Precast concrete diaphragm design by ASCE 7-16

by

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A REPORT

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MASTER OF SCIENCE

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Abstract

Floor and roof diaphragms are an important aspect of a building's structural lateral system. The diaphragm resists in-plane seismic loads and transfers them to the vertical lateral force resisting system. If they are not properly designed, the failure could be devastating. After the 1994 Northridge earthquake the failures of multiple precast structures were investigated, and it was determined that inadequate diaphragm design was the cause of the failures. It was found that previous code provisions underestimated the diaphragm design forces. Additionally, diaphragms were assumed to behave elastically during seismic events, but it was observed that a large earthquake could cause inelastic behavior of diaphragms.

Code provisions prior to the ASCE 7-16 determined diaphragm design forces based solely on the fundamental modal response of structures. However, further research proved that higher modes also influenced diaphragm forces. Therefore, a change was warranted to diaphragm design provisions in the ASCE 7-16. The diaphragm design provisions in the ASCE 7-16 ensure adequate ductility and strength of diaphragms and provides different options for elastic or inelastic diaphragm design. Not all design options are applicable and permitted for every scenario. The selection of the design option will depend on the seismic design category, the geometry of the diaphragm, and the number of stories of the building. The new diaphragm design provisions in the ASCE 7-16 are required for precast diaphragm design.

This report provides a detailed design procedure for precast diaphragms and briefly discusses the research done on precast diaphragms that led to the provisions in the ASCE 7-16. In addition, a parametric study was conducted to illustrate how the design options change the diaphragm design forces and required reinforcement.

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Chapter 1: Introduction

1.1 Precast Concrete Diaphragms

Structural diaphragms are the horizontal components of a building's lateral force resisting system. The diaphragm plays a key role in a structure's lateral performance and response. Diaphragms should have enough strength, stiffness, and ductility to resist in-plane seismic loads as well as safely transfer loads to the vertical lateral force resisting system (Rodriguez et al., 2002). In a building, the structural floor or roof system will act as the diaphragm. A precast diaphragm system consists of concrete shapes or members cast off site and specified through a precast manufacturer. Examples of precast members include precast double-tee beams and hollow-core planks, see Figure 1.

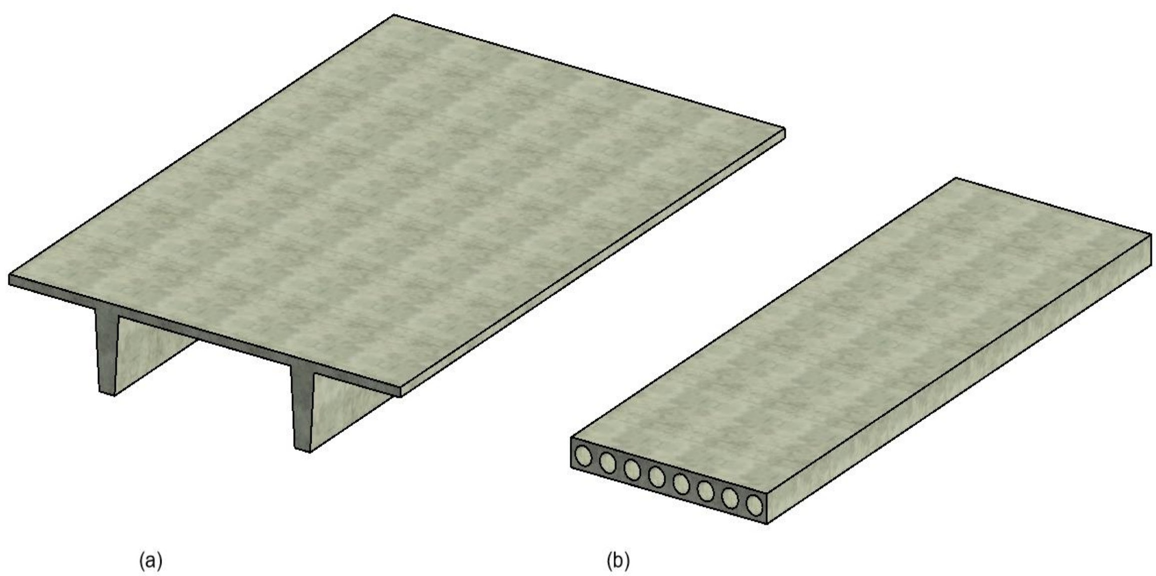

Figure 1: (a) Precast double-tee (b) Precast hollow-core plank

Furthermore, precast systems can be classified as topped or untopped. Untopped precast systems use a typical shape or member without a cast-in-place slab. On the contrary, topped precast floor systems have a cast-in-place concrete slab poured on the top of the precast members. Regardless of the topping of the system, the joints and connections between the precast units are the weak points in the system and affect the flexibility and capacity of the diaphragm (Zhang, 2011). Knowing whether a topped or untopped system is being utilized is crucial when designing a diaphragm because different systems will behave differently under a lateral load. An untopped precast system will rely on the precast units and the connection between the units for diaphragm action, whereas a topped precast system will utilize the topping slab for the diaphragm action. However, if the system is detailed where the topping slab works compositely with the precast members, both the precast members and the topping slab will resist the lateral forces.

1.2 Past Code Diaphragm Design

In codes prior to the *ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, seismic design practice for precast structures was based on the Equivalent Lateral Force procedure. If the elements are detailed correctly, this procedure assumes that vertical elements of the Lateral Force Resisting System will be loaded beyond the elastic range and into inelastic behavior under design and maximum considered earthquakes. However, in common design practice, the diaphragm is assumed to remain in the elastic range and will not be detailed to behave inelastically (Fleischman et al., 1998).

The *ASCE 7-10 Minimum Design Loads for Buildings and Other Structures*, which is the design code prior to the ASCE 7-16, provides few provisions for which the diaphragm should be designed. These provisions include the chord design of the diaphragm, as well as the collector design. The provisions provided were universally permitted to be used for diaphragms of any material. Diaphragm design was often overlooked and taken for granted. However, after a major earthquake in 1994 in the San Fernando Valley region in California (known as the Northridge earthquake) resulted in the failure of multiple precast concrete structures, structural researchers and practicing engineers investigated the failures in the components of these precast structures. After investigating the cause of failure, researchers agreed that further development of the design provisions for precast diaphragms was warranted (Nakaki, 2000). With the need for improved precast diaphragm provisions, research began to take place from the early 2000s on the behavior and strength of precast concrete diaphragms. Research has shown that diaphragm forces are underestimated with code provisions and diaphragms will behave inelastically during large seismic events (Fleischman et al., 1998; Rodriguez et al., 2002). According to investigation and research by Fleischman and his team, this was the cause of several precast structural failures during the 1994 Northridge earthquake (Fleischman et al., 1998).

1.3 Diaphragm Internal Forces and Design

Standard practice for diaphragm design involves the beam analogy method. With this method, the diaphragm is idealized as a deep beam resisting the lateral loads on the structure. The chord elements of the diaphragm are designed to withstand the resultant couple from the applied lateral load. This method has been widely accepted for diaphragm design for many years. However, further research has shown that the resultant design can be uneconomical and inaccurate when observing the forces in the connections between precast panels (Fleischman, 2014). Fleischman and his team found that the free body method is a more accurate way to find the forces acting on the precast diaphragm joints. The free body method takes a cut of the structure at each panel joint and develops a free body diagram for each cut. The designer then determines the forces acting on the joint from the applied lateral loading by using basic statics so that the forces are in equilibrium.

1.4 Scope

This report will focus on the seismic provisions for precast diaphragm design found in the ASCE 7-16. Along with a brief review of the previous codes and the research which led to the newest code, a design example will be provided comparing the new code requirements to the previous version of the code. The report will also use the free body method to determine the internal forces acting on the precast joints in the design example. The connections between precast members and the reinforcement required in the topping slab are compared for different design options by the code. The derivation of factors to amplify the diaphragm design force of previous codes is outside of the scope of this report. Additionally, the free body method example is only presented for the layout chosen in the design example.

Chapter 2: Literature Review

2.1 Background

It is worth noting that although much research has been conducted on structural responses during seismic events over the past 20 years, diaphragm design provisions in the code were not significantly changed during that time period until the publication of the ASCE 7-16. After the Northridge earthquake in 1994, insufficient diaphragm design was determined to be the cause of severe damage and collapse of multiple precast structures (Fleischman et al., 1998). With this information, researchers began to investigate diaphragms and their behavior. It was found that horizontal floor accelerations were amplified with respect to the ground acceleration on different levels of failed precast structures (Hall, 1995). These large accelerations observed during large seismic events are attributed to higher mode effects (Rodriguez et al., 2002). The amplification of floor accelerations observed by Hall after the 1994 Northridge earthquake is unaccounted for in the building design codes prior to the ASCE 7-16 and therefore a change to the code provisions was warranted. This report will cover the research that developed a more accurate precast diaphragm design approach and include the proposed and adopted equations for the alternative design provisions for diaphragms including chords and collectors from *ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*.

Many ideas from this report were obtained through the literature published within the Diaphragm Seismic Design Methodology Project (DSDM). The DSDM project is part of a multi-university research project to develop an industry-wide standardized precast diaphragm design procedure. The DSDM research was initiated, guided, and co-funded by the Precast/Prestressed Concrete Institute (PCI). Additional funding was provided by the National Science Foundation (NSF) and the Charles Pankow Foundation (CPF)(Fleischman, 2014).

2.2 Previous Code Diaphragm Design

2.2.1 Elastic Design Assumption for Diaphragms

In previous design practice, the diaphragm was designed to remain elastic while the vertical elements of the LFRS were designed to behave inelastically during design and maximum level seismic events. To design the vertical elements for inelastic behavior, the response modification factor (R) for the chosen system must be identified. This factor is based on the inelastic capacity of the vertical element of the LFRS with respect to the system's ductility, strength, and toughness. During a seismic event, the vertical LFRS is assumed to have an inelastic behavior through energy dissipation and post-yield deformation capacities (Precast/Prestressed Concrete Institution, 2010). In previous code provisions and design practice, the diaphragm must remain elastic as it resists and transfers the lateral loads to the vertical LFRS. The diaphragm should not be pushed past the yielding point so that the anticipated force in the vertical LFRS does not get distributed to stiffer elements in the diaphragm and cause other failure mechanisms in the system. (Precast/Prestressed Concrete Institution, 2010). However, after investigating precast failures due to earthquake events, older code provisions were found to underestimate lateral diaphragm design forces and this caused inelastic diaphragm behavior.

2.2.2 Previous Code Diaphragm Design Forces

Researchers were investigating The Uniform Building Code (UBC) since the precast structures which failed during the Northridge earthquake would have been designed in accordance with the UBC at the time of construction. The UBC, ASCE7-5, and ASCE 7-10 all utilize the Equivalent Lateral Force (ELF) procedure, a capacity procedure, to determine the seismic design force for diaphragms (Fleischman et al., 1998; Rodriguez et al., 2002, 2007). This procedure assumes a linear variation of lateral forces along the height of the building. The force at each level, F_i, is proportional to the seismic weight at that level and the relative height of the level to the ground. The diaphragm design force at each floor is determined by taking the ratio of the weight tributary to the diaphragm to the total seismic weights at and above that level. The column on the right side indicates the ASCE 7-10 equation number. The diaphragm design force equation per the ASCE 7-10 can be found below:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (12.10\text{-}1)$$

$Where$:

F_{px} $is\ the\ diaphragm\ design\ force\ at\ level\ x$

F_i $is\ the\ design\ force\ applied\ ot\ Level\ i$

w_i $the\ weight\ tributary\ to\ Level\ i$

W_{px} $is\ the\ weight\ tributary\ to\ level\ x$

The equation above is still permitted in the ASCE 7-16 for all diaphragms other than precast. This approach targets a strong and stiff diaphragm design, relative to the specified ductile vertical LFRS element (Fleischman, 2014; Nakaki, 2000). If the diaphragm design force was designed using the ELF procedure, the diaphragms would be designed to have higher forces than the vertical LFRS to remain elastic during a seismic event, while the vertical systems are assumed to yield. However, the linear variation of floor acceleration along the height of a building on which the above method is based is not an accurate representation of the force distribution that a large ground motion would induce on diaphragms at different heights. The ELF procedure provides diaphragm forces based solely on the first, or fundamental mode. The actual floor acceleration at different heights of a building is affected by the periods and mode shapes of higher modes of the building’s vibration response (Chaudhuri, 2004; Fleischman, 2014; Rodriguez et al., 2002, 2007). Chaudhuri further states that studies used to develop the linear variation of floor acceleration did not consider earthquakes large enough to induce non-linear behavior on a structure. Therefore, the code provisions underestimate diaphragm forces and ignore forces due to higher modes, hence, change to the provisions was necessary.

2.3 ASCE 7-16 Diaphragm Design Provisions

2.3.1 Precast Diaphragm Flexibility and Deformation

Precast diaphragms are often classified as rigid diaphragms. However, it was observed that precast diaphragms were behaving inelastically during seismic events, therefore the flexibility of precast diaphragms was investigated. In precast parking structures, there is often a ramp in the center of the structure to allow vehicles to access all levels, resulting in diaphragm segments of large aspect ratios. With large aspect ratios, the diaphragm segments will have larger deformations than the rigid assumption (Paulay, 1995). Therefore, flexible behavior and ductility of precast diaphragms became a concern for future code development. Fleischman et al (Fleischman et al., 1998) were among the first to produce research on the lateral system of precast parking garages after the Northridge earthquake. A prototype parking garage structure similar to one of the many collapsed during the Northridge earthquake was set up for analysis. Applying ground acceleration of similar magnitude of the Northridge earthquake to the structure, significant diaphragm deformations were observed in both the longitudinal direction and transverse direction. In addition, joint deformations between precast members were substantial , and therefore more ductile detailing and regulation of precast diaphragm connections will be required to ensure elastic behavior of diaphragms (Farrow & Fleischman, 2003). If inelastic behavior occurs within the diaphragm or the joints between the precast members fail, unanticipated failure mechanisms will occur (Fleischman & Farrow, 2001). Since the reinforcement and connectors of precast structures proved to be critical to adequate diaphragm design, regulation of reinforcement and connector ductility was needed. To regulate the ductility of connectors and reinforcement, a diaphragm seismic demand level was determined and adopted into the ASCE 7-16 code.

2.3.2 Diaphragm Seismic Demand Level

The seismic demand level will determine whether a diaphragm is permitted to be designed in the elastic or inelastic range, and in turn, how much ductility is required in the connectors and reinforcement for a precast diaphragm. Research has found that three parameters will affect the seismic demand level for precast structures: the number of stories of a structure, the span of the diaphragm, and the aspect ratio of the diaphragm. To determine how these parameters affect the seismic demand of a diaphragm, a set of precast diaphragm designs which targeted elastic behavior under Design Basis Earthquake (DBE) loads and inelastic behavior under Maximum Considered Earthquake (MCE) loads were tested. It was observed that even when hitting the DBE target of elastic deformation, the inelastic deformation under MCE can vary depending on parameters previously stated (Fleischman, 2014). As the number of stories and diaphragm span increase, the required ductility of connections will increase. Figure 2 shows testing results and the local deformation of the joints, δ, plotted with the diaphragm span and number of stories. The two lines observed on the figure were adopted into the ASCE 7-16 section 14.2.4.1 for the determination of the diaphragm seismic demand level. As discussed in the previous section, diaphragms with higher aspect ratios will behave as flexible diaphragms and therefore the AR of a diaphragm will also affect the diaphragm seismic demand level.

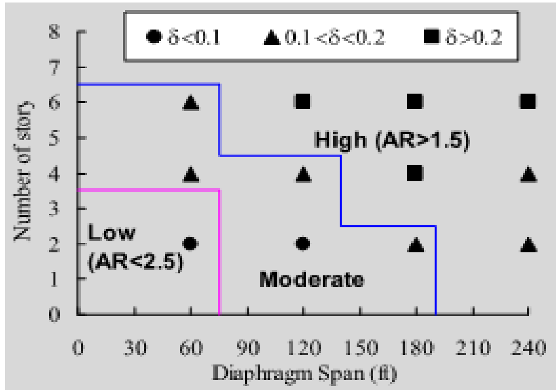

Figure 2: Diaphragm maximum joint opening for BDO designs under MCE (Fleischman, 2014)

With a determined seismic demand level based on ductility requirements, different design options which allow for inelastic or elastic response of the diaphragm are defined by the ASCE 7-16.

2.3.3 Diaphragm Design Options and Reinforcement Ductility

ASCE 7-16 allows for the inelastic behavior of diaphragms by including three different seismic design options. Each design option is associated with different response behavior of a diaphragm under the design basis earthquake or maximum considered earthquake, and therefore, will result in different design forces and deformation limitations (Zhang, 2011). Testing was done to determine the force levels required to achieve the target behavior in each of the design options. Even though some of the following design options involve the inelastic behavior of diaphragms, the goal is to provide enough ductility to the diaphragm to remain intact during a strong seismic event, rather than serve as an energy dissipation mechanism (Fleischman, 2014). The different options are a Basic Design Option (BDO), an Elastic Design Option (EDO), and a Reduced Design Option (RDO).

1. The Basic Design Option targets elastic diaphragm behavior under the design basis earthquake (DBE) and sufficient connection inelastic deformation capacity to withstand the maximum considered earthquake (MCE).

2. An Elastic Design Option targets elastic diaphragm behavior under the maximum considered earthquake (MCE).
3. A Reduced Design Option permits diaphragm yielding during the DBE.

Furthermore, all of the design options target an elastic shear response. See Table 1 for a summary of the performance targets. These three seismic design options for precast concrete diaphragms have been accepted in the ASCE 7-16 and descriptions can be found in section 14.2.4.2. The BDO was intended to be the benchmark option. The design options will allow designers to have some control over the forces and ductility of connections used in the diaphragm design. Since precast diaphragms are not monolithic and connected with steel components, the ductility of the diaphragm will come from the ductility of connections.

Table 1: Design option performance targets, adapted from (Fleischman, 2014)

Diaphragm Design Option	Diaphragm Strength Performance Target			
	Flexure		Shear	
	DBE	MCE	DBE	MCE
EDO	Elastic	Elastic	Elastic	Elastic
BDO	Elastic	Inelastic	Elastic	Elastic
RDO	Inelastic	Inelastic	Elastic	Elastic

Precast connections vary depending on whether a precast system is topped or untopped. Therefore, it is important to determine which connections have adequate ductility for the inelastic behavior of the diaphragm. Extensive research was done on precast double tee connections and their performances as part of a diaphragm (Wan et al., 2012). The goal of researching connection deformation was to classify reinforcement deformability to achieve the target behavior of the diaphragm design options (Fleischman, 2014). Large deformations of the joints between precast members can cause failure mechanisms that differ from the intended failure mechanisms. Therefore, connections of precast diaphragms need to have limited deformations. With a need to regulate precast connection deformability, ASCE 7-16 included the testing standards for reinforcement and their classification in terms of ductility. The testing requirements are found in Section 14 of the ASCE 7-16. ASCE 7-16 defines three ductility classifications for reinforcement and connectors for precast concrete diaphragms: low-deformability elements (LDE), moderate-deformability elements (MDE), and high-deformability elements (HDE). Classifications of deformability elements are based on cyclic tension tests per ASCE 7-16. The testing methods are a result of multiple research projects by Naito et al from 2006-2009 investigating the behavior of precast diaphragm connections under seismic loading (Ren & Naito, 2013). To achieve adequate precast diaphragm ductility, the studies provided the following definitions for the classification of precast diaphragm reinforcement: LDE is defined as having deformability less than 3 inches, MDE is deformability between 3 inches and 6 inches, and HDE is deformability greater than 6 inches. After further research on a variety of different connectors in precast diaphragm construction, a database of multiple experiments was created to help establish a reference for ductility of precast connections (Ren & Naito, 2013). Knowing the required ductility of precast connections is critical to providing adequate strength in precast diaphragm design and will require close coordination with the contractor and precast manufacturer. The ductility of reinforcement and connectors should correspond to the diaphragm design option. For example, the EDO will result in the highest forces of all the design options since the diaphragm would be designed to remain elastic during MCE, therefore low-deformability elements (LDE) connections are permitted to be used. Since specific behavior of the diaphragm is targeted with the use of the design options, ductility of the connections will also correspond with the design options and be permitted or restricted under certain options. Section 14.2.4 in ASCE 7-16 defines the allowed connection ductility for diaphragm design option as follows: any connection deformability classification is allowed in the EDO, moderate-deformability elements (MDE) or high-deformability elements (HDE) connections are allowed for BDO, and HDE connections must be used for RDO.

2.3.4 ASCE 7-16 Diaphragm Design Force

There are two main changes in ASCE 7-16 to the diaphragm design force for precast structures: the design acceleration coefficient, C_{px}, and the diaphragm force reduction factor, R_s. The change in C_{px} accounts for the higher mode responses on the design force, while R_s accounts for the targeted response and ductility of the diaphragm. Equation 12.10-4 was adopted into the ASCE 7-16 which includes higher mode effects and diaphragm flexibility. The column on the right-hand side indicates the equation number in ASCE 7-16:

$$F_{px} = \frac{C_{px}}{R_s} W_{px} \qquad (12.10\text{-}4)$$

$Where$:

$F_{px}\ is\ the\ diaphragm\ design\ force\ at\ level\ x$

$C_{px}\ is\ the\ design\ acceleration\ coefficient$

$W_{px}\ is\ the\ weight\ tributary\ to\ the\ diaphragm\ as\ level\ x$

$R_s\ is\ the\ \ diaphragm\ reduction\ factor$

2.3.4.1 Diaphragm Acceleration Coefficient and Mode Shape Factor, Z_s

The provisions in the ASCE 7-16 are based on the First Mode Reduce (FMR) method proposed by Rodriguez in 2002 (Fleischman, 2014; Rodriguez et al., 2002). See Figure 3 below for the determination of the floor acceleration coefficient, C_{px} and the variables which affect it in ASCE 7-16.

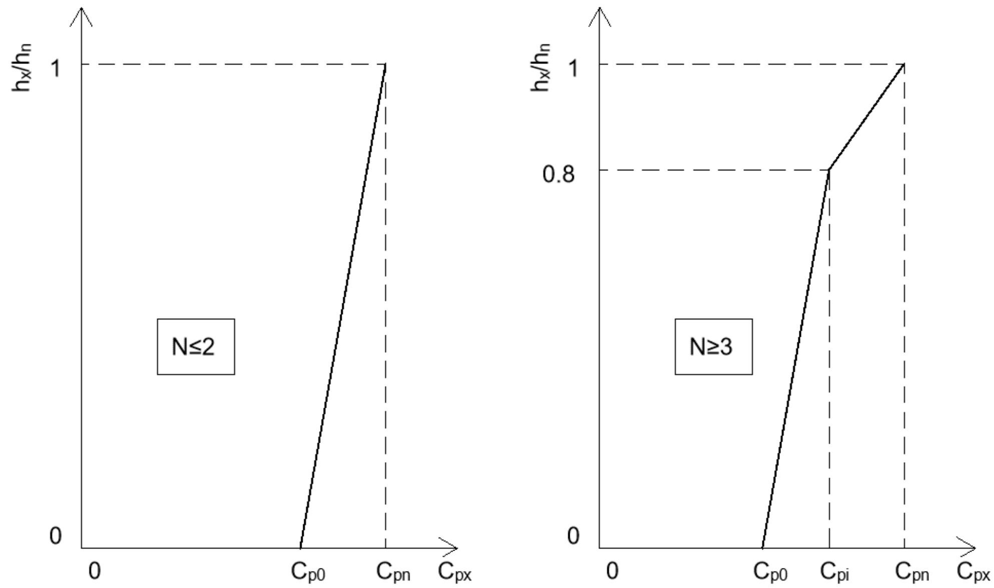

Figure 3 Calculating the Design Acceleration Coefficient, C_{px} adapted from ASCE 7-16

$Where$:

$N = Number\ of\ stories$

$I_e = Seismic\ importance\ factor$

$$C_{pn} = \sqrt{(\Gamma_{m1}\Omega_o C_s)^2 + (\Gamma_{m2}C_{s2})^2} \geq C_{pi} \quad (12.10\text{-}7)$$

$$C_{p0} = 0.4 S_{DS} I_e \quad (12.10\text{-}6)$$

$and\ C_{pi}\ is\ the\ maximum\ of$:

$$C_{pi} = 0.8 C_{p0} \quad (12.10\text{-}8)$$

$$C_{pi} = 0.9 \Gamma_{m1} C_s \Omega_O \quad (12.10\text{-}9)$$

$and\ C_{s2}\ is\ the\ minumum\ of$

$$C_{s2} = (0.15N + 0.25) I_e S_{DS} \quad (12.10\text{-}10)$$

$$C_{s2} = I_e S_{DS} \quad (12.10\text{-}11)$$

$$C_{s2} = \frac{I_e S_{D1}}{0.03(N-1)} \quad (12.10\text{-}12a)$$

$\Gamma_{m1} = Fundamental\ mode\ response\ coefficient$

$$\Gamma_{m1} = 1 + \frac{z_s}{2}\left(1 - \frac{1}{N}\right) \tag{12.10-13}$$

$\Gamma_{m2} = Higher\ mode\ response\ coefficient$

$$\Gamma_{m2} = 0.9z_s\left(1 - \frac{1}{N}\right)^2 \tag{12.10-14}$$

and

$C_s = Building\ seismic\ response\ coeffiecient\ determined\ from$

$ASCE\ 7 - 16\ section\ 12.8$

$C_p = Design\ acceleration\ coefficient\ with\ respect\ to\ height$

$z_s = Mode\ Shape\ Factor$

Rodriguez performed tests relating the different modal responses of buildings subjected to ground motions similar to the Northridge earthquake. It was concluded that previous code versions neglected the higher modal responses of a building during a large seismic event. However, the study to develop the FMR method showed that higher modes will contribute to the horizontal floor acceleration in addition to the fundamental mode when subject to a large seismic load (Rodriguez et al., 2002). The contribution of modal response, Γ, to the design acceleration coefficient is determined by ASCE Equation 12.10-13 and 12.10-14 and represented in Figure 3. The proposed modal coefficient equations developed by the study as a part of the FMR method were adopted into the ASCE 7-16. Further testing was done on more structures after the findings by Rodriguez and the results were confirmed. The results of the testing are plotted in the ASCE 7-16 commentary showing the same envelope of the modal coefficient equations as developed by Rodriguez for the FMR method, see Figure 4. It can be observed that buildings are affected by the higher modal responses, in addition to the fundamental mode. Since the FMR method includes higher modal responses in alignment with testing, it was adopted into the ASCE 7-16.

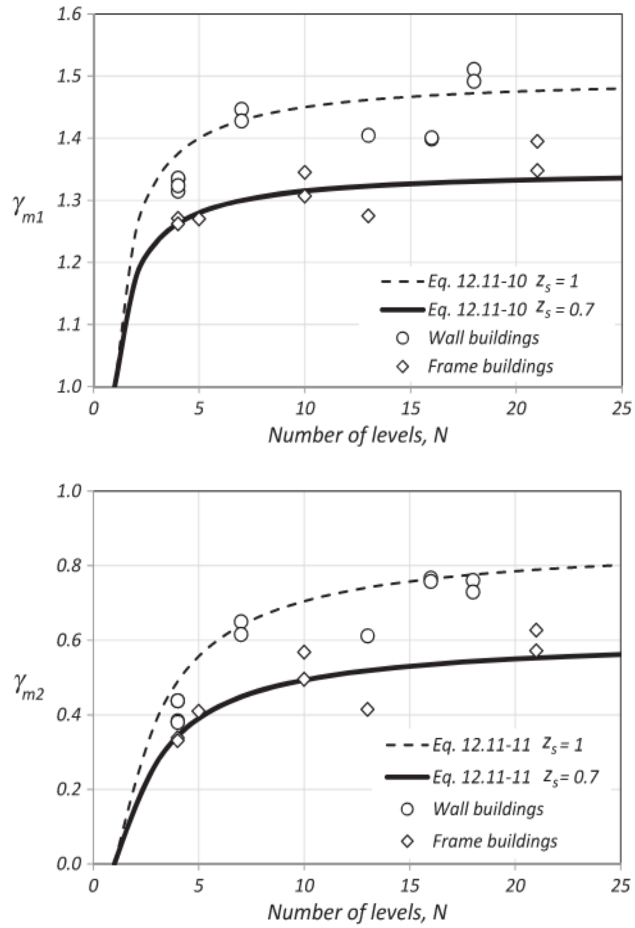

Figure 4: Modal Response Coefficient (ASCE 7-16 Figure C12.10-3)

From the above figure, it can be seen that different vertical LFRS will result in different higher mode responses. To determine the different effect of modal responses with respect to the vertical LFRS elements, studies were conducted. It was found that buckling-restrained braced frame (BRBF) systems are highly effective in limiting the horizontal floor accelerations from higher modes (Choi, 2008). Additionally, steel moment frames (SMRF) are moderately effective in limiting horizontal floor accelerations from higher modes. As a result of these findings, a modal shape factor, Z_s, is used to represent the differences. BRBF systems were assigned the lowest Z_s value of 0.3, followed by SMRF systems with a Z_s value of 0.7. All other systems were assigned a value of Z_s of 1.0. This is because bearing wall systems showed larger higher mode effects when compared to frame systems as seen in Figure 4.

2.3.4.2 Diaphragm Force Reduction Factor, Rs

In addition to accounting for the ductility of the diaphragm with respect to the chosen design option, it was apparent that the increase in diaphragm force observed was found to be material-dependent since other structures with non-precast diaphragms performed adequately under high seismic events such as the Northridge earthquake. The diaphragm force reduction factor, Rs, is a material-dependent factor calibrated to achieve the target behavior of the diaphragm design options. The Rs value which corresponds to the different design options and their behavior can be seen in Figure 5 and Table 2. To ensure the expected diaphragm response, the global ductility demand needs to be related to the local ductility demand of the material and connections(Fleischman, 2014). The values are calibrated based on precast diaphragm testing in which the targeted behavior is achieved. The R_s values for other materials are not covered in this report but can be found in the ASCE 7-16. The calibration of the factors makes sense since an RDO would require more ductility, but a decrease in design force to ensure inelastic behavior.

Table 2: Diaphragm force reduction factors

Option	Diaphragm Connector Category	R_s
EDO	LDE	0.7
BDO	MDE	1
RDO	HDE	1.4

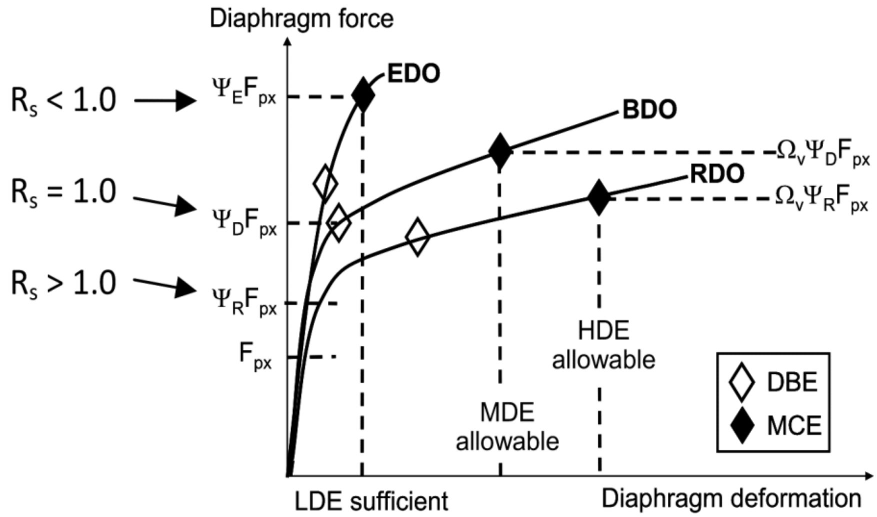

Figure 5: Schematic of Diaphragm response indicating diaphragms design options (Fleischman, 2014)

2.3.4.3 Testing Results Against Diaphragm Design Force

Testing was done to compare the proposed design procedure to observed forces on a full-scale level. The design envelopes for the proposed equations are adequate to resist motions observed from a full-scale test, see Figure 6. This test included an induced ground acceleration at the base of the shake-table similar to the Northridge earthquake (Choi, 2008). Additionally, under the DSDM project prototype, precast parking structures were evaluated using nonlinear transient dynamic analysis of three-dimensional finite element models(Fleischman, 2014; Zhang, 2011). The result of the analysis models was similar to that observed in Chen's model which can be seen in Figure 7 and Figure 8. These figures show the amplification factor, Ψ, applied to the ASCE 7-10 forces. Prior to the proposal to amend the ASCE 7-10, an amplification factor was developed to be applied to the then current ASCE 7-10 diaphragm design forces (Fleischman, 2014; Zhang, 2011). The amplification factor was developed with the intent to amplify the design forces in the ASCE 7-10 to more accurately represent the diaphragm design forces as a result of higher modal response. However, this amplification factor was not adopted into the

ASCE 7-16 since the acceleration coefficient now includes higher modal responses. Since the provisions in the ASCE 7-16 now include the higher modal responses which the amplification factor was calibrated to include, the diaphragm design capacity is still an accurate representation. Diaphragm design procedure using ψ and α_{fx} is outside of the scope of this report but can be found in (Fleischman, 2014; Zhang, 2011).

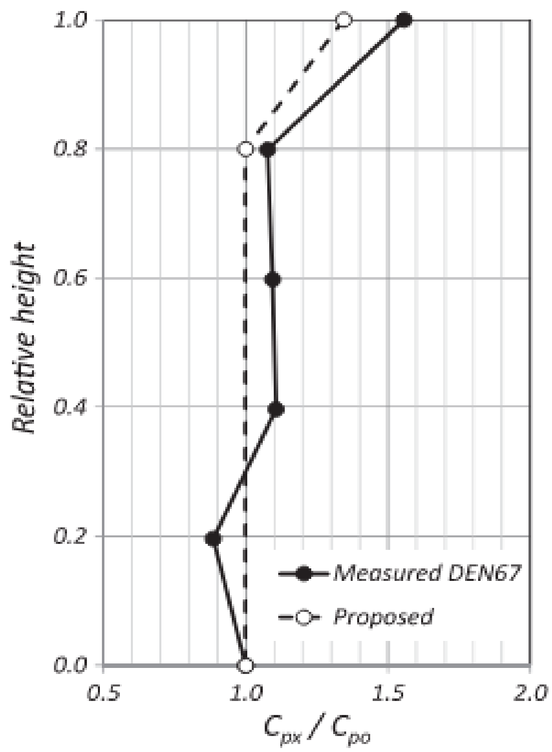

Figure 6: Comparison of measured floor accelerations from (Choi, 2008) to the acceleration predicted by Equation 12.10-4 ASCE 7-16 (ASCE 7-16 Figure C12-10.5)

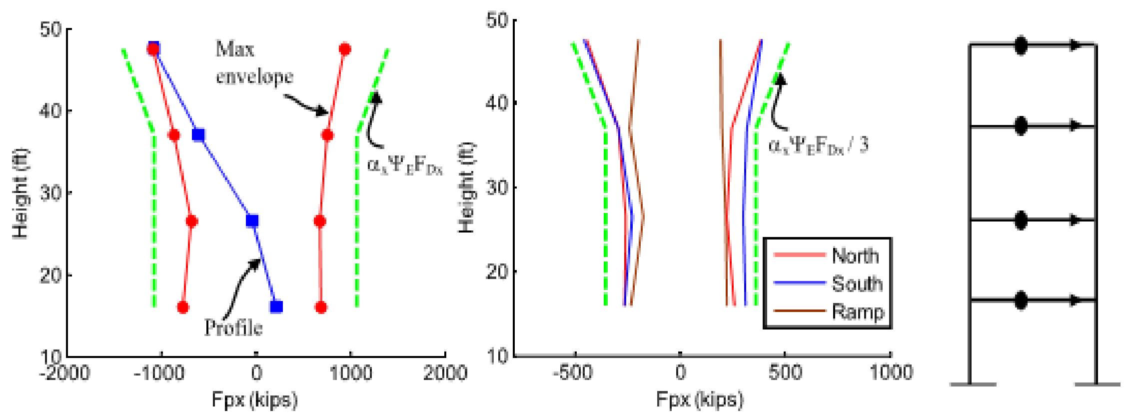

Figure 7: Diaphragm inertial force in the transverse direction: floor total (left); sub-diaphragm (right) (Fleischman, 2014)

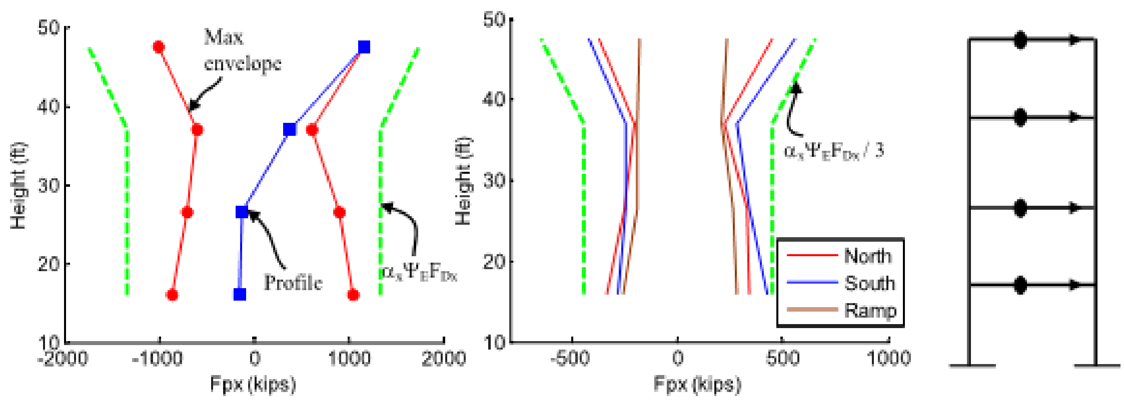

Figure 8: Diaphragm inertial force in the longitudinal direction: floor total (left); sub-diaphragm (right) (Fleischman, 2014)

2.3.6 Diaphragm Shear Overstrength Factor, Ω_v

It was observed through studies and reports of past seismic events that precast diaphragms will show a ductile response under flexure, but a brittle failure under shear. The shear overstrength factor must be applied to ensure the precast diaphragm system does not have a brittle shear failure prior to the joints reaching their intended response targets. Only the RDO and BDO design options require that the shear overstrength factor be applied since the diaphragm will remain elastic under the MCE in EDO (Fleischman, 2014). The shear overstrength factor, Ω_v, is calculated by ASCE 7-16 Equation C12.10-3 and can be seen on the graph in Figure 9.

The shear overstrength factor was derived by modeling a structure designed in accordance with the ASCE 7-16 and applying an MCE seismic event. By scaling the shear capacity to the shear demand, the overstrength factor is identified. The shear overstrength factor is directly proportional to the diaphragm force reduction factor and the equation is sufficiently conservative for all cases observed, see Figure 9. The green line in the figure represents the shear overstrength factor adopted into the ASCE 7-16, while the plotted points correspond with the required overstrength factor for a variety of buildings with different numbers of stories subjected to an MCE load (Fleischman, 2014).

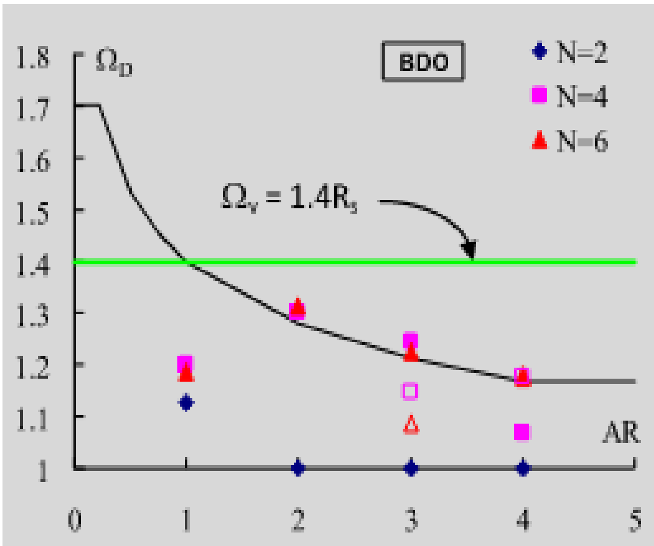

Figure 9: Diaphragm shear overstrength factor equations (Fleischman, 2014)

Chapter 3: ASCE 7-16 Design Procedure

3.1 Diaphragm Design Force

This procedure follows the new provisions in the ASCE 7-16 required for precast concrete diaphragms in buildings classified with Seismic Design Category C or higher. The procedure includes all of the steps necessary for designing a precast concrete diaphragm. The equations and provisions can be found in the ASCE 7-16 section 12.10.3 *Alternate Design Provisions for Diaphragms, Including Chords and Collectors* and 14.2.4 *Additional Design and Detailing Requirements for Precast Concrete Diaphragms.*

3.1.1 Seismic Demand Level

The first step to designing a precast concrete diaphragm is determining the seismic design category (SDC) per ASCE 7-16 section 11.6. SDC is based on the building risk category and design response spectral accelerations, S_{DS} and S_{D1}. Both of these parameters vary by the location and site geotechnical condition. In addition to determining SDC, the seismic response coefficient, C_s, should be determined in accordance to ASCE 7-16 section 12.8. This value will be used to determine some of the other design coefficients.

As previously mentioned, the procedure used for precast diaphragm design in the ASCE 7-16 utilizes a seismic demand level dependent on the building's number of stories, diaphragm length, aspect ratio, and seismic design category. The seismic demand has three levels; low, moderate, and high. The diaphragm seismic design level can be obtained for SDC D, E, and F by following Figure 10. Structures with SDC B and C are considered low seismic demand. This procedure can also be found in the ACI 550.5-18 section 5.3 and the ASCE 7-16 section 14.2.4.

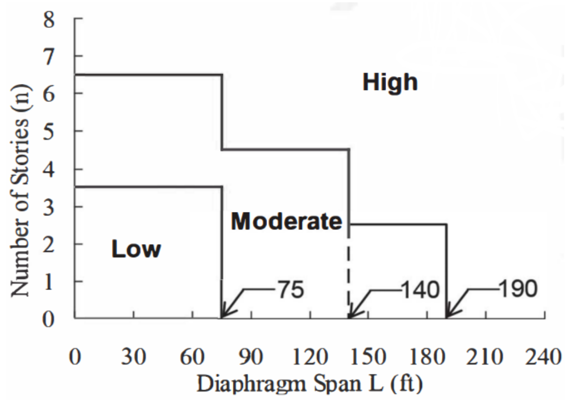

Figure 10: Seismic demand level (ASCE 7-16 Figure 14.2-1)

There are a few exceptions to the determination of the seismic demand level using Figure 10. The exceptions are as follows:

- If the seismic demand level is low according to Figure 10, but the aspect ratio (AR) is greater than or equal to 2.5, then the seismic demand level must be increased to moderate.
- If the seismic demand level is high according to Figure 10, but the AR is less than 1.5, then the seismic demand level is permitted to decrease to moderate.

The exceptions are a result of testing which showed how the aspect ratio affects the flexibility of precast diaphragms as discussed in Chapter 2 (Fleischman, 2014).

Diaphragm span shall be defined as the maximum diaphragm span in either direction. For any given direction, the span shall be taken as the larger of the maximum distance between vertical LFRS elements and two times the distance from the outer vertical LFRS element and the building edge. If an opening is present in the structure, such as a parking garage ramp as seen in (b) or (c) of Figure 11, the definitions above remain the same. However, the depth of the diaphragm will vary based on the layout of the floor plan and openings or interior vertical LFRS elements. The diaphragm aspect ratio (AR) is the diaphragm span-to-depth ratio. The depth of the diaphragm is defined as the distance between the chords of the diaphragm. The definitions above are illustrated and defined in Figure 11 for various precast diaphragm layouts.

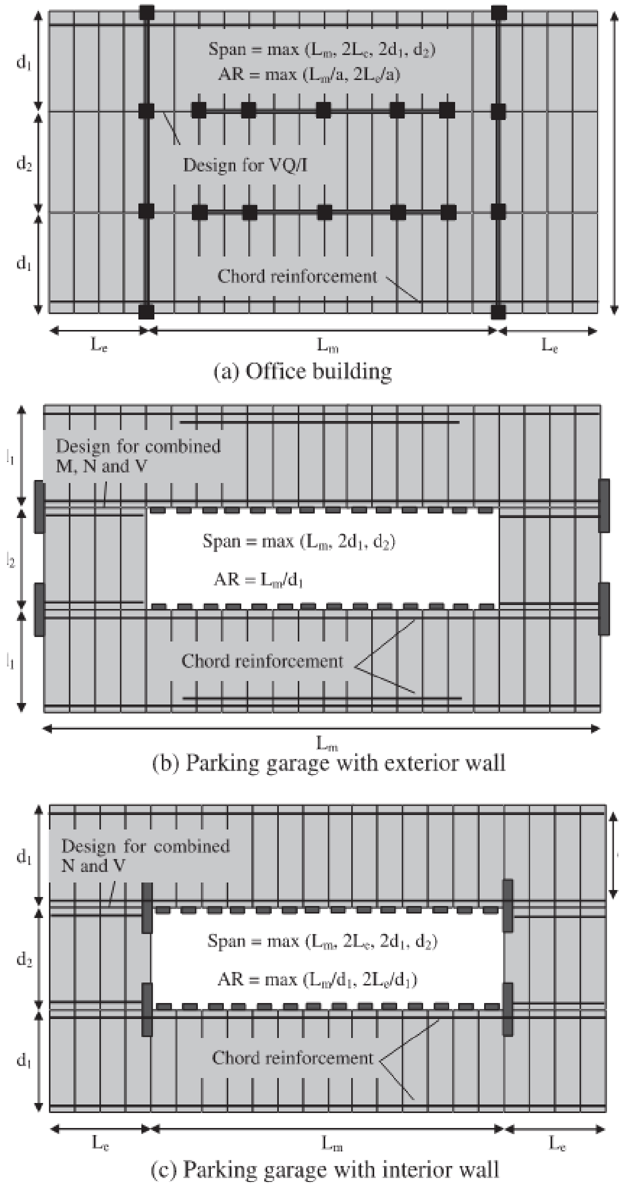

Figure 11: Diaphragm dimensions (ASCE 7-16 Figure C14.2-1)

3.1.2 Design Option

As discussed in Chapter 2, there are three diaphragm design options: Elastic, Basic, and Reduced. The design option must be identified in order to use the correct diaphragm reduction factor, R_s, when determining the design force. Selection of a design option is based on the seismic demand level and dictates the deformability requirement for connectors and reinforcement. Some design options are not permitted for use depending on the seismic demand level. See Table 3 for the permitted design options per seismic demand level.

Table 3 Diaphragm Design Option, adapted from (ASCE 7-16)

Design Option	Diaphragm Seismic Demand Level		
	Low	Moderate	High
Elastic	Recommended	With Penalty[1]	Not Allowed
Basic	Alternative	Recommended	With Penalty[1]
Reduced	Alternative	Alternative	Recommended

[1]Design force shall be increased by 15%

Additionally, 12.10.3.4 in the ASCE 7-16 states that when designing collectors, the design force needs to be amplified by 1.5. Next, the diaphragm shear overstrength factor needs to be determined per equation (C12.10.3). This factor is directly proportional to the diaphragm reduction factor and needs to be applied to the required diaphragm shear force when designed with an RDO or BDO option. It does not need to be included in an EDO design since the diaphragm would remain elastic.

ASCE 7-16

$$\Omega_v = 1.4R_s \qquad \text{(C12.10.3)}$$

3.1.3 Design Force

Diaphragm design forces are calculated in accordance to the ASCE 7-16 section 12.10.3. First, the diaphragm design acceleration coefficient, $C_{px,}$ must be found. This coefficient varies based on the number of stories, type of Lateral Force Resisting System, and height of diaphragm relative to total height of the structure, h_x/h_n. C_{px} varies linearly or bi-linearly along the height depending on the number of stories, as shown in Figure 12. The calculation of C_{po} and C_{pn} follows equations 12.10-6 through 12.10-14 in the ASCE 7-16. The column on the right indicates the corresponding equation number in ASCE 7-16.

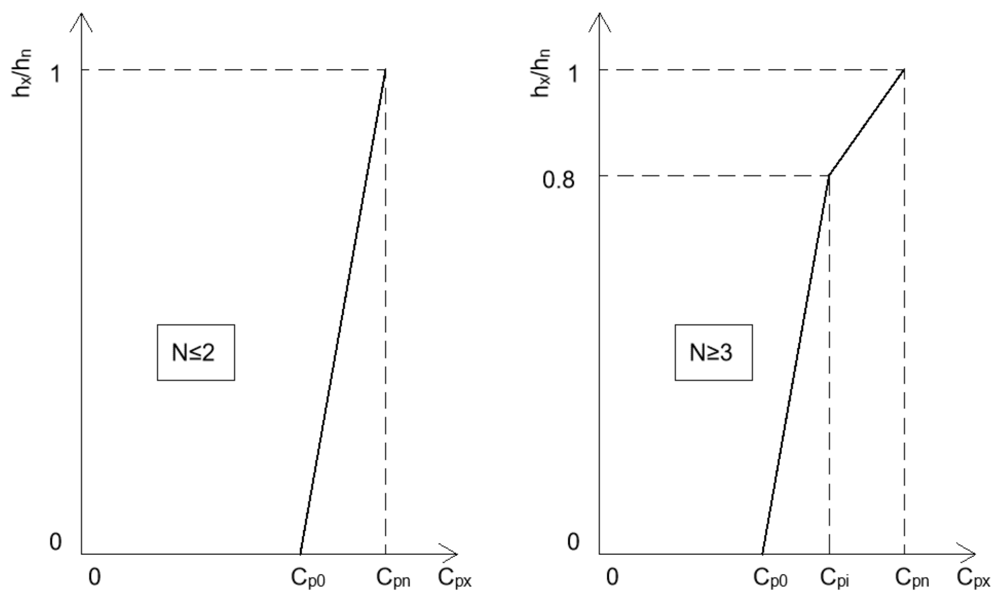

Figure 12 Calculating the Design Acceleration Coefficient, C_{px} adapted from ASCE 7-16 Figure 12.10-2

$Where$: ASCE 7-16

$N = Number\ of\ stories$

$I_e = Seismic\ importance\ factor$

$$C_{pn} = \sqrt{(\Gamma_{m1}\Omega_o C_s)^2 + (\Gamma_{m2}C_{s2})^2} \geq C_{pi} \qquad (12.10\text{-}7)$$

$$C_{p0} = 0.4S_{DS}I_e \quad (12.10\text{-}6)$$

$and\ C_{pi}\ is\ the\ maximum\ of:$

$$C_{pi} = 0.8C_{p0} \quad (12.10\text{-}8)$$

$$C_{pi} = 0.9\Gamma_{m1}C_s\Omega_O \quad (12.10\text{-}9)$$

$and\ C_{s2}\ is\ the\ minumum\ of$

$$C_{s2} = (0.15N + 0.25)I_eS_{DS} \quad (12.10\text{-}10)$$

$$C_{s2} = I_eS_{DS} \quad (12.10\text{-}11)$$

$$C_{s2} = \frac{I_eS_{D1}}{0.03(N-1)} \quad (12.10\text{-}12a)$$

$\Gamma_{m1} = Fundamental\ mode\ response\ coefficient$

$$\Gamma_{m1} = 1 + \frac{z_s}{2}\left(1 - \frac{1}{N}\right) \quad (12.10\text{-}13)$$

$\Gamma_{m2} = Higher\ mode\ response\ coefficient$

$$\Gamma_{m2} = 0.9z_s\left(1 - \frac{1}{N}\right)^2 \quad (12.10\text{-}14)$$

and

$C_s = Building\ seismic\ response\ coeffiecient\ determined\ from$

$ASCE\ 7-16\ section\ 12.8$

$C_{px} = Design\ acceleration\ coefficient\ with\ respect\ to\ height$

$z_s = Mode\ Shape\ Factor$

$= 0.3\ for\ buckling-restrained\ braced\ frame\ systems$

$= 0.7\ for\ steel\ moment\ frame\ systems$

$= 1.0\ for\ all\ other\ systems$

The definitions and background on variables are previously discussed in Chapter 2. Next, the diaphragm design force needs to be determined using equation 12.10-4 and 12.10-5 in the ASCE 7-16:

$$F_{px} = \frac{C_{px}}{R_s} W_{px} \tag{12.10-4}$$

$Where$:

F_{px} $is\ the\ diaphragm\ design\ force\ at\ level\ x$

C_{px} $is\ the\ design\ acceleration\ coefficient$

W_{px} $is\ the\ weight\ tributary\ to\ the\ diaphragm\ as\ level\ x$

R_s $is\ the\ \ diaphragm\ reduction\ factor$:

$= 0.7\ for\ EDO$

$= 1.0\ for\ BDO$

$= 1.4\ for\ RDO$

F_{px} $should\ not\ be\ taken\ less\ than$:

$$F_{px} = 0.2 S_{DS} I_e W_{px} \tag{12.10-5}$$

The determined design force is then applied to the diaphragm. The force is distributed along the length of the diaphragm and the diaphragm is analyzed as a horizontally loaded beam. Since precast diaphragms are separate pieces connected in few locations, the internal forces will be determined at each precast panel joint.

Chapter 4: Parametric Study

A parametric study is conducted to investigate the effects of the following variables on diaphragm design for a multi-story precast office building:

- Seismic design category (SDC)
- The diaphragm design option

These parameters will vary from project to project and therefore it is important to understand the change in design when the changing parameters are observed. It is crucial to coordinate the reinforcement and connectors with the contractor as the ductility of the diaphragm and diaphragm response depend on the type of connectors and reinforcement. The goal of this study is to observe and comment on how the diaphragm design force and required reinforcement for the diaphragm design will vary based on the chosen parameters. In addition, the design force of the diaphragm will be compared to the ASCE 7-10 design forces. An example of calculations and diaphragm reinforcement tables can be found in Appendix A.

The building being investigated is a 6-story office building comprised of precast double-tees with 3” topping and perimeter special precast reinforced concrete shear walls for the LFRS. The double-tees are 10 feet wide in bays of 3 and span 50 feet. There are two shear walls 30 feet long in the East/West direction on each side of the building and one shear wall 50 feet long in the North/South direction on each side of the building, see Figure 13. The gravity system is assumed to be adequate along with the vertical elements of the LFRS. The typical story height for the building is 15 feet. The total height of the building is 90 feet. The dead load was assumed to be 100 psf including the precast members and the topping per the PCI Design Handbook. Soil conditions were taken as D for default soil. The building is classified as Risk Category II. See Figure 13,Figure 14, andFigure 15 for the plan and elevation views of the building.

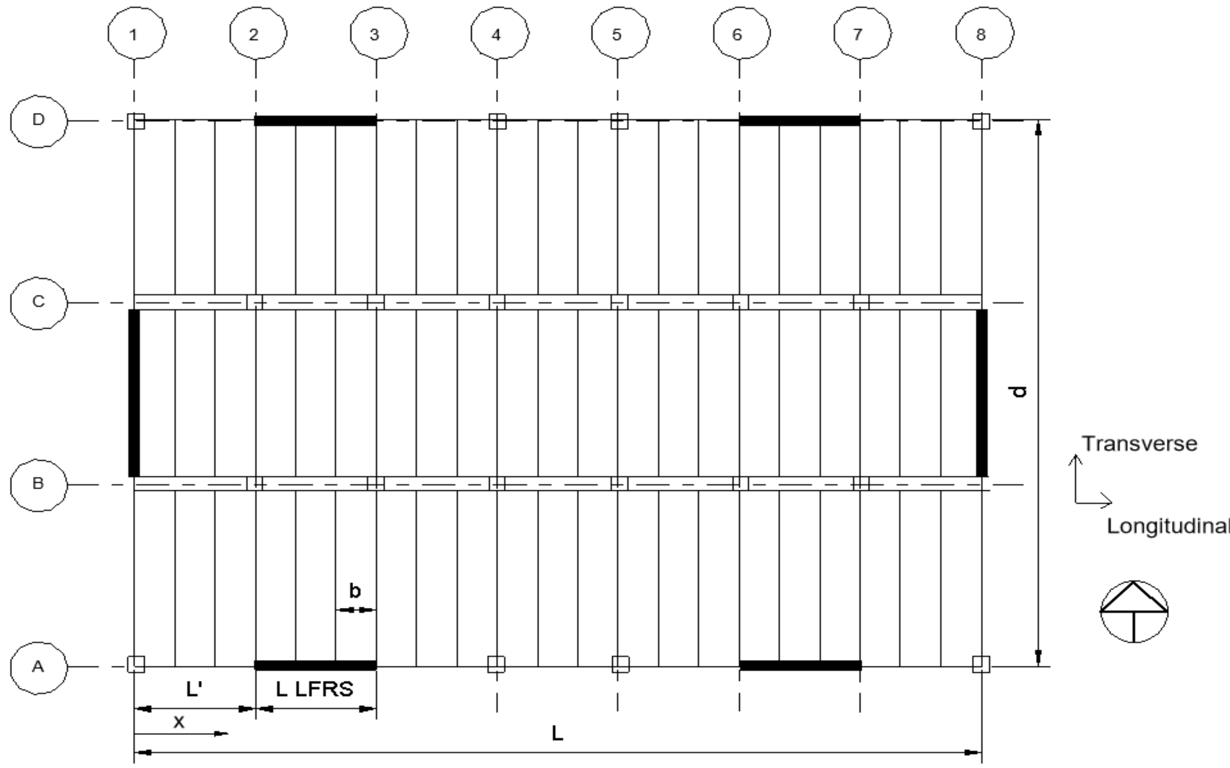

Figure 13: Plan view for Parametric Study

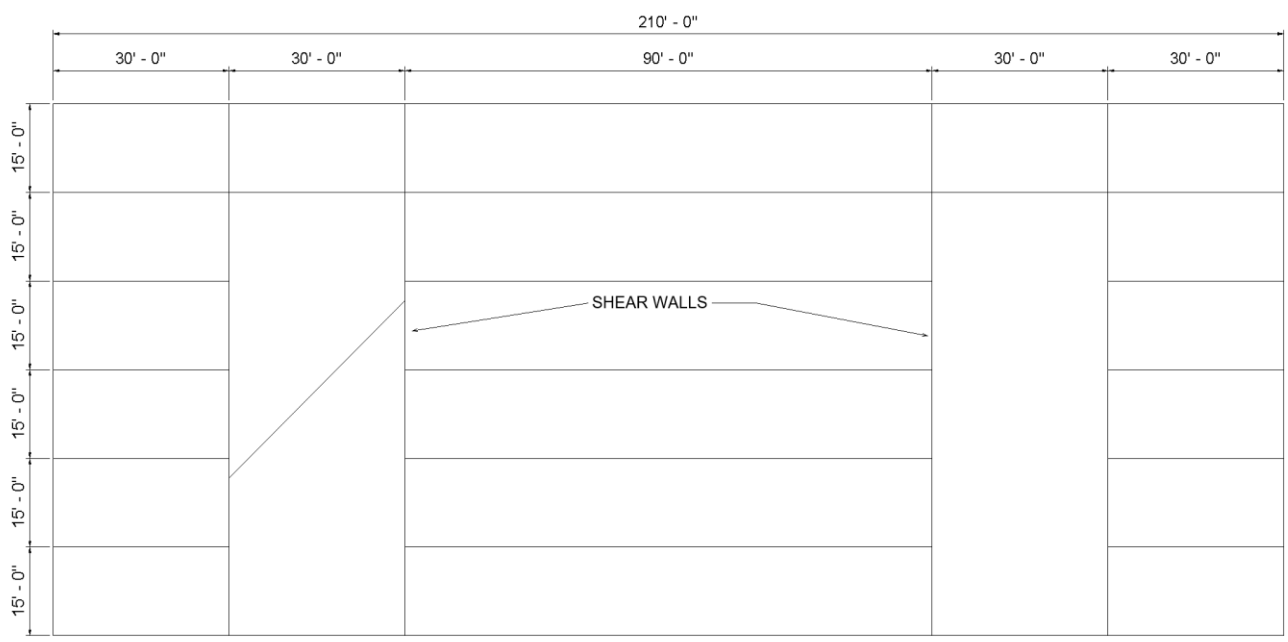

Figure 14:E/W Elevation for Parametric Study

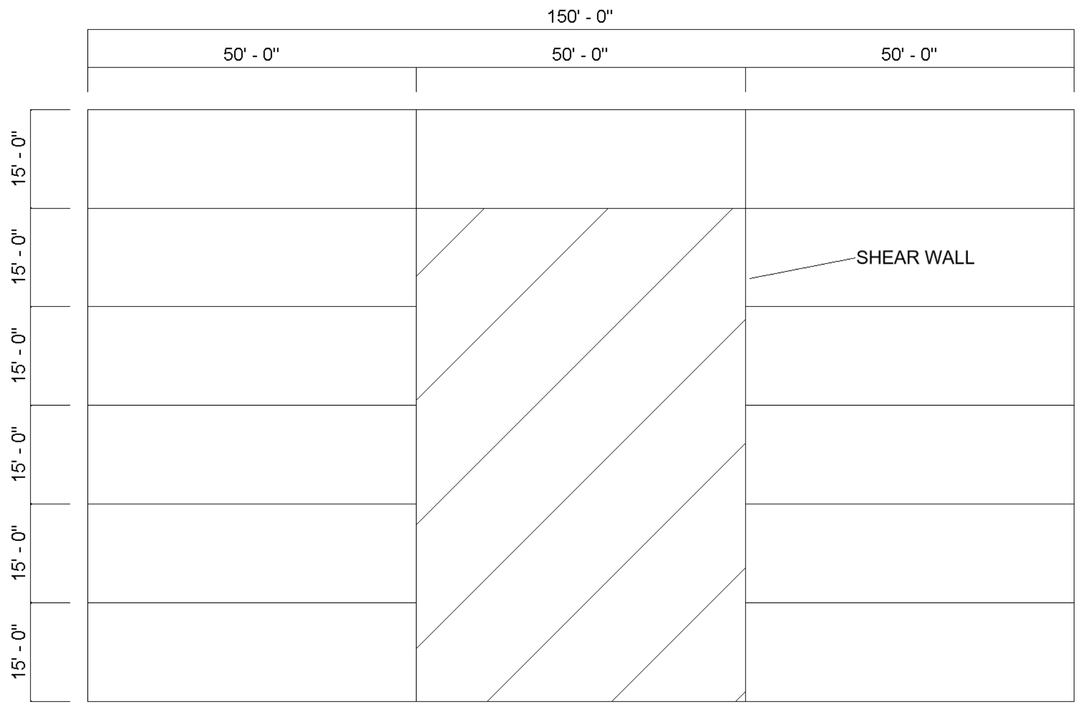

Figure 15: N/S Elevation for Parametric Study

Since precast floor members are topped, the chord reinforcement will be a pour-strip. Connectors between precast members will be Hairpin (#4) connectors, see Figure 16. Additionally, the slab reinforcement will consist of standard ASTM A185 Welded Wire Reinforcement (WWR). Different design options require different ductility requirements for connectors and reinforcement, see Table 4. When designing for the Reduced Design Option, HDE reinforcement is required, and therefore a ductile ladder Gr. 1018 will be utilized, see Figure 17.

Table 4 Required Diaphragm Reinforcement Classification, adapted from (Fleischman, 2014)

Design Option	Diaphragm Reinforcement Classification		
	Low	Moderate	High
Elastic	Recommended	Allowable	Allowable
Basic	Not Allowed	Recommended	Allowable
Reduced	Not Allowed	Not Allowed	Recommended

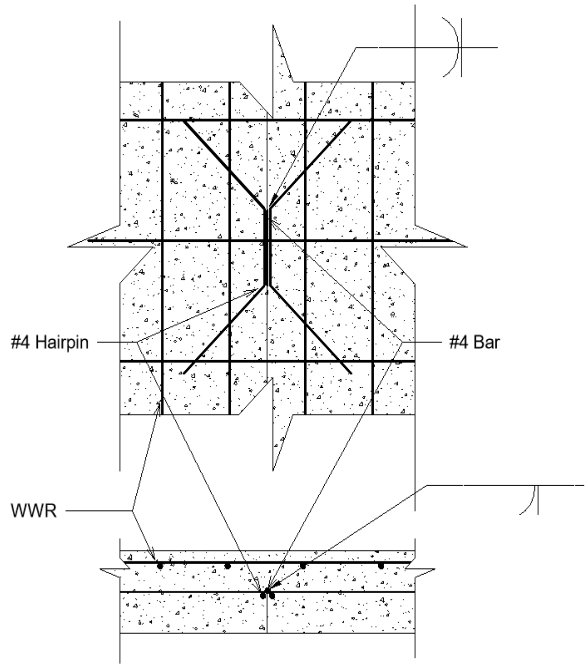

Figure 16: Hairpin Connection

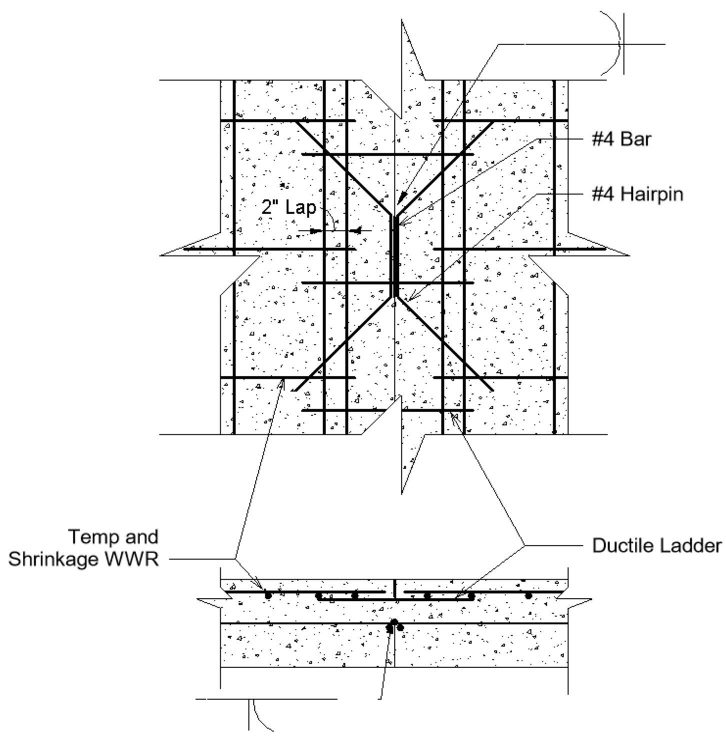

Figure 17: Hairpin with Ductile Ladder Connection

From the building parameters a high seismic demand level is determined using Figure 10 with a height of 90 feet and diaphragm span of 210 feet. However, since the AR<1.5, the building is permitted to be classified under a moderate seismic demand level for SDC D. As previously stated, buildings with SDC C or B shall be designed as low seismic demand level.

4.1 Lateral Loading

This parametric study neglects wind load and assumes seismic loads will govern the lateral design of the diaphragm due to the heavy weight of the structure. Seismic criteria were applied based on the location of the project. Basic seismic parameters such as S_{DS}, S_{D1}, F_a, F_v, and C_s were calculated in accordance with the ASCE 7-16 Chapters 11 and 12. These parameters are used to show previous diaphragm design forces from the ASCE 7-10. The study assumes no horizontal irregularities in the diaphragm. Lateral loads were investigated in both directions to obtain forces in the precast panel joints. The equations to determine the diaphragm force per ASCE 7-16 can be found in Chapter 3.

The building determined to be in SDC C is located in Chattanooga, TN, and the building determined to be in SDC D is located in Knoxville, TN. The SDC was determined from seismic parameters as stipulated in Chapter 11 in ASCE 7-16. The building has a layout as described above with a low seismic demand level for SDC C and a moderate seismic demand level for SDC D. The following seismic properties are calculated in accordance with ASCE 7-16. The column on the right indicates the corresponding equation or table number in ASCE 7-16.

Table 5: ASCE 7-16 Chapter 11 and 12 Seismic Properties

ASCE 7-16 Chapter 11 and 12 Properties			
	SDC C	SDC D	ASCE 7-16 Ref.
S_s	0.467	0.627	
S_1	0.122	0.134	
F_a	0.143	1.298	T11.4-1
F_v	2.356	2.331	T11.4-2
S_{MS}	0.666	0.814	(11.4-1)
S_{M1}	0.287	0.312	(11.4-2)
S_{DS}	0.444	0.543	(11.4-3)
S_{D1}	0.192	0.208	(11.4-4)
C_s	0.0546	0.0594	(12.8-2/3/5)
T_a	0.584	0.584	(12.8-7)

The parameters used to calculate the diaphragm acceleration coefficient C_{px} are shown in Table 6. Both buildings are comprised of the same structure and therefore $Z_s = 1.0$. Similarly, Γ_{m1} and Γ_{m2} are the same for both buildings. Table 7 shows the diaphragm design acceleration coefficients at different levels for both buildings, as compared side-by-side in Figure 18.

Table 6: ASCE 7-16 Cpx Properties

ASCE 7-16 C_{px} Properties			
	SDC C	SDC D	ASCE 7-16 Ref.
Γ_1	1.417	1.417	(12.10-13)
Γ_2	0.625	0.625	(12.10-14)
C_{s2}	0.444	0.543	(12.10-10/11/12)
C_{po}	0.178	0.217	(12.10-6)
C_{pi}	0.174	0.189	(12.10-8/9)
C_{pn}	0.338	0.399	(12.10-7)

Table 7: SDC C Design Acceleration Coefficient, C_{px}

Level	h_x(ft)	hn/hx	SDC C C_{px}	SDC D C_{px}
2	15	0.17	0.177	0.211
3	30	0.33	0.176	0.206
4	45	0.50	0.176	0.200
5	60	0.67	0.175	0.194
6	75	0.83	0.202	0.224
Roof	90	1.00	0.338	0.399

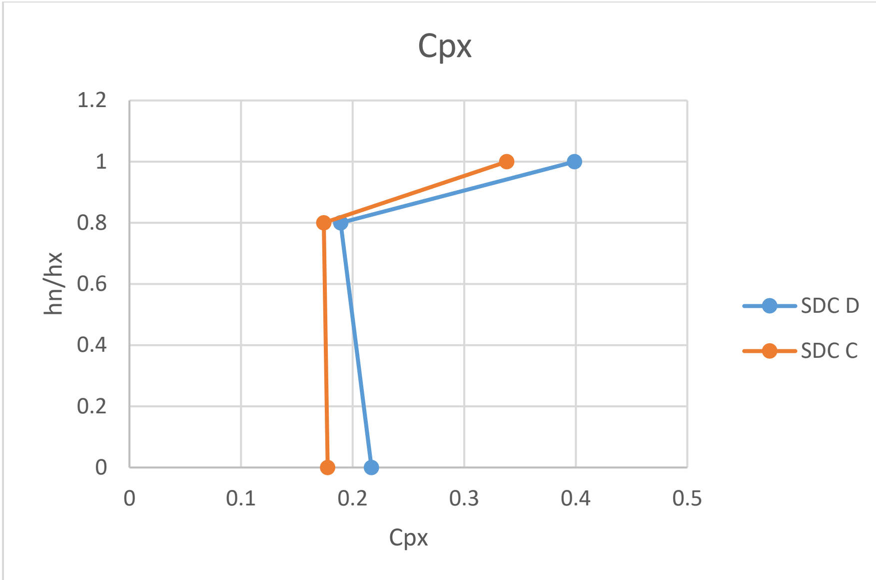

Figure 18: Design Acceleration Coefficient, C_{px}

Table 8 shows the building seismic force distribution along building height for SDC C. Table 9 shows diaphragm design force distribution at different levels for different design options for SDC C. The ASCE 7-10 diaphragm design forces are also compared can also be found in Table 9 and are graphically compared in Figure 19.

Table 8: SDC C Vertical Force Distribution

Level	h_x(ft)	W_x(k)	$W_x h_x^k$ *	C_{vx}	F_x(k)	V_x(k)
2	15	3150	52970	0.045	46.4	1032.9
3	30	3150	109085	0.092	95.5	9865
4	45	3150	166452	0.141	145.8	890.9
5	60	3150	224647	0.190	196.7	745.2
6	75	3150	283465	0.240	248.2	548.4
Roof	90	3150	342785	0.291	300.2	300.2

*k=1.042

Table 9: SDC C Diaphragm Design Forces Per Level

Level	C_{px}/R_{EDO}	EDO (k)	C_{px}/R_{BDO}	BDO (k)	C_{px}/R_{RDO}	RDO (k)	ΣFi/ΣWi	ASCE 7-10 (k)
2	0.253	796.0	0.177	557.2	0.126	398.0	0.089	279.7
3	0.252	792.8	0.176	554.9	0.126	396.4	0.089	279.7
4	0.251	789.6	0.175	552.7	0.125	394.8	0.089	279.7
5	0.250	786.4	0.175	550.5	0.125	393.2	0.089	279.7
6	0.288	907.0	0.202	634.9	0.144	453.5	0.089	279.7
Roof	0.483	1522.4	0.338	1065.7	0.242	761.2	0.095	300.2

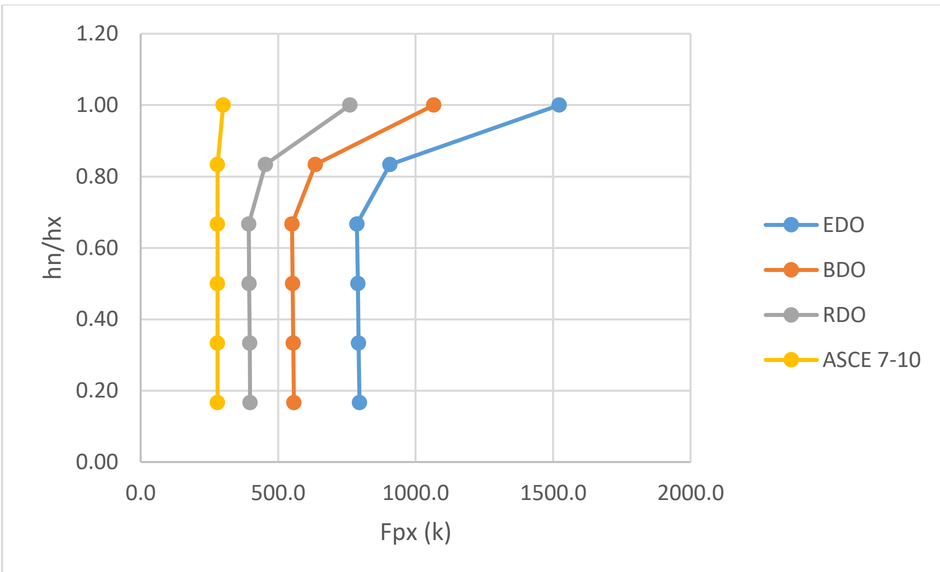

Figure 19: SDC C Diaphragm Design Forces

Similarly, Table 10 and Table 11 show the results for the SDC D building. The graphical comparison of diaphragm design forces can also be seen in Figure 20

Table 10: SDC D Vertical Force Distribution

Level	h_x(ft)	W_x(k)	$W_x h_x^k$	C_{vx}	F_x(k)	V_x(k)
2	15	3150	52970	0.045	50.4	1122.4
3	30	3150	109085	0.092	103.8	1072.0
4	45	3150	166452	0.141	158.4	968.2
5	60	3150	224647	0.190	213.8	809.8
6	75	3150	283465	0.240	269.8	596.0
Roof	90	3150	342785	0.291	326.2	326.2

Table 11: SDC D Diaphragm Design Force

Level	C_{px}/R_{ED}	EDO (k)	C_{px}/R_{BDO}	BDO (k)	C_{px}/R_{RDO}	RDO (k)	ΣFi/ΣWi	ASCE 7-10 (k)
2	0.347	1093.2	0.211	665.4	0.151	475.3	0.109	341.8
3	0.338	1063.3	0.205	647.2	0.147	462.3	0.109	341.8
4	0.328	1033.4	0.200	629.0	0.143	449.3	0.109	341.8
5	0.319	1003.5	0.194	610.8	0.139	436.3	0.109	341.8
6	0.368	1160.5	0.224	706.4	0.160	504.6	0.109	341.8
Roof	0.656	2065.0	0.399	1257.0	0.285	897.8	0.109	341.8

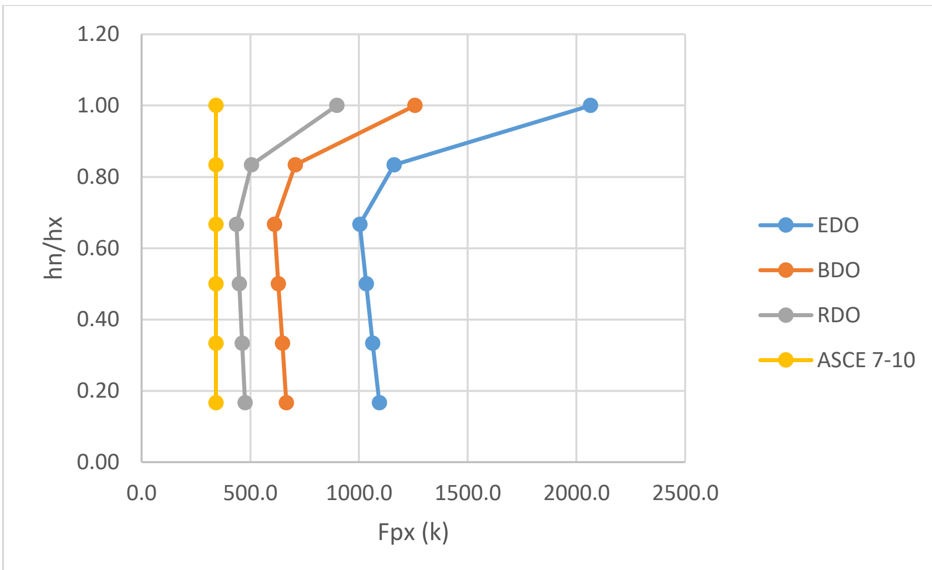

Figure 20: SDC D Diaphragm Design Forces

As seen in the above figures and tables the ASCE 7-10 design forces are smaller than all the ASCE 7-16 design option forces. This is due to the contribution of higher modes on the diaphragm force as well as diaphragm inelastic behavior. Since the ASCE 7-10 design force is also lower than the RDO which allows elastic behavior, it can be concluded that precast diaphragms will behave inelastically under ASCE 7-10 design loads.

When comparing the graphs of design forces from SDC C to SDC D, the gap between the EDO and BDO forces is larger in SDC D. This is due to the difference in seismic demand level.

Since the building in SDC D is in a moderate seismic demand level, the EDO force is to be increased by 15%.

4.2 Internal Forces and Strength Check

The diaphragm design force is applied to the diaphragm to determine the internal forces at the joints between precast panels. Traditionally, the horizontal beam analogy is used to determine diaphragm internal forces. However, it was found that this method does not consider deep beam effects and can result in non-accurate joint forces due to the lack of continuity in precast diaphragms (Fleischman, 2014). Since connections are the weak points in precast diaphragms, the forces at each joint become critical. The free-body method used in this study still treats the diaphragm as a beam but takes a cut at the joints between the precast panels and analyzes the forces on the joint using statics. The cut is taken across the whole depth of the diaphragm. Moment in the joint will be resisted by the chords of the diaphragm while the shear will be resisted by the connectors between the precast panels. However, since the connection between the panels is critical to the diaphragm, interaction of all of the forces acting on the joint will be checked. This report will follow the assumptions laid out by (Fleischman, 2014) since the results were verified to be accurate with these assumptions:

- The shear flow in the perimeter wall (qs_w) will be taken as 5% of the diaphragm inertial force
- The perimeter walls form a force couple which reduces the diaphragm moment between the perimeter wall span. These assumptions are only applicable to the layout used in the parametric study.

Figure 13 above shows the plan view for the building and defined variables used to determine the internal forces at each joint. Figure 21 provides a visual representation of the diaphragm force applied in the transverse direction. Since the cut is taken parallel to the joint, there will be no axial load in the joints under transverse loading. The internal forces were calculated based on the diaphragm design forces by calculated above

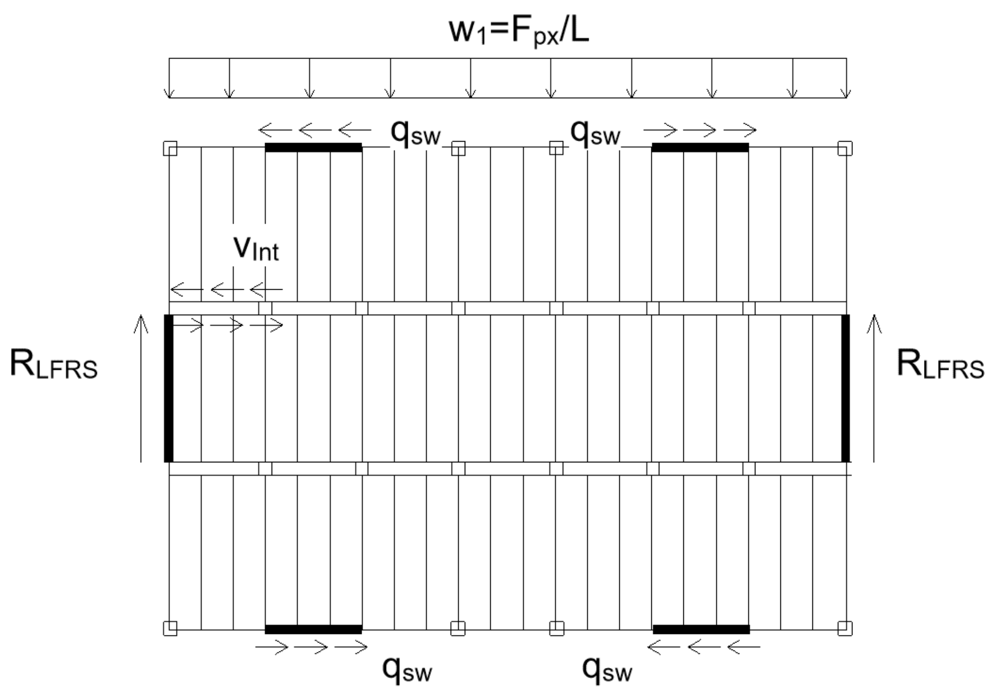

Figure 21: Transverse Loading with Internal Forces

$\underline{\textit{Distributed Load}}$

$w_1 = F_{px}/L$

$\underline{\textit{Boundary Reactions}}$

$R_{LFRS} = w_1 L/2$

$q_{sw} = 0.05 w_1 L/L_{LFRS}$

$\underline{\textit{Primary Diaphragm Joint}}$

$\boldsymbol{0 \leq x \leq L'}$:

$V_u = R_{LFRS} - w_1 x$

$M_u = x R_{LFRS} - w_1 x^2/2$

$\boldsymbol{L' \leq x \leq L' + L_{LFRS}}$:

$V_u = R_{LFRS} - w_1 x$

$$M_u = xR_{LFRS} - \frac{w_1 x^2}{2} - q_{sw}(x - L')d$$

$\boldsymbol{L' + L_{LFRS} \leq x \leq L/2}$:

$$V_u = R_{LFRS} - w_1 x$$

$$M_u = xR_{LFRS} - \frac{w_1 x^2}{2} - q_{sw} L_{LFRS} d$$

$\underline{Internal\ Beam\ Joint}$

$$V_{int} = \frac{V_u Q b}{I} = V_u b(\frac{4}{3d})$$

$\underline{Diaphragm\ to\ Transverse\ Wall\ Joint\ Per\ Wall}$

$$V_u = R_{LFRS}$$

The diaphragm design force must be applied and checked in both directions. Figure 22 depicts the diaphragm force applied in the longitudinal direction. The resultant shear and moment in the joints due to this loading will be zero.

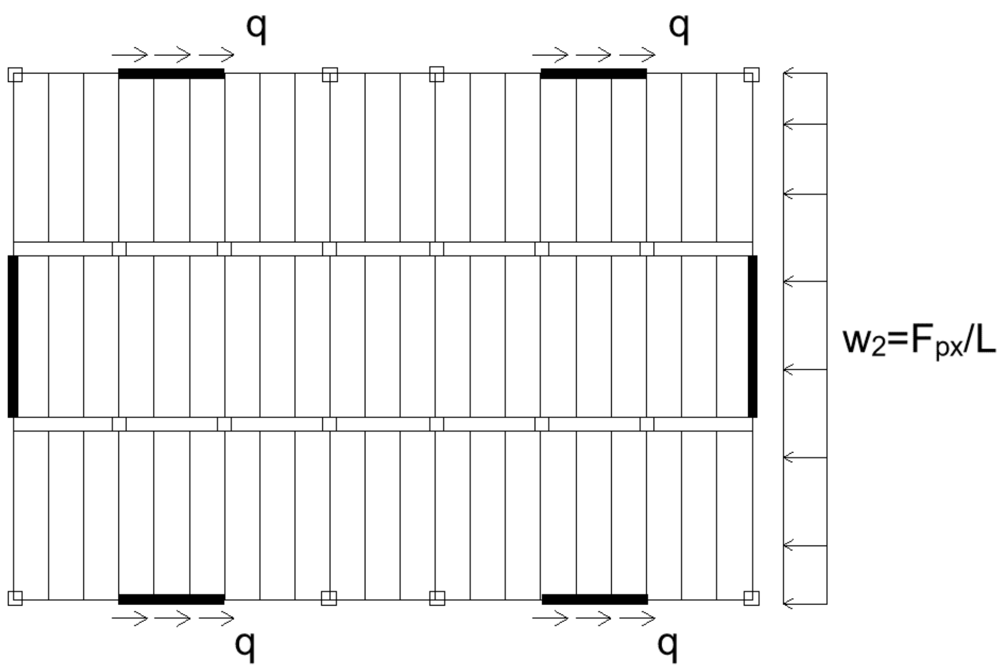

Figure 22: Longitudinal Loading with Internal Forces

$\underline{Distributed\ Load}$

$w_2 = F_{px}/d$

$\underline{Boundary\ Reactions}$

$q = w_2 d/4L_{LFRS}$

$\underline{Primary\ Diaphragm\ Joint}$

$\mathbf{0 \leq x \leq L'}$:

$N_u = w_2 x$

$\mathbf{L' \leq x \leq L' + L_{LFRS}}$:

$N_u = w_2 x - 2q(x - L')$

$\mathbf{L' + L_{LFRS} \leq x \leq L/2}$:

$N_u = w_2 x - 2qL_{LFRS}$

$\underline{Diaphragm\ to\ Longitudinal\ Wall\ Joint\ Per\ Wall}$

$N_u = 0$

$V_u = qL_{LFRS}$

$M_u = 0$

Once diaphragm internal forces are known, the capacity of the connectors and reinforcement must be determined for a strength check. Since diaphragm joints can be under combined tension force, shear force, and moment depending on the layout of the structure. Therefore, the following interaction equation is permitted (Fleischman, 2014):

$$\sqrt{\left(\frac{|M_u|}{\phi