at the bottom of the dialog box unchecked that stated, *Do not transform frame stiffness for offsets from centroid of non-P/T floors*, as seen in Figure 7. When the box was unchecked for transforming the moment of inertia, it resulted in a natural frequency of the 10^{th} floor of 4.18 Hz. From looking at CSI Knowledge Base, when this box is left unchecked the stiffness of the member is transformed to account for the offset of the element from the line where it was drawn into the model. To see what effect checking this box has on the model properties, the model was re-executed with the box checked. With the box checked, it produced a natural frequency of the 10^{th} floor of 3.45 Hz. This natural frequency is lower than the value that was obtained from the when the box is unchecked for transforming the moment of inertia, indicating non-composite behavior of the frame elements. To accurately model composite behavior, all floor frame elements in the model should have their insertion point assigned to be at the top center, with the bottom box unchecked for not transforming the moment of inertia. This also applies to floor slabs with the insertion point being defined as bottom with the box at the bottom of the dialog box left unchecked. Given that when having the box unchecked for not transforming frame stiffness produced a modal result much closer to the results of the in-situ testing, it was concluded that for ETABS models used for vibrational analysis that this box at the bottom of the *Frame Assignment – Insertion Point* dialog box should be unchecked.

CHAPTER 6.0 COMPARISON OF RESULTS WITH IN-SITU TESTING

This section of the thesis will go over the time domain analysis results obtained from the case study building that was constructed in ETABS. As noted previously, the final version of the model created produced a natural frequency of the 10th floor of 4.18 Hz which is close to the natural frequency of the actual structure which was calculated to be 4.0 Hz from the in-situ testing. Images of the various mode shapes from the modal analysis of the final ETABS model are provided in Appendix G.

6.1 Dynamic Force Function

The dynamic force function that is discussed in Section 5.2.7 came from equation 2.3.1.1. The values for ϕ_n and r_n used in equation 2.3.1.1 came from Table 1 using a contact ratio (α) of ½ are those variables are shown in Table 4. Additionally, G was defined as 150 lbs., the estimated weight of a measured participant jumping on the force plate.

$$F(t) = G\left(1 + \sum_{n=1}^{\infty} r_n \sin(\frac{2n\pi}{T_p} t + \phi_n)\right) \qquad \text{Eq. 2.3.1.1}$$

Table 4: Coefficient Values Used in Equation 2.3.1.1

α = 1/2	r_n	1.57080	0.66667	0.00000	0.13333	0.00000	0.05714
	ϕ_n	0	-π/2	0	-π/2	0	-π/2

This equation plots the dynamic force induced from rhythmic activity. To compare the results of the ETABS model with the in-situ results, it is important that the dynamic force for both is comparable. Data was available on the dynamic force of a single person from the group jumping. Figure 15 shows the force plate data from a single male jumping.

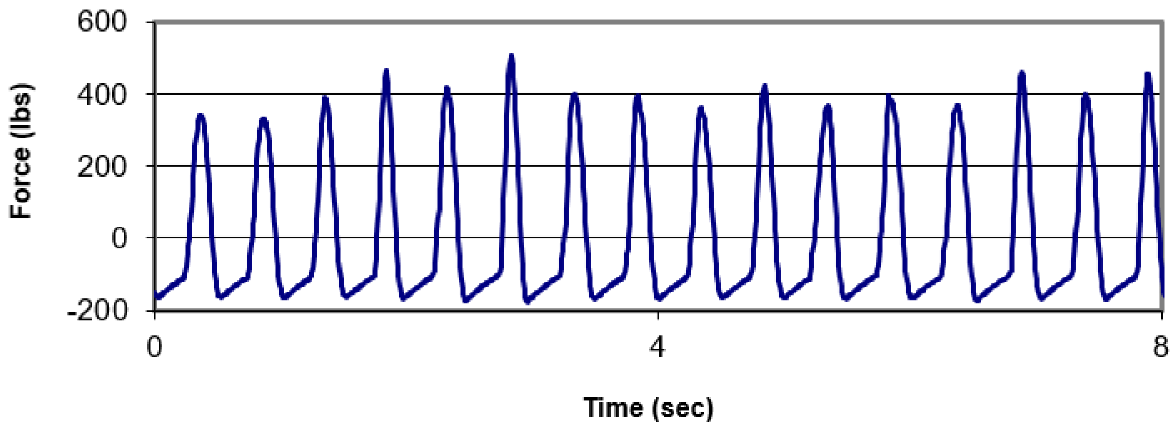

Figure 15: In-Situ Data of the Force from a Single Male Jumping

From Figure 15, it was concluded that the person weighed around 150 lbs. With that information, the calculated force function the 150 lb. person jumping is shown in Figure 16.

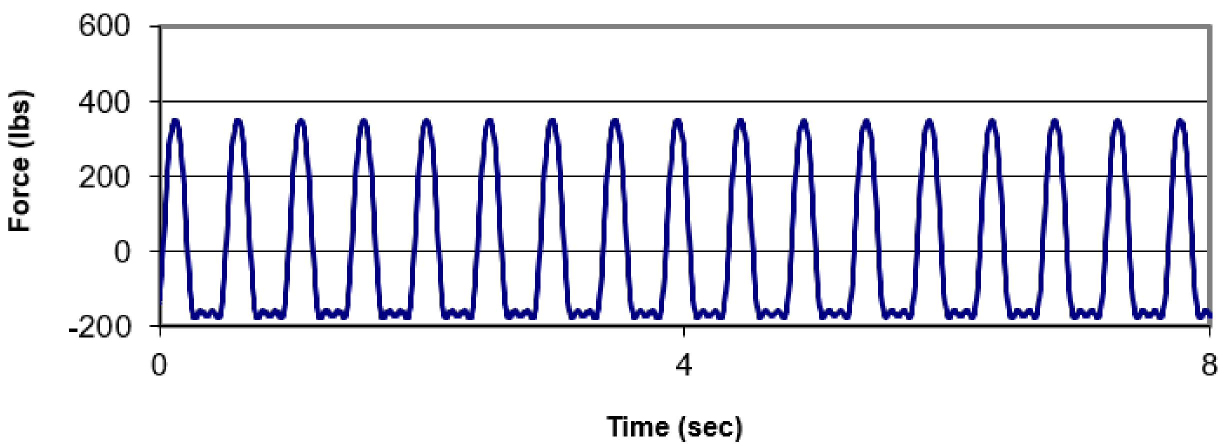

Figure 16: Calculated Force Function of 1 150 lb. person Jumping

To account for seven people jumping, the factor G in the function applied to the 9th floor was set to be 0.98 kips using the same fundamental forcing function. This was done because the estimated average weight for each person was 140 lbs., resulting in a total group weight of 980 lbs.

6.2 Dynamic Response of the Floors

Once the final model was constructed and had a dynamic force function, the dynamic response of the 9th and 10th floor could be evaluated. When looking at the dynamic response of floors in ETABS, it is done by looking at specific joint locations on various floors. Therefore, it is important to be able to identify which joint on each floor corresponds to the location of the accelerometers placed from the in-situ testing. Two joints were viewed for comparing the results of the model with the in-situ data. Figure 17 shows the 10th floor of the model with joint 88 called out alongside the case study floor plans of the 10th floor with the corresponding accelerometer (A7) for the location at joint 88.

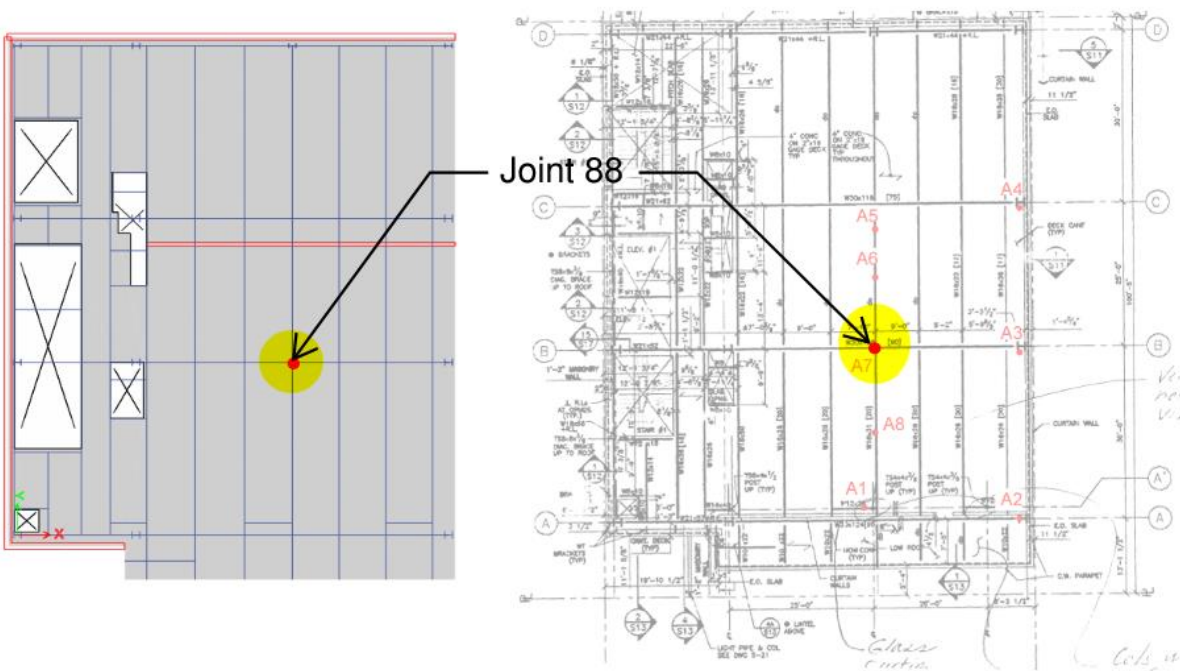

Figure 17: Location of Joint 88 on the 10th Floor and Corresponding Accelerometer Location

The second accelerometer used for comparison of results is A15 which is located on the 9th floor. This accelerometer corresponds to joint 88 on the 9th floor in the ETABS model. Figure 18 depicts the location of accelerometer A15 located on the 9th floor which also happens to be the location for which the group of seven people jumping was located.

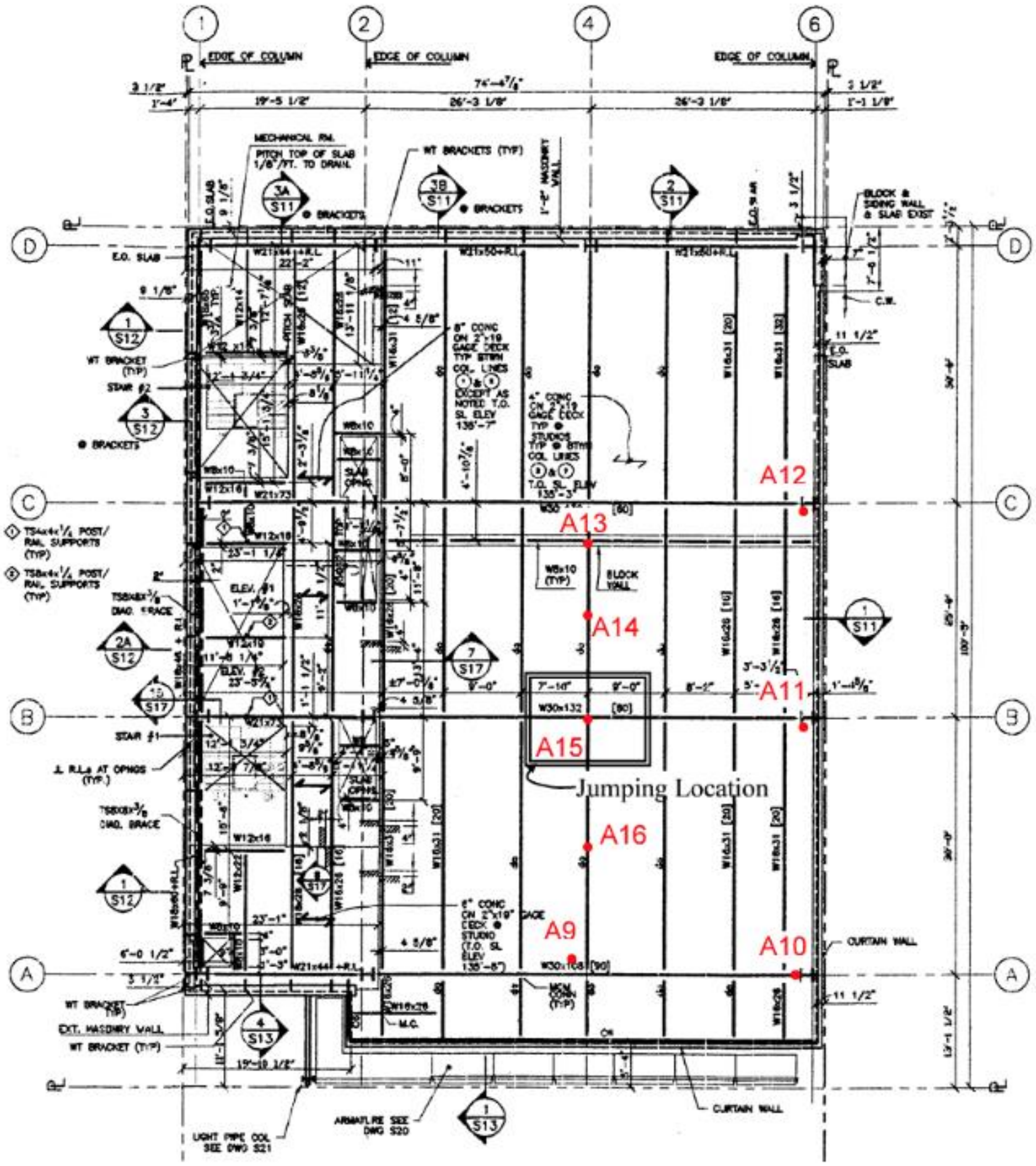

Figure 18: Location of Accelerometers on the 9th Floor from In-Situ Testing

To view the dynamic response of the floors in the ETABS model, the time history plots were viewed. This tool allows the user to view various aspects of a specific joint under certain time history cases over a period of time. For the purpose of this thesis, the vertical (Z) acceleration was of interest. These time history plots were viewed for the two different accelerometer locations mentioned before to compare the peak acceleration obtained from the model to the in-situ results. The data from this plot was exported into excel so that it could be graphed on a scale comparable to the in-situ data. Figures 19 and 20 show the comparison in results from the ETABS model and the in-situ data from joint 88 on the 10th floor.

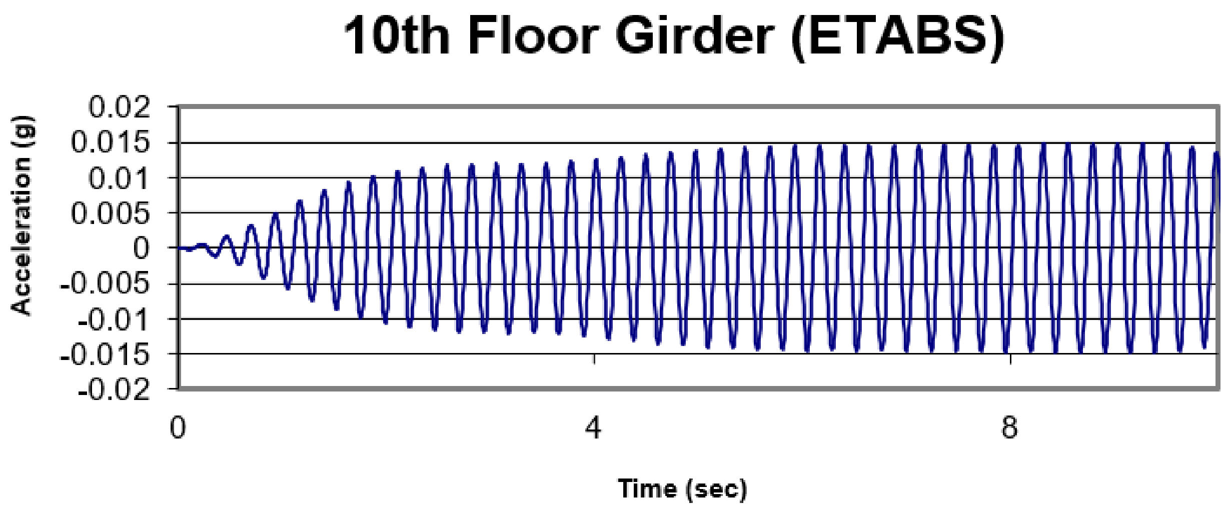

Figure 19: ETABS Values for Acceleration of the 10th Floor

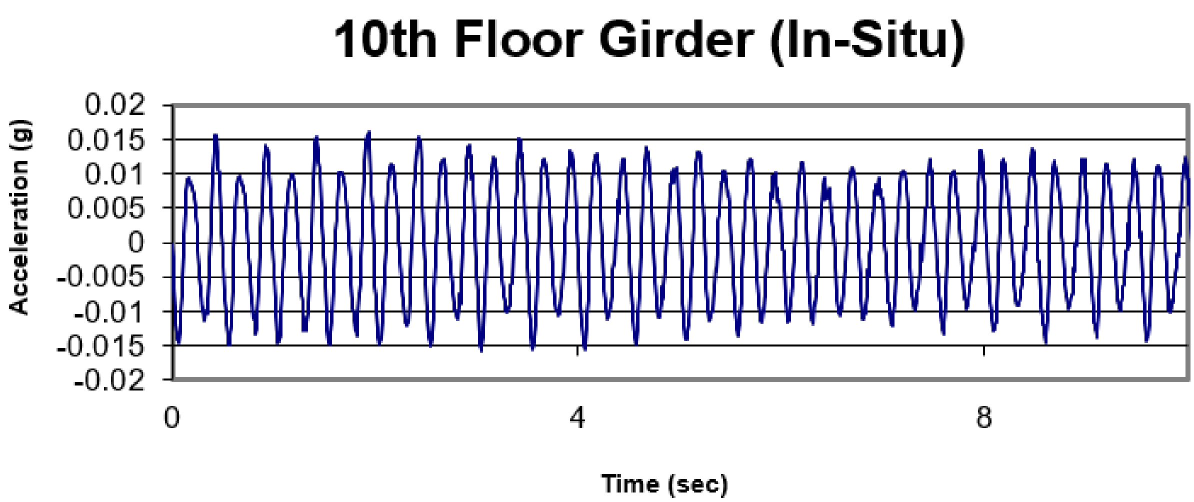

Figure 20: In-Situ Values for Acceleration of the 10th Floor

From the response plots, it can be seen that the peak acceleration at this location is around 0.0134g for the ETABS model and around 0.0142g for the in-situ data. These values are very close to one another and Figures 20 and 21 show how the two time history plots closely match in shape. The ETABS plot shows the floor building up to a resonance response in the first few seconds. This is not captured in the in-situ plot because the data started to be collected once the floor was already at resonance.

Joint 88 on the 9th floor, the location where the people were jumping, was also evaluated. This data was also then exported to excel so that it could be graphed and compared to the in-situ results. Figures 21 and 22 show the comparison in results from the ETABS model and the in-situ data from joint 88 on the 9th floor.

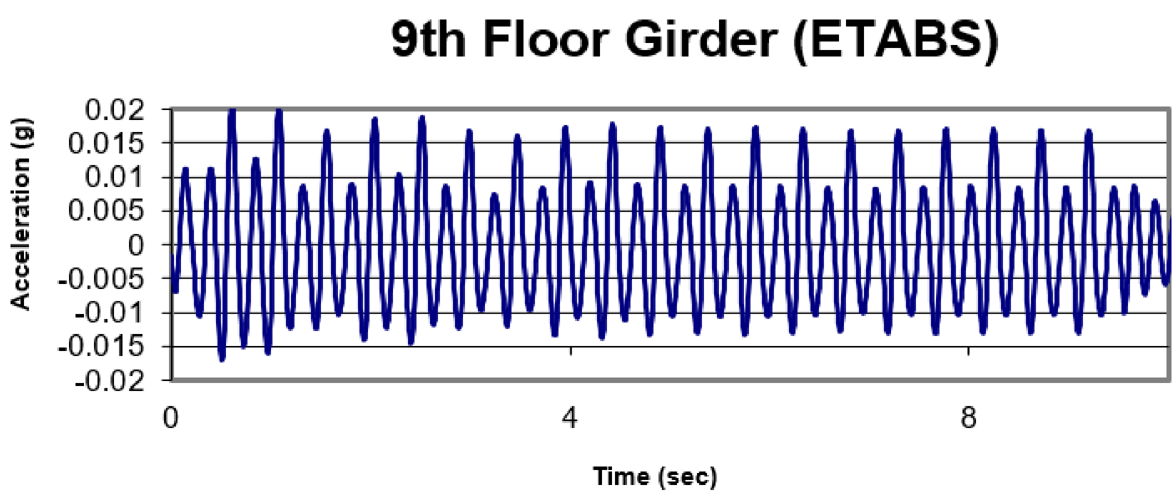

Figure 21: ETABS Values for Acceleration of the 9th Floor

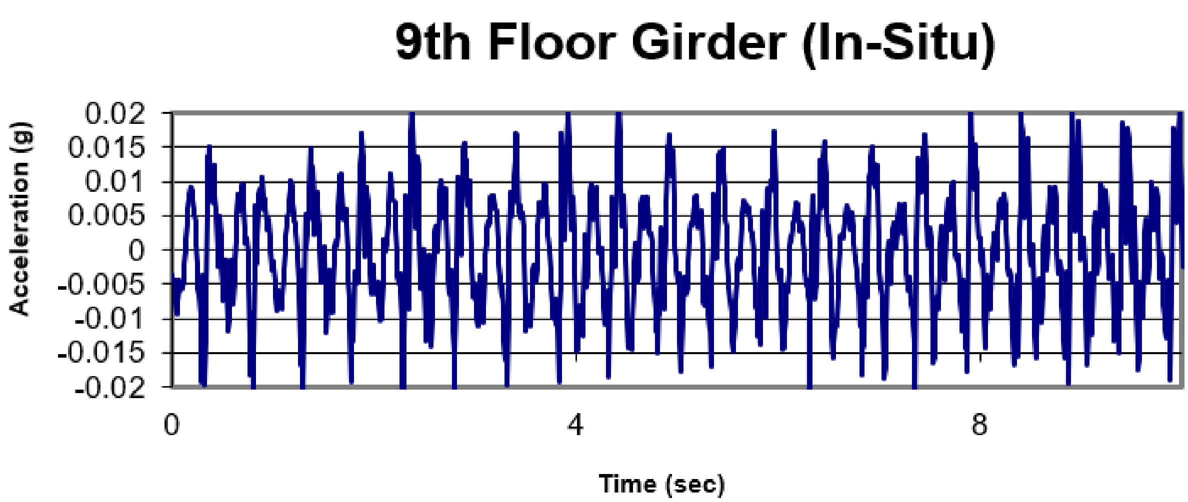

Figure 22: In-Situ Values for Acceleration of the 9th Floor

From the response plots, it can be seen that the peak acceleration at this location is around

0.0175g for the ETABS model and around 0.0164g for the in-situ data. These values are very close to one another and Figures 24 and 25 show how the two time history plots closely match in shape. Particularly, the occurrence of two distinct different amplitude peaks caused from the first two harmonics of the dynamic force.

6.3 Spectral Response

Another comparison of interest is to show the two locations in the spectral domain. Figures 23 and 24 show the response spectrum curves of the 10th floor for a damping level of 0.017 from the ETABS and in-situ results respectively. It can be seen that the 10th floor has a resonance response at 4.18 Hz from the dynamic load applied to the 9th floor. Both response spectrums show a dominant frequency at the second harmonic of jumping.

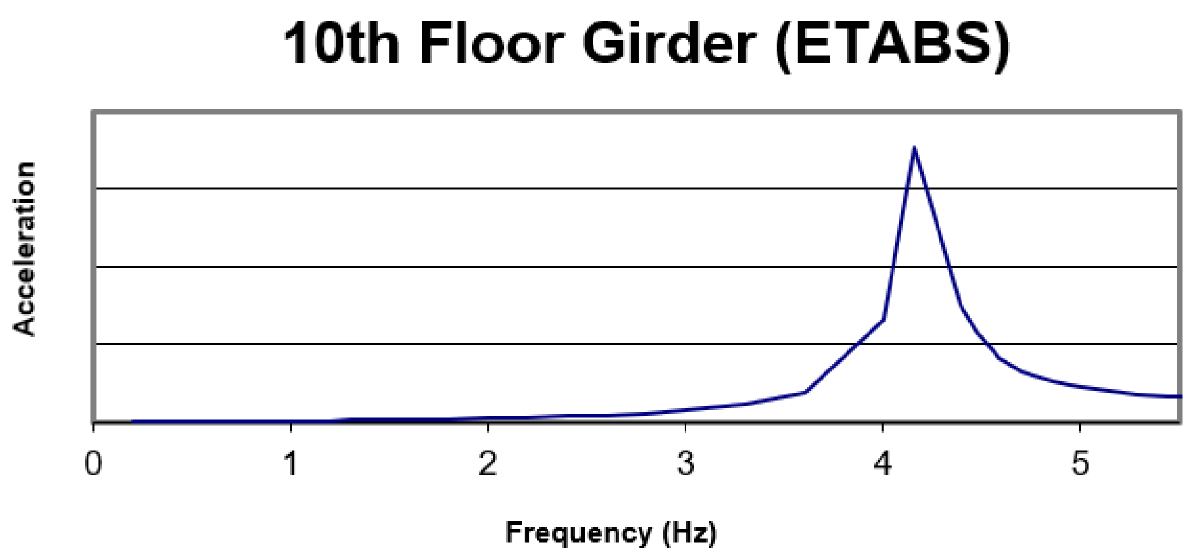

Figure 23: ETABS Response Spectrum Curve of the 10th Floor

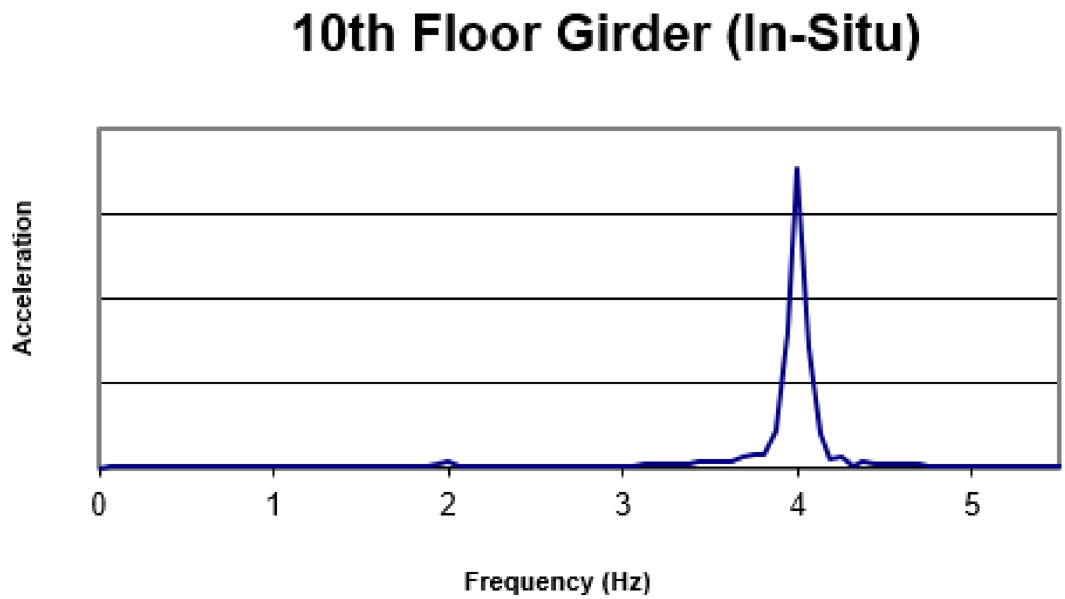

Figure 24: In-Situ Response Spectrum Curve of the 10th Floor

Figures 25 and 26 show the response spectrum curves of the 9th floor from the ETABS and in-situ results respectively. These two images show that the in-situ and ETABS response spectrum curves closely match one another in their shape with spectral content at the first and second harmonics of the forcing frequency.

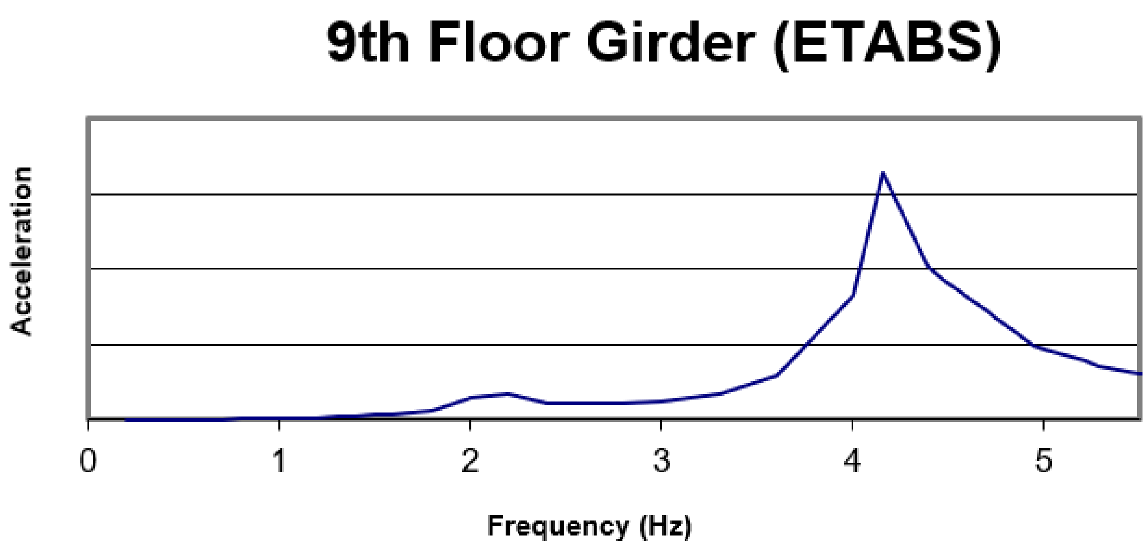

Figure 25: ETABS Response Spectrum Curve of the 9th Floor

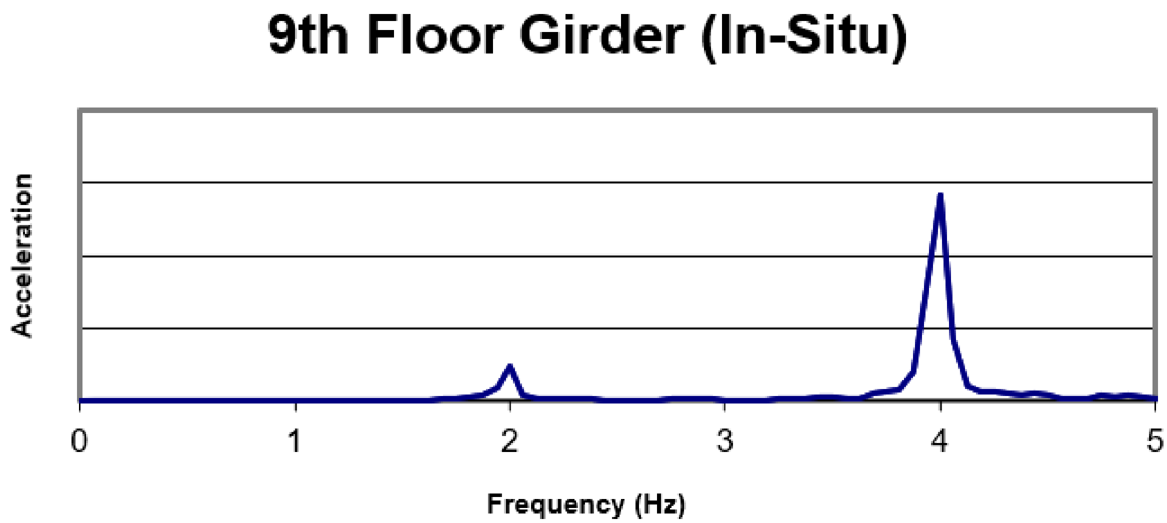

Figure 26: In-Situ Response Spectrum Curve of the 9th Floor

The response spectrum curves for all of the levels were inspected to see what the dynamic response of each floor was from the dynamic load applied to the 9th floor. These plots are included in Appendix H. These plots indicated that there were larger dynamic responses on the floors close to the 9th floor and the peak acceleration of the floors decreased as the distance between the floors increased. Going down the floors in the building, the dynamic response of the floors kept decreasing until the peak acceleration of the floor was nearly zero on the bottom levels of the building. This is what is to be expected from the model results as the axial column shortening becomes smaller as the length of the columns decreases. These results confirmed that ETABS is capable of producing a multi-level vibrational analysis.

CHAPTER 7.0 DISCUSSION

The impact of this research is that it shows designers that it is possible, with the use of modern analysis procedures and techniques, to predict multistory structures that, if actually built, would be susceptible to vibrational issues due to resonant rhythmic excitations. This is a potentially important consideration in studio buildings, such as the case study building, and also in office buildings that house gym areas where aerobics classes are conducted. Many designers do account for and analyze the dynamic response of their designs when it comes to looking at vibrational considerations, however, the current guidelines are silent on this inter-story phenomenon. The structure investigated in this thesis was a design that ultimately got built but was susceptible to vibrational issues due to rhythmic activity on one floor at resonance leading to annoying vibrations on another floor. Such a scenario is one that should be considered in the design, particularly when structures support multiple differing program spaces of different human activity.

The hope is that designers can refer to the procedure for the creation of an ETABS model and use it to create finite element models for their proposed structural designs to evaluate the vibrational response before the building is built. Alternatively, an existing structure can be evaluated before a space is remodeled to include an aerobics studio or other spaces that will include rhythmic activities like running on treadmills. Additionally, this thesis shows that there are special considerations in structures where there are conflicting program spaces in terms of occupant activities. While a designer might be able to meet all of the current design standards out there when it comes to vibrational considerations and best design practices, it does not necessarily mean the design will not one day have vibrational issues if conditions change. Things like architectural layout, number of inhabitants, and inhabitant activity in various spaces can all impact if the design will have annoying vibrations or not.

Given the complexity of structural dynamics and the relationship between damping, natural frequency, and stiffness, no source can definitely layout what to do and what not to do to have a building devoid of any vibrational issues. Rather, the structural engineer has to use their best judgment on how to avoid vibrational issues within their structure. This thesis, along with other literature on floor vibrations, should be used as a guide to see what has and has not worked in the past in terms of structural designs susceptible to vibrational issues. However, there are still unknowns on how to deal with floor vibrations and this thesis serves as an additional reference for designers.

CHAPTER 8.0 SUMMARY AND CONCLUSIONS

It has been shown in this thesis that it is possible to get a multi-story vibrational response in ETABS. When looking at the response spectrum curves provided in Appendix H, it is clear that the model is capturing the vibrational response of multiple floors produced from a dynamic load applied to the 9th floor. The model created was able to produce accelerations of floor elements on multiple levels from applying a force function on a single floor. The data showed that it is possible to get reasonable results for predicating multi-story vibrations induced by human inhabitants. Additionally, from looking at the response spectrum plots, it is clear it is possible to accurately predict the peak acceleration of floors. These plots show the dynamic response of the floors from a dynamic load applied in the model.

From the results found through the exploration of modeling assumptions, it is clear that more research is needed into how combining certain assumptions effects the result of the model results when looking at the dynamic response of the structure. Obtaining accurate dynamic response results of structures in FEM's is very difficult given the complex relationship of stiffness, mass, and damping. The three variables are interconnected and effect the natural frequency and amplitude of acceleration of floors in various ways. While this study indicates a strong correlation between model and in-situ results, more research needs to be done for other case study buildings to confirm that ETABS is effective in determining the dynamic response of floors to human induced loading in multi-story buildings.

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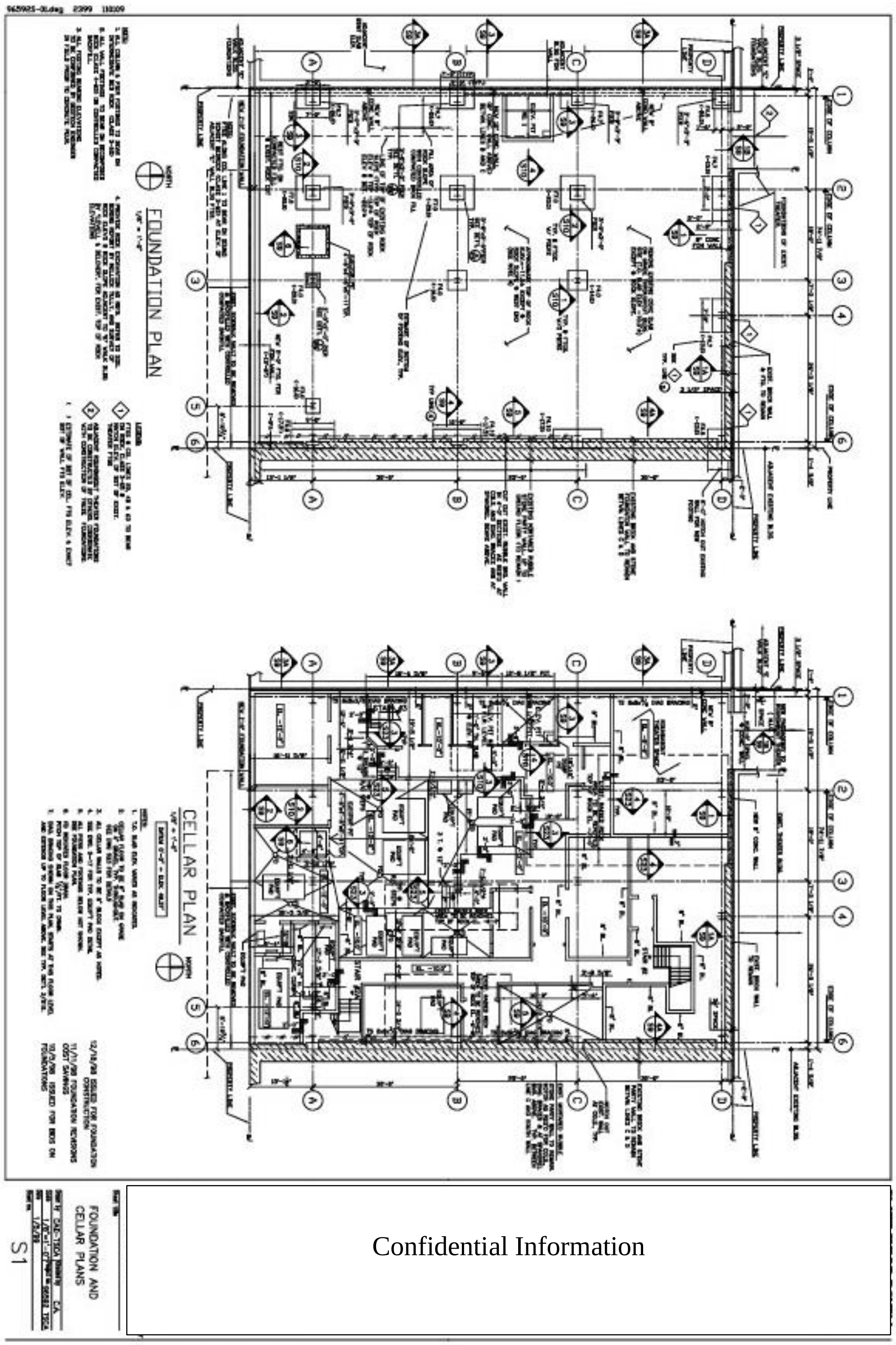
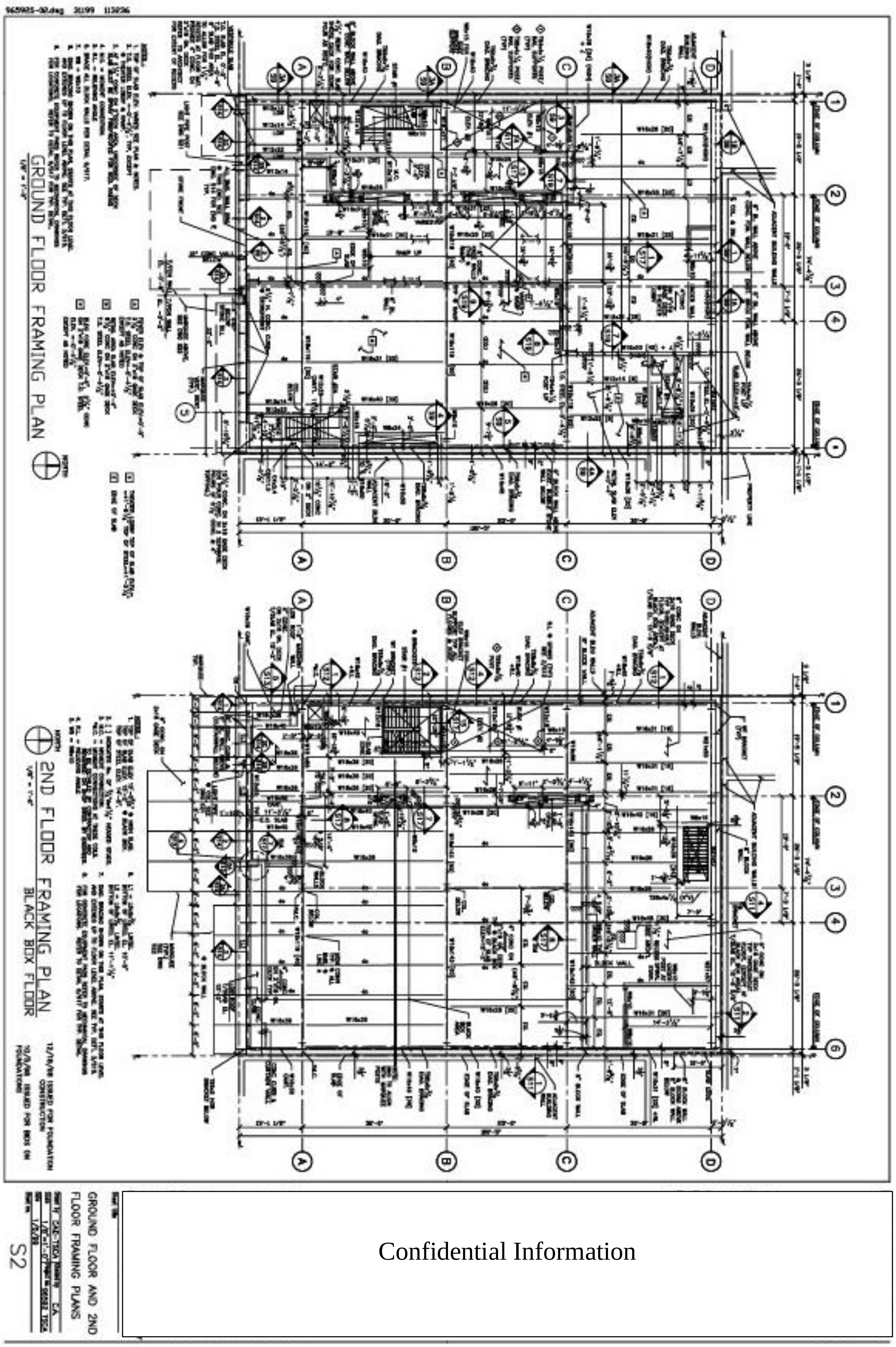
APPENDIX A - PROVIDED IMAGES OF FLOORPLANS

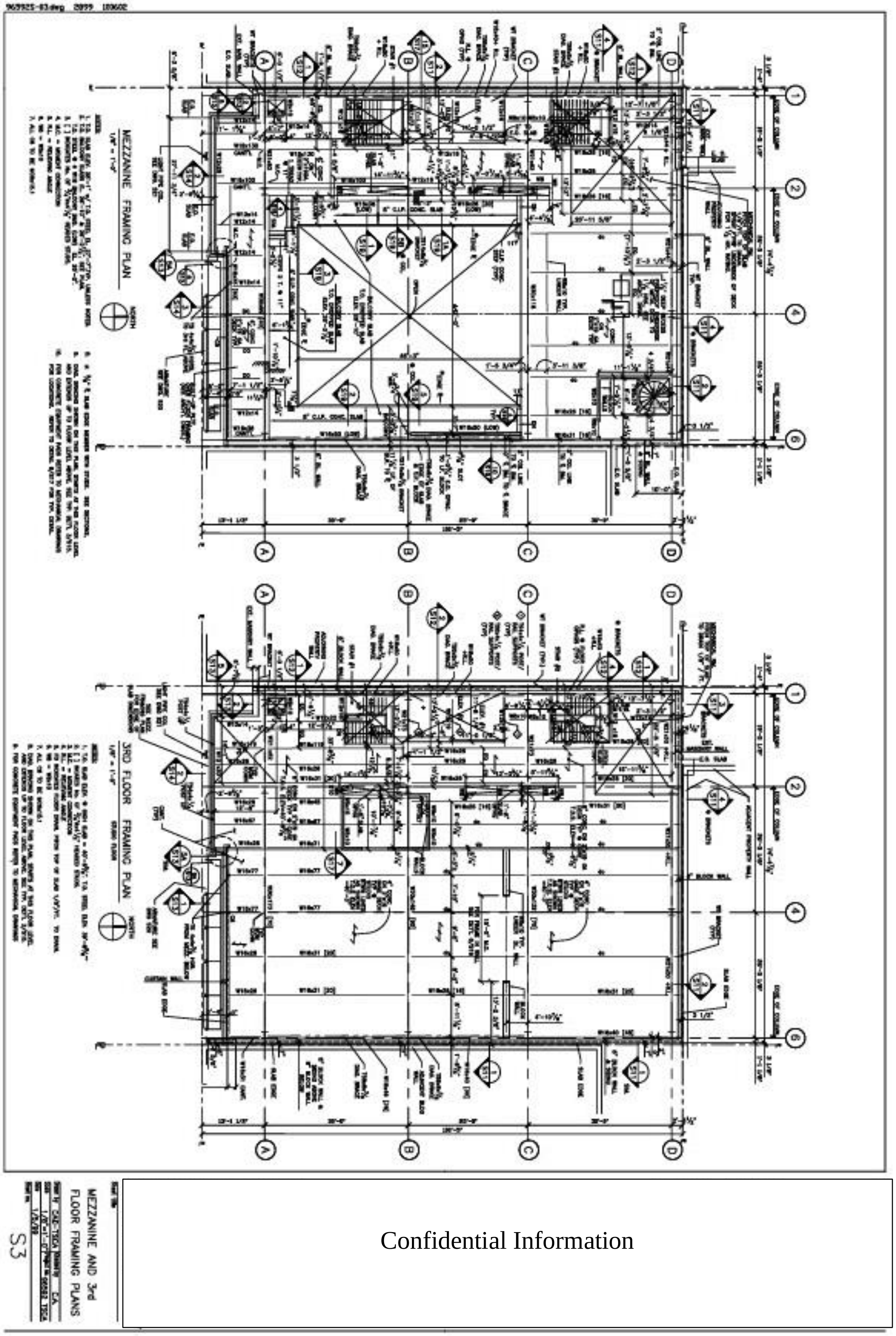


Drawing Sheet 1



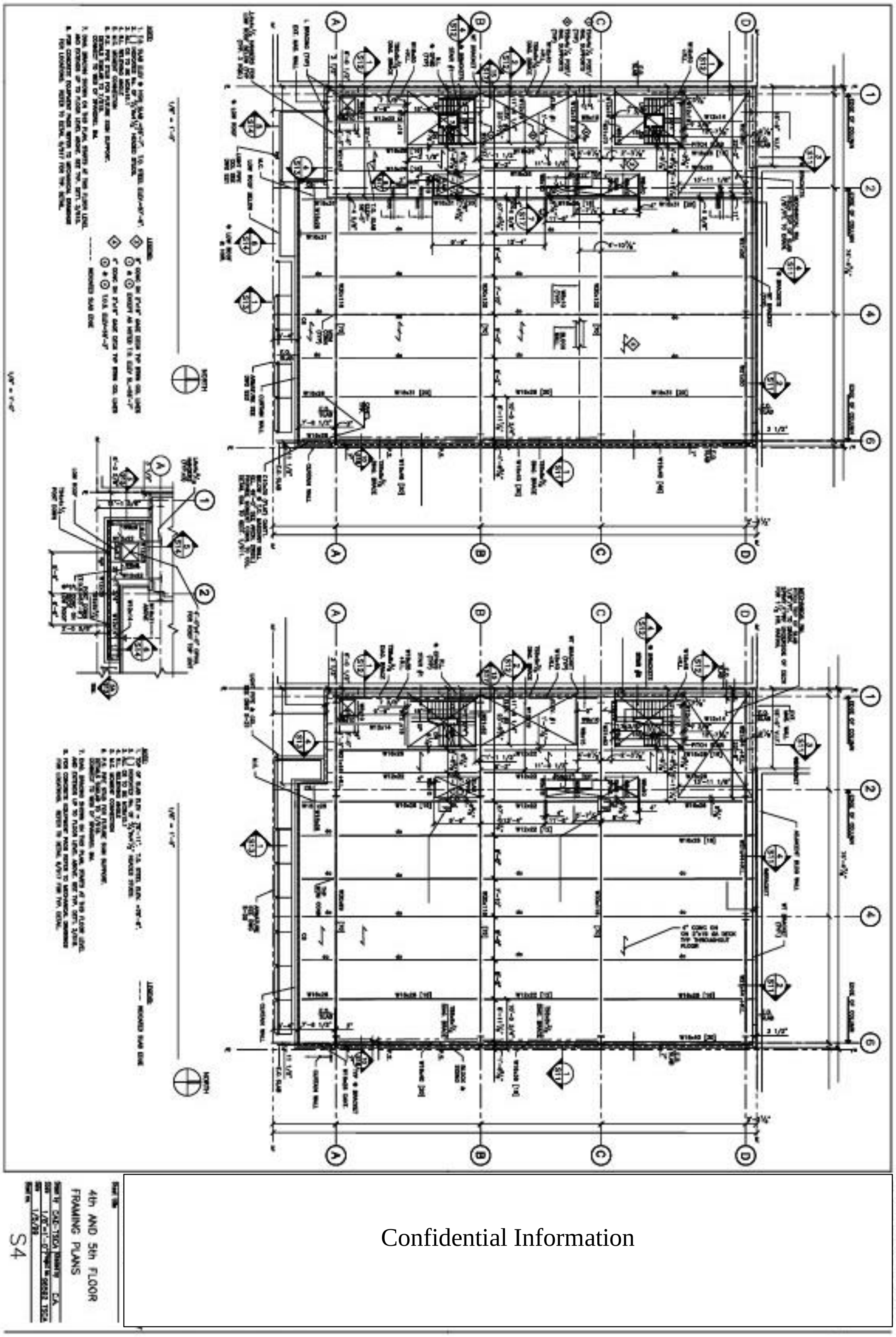
Confidential Information

Drawing Sheet 2



Confidential Information

Drawing Sheet 3



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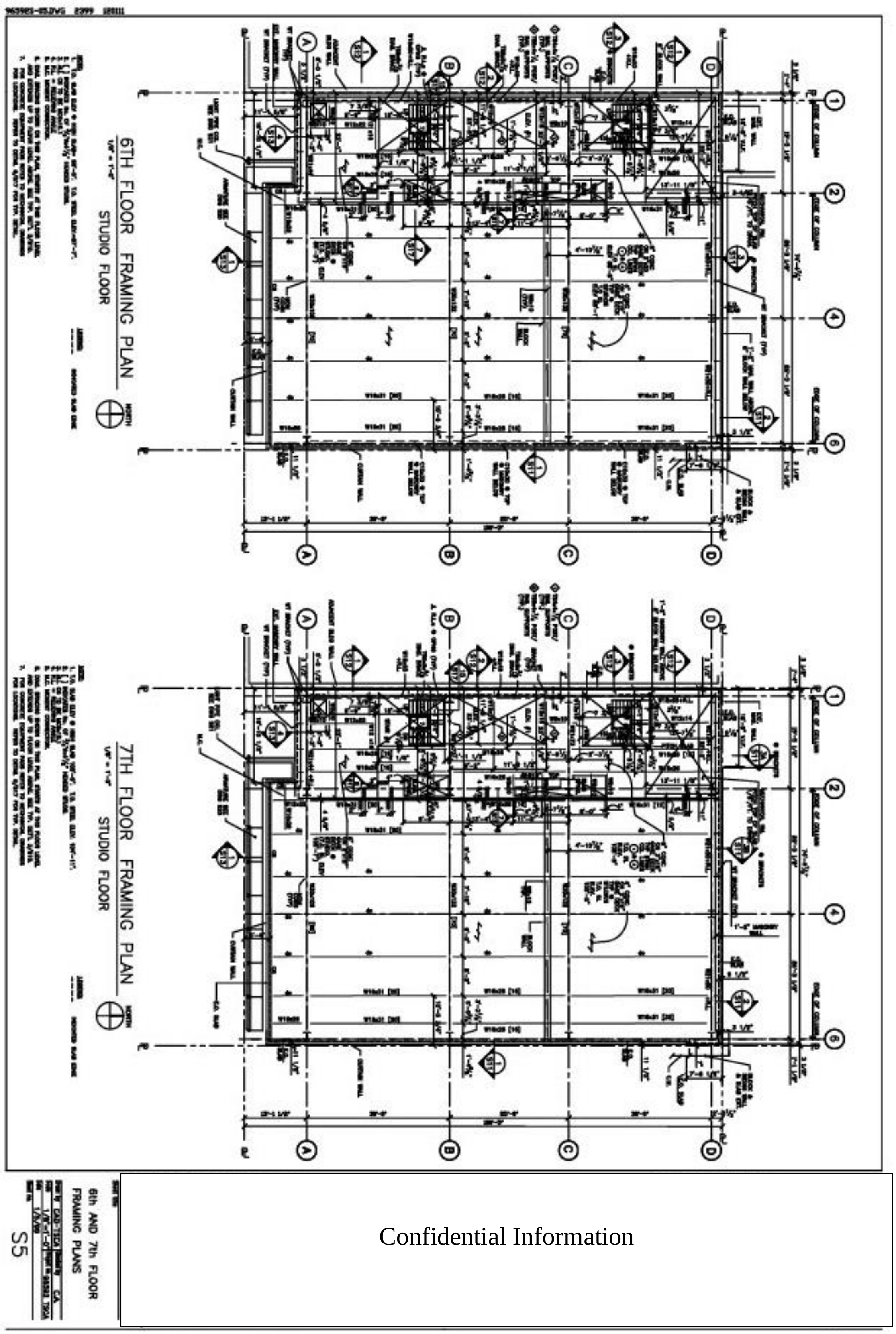

Confidential Information

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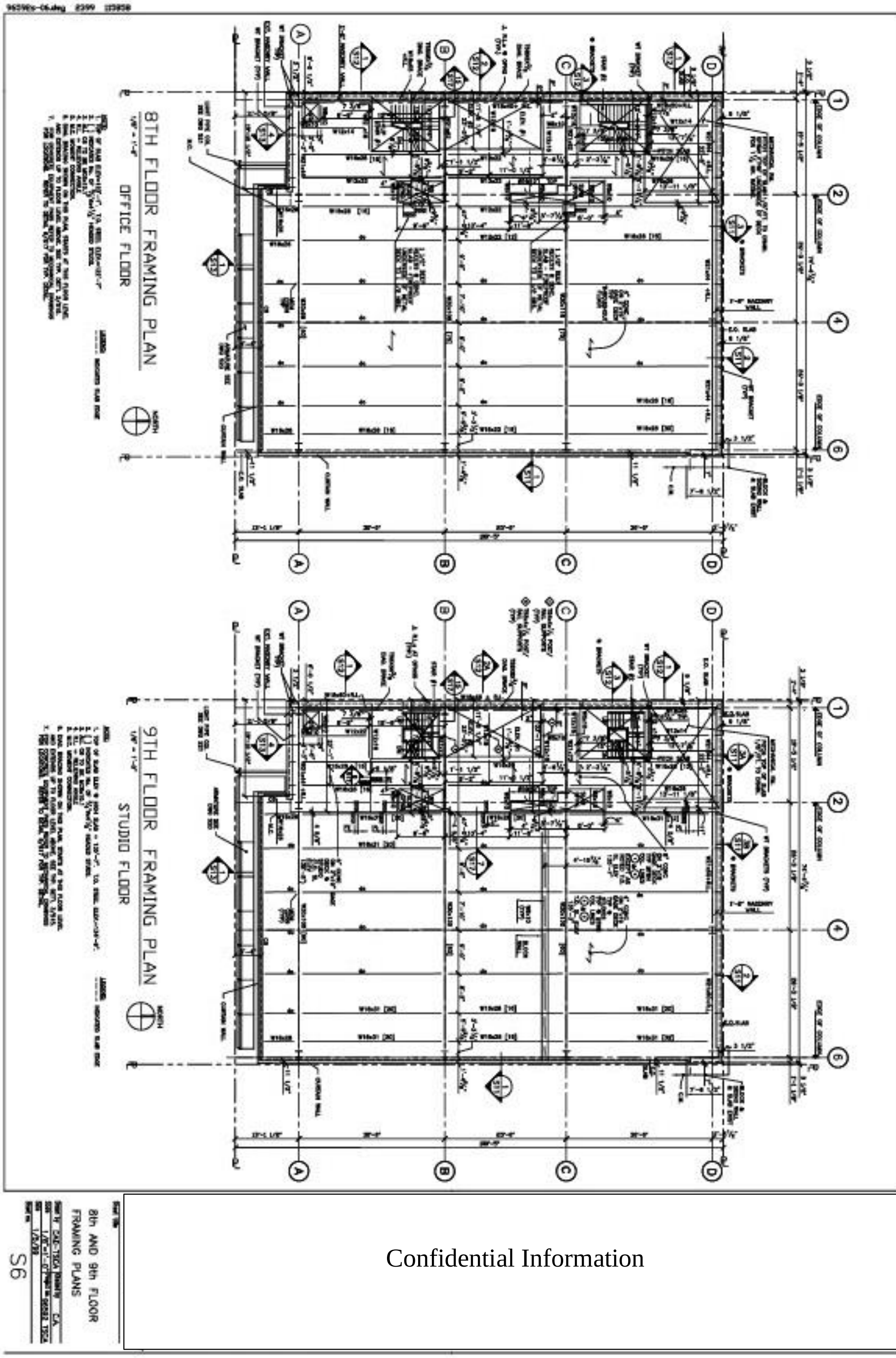

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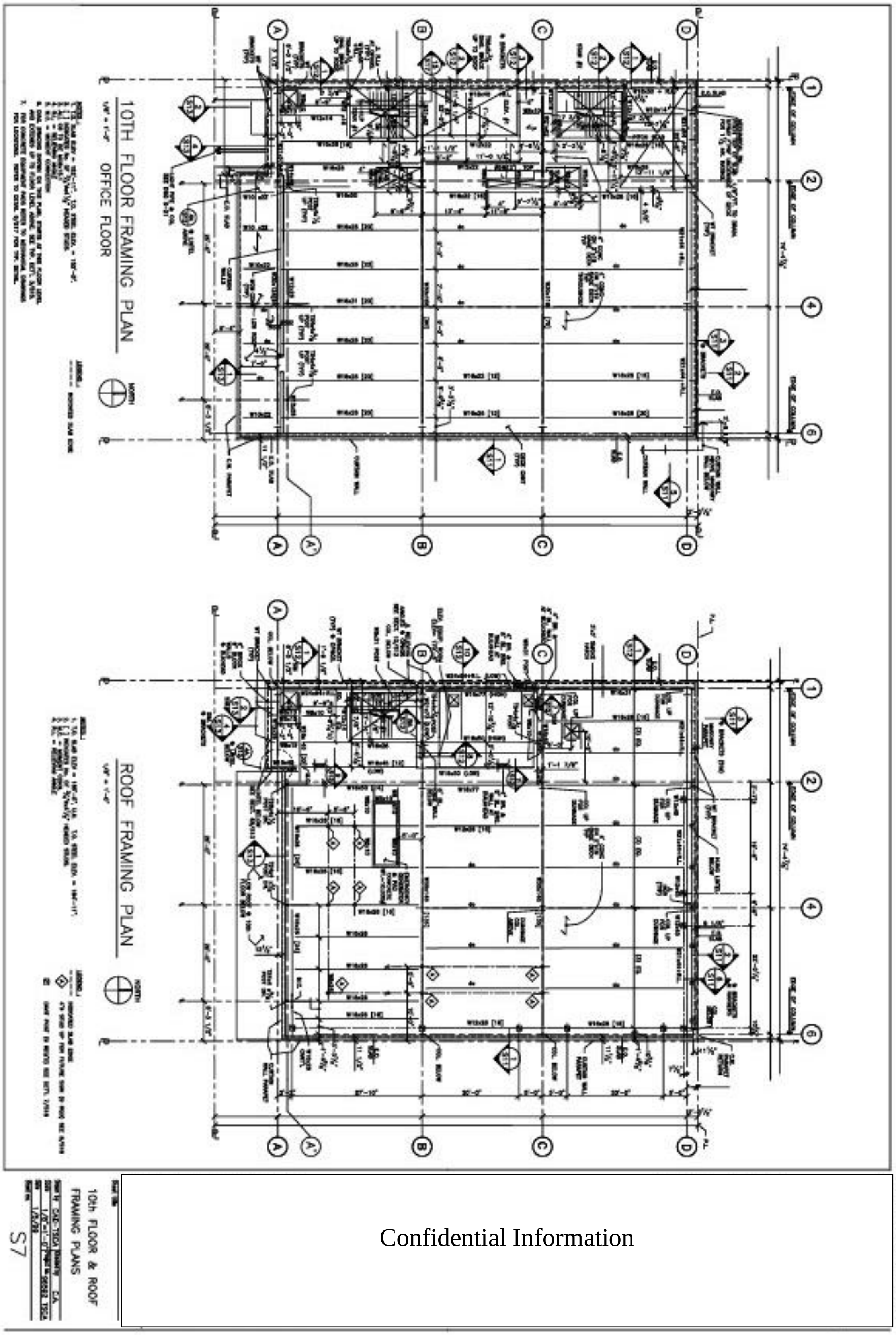

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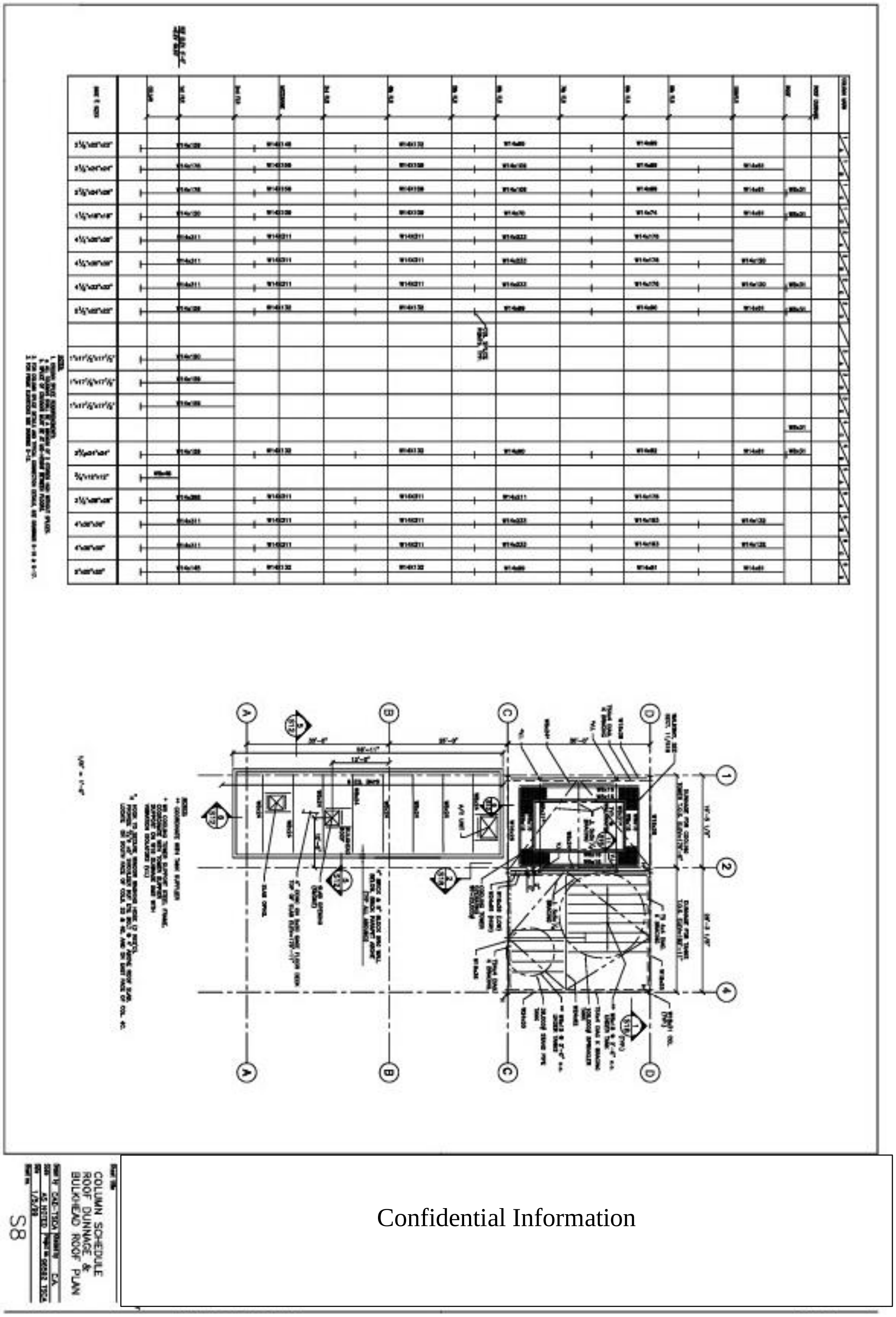
Drawing Sheet 7



Confidential Information

Drawing Sheet 8

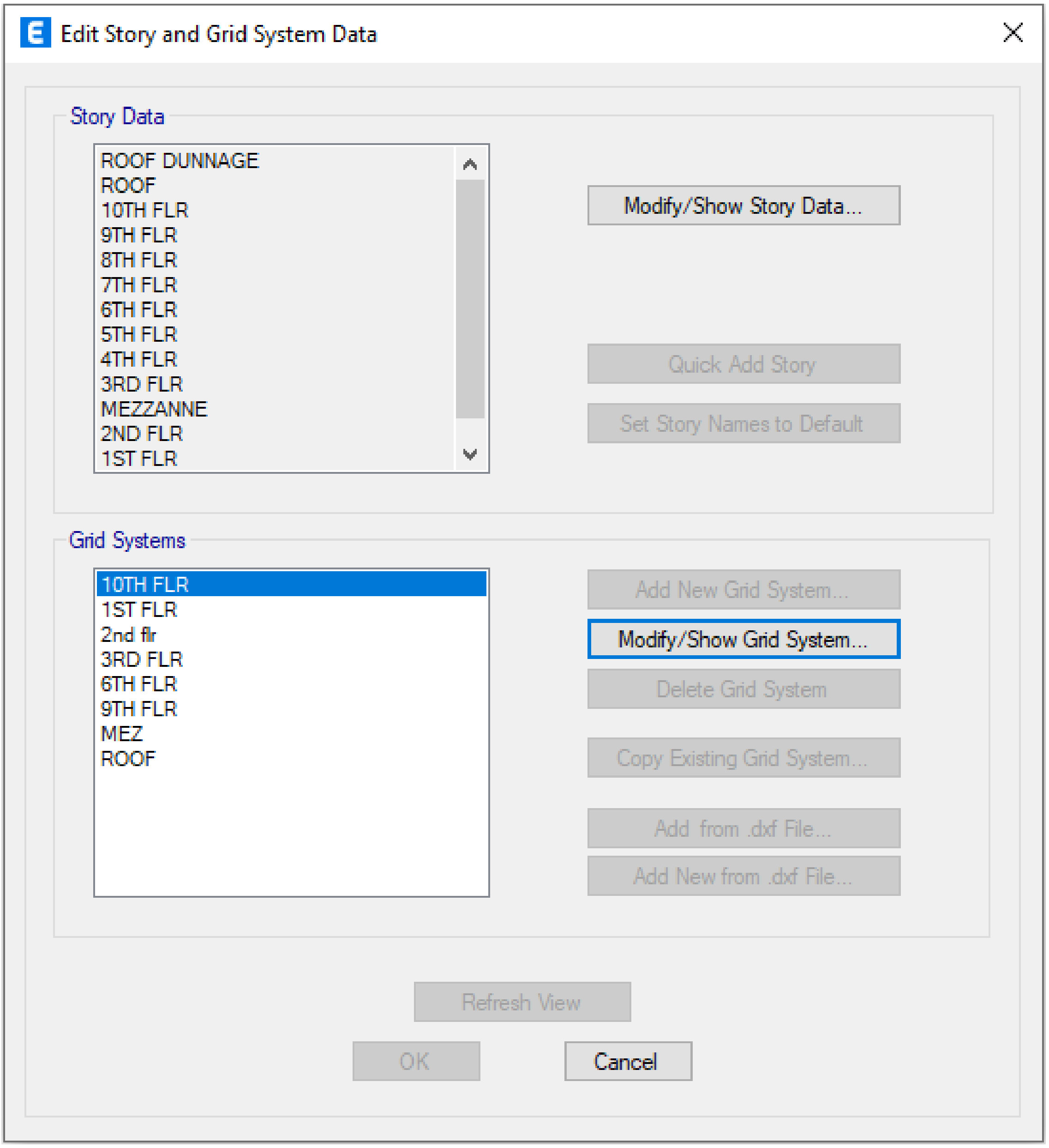
APPENDIX B – STORIES AND GRIDS CREATED IN ETABS

Story Data

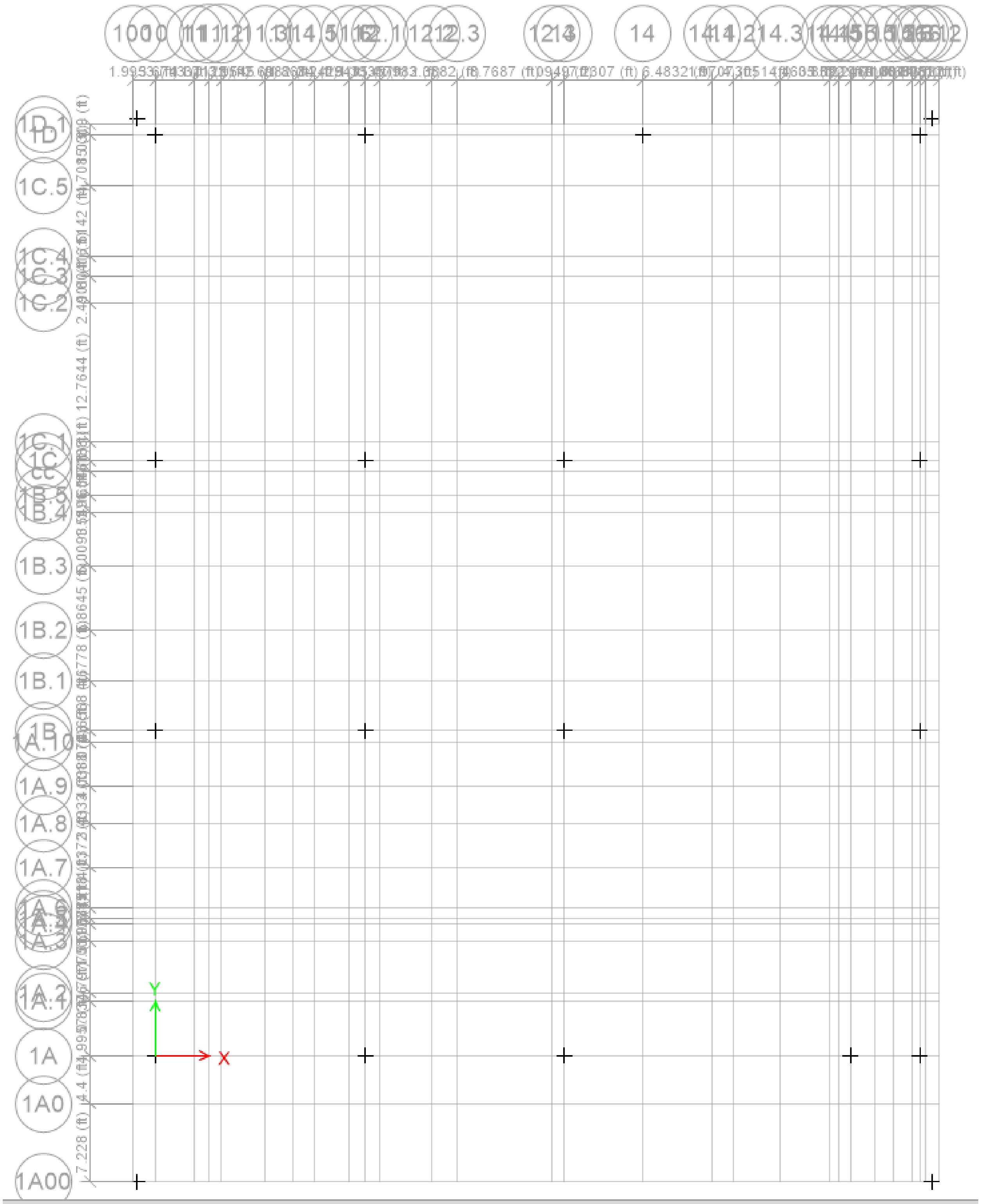
Story	Height ft	Elevation ft	Master Story	Similar To	Splice Story	Splice Height ft	Story Color
ROOF	12.5	182.0367	No	None	No	0	
10TH FLR	17.6667	169.5367	No	None	No	0	
9TH FLR	12.1667	151.87	No	None	No	0	
8TH FLR	17.6666	139.7033	No	None	No	0	
7TH FLR	17.3334	122.0367	No	None	No	0	
6TH FLR	12.1666	104.7033	No	None	No	0	
5TH FLR	17.6667	92.5367	No	None	No	0	
4TH FLR	18.0313	74.87	No	None	No	0	
3RD FLR	12.1355	56.8387	No	None	No	0	
MEZZANNE	12.1355	44.7033	No	None	No	0	
2ND FLR	15.1094	32.5678	No	None	No	0	
1ST FLR	8.7292	17.4584	No	None	No	0	
CELLAR		8.7292					

Note: Right Click on Grid for Options

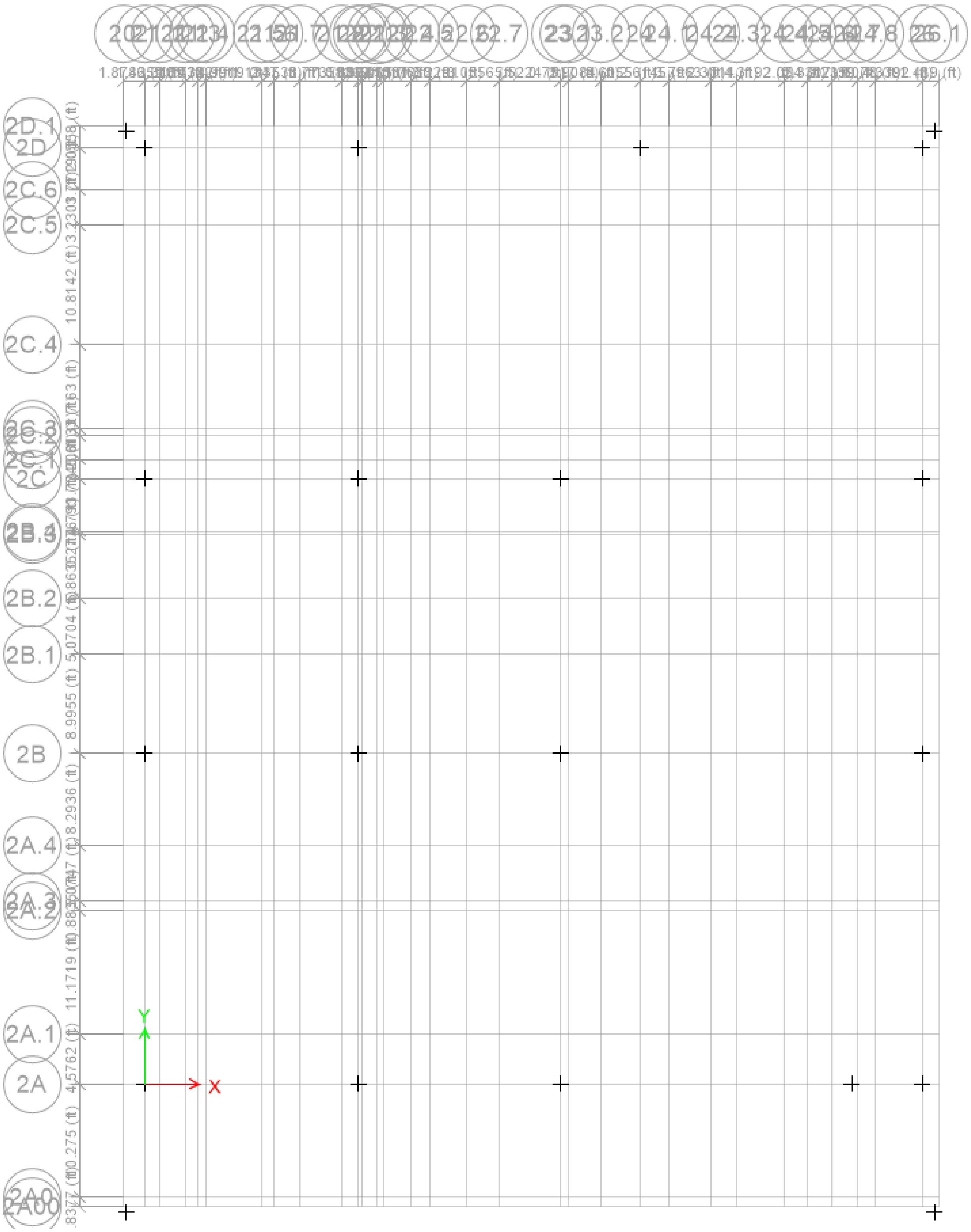
Story Elevations Created in the ETABS Model



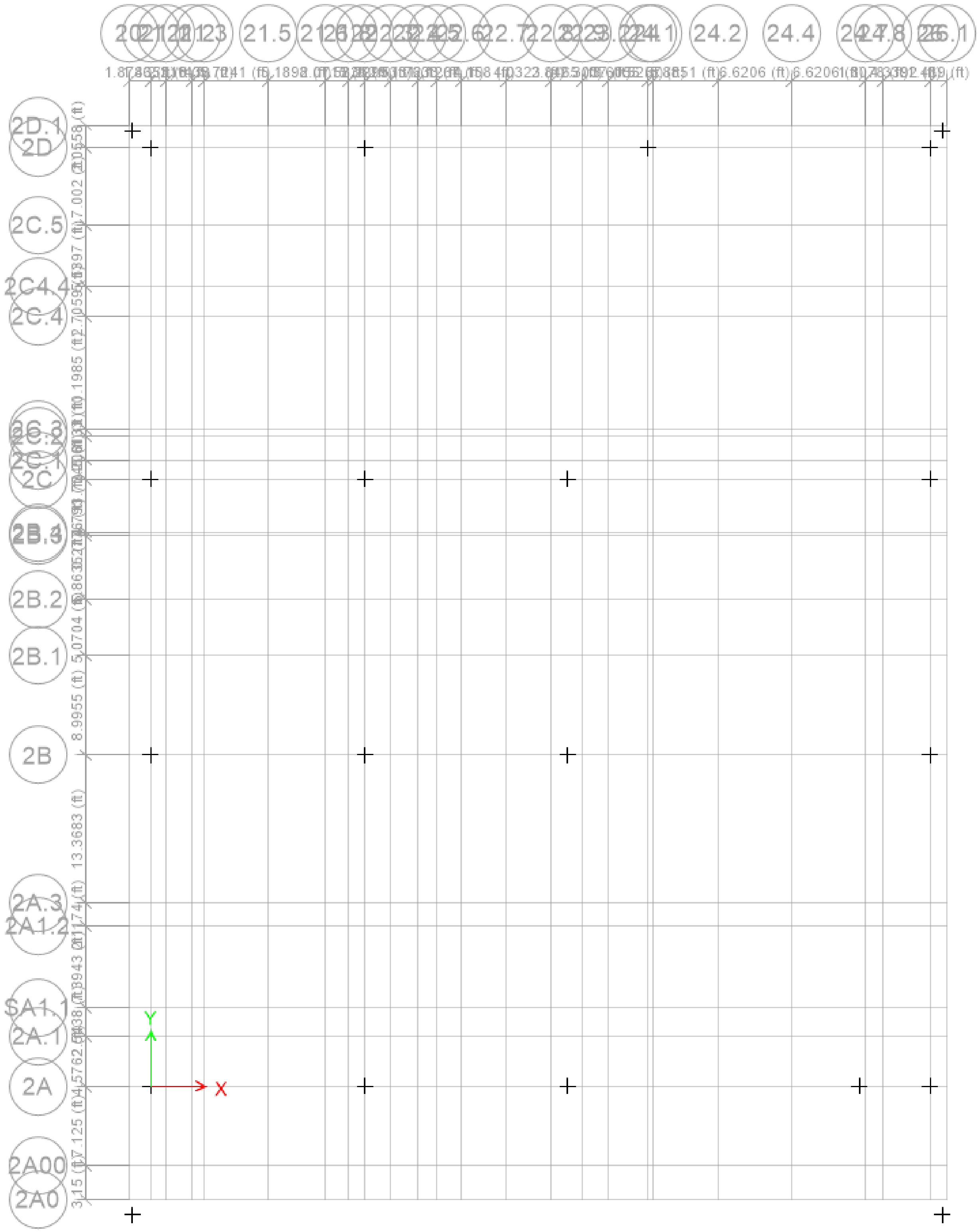
The Various Girds Created Within the ETABS Model



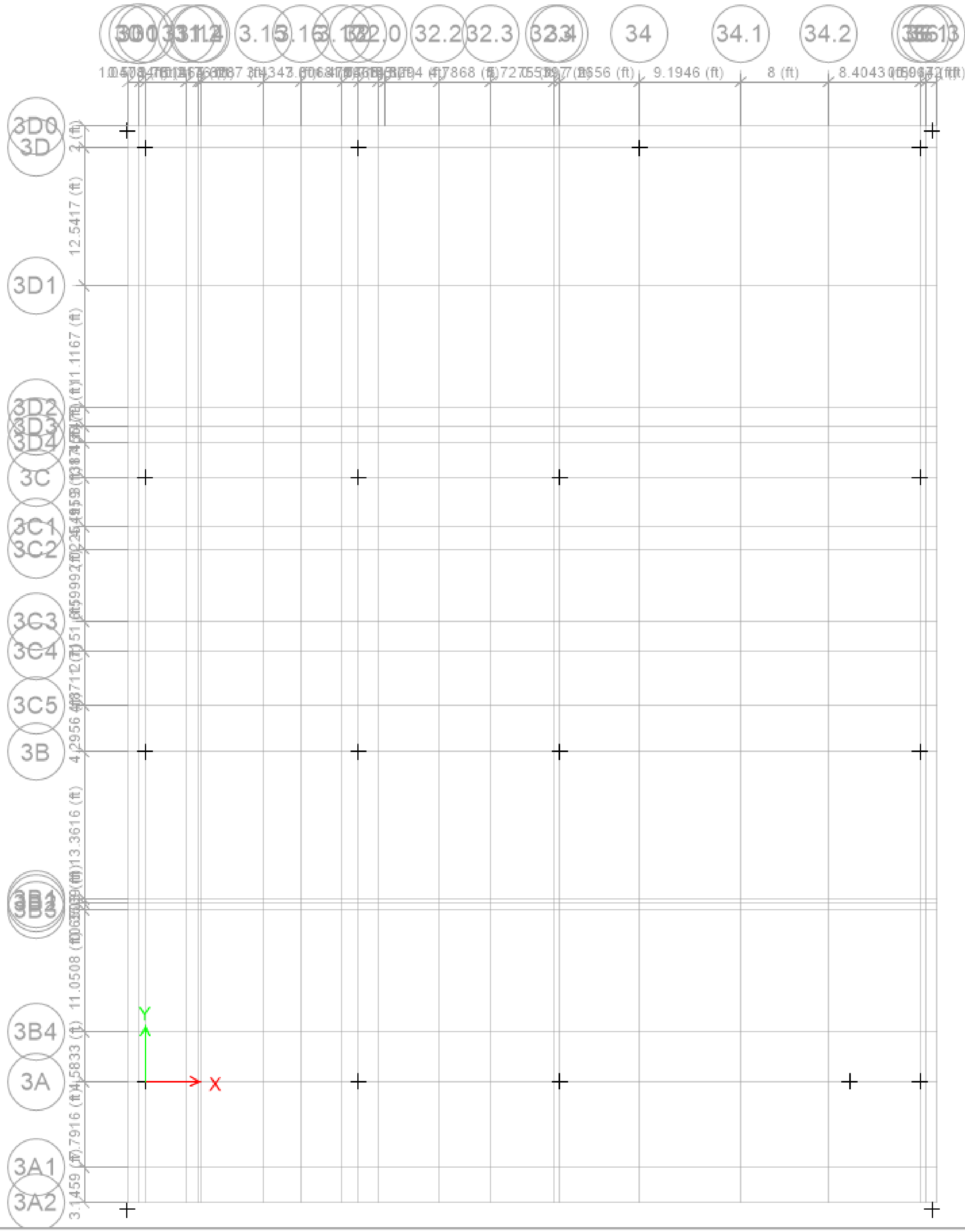
First Floor Grid System



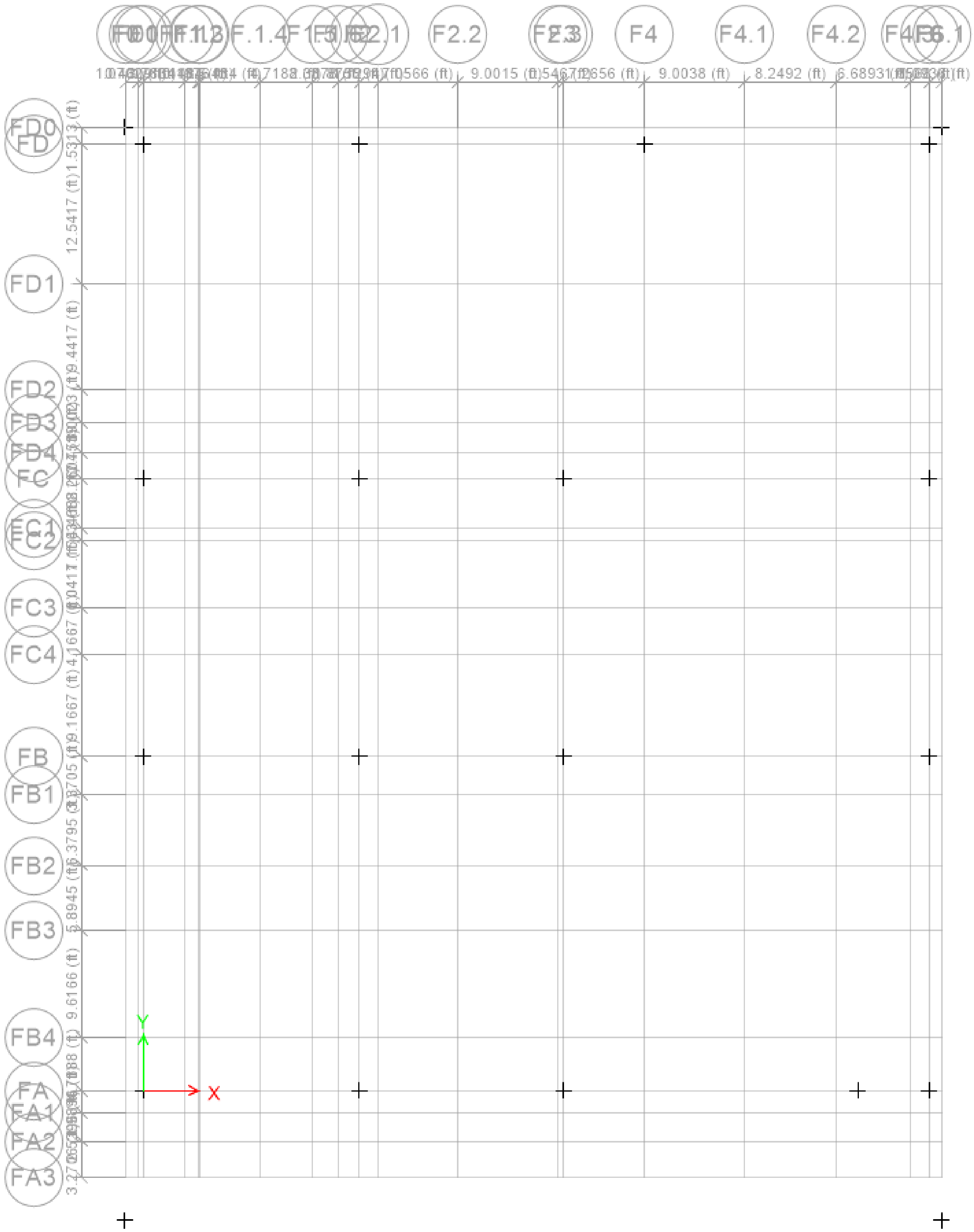
Second Floor Grid System



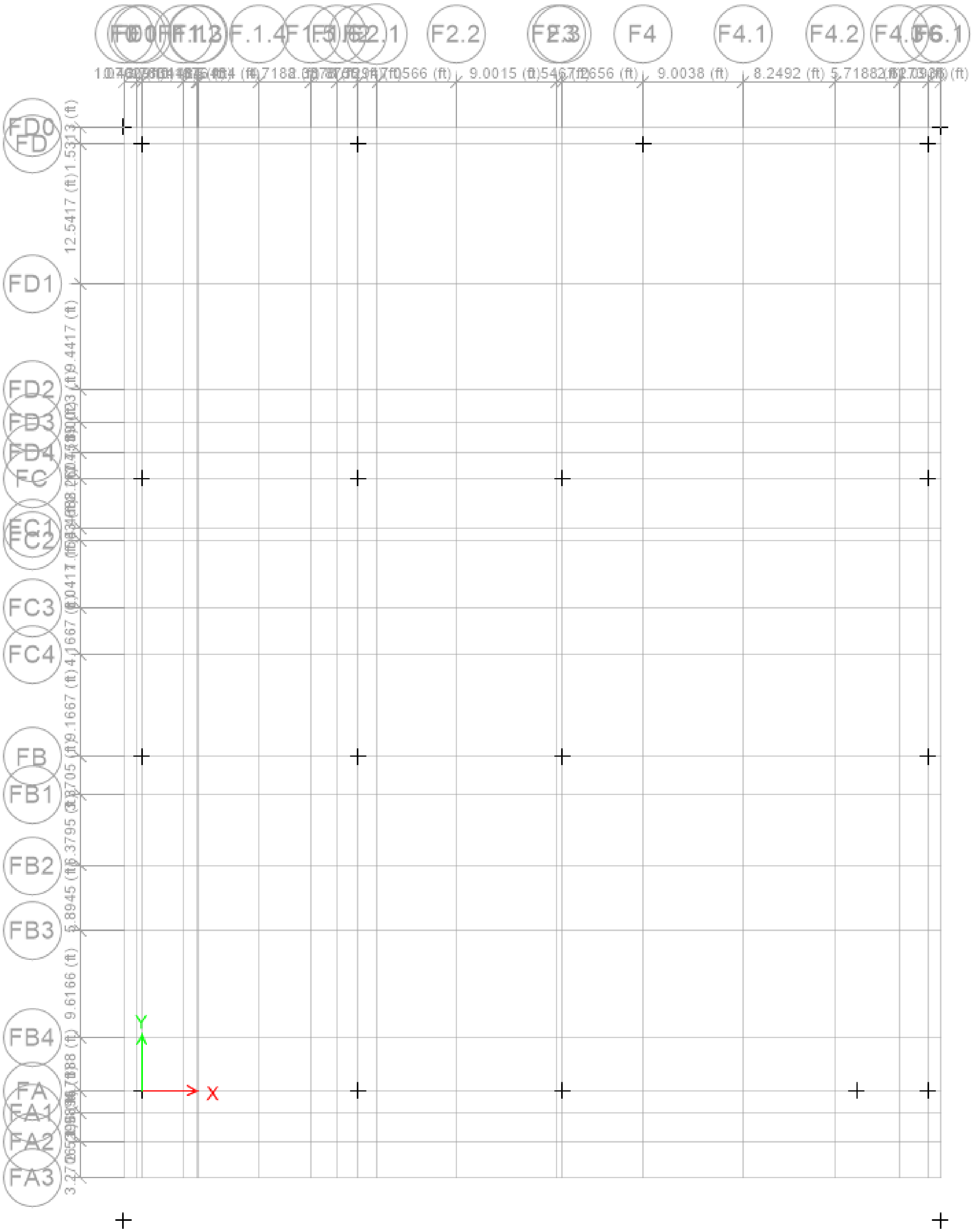
Mezzanine Grid System



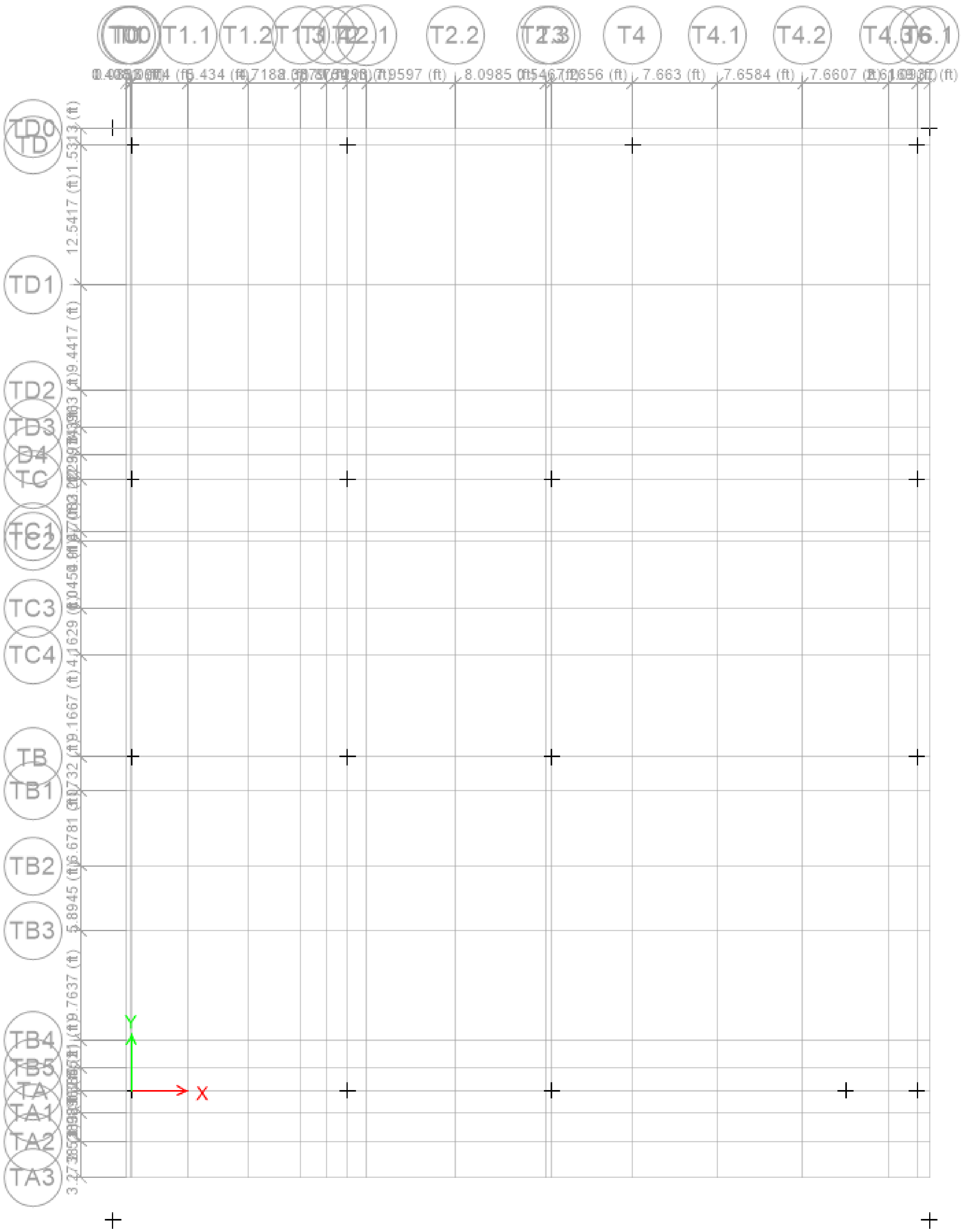
Third Floor Grid System



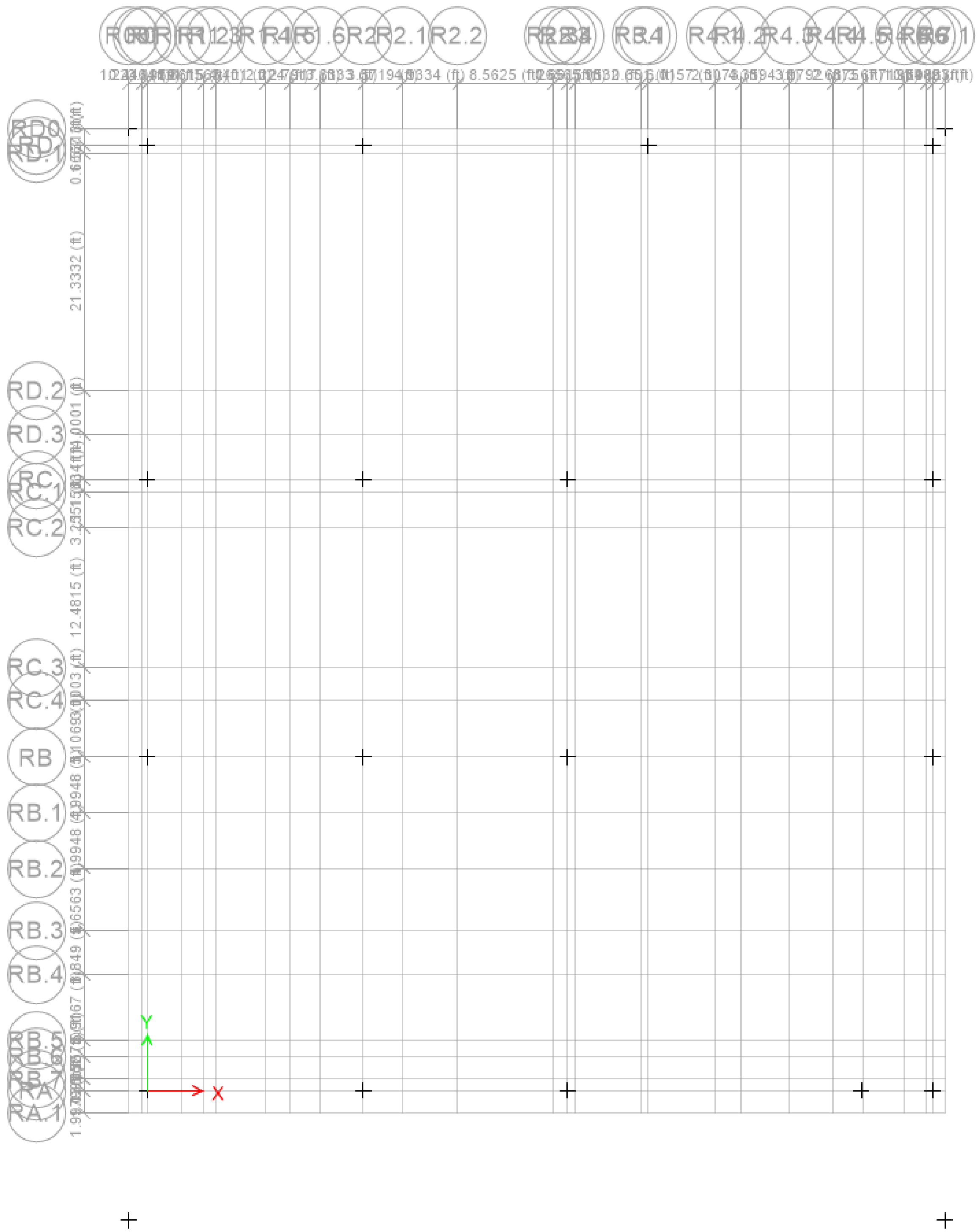
Fourth, Fifth, and Sixth Floor Grid System



Seventh, Eight, and Ninth Floor Grid System

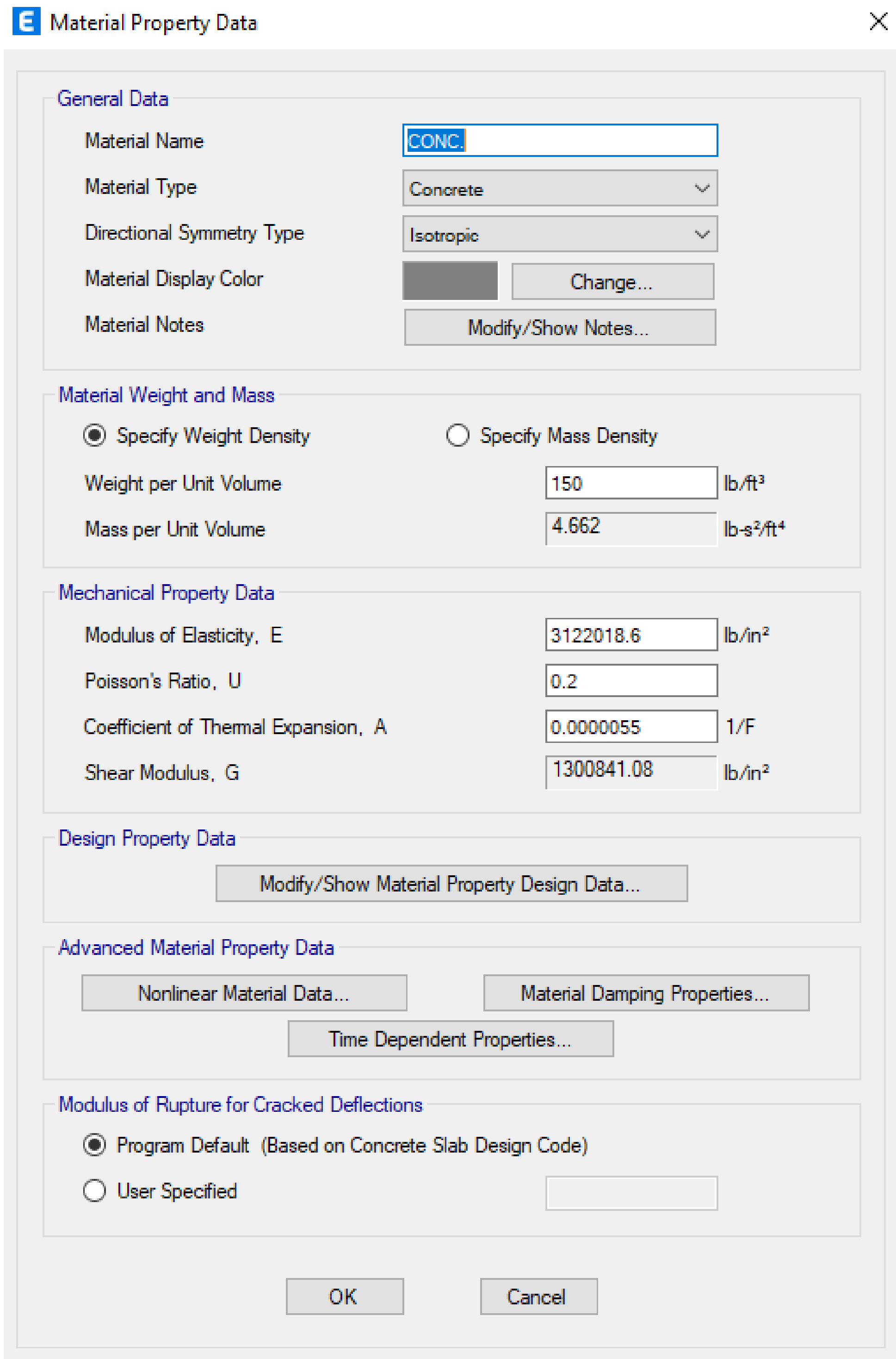


Tenth Floor Grid System

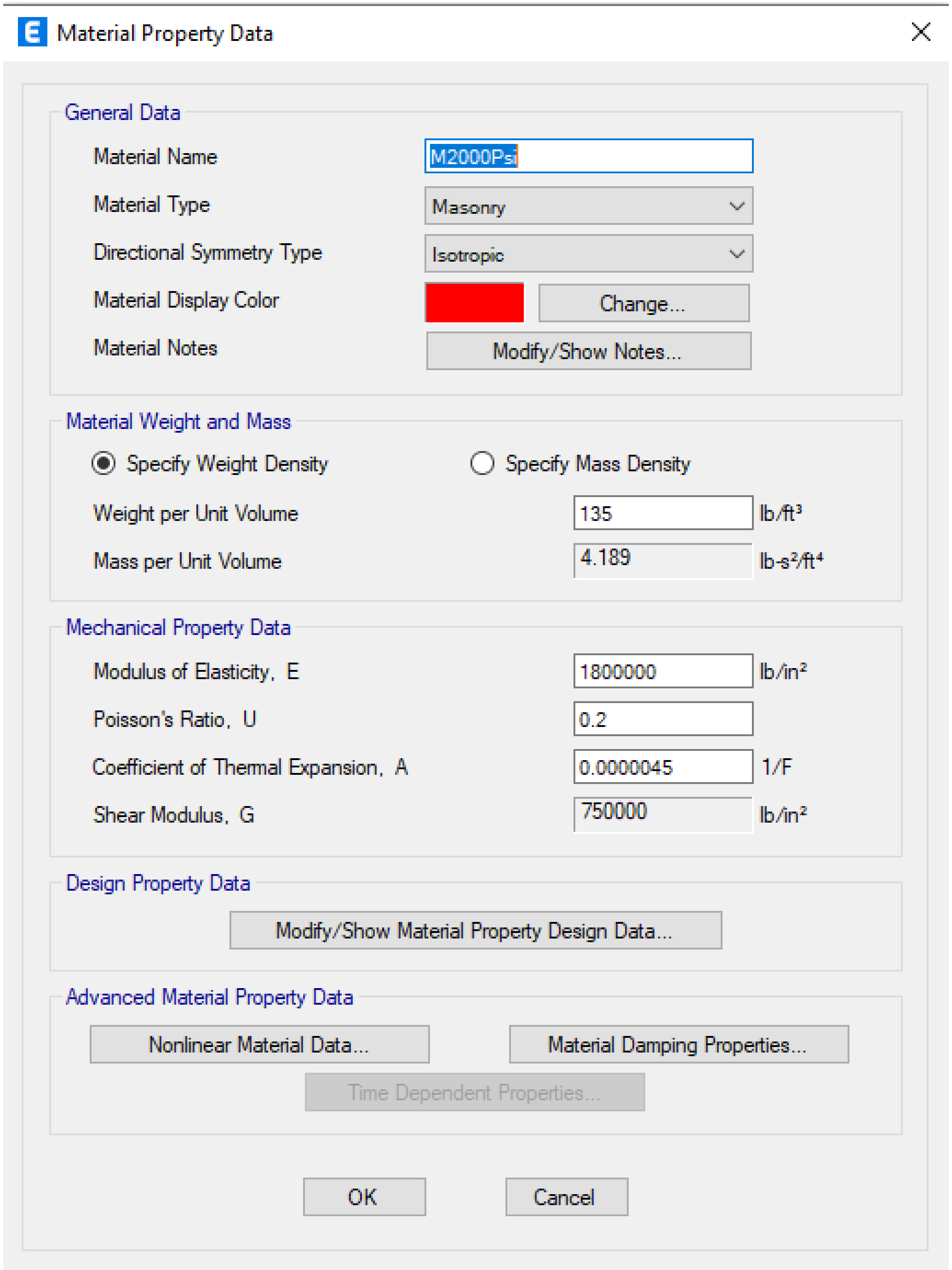


Roof Grid System

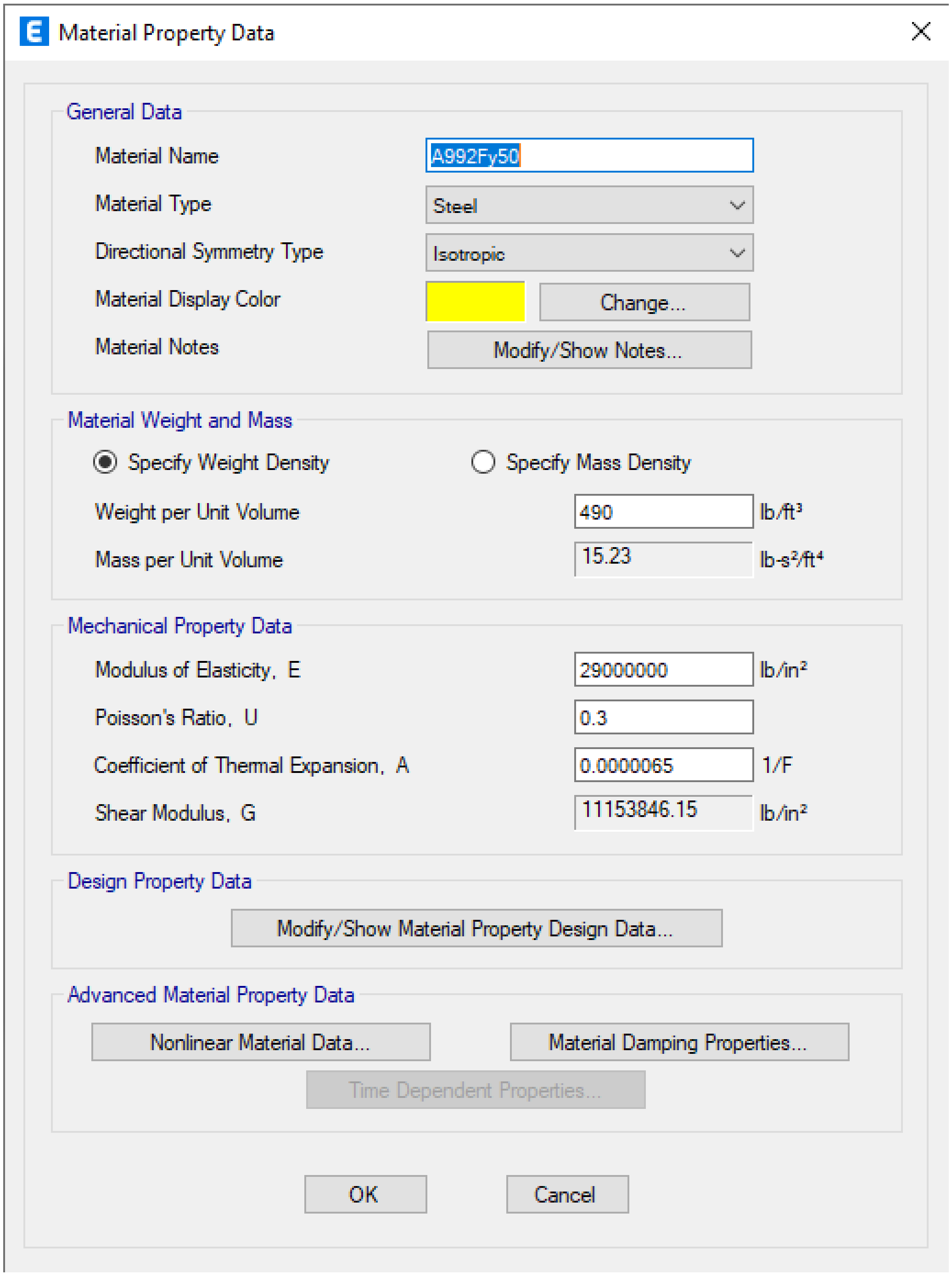
APPENDIX C – MATERIAL PROPERTIES



Concrete Material Property

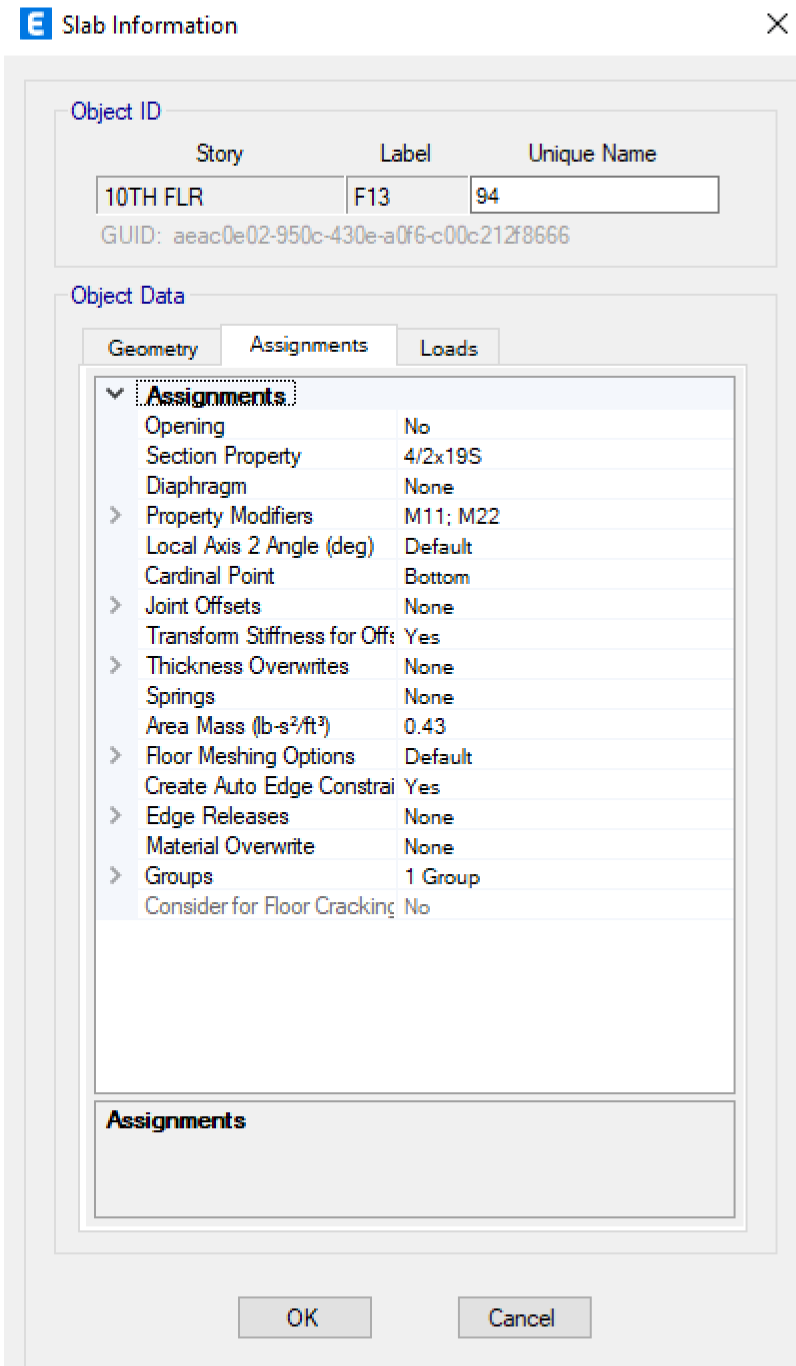


Masonry Material Property

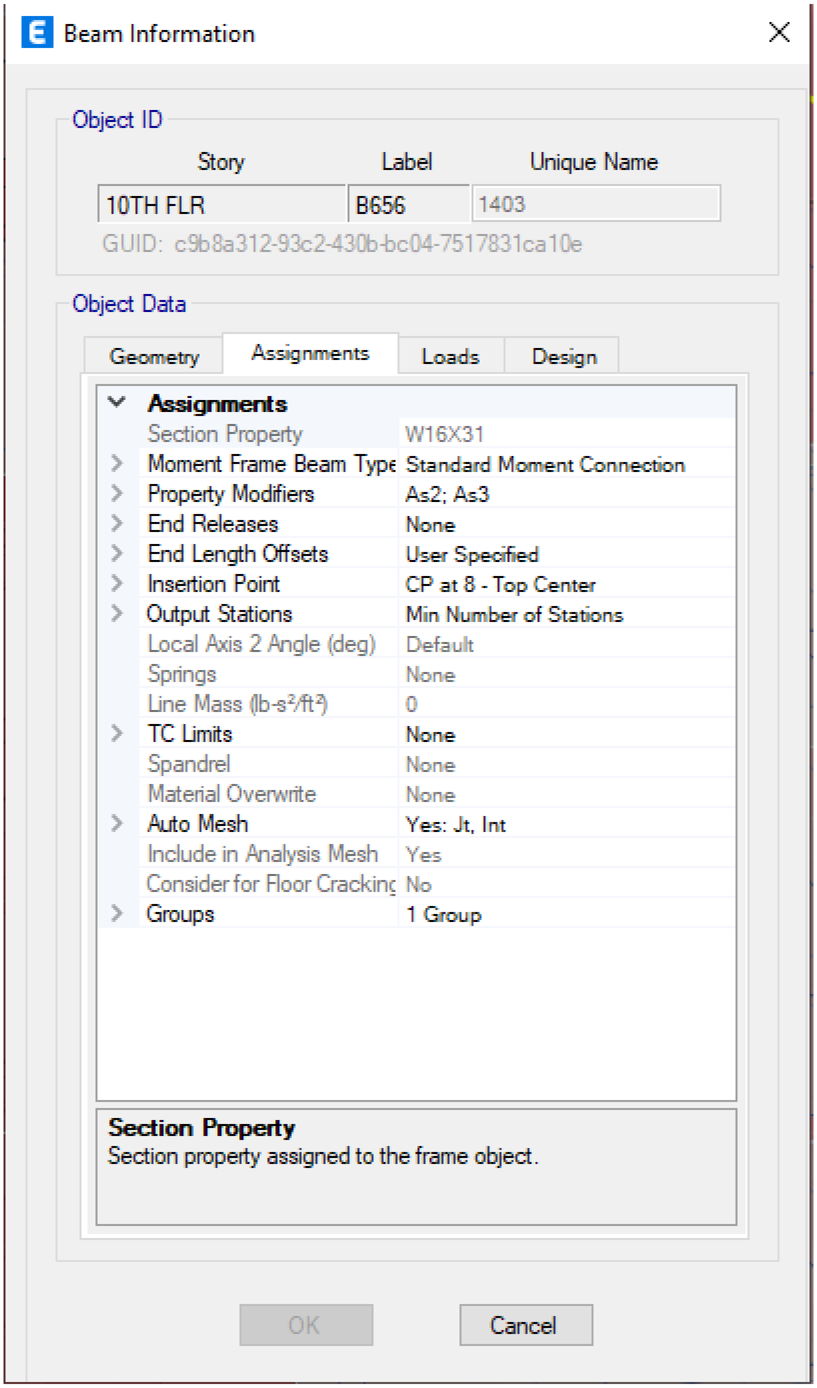


Steel Material Property

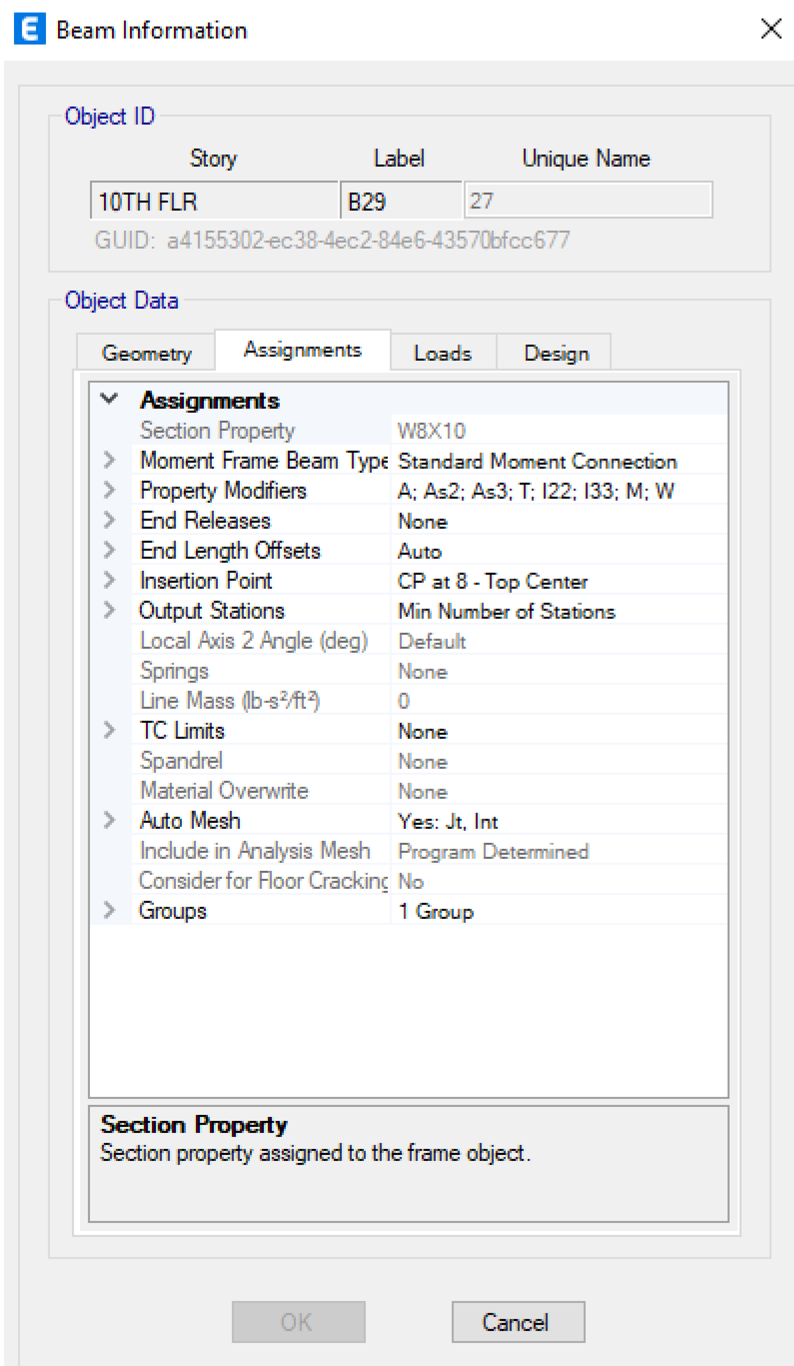
APPENDIX D – FINAL ETABS MODEL PROPERTIES



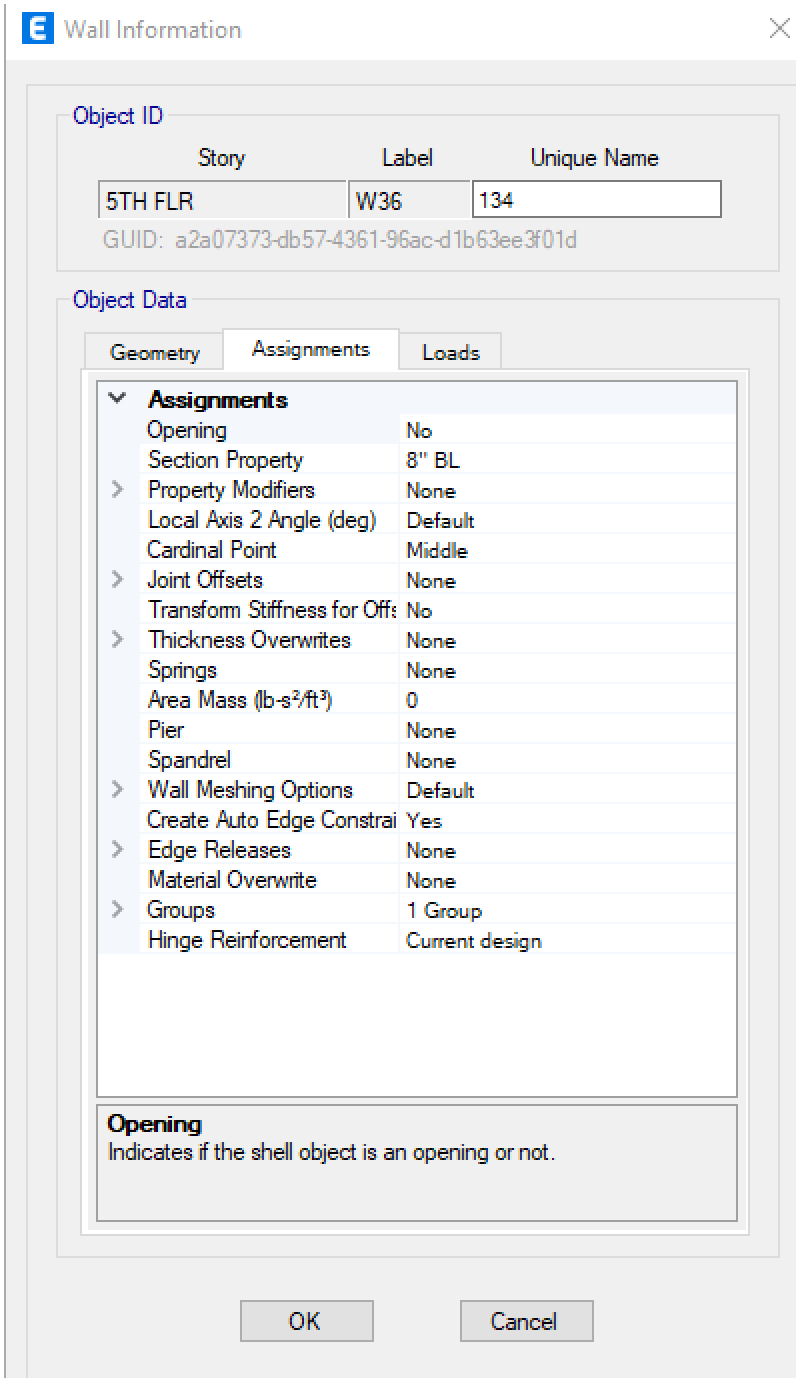
Typical Properties of Shell Floor Elements



Typical Properties of Frame Floor Elements

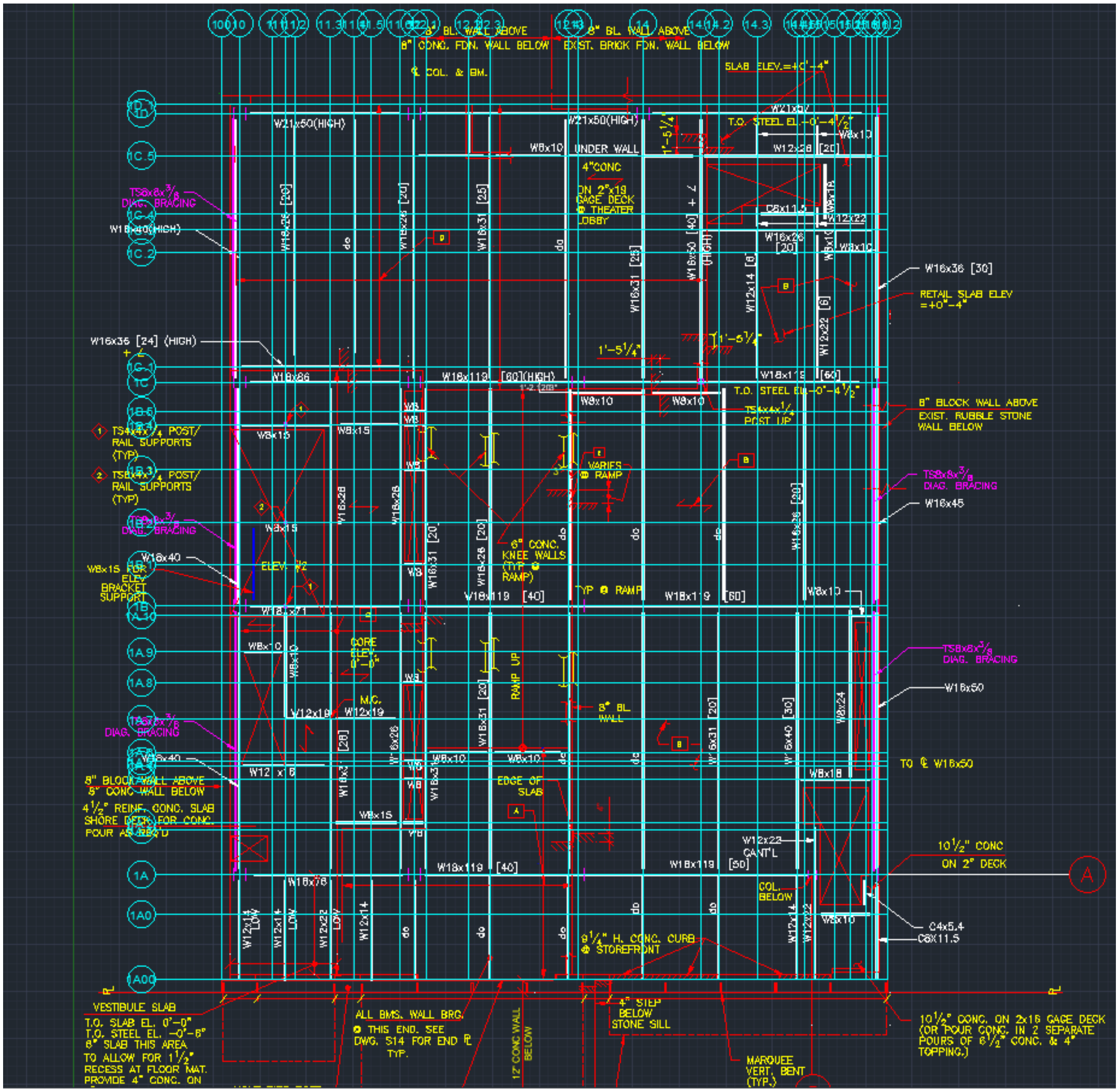


Typical Properties of Spandral Frame Floor Elements

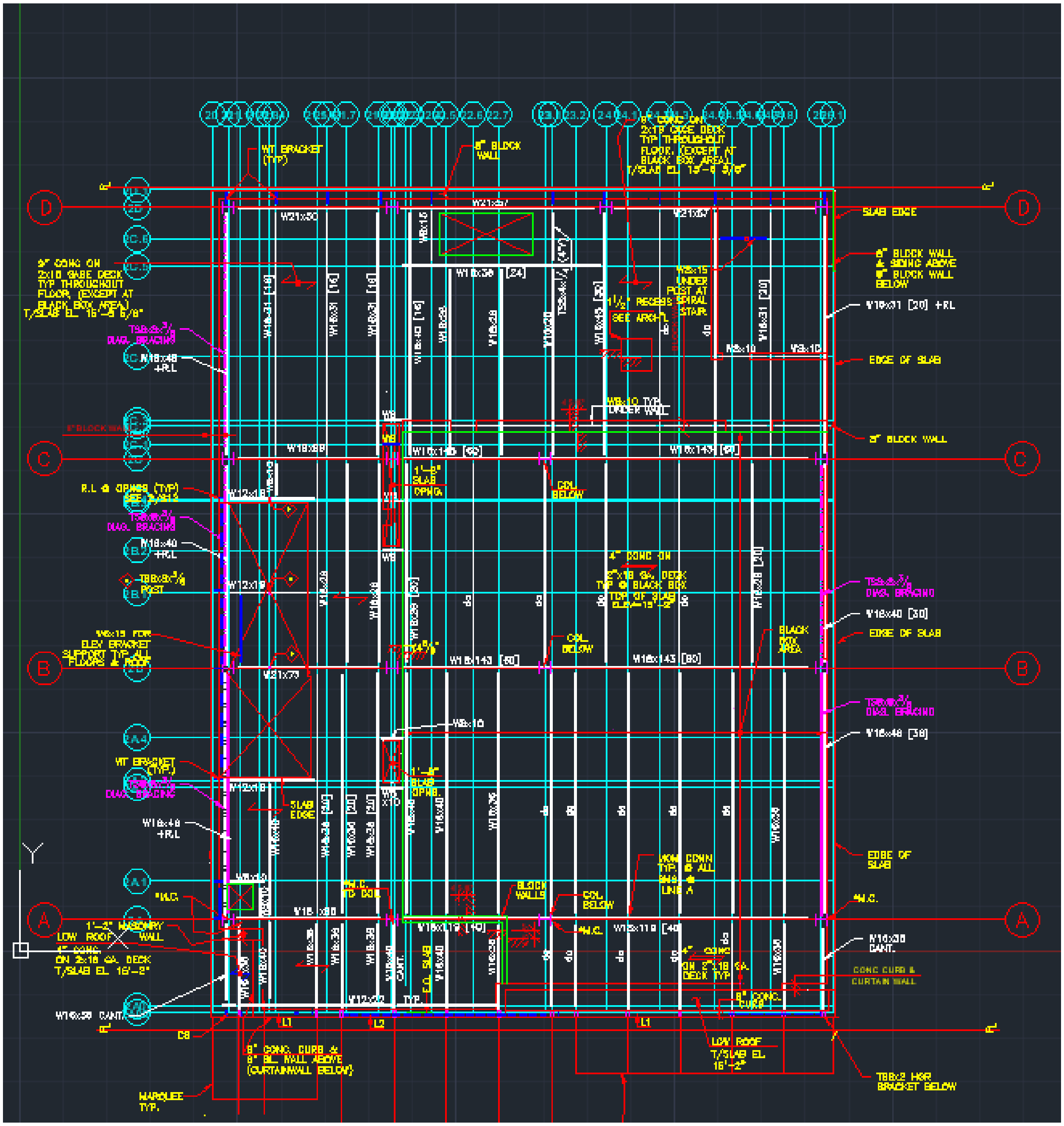


Typical Properties of Bearing Wall Shell Elements

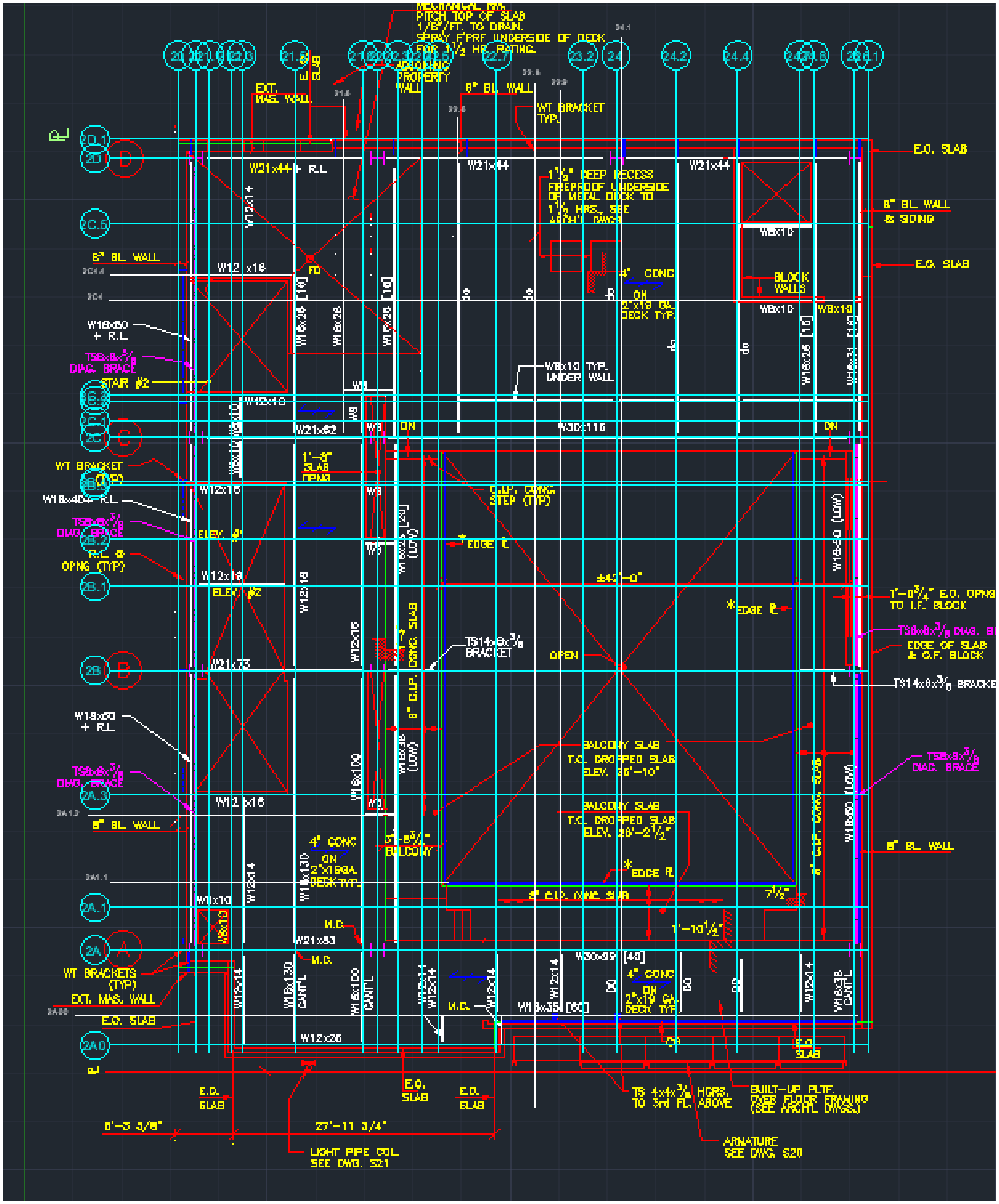
APPENDIX E - EDITED FLOORPLANS USED FOR ETABS MODEL CONSTRUCTION



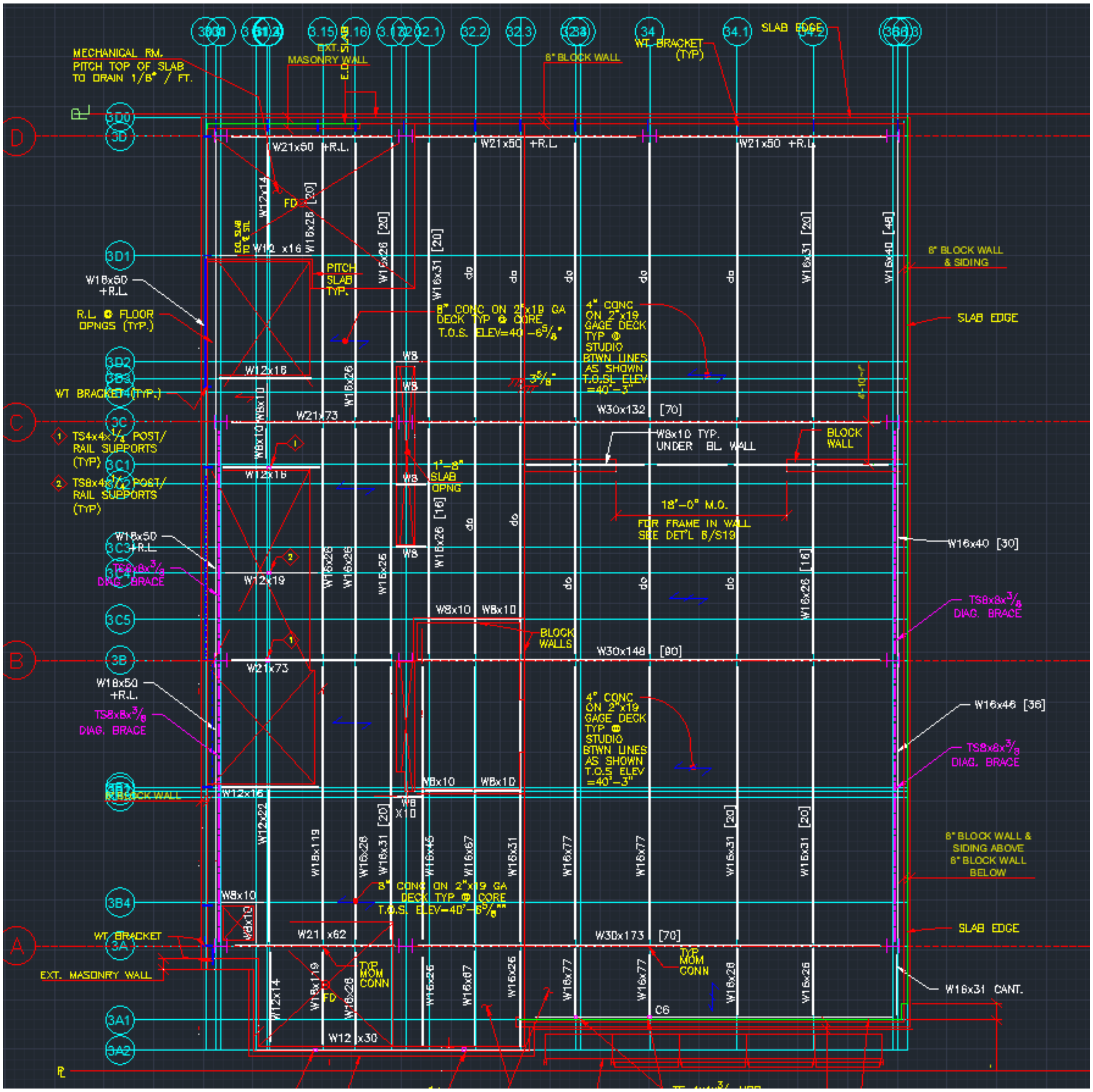
First Floor AutoCAD Floor Plan



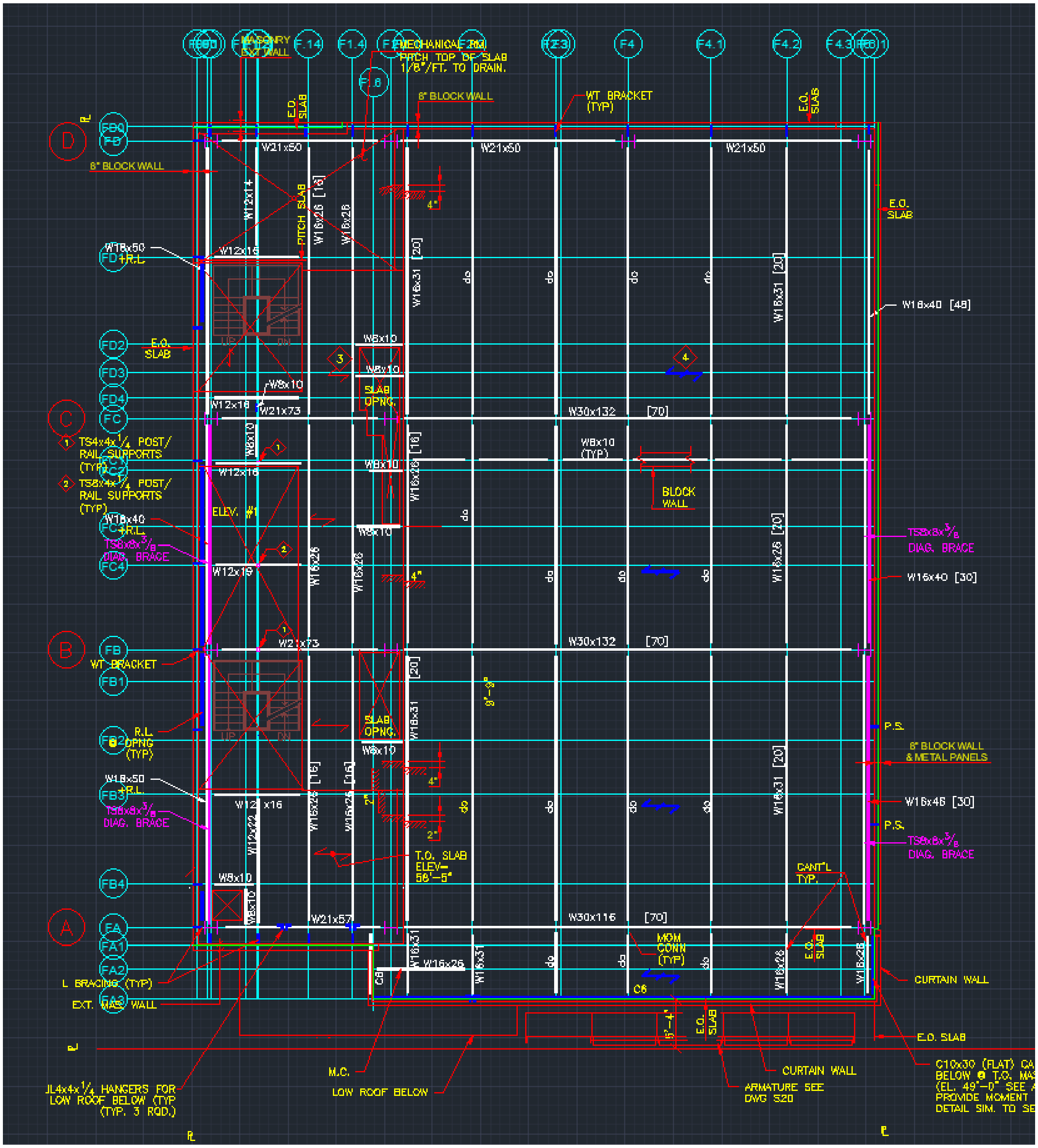
Second Floor AutoCAD Floor Plan



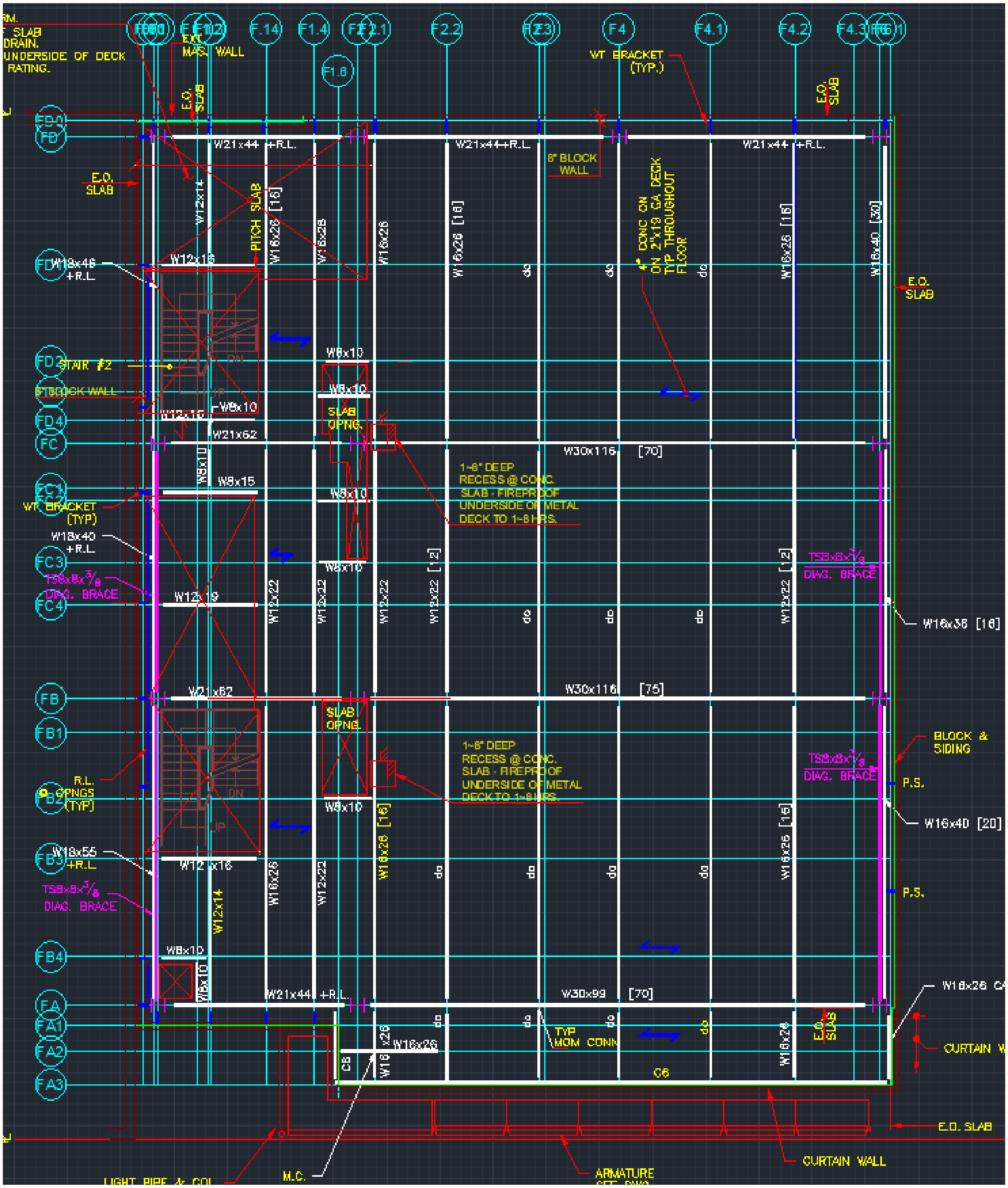
Mezzanine AutoCAD Floor Plan



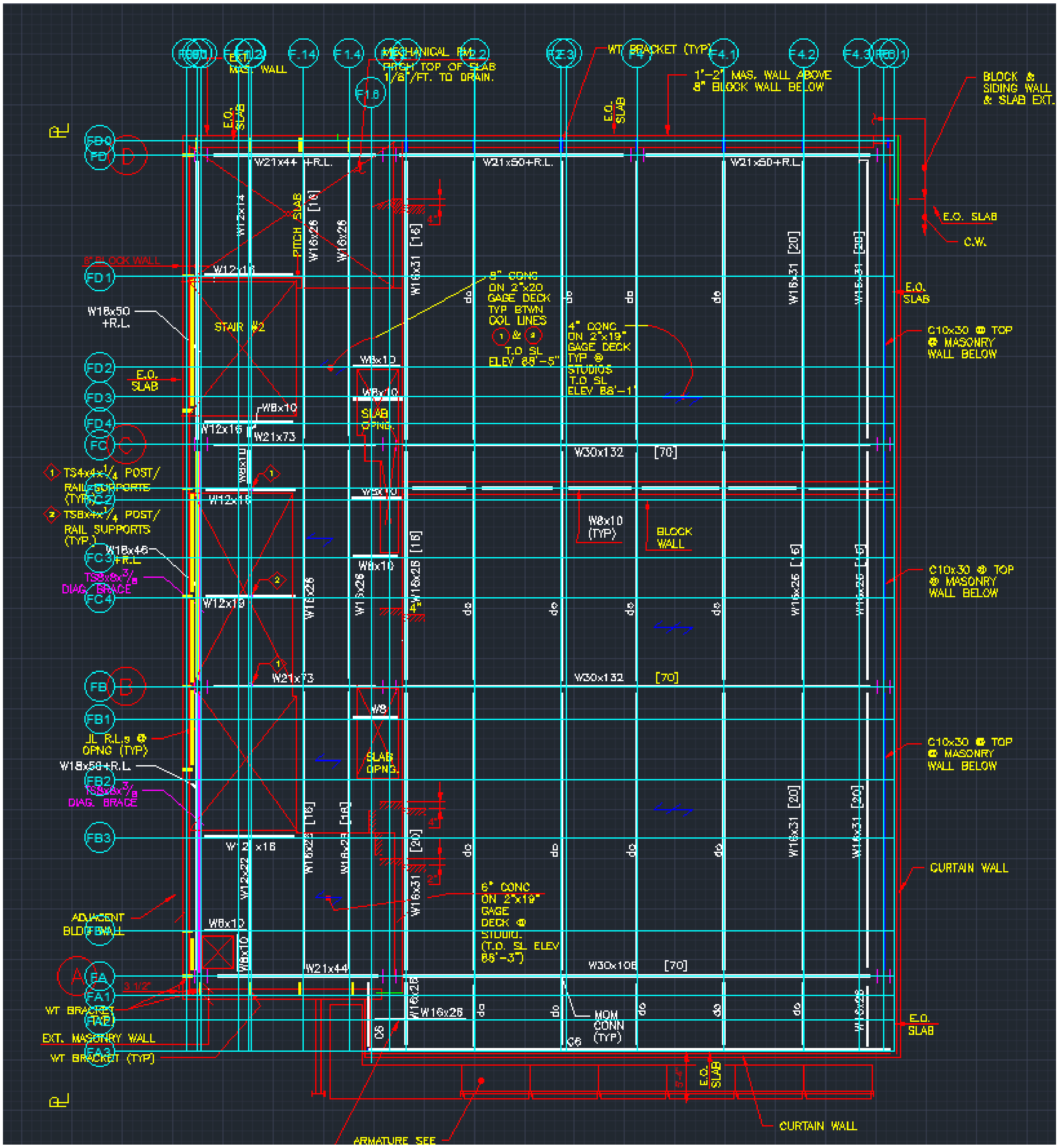
Third Floor AutoCAD Floor Plan



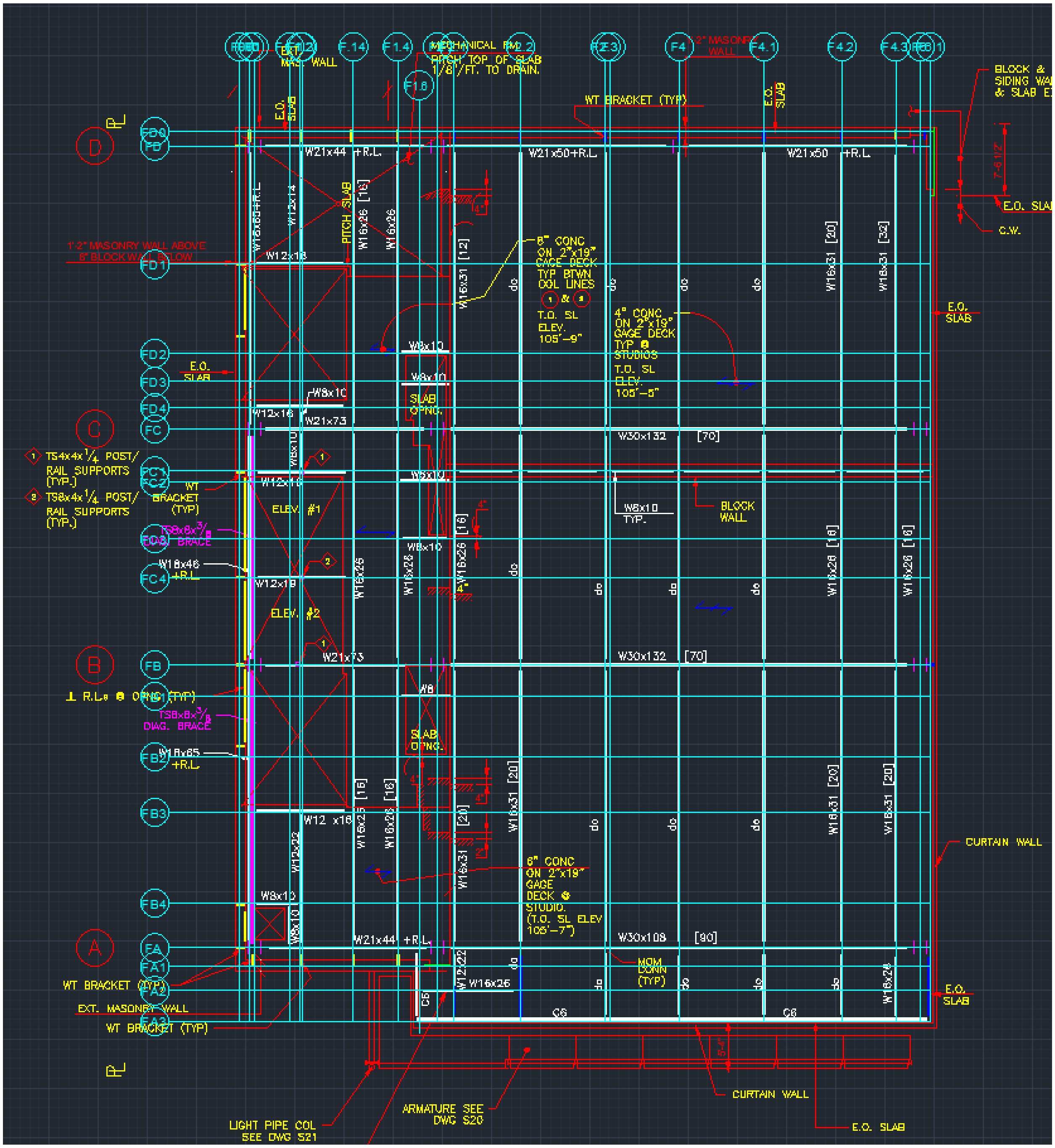
Fourth Floor AutoCAD Floor Plan



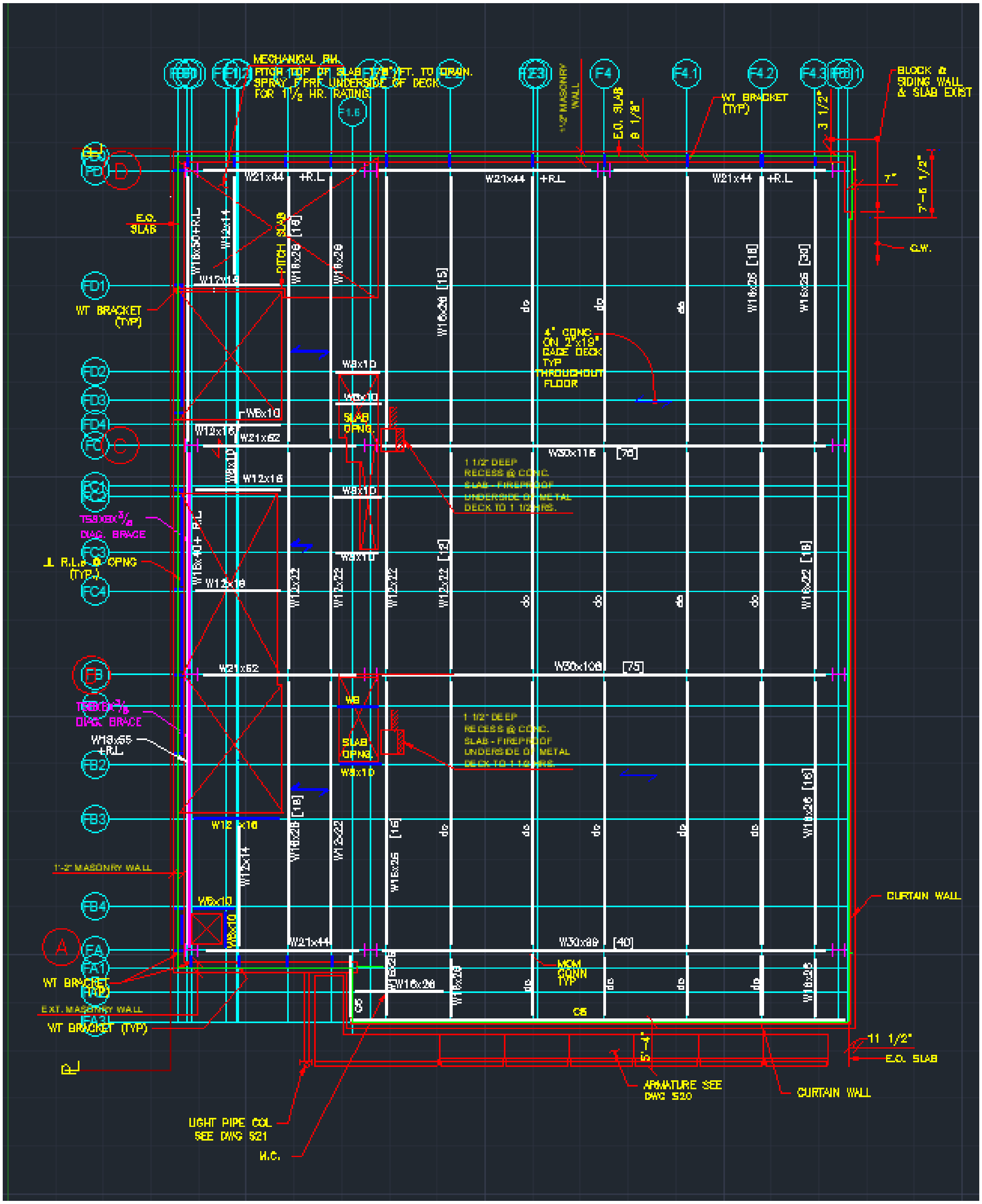
Fifth Floor AutoCAD Floor Plan



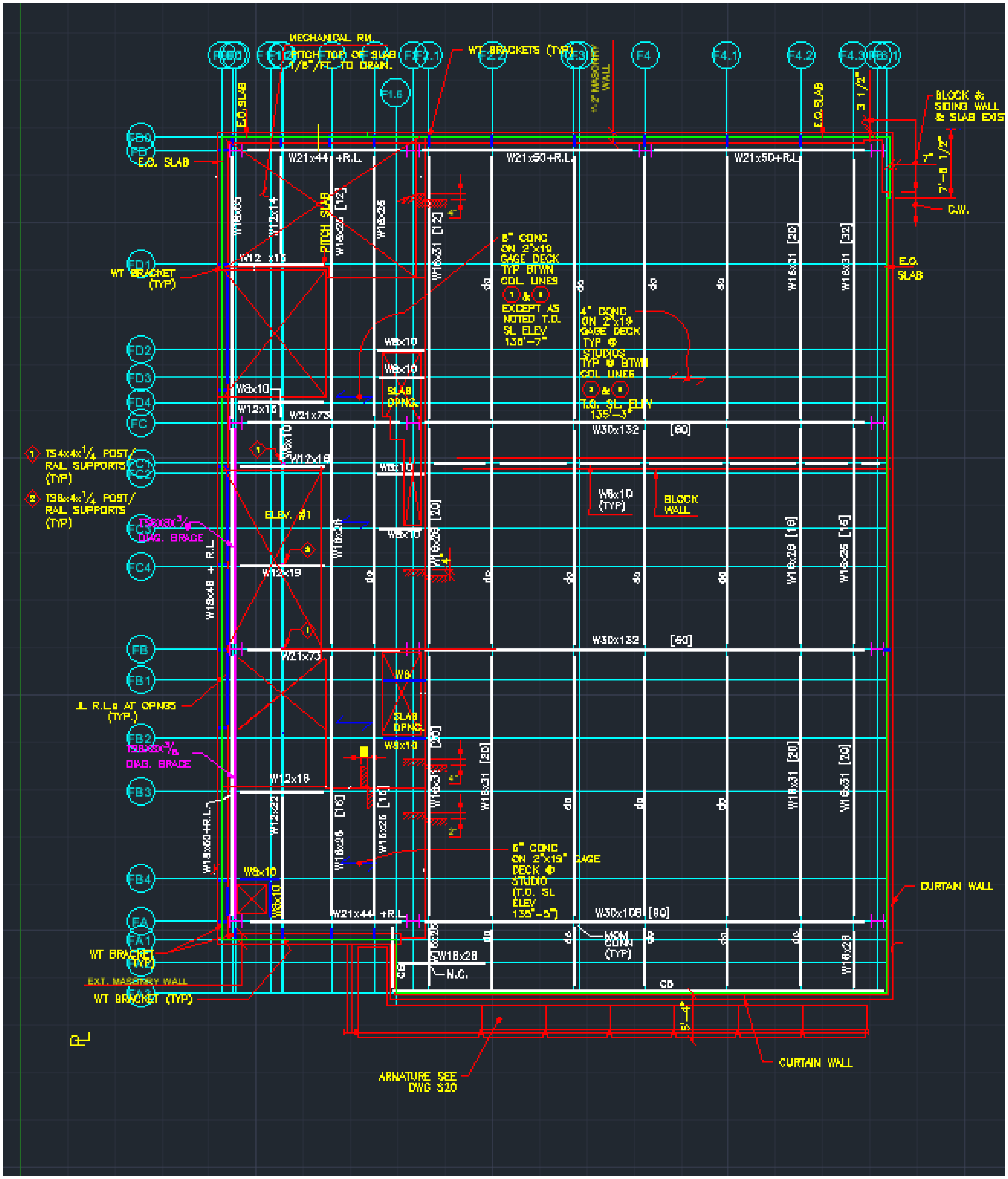
Sixth Floor AutoCAD Floor Plan



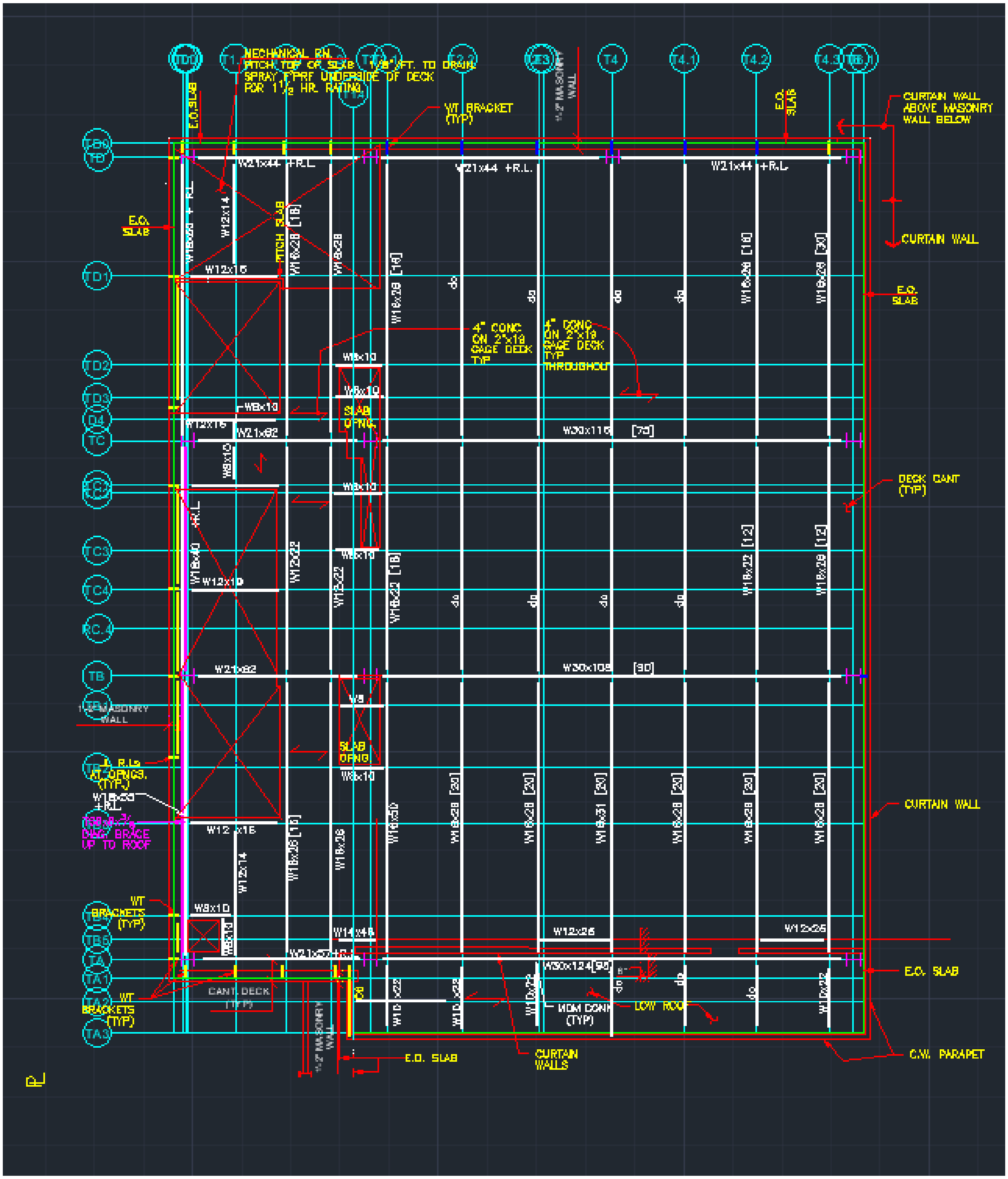
Seventh Floor AutoCAD Floor Plan



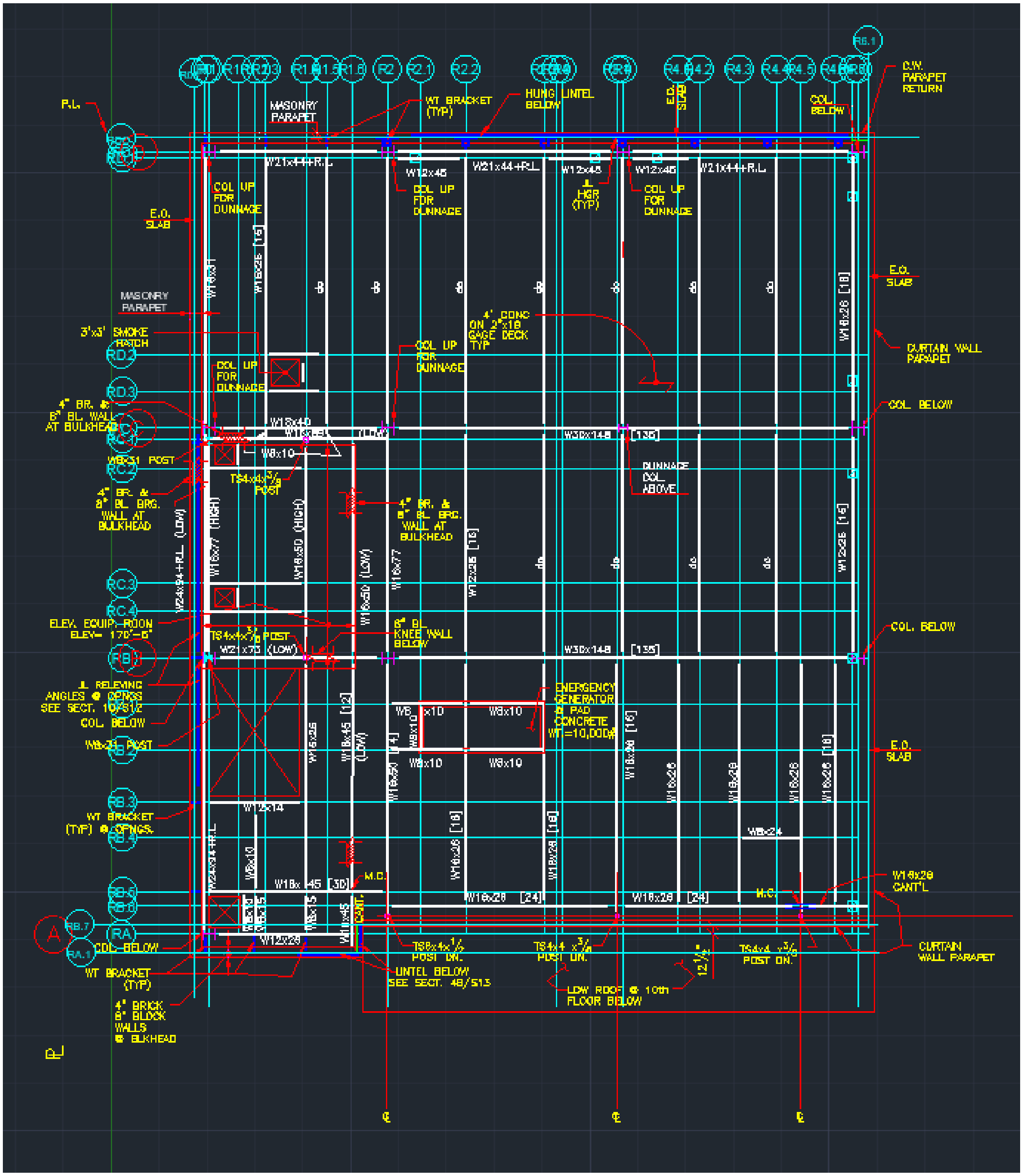
Eight Floor AutoCAD Floor Plan



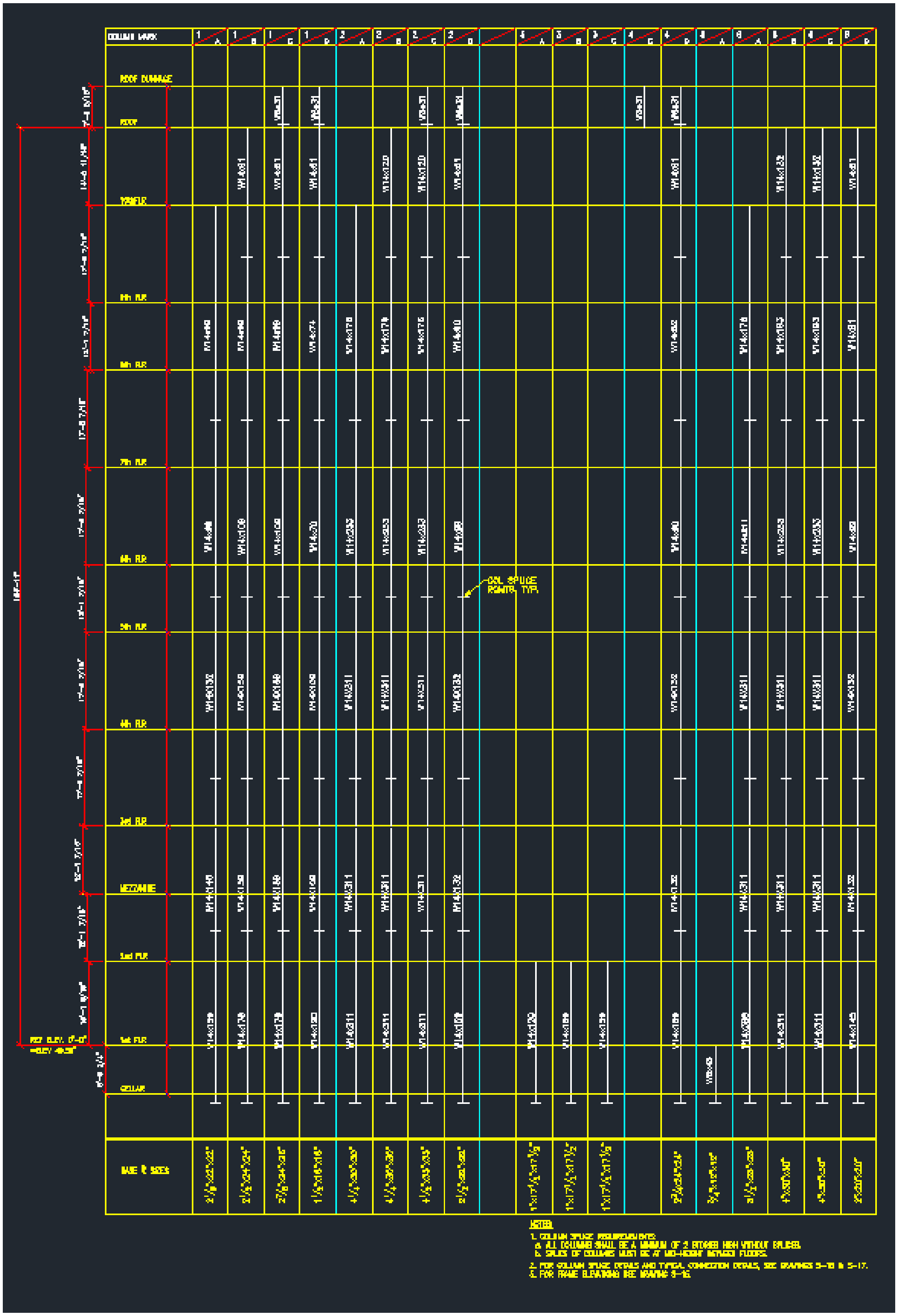
Ninth Floor AutoCAD Floor Plan



Tenth Floor AutoCAD Floor Plan

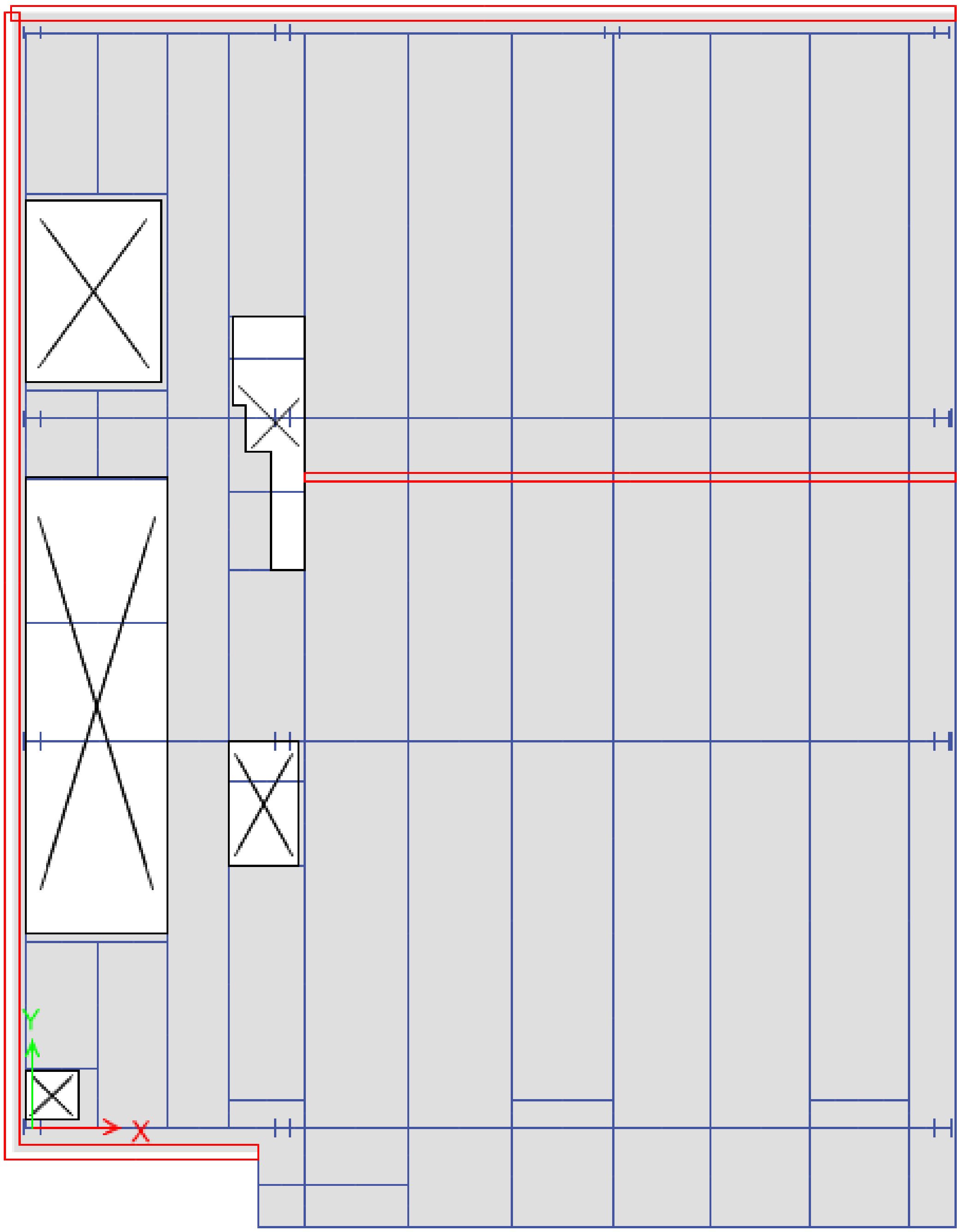


Roof AutoCAD Floor Plan

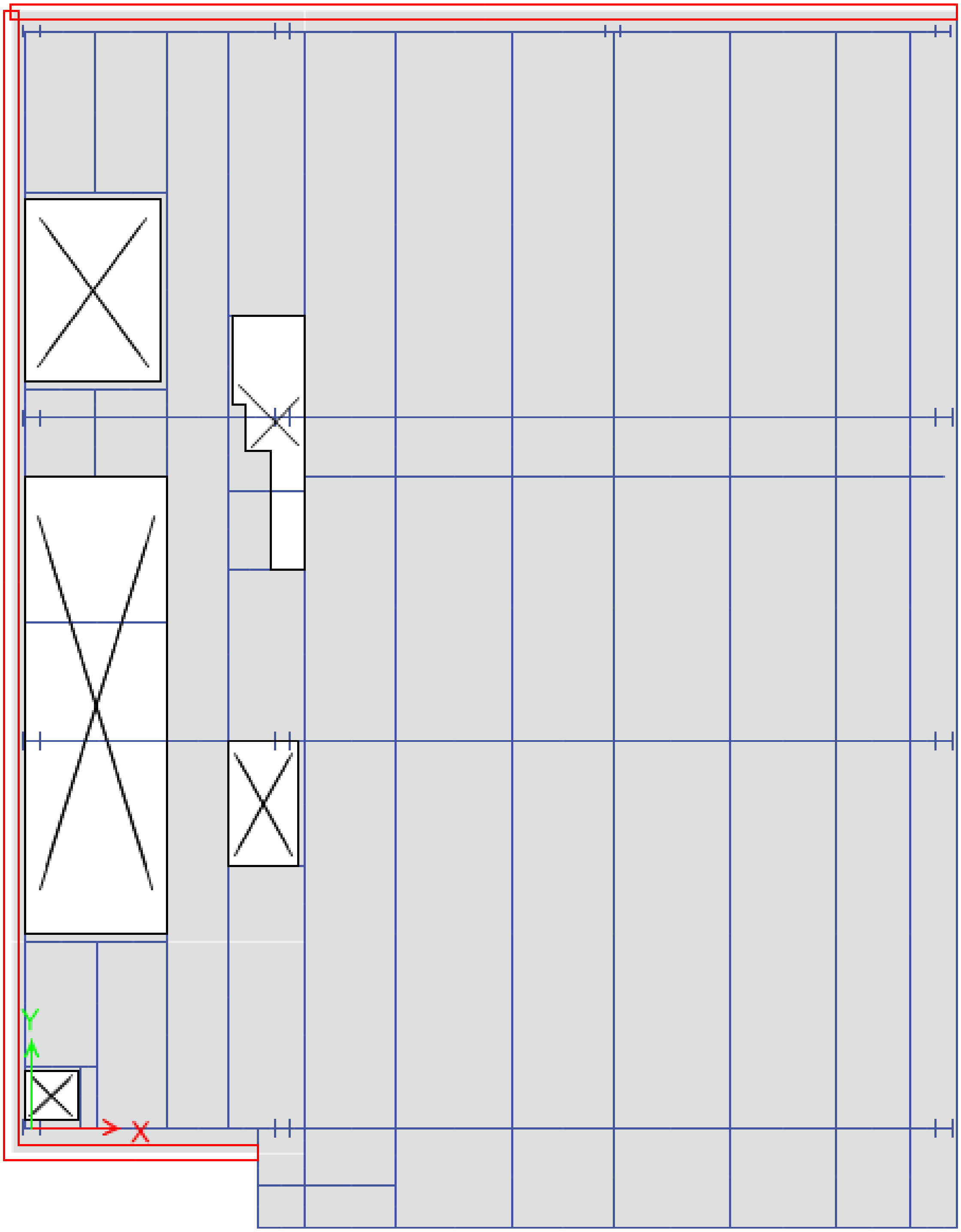


Column Schedule AutoCAD schedule

APPENDIX F: ELEMENTS IN THE MODEL ON THE 9TH AND 10TH FLOORS

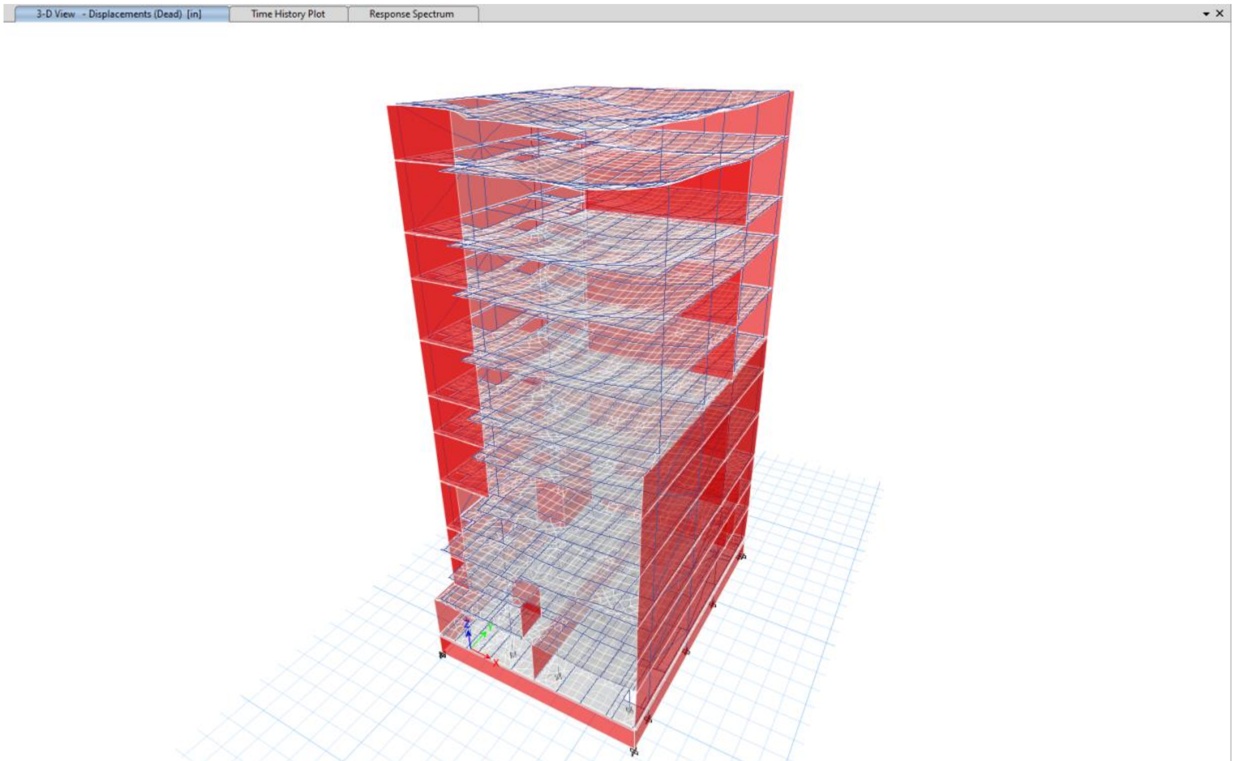


10th Floor

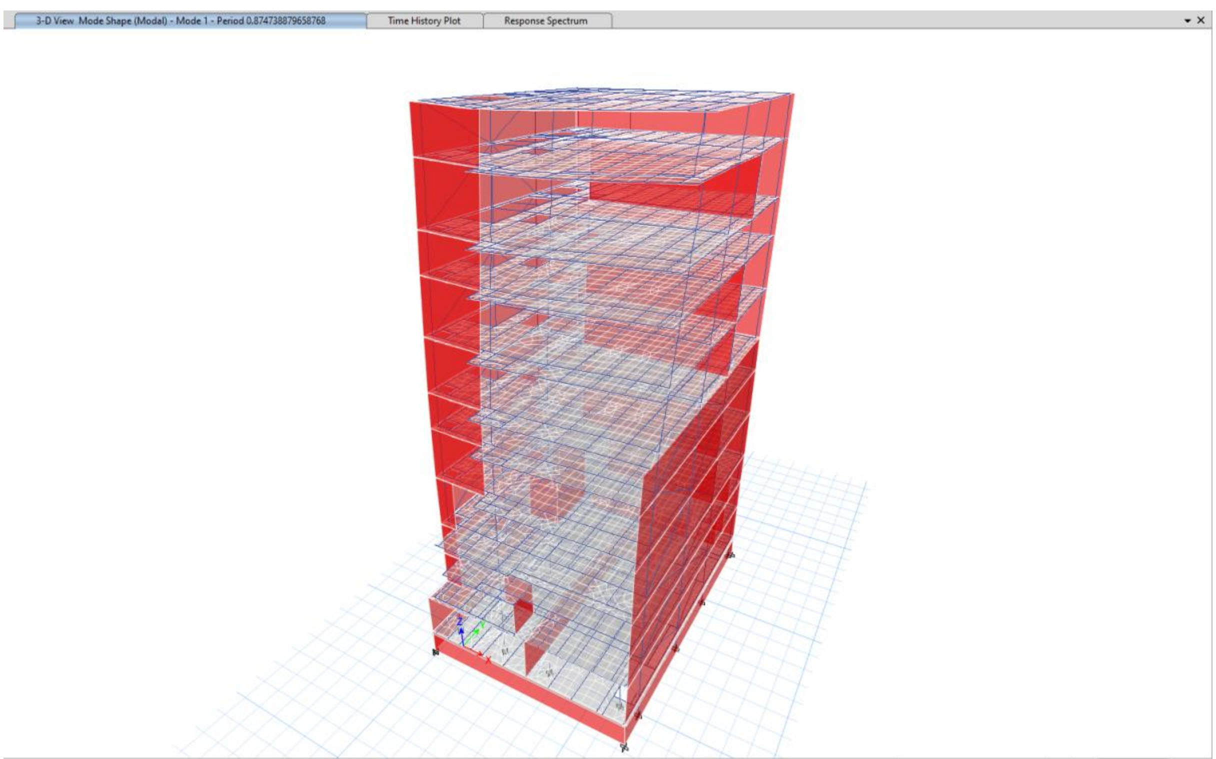


9th Floor

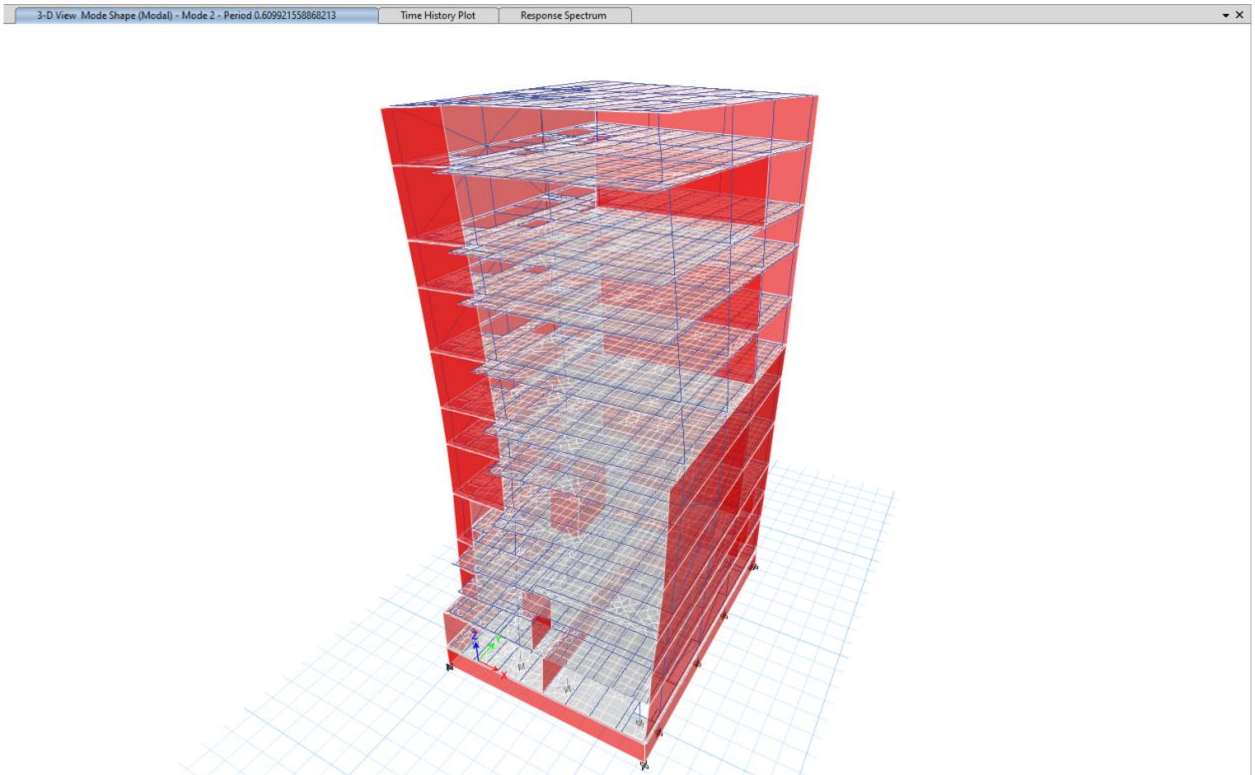
APPENDIX G: DEFLECTED SHAPES OF THE FINAL MODEL



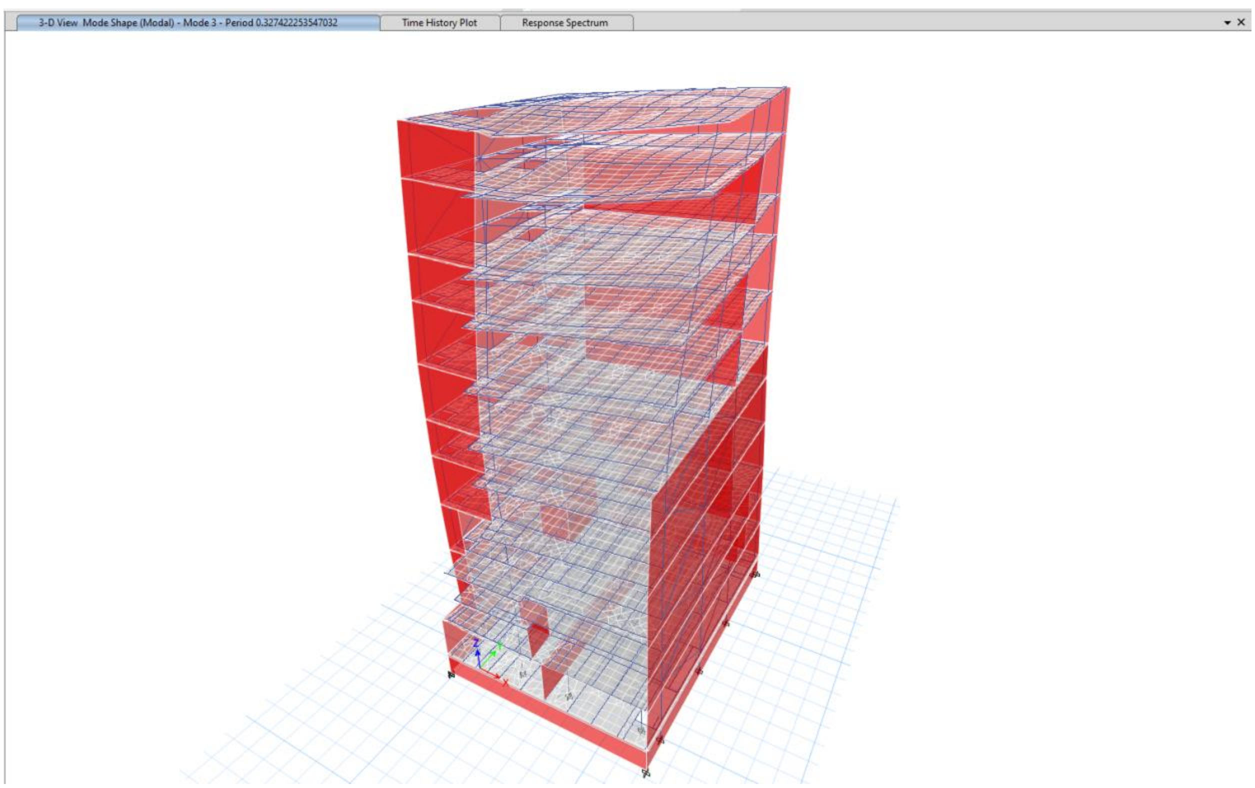
Dead Load Deflected Shape



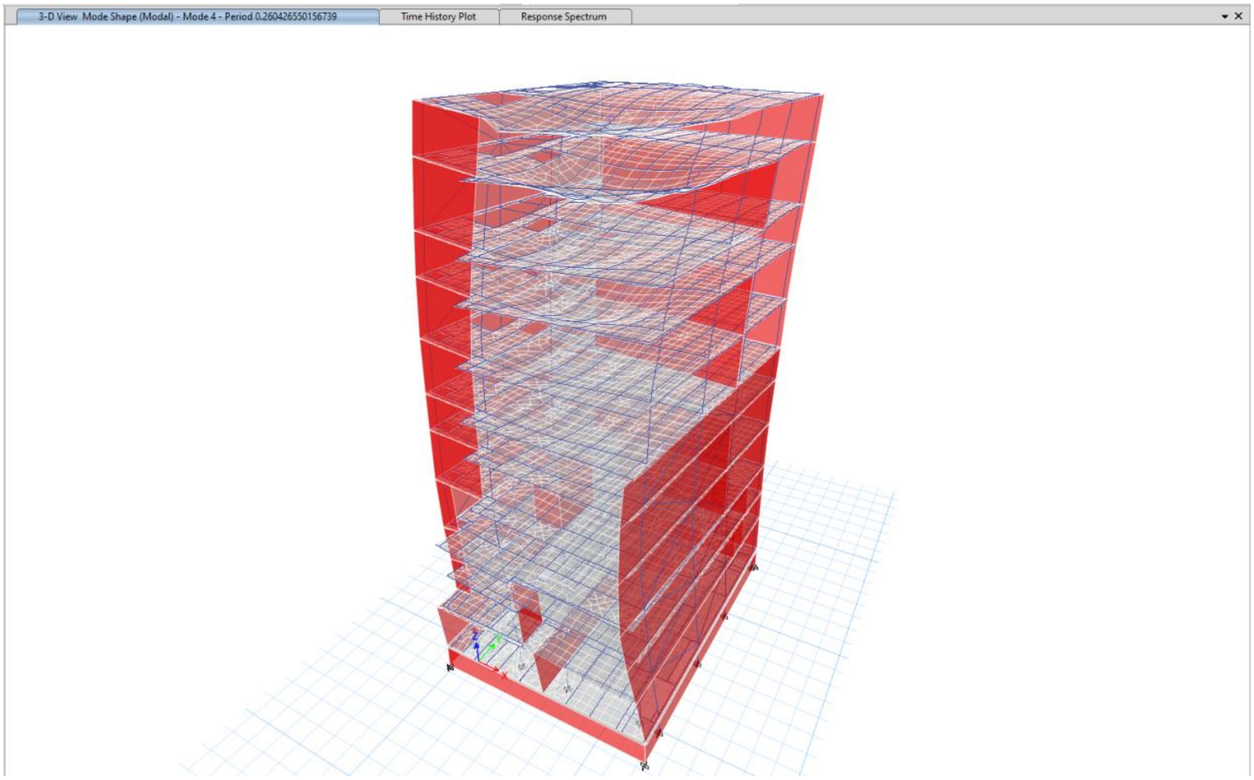
First Mode Deflected Shape



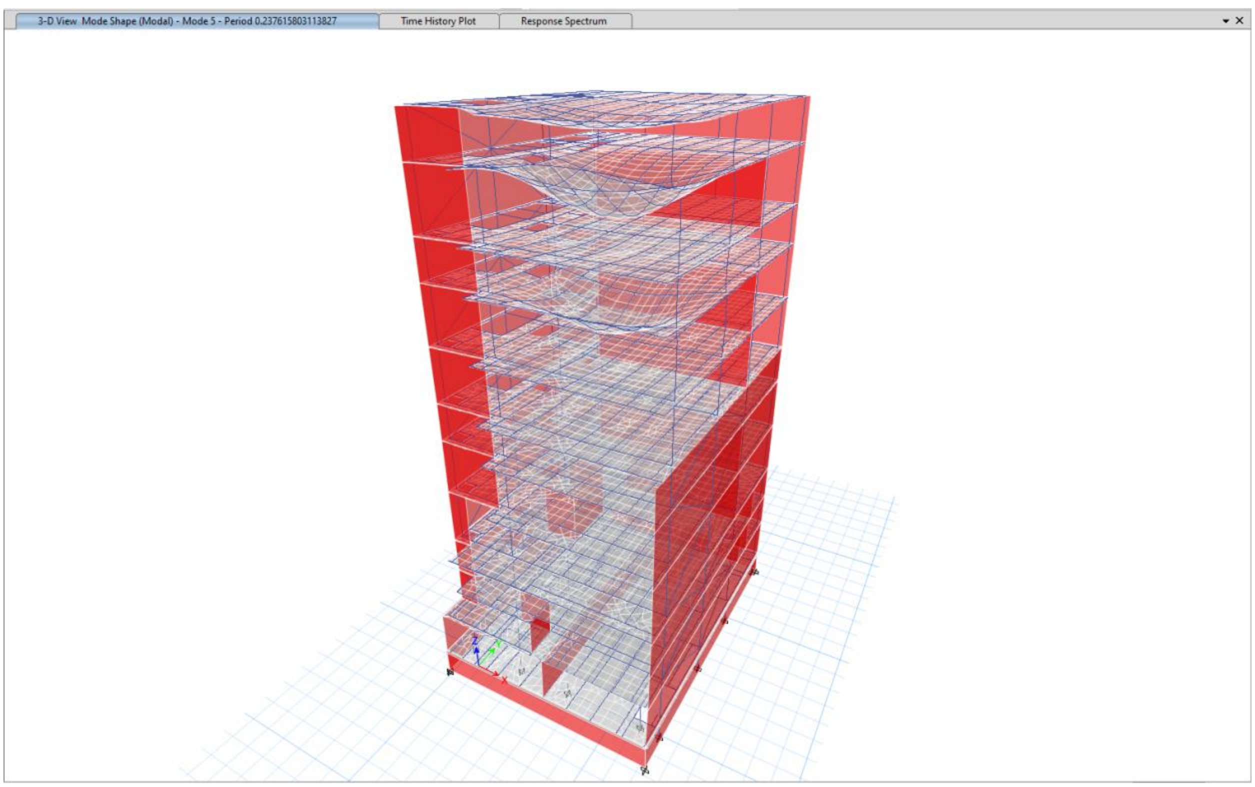
Second Mode Deflected Shape



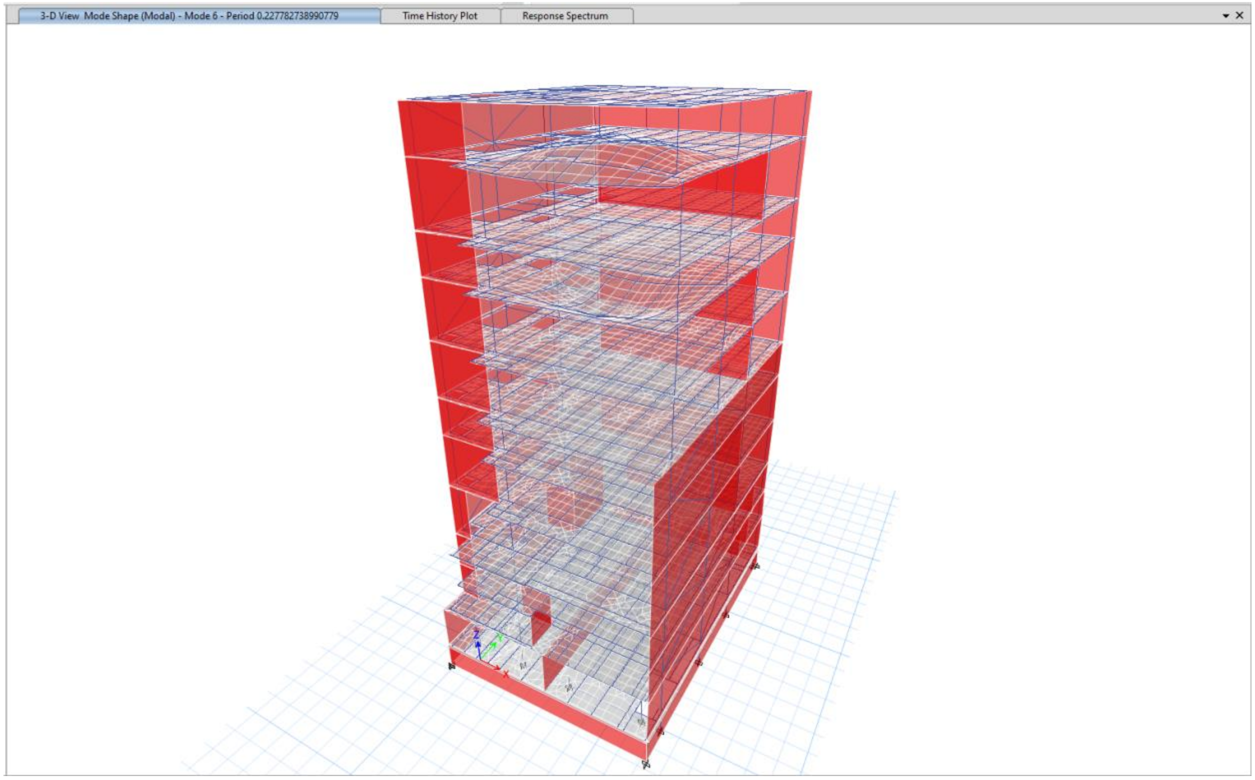
Third Mode Deflected Shape



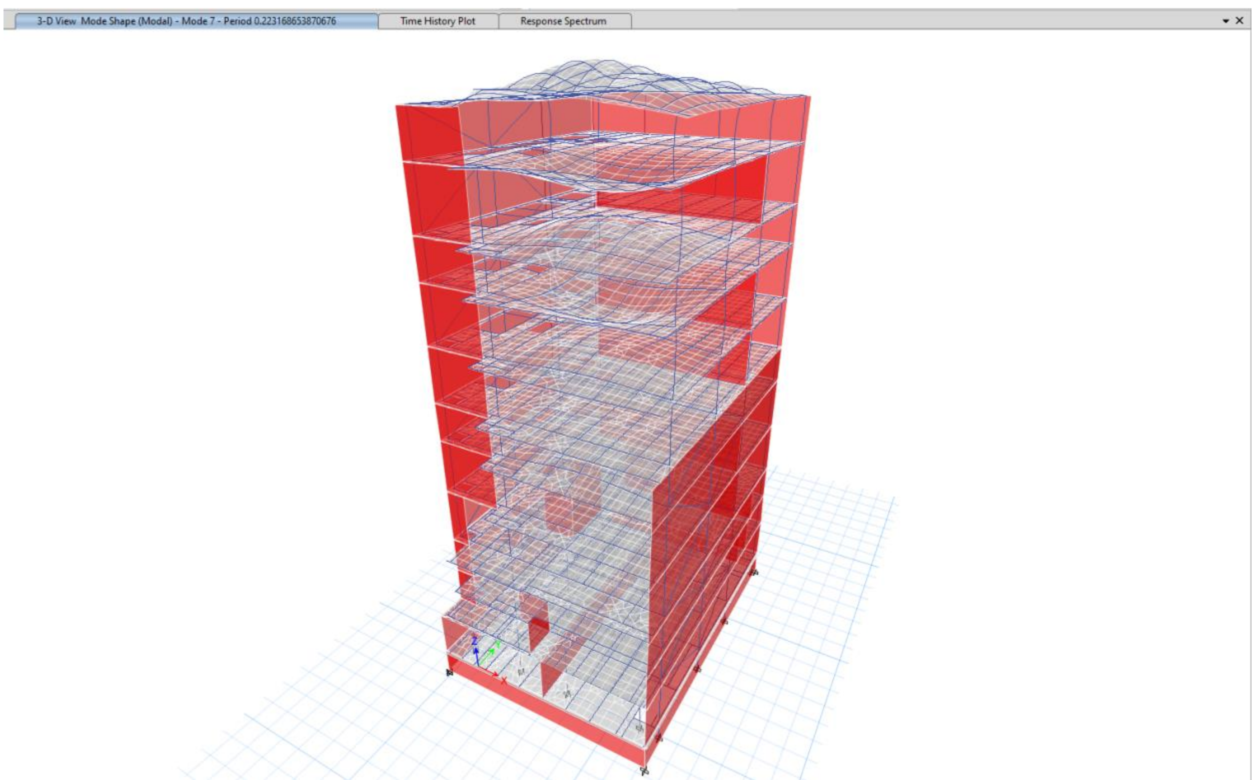
Fourth Mode Deflected Shape



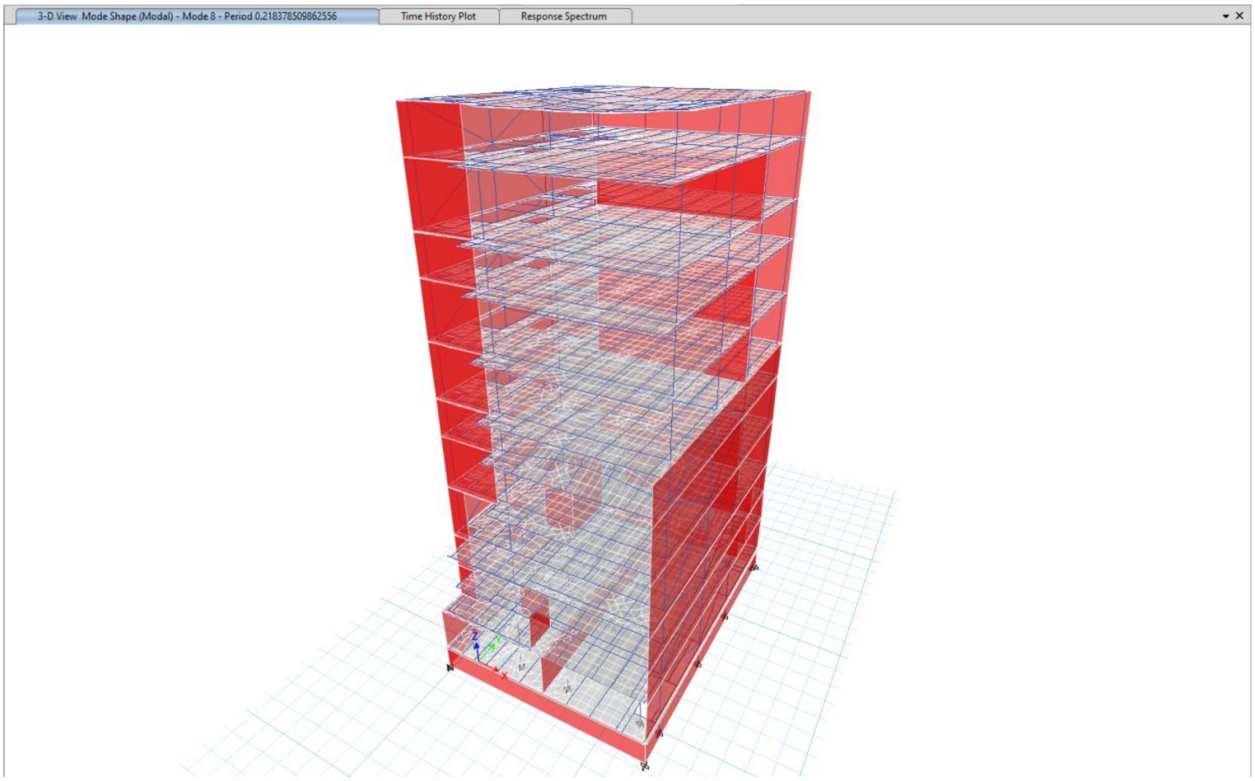
Fifth Mode Deflected Shape



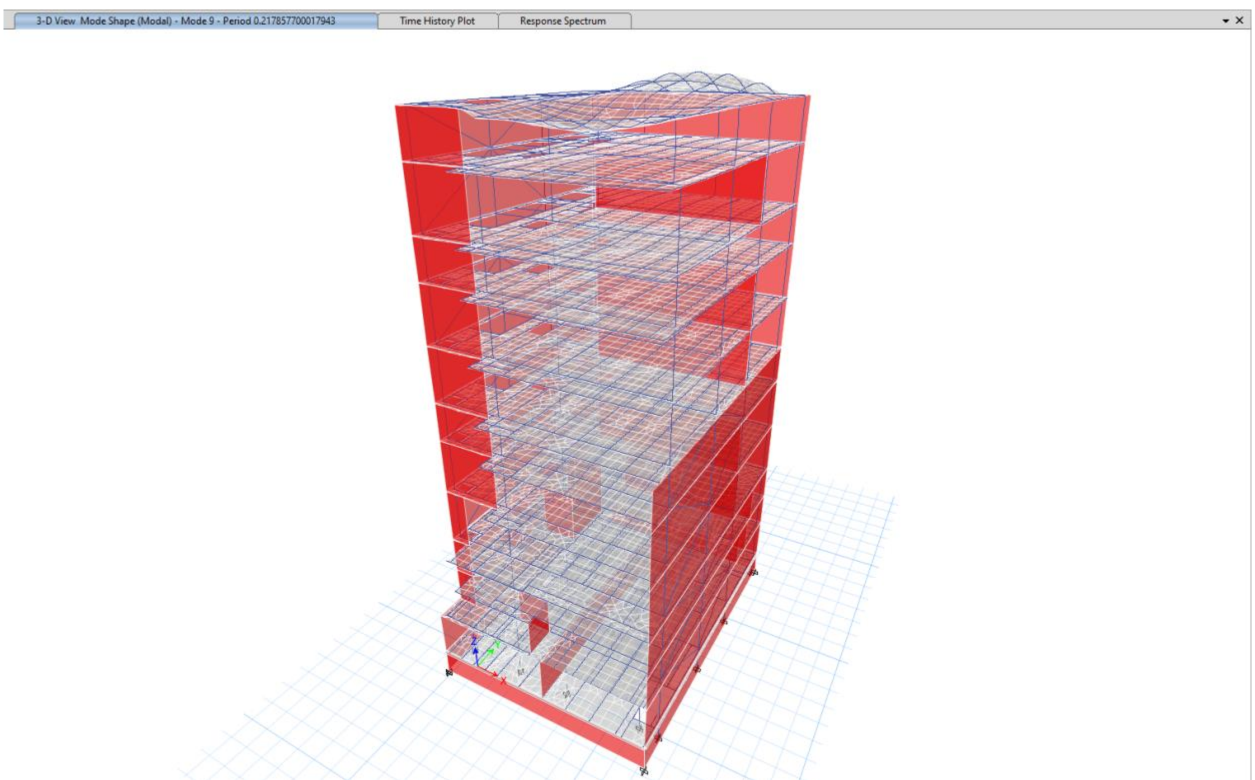
Sixth Mode Deflected Shape



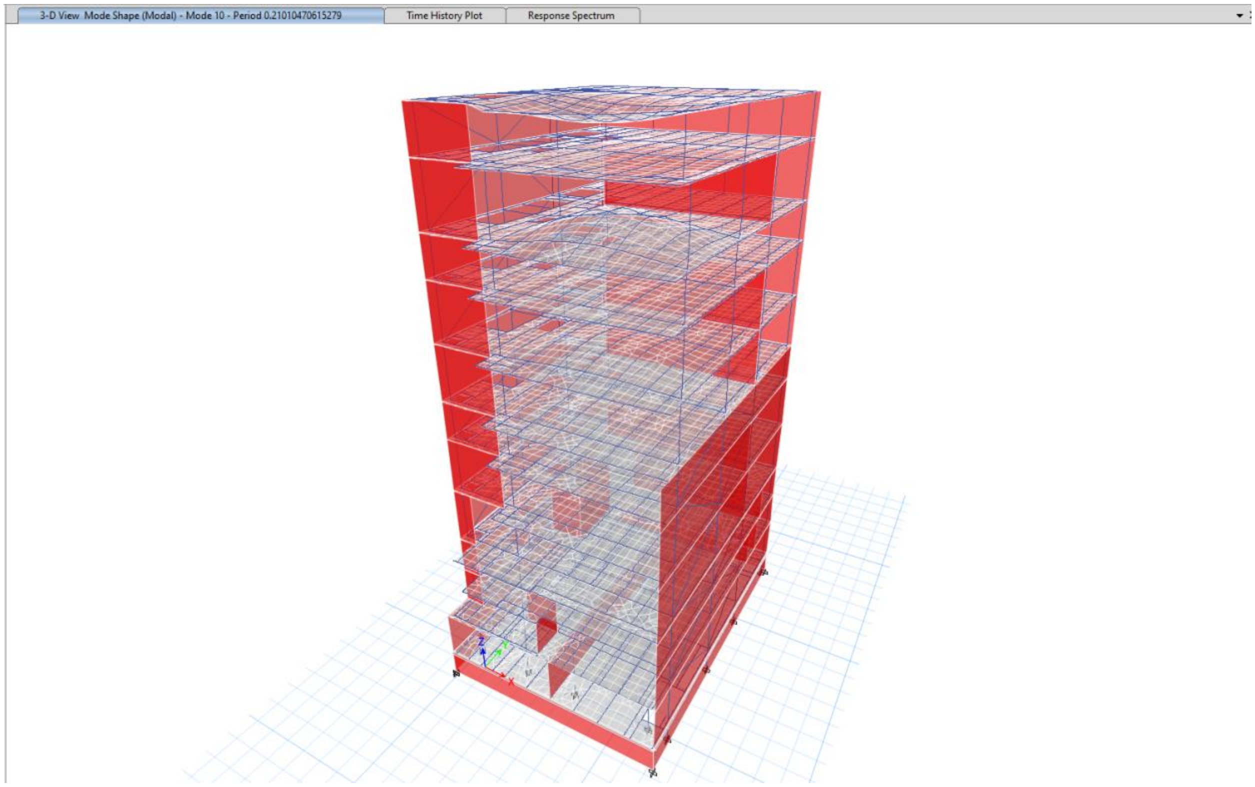
Seventh Mode Deflected Shape



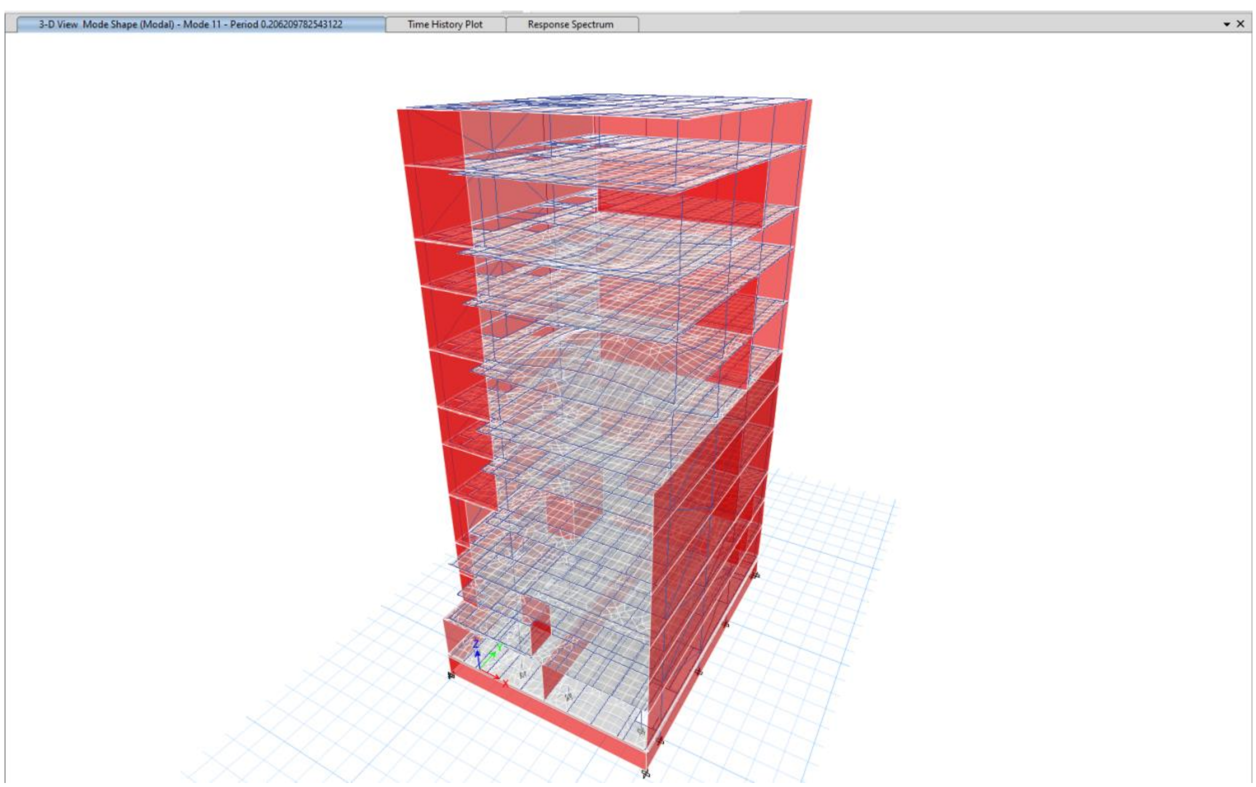
Eight Mode Deflected Shape



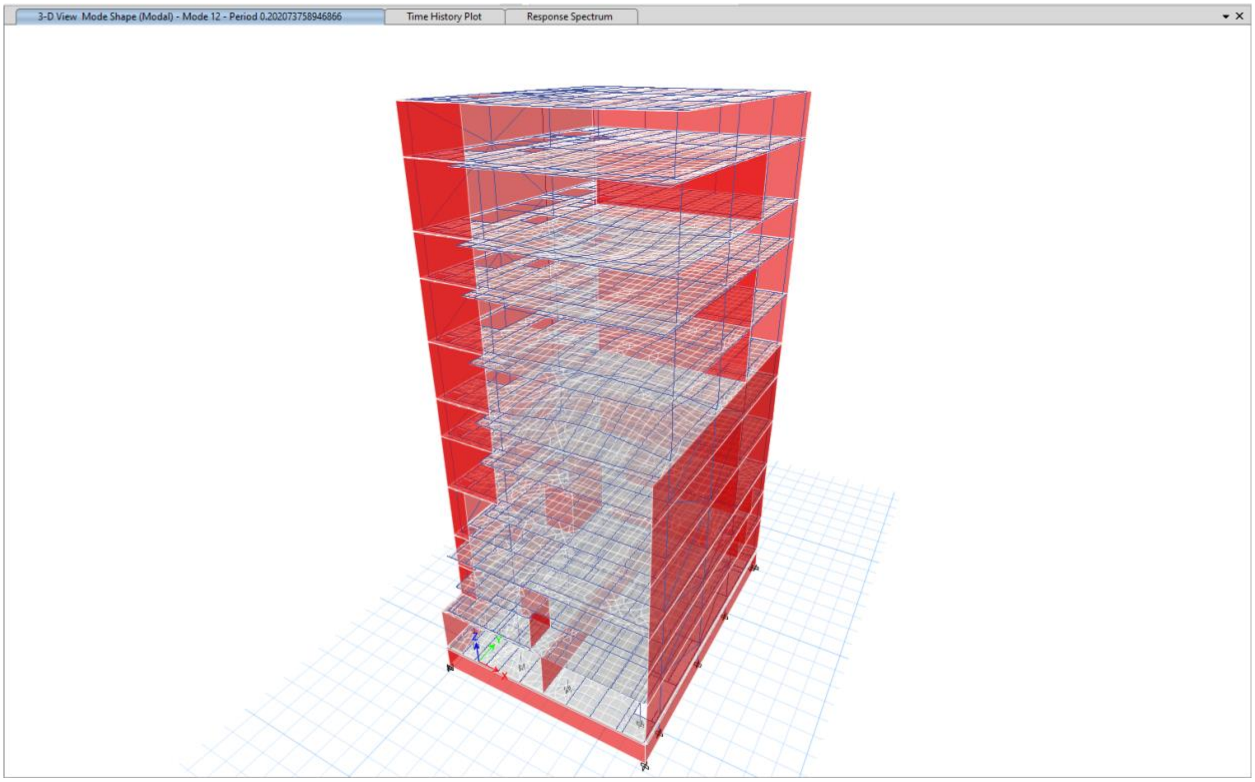
Ninth Mode Deflected Shape



Tenth Mode Deflected Shape

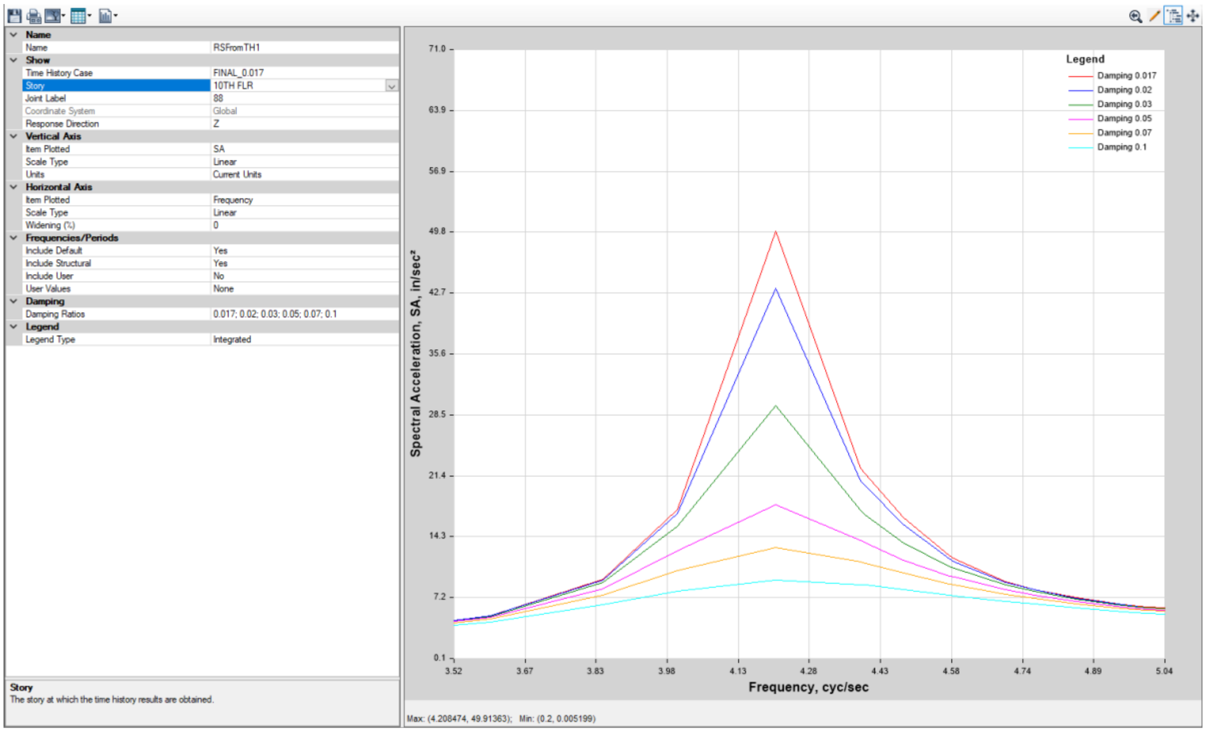


Eleventh Mode Deflected Shape

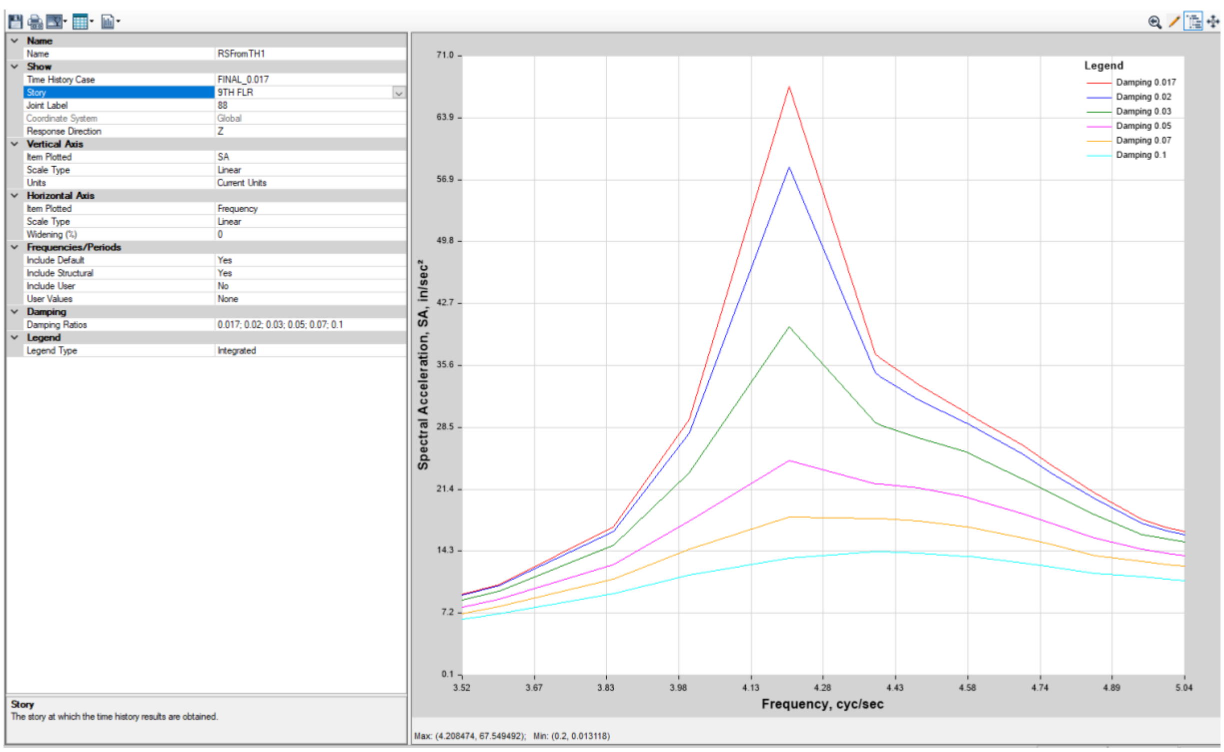


Twelfth Mode Deflected Shape

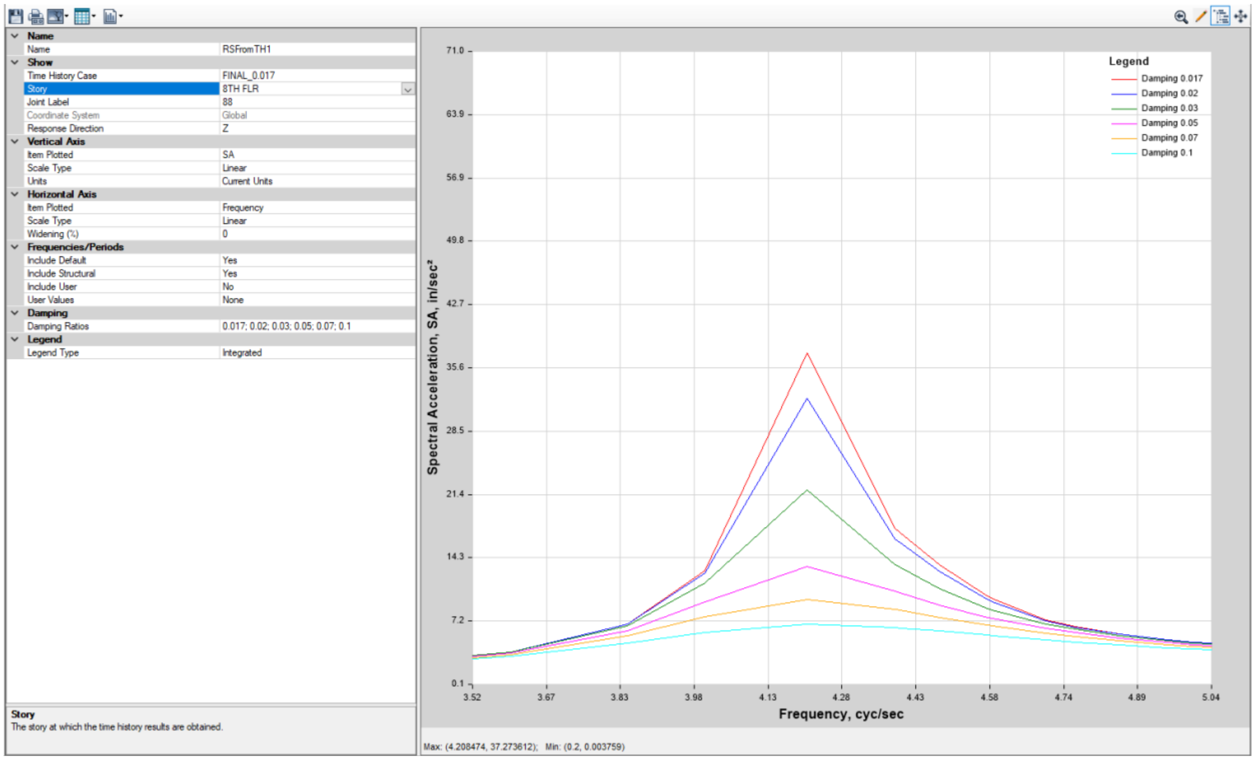
APPENDIX H: RESPONSE SPECTRUM CURVES OF THE FLOORS



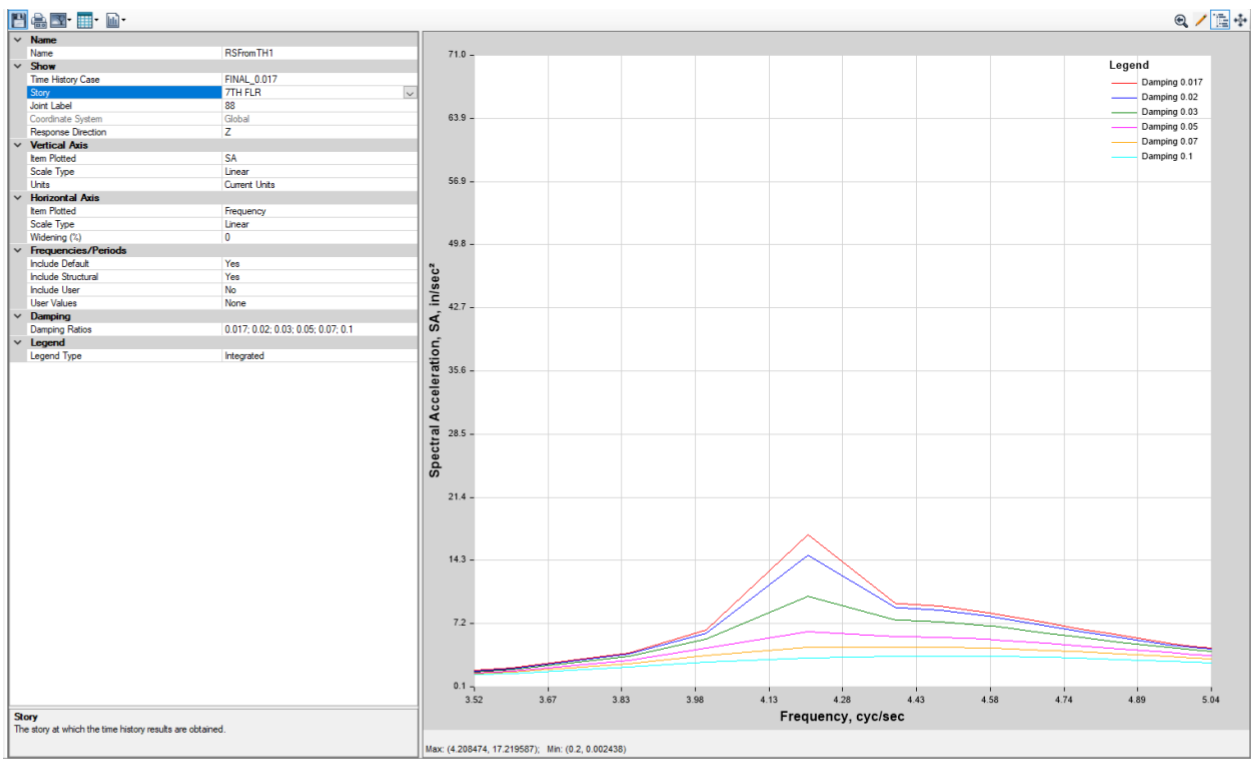
10th Floor



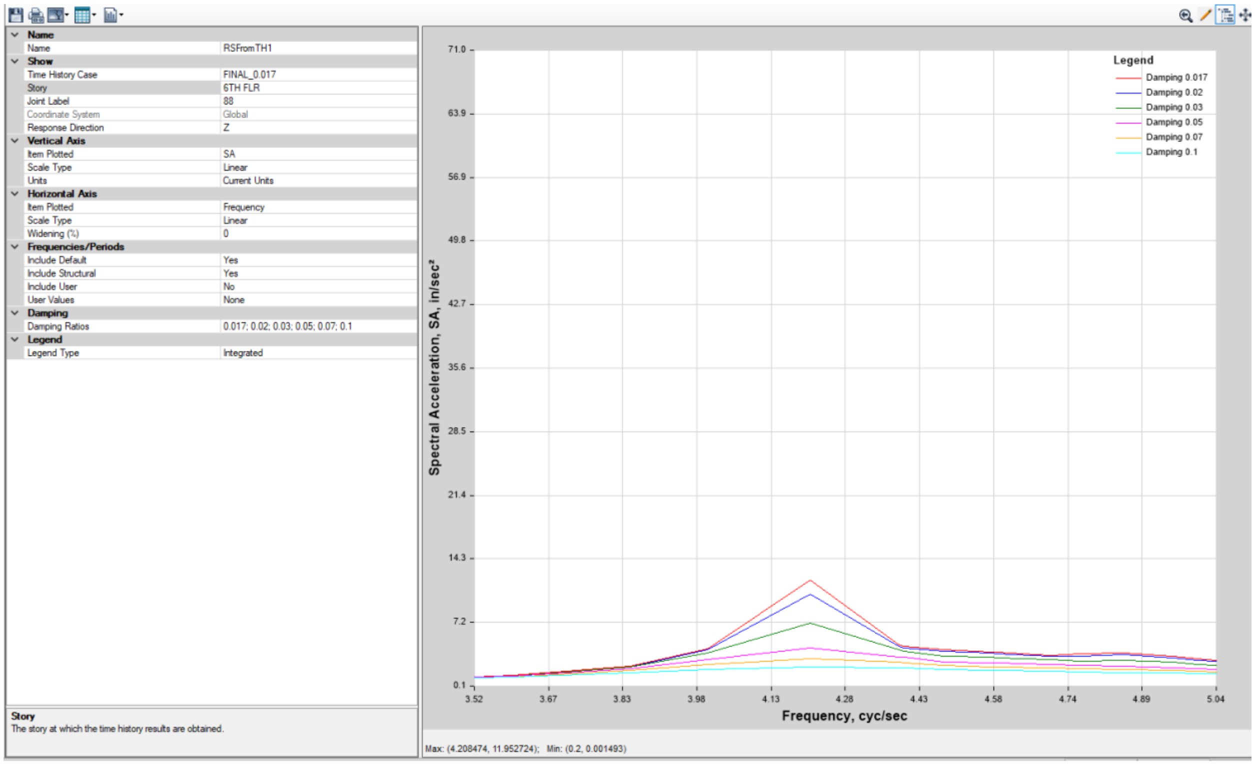
9th Floor



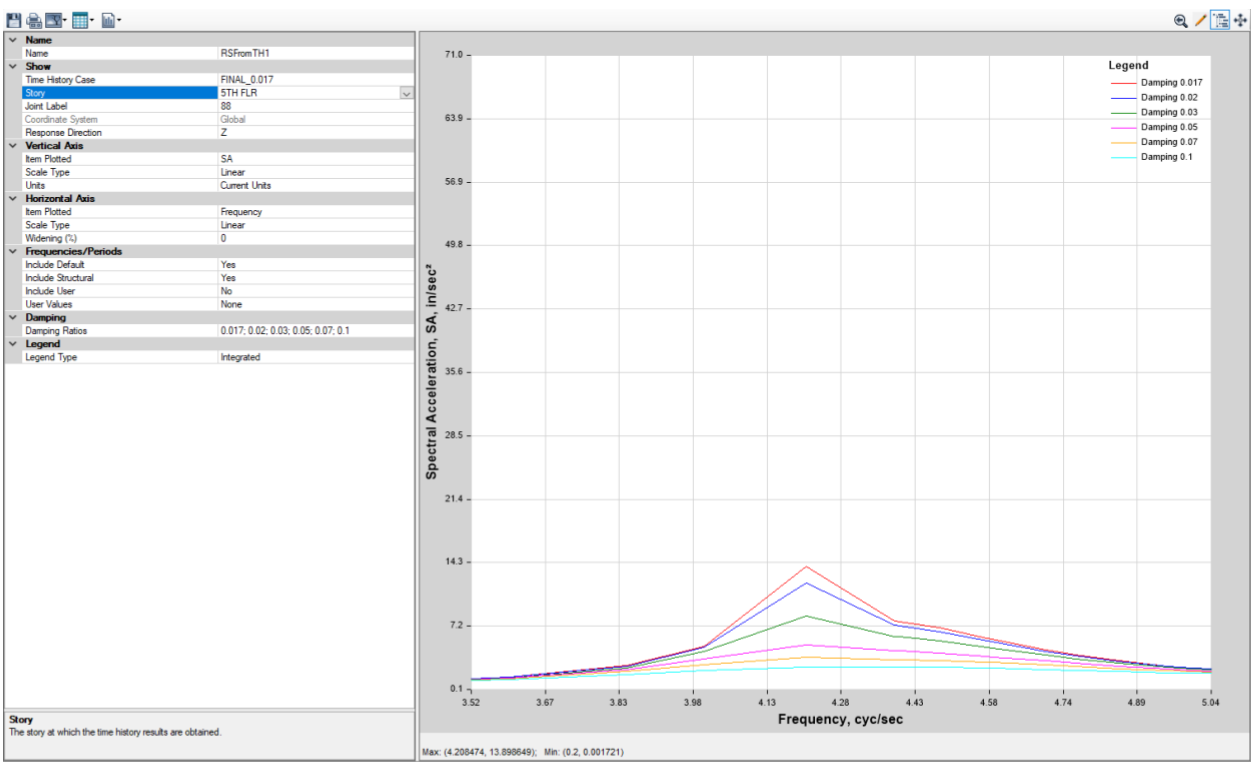
8th Floor



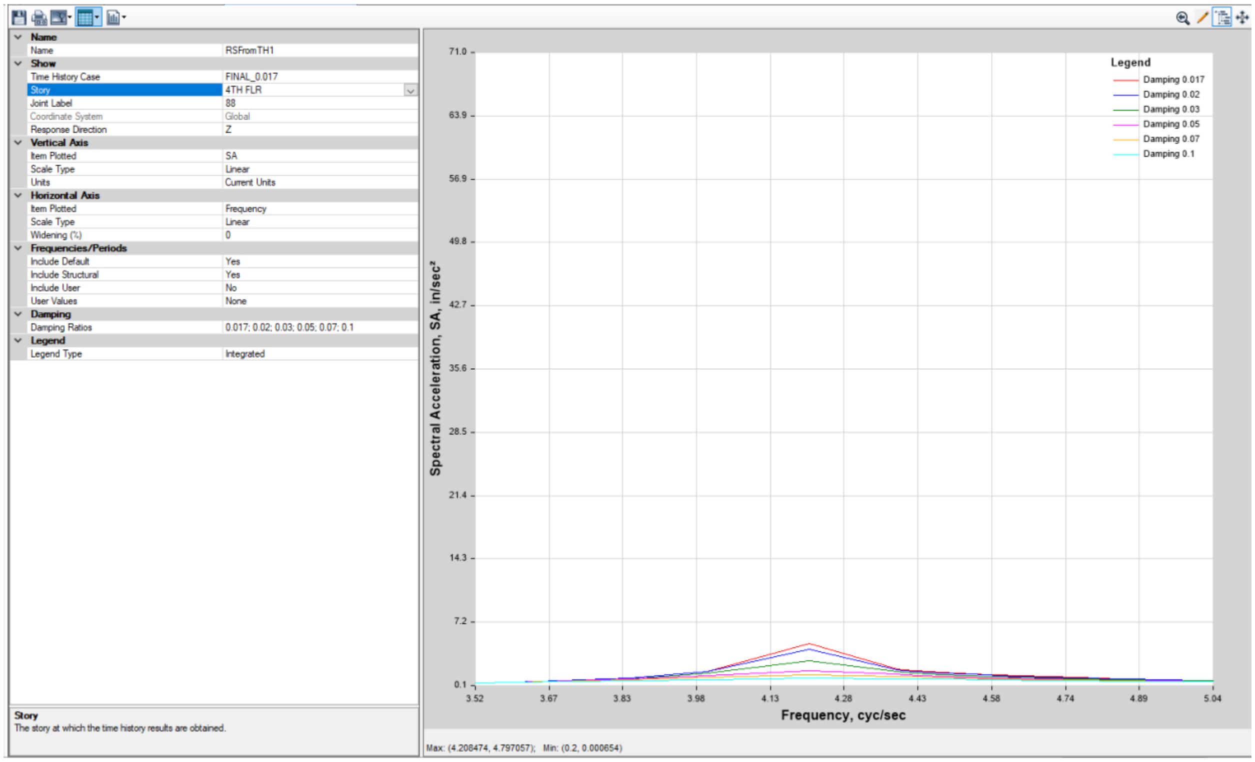
7th Floor



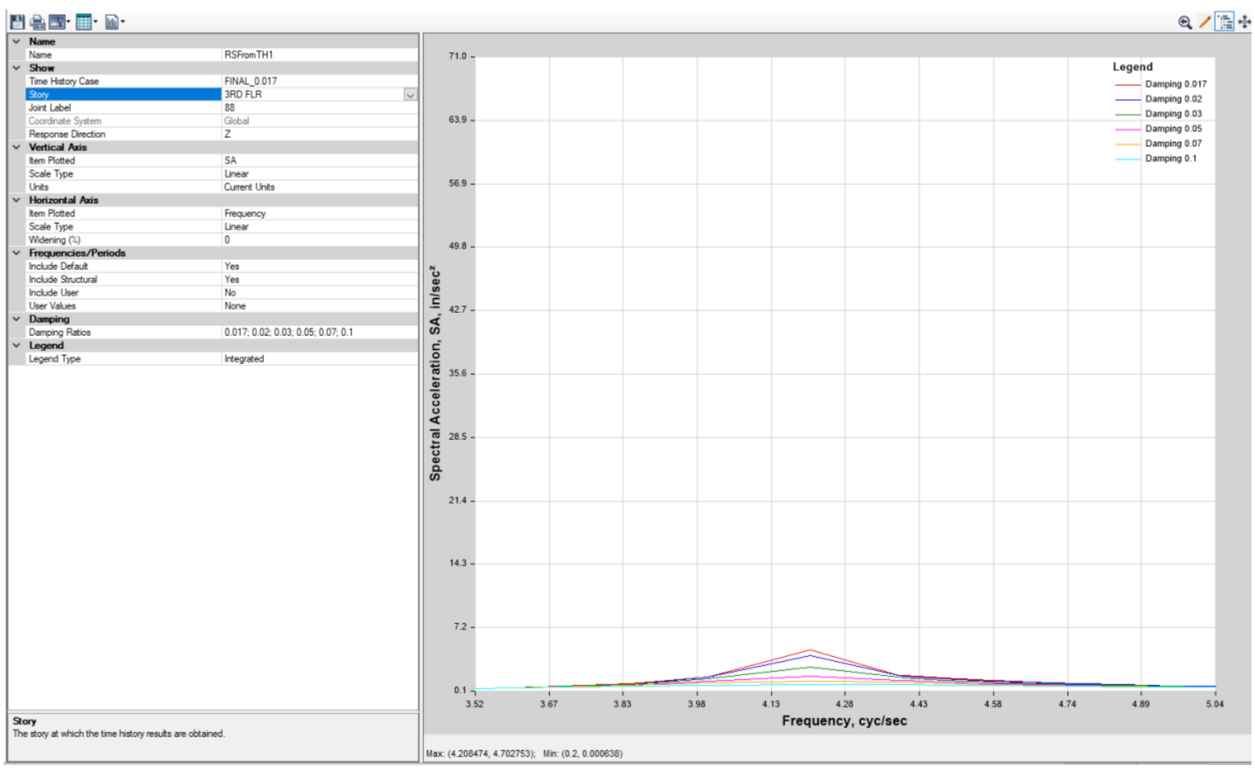
6th Floor



5th Floor



4th Floor



3rd Floor

ACADEMIC VITA

Kyle P. McKelvey

Education

Master of Science in Architectural Engineering; The Pennsylvania State University, University Park, PA
Integrated Undergraduate and Graduate Program; Anticipated Graduation: May 2022

Bachelor of Architectural Engineering, The Pennsylvania State University, University Park, PA
Schreyer Honors College; Anticipated Graduation: May 2022
China Study Abroad; Summer 2019

Passed Fundamentals of Engineering Exam (EIT Pending Upon Graduation); January 2021

Work Experience & Volunteering

LERA Consulting Structural Engineers, New York, NY — Summer 2021

- Designed concrete transfer beams for the second level of a multipurpose academic building that included the preparation of a structural calculation package for submittal
- Developed ETABS models for a bridge, boardwalk, and pergola for a project located in Central Park that were used for analysis of pile elements for foundation design

Jacobs, Arlington, VA — Summer 2020

- Preformed structural calculations for different load design criteria along with calculating member sizes by following various codes for steel and concrete, including codes specific to DoD buildings
- Modeled structures and created construction drawings within Revit along with ensuring coordination between all other disciplines within the BIM model
- Conducted submittal reviews of shop drawings to verify steel fabrication drawings

Researcher, University Park, PA — Nov. 2019 – Present

- Analyzing the effect rhythmic activity has on multi story structures and how to account for the dynamic response of the structure
- Assisted by Dr. Linda Hanagan with an extensive background in floor vibrations and serviceability

Teaching Assistant; AE 308: Intro Structural Analysis, University Park, PA — Fall 2020/2021

- Instruct students in the basics of structural design and analysis through practice problems and examples
- Grade assignments and provide feedback on work

IT Service Desk Supervisor, University Park, PA — Jan. 2019 – Nov. 2019

- Worked on the customer service side of the university's IT support, assisting all that have a Penn State account
- Supervised student workers across the different locations on campus as a Student Supervisor

Leadership & Involvement

SHO Time Leader and Mentor, Logistics Committee — Jan. 2018 - Present

- Lead 60 current students charged with introducing the 300 new Scholars to the university
- Organize and plan events throughout the year for the fast-paced three-day orientation program

Schreyer Ambassador — Apr. 2018 - Present

- Represent the college in events for prospective students, distinguished alumni, and donors
- Support staff in implementing and planning events throughout the year

School Activities

Alpha Rho Chi Inc., Professional Architecture Fraternity, University Park, PA — Jan. 2018 - Present

- Executive board member as *Worthy Estimator* where I oversee all of the financials for the chapter
- Managed the organization's house fulfilling the role of House Manager

Penn State Crew Club, University Park, PA — Aug. 2017 – May 2020

- Represented Penn State twice by competing at Head of the Charles, the largest regatta in the world
- Competed in regattas throughout the semester on weekends and practiced daily starting at 5:30am

Student Society of Architectural Engineers, University Park, PA — Aug. 2017 – Present

Structural Engineers Association of Penn State, University Park, PA — Aug. 2020 – Present

Skills

Revit • AutoCAD • Sketchup • Adobe Photoshop and Illustrator • Microsoft Office • ETABS • Mathcad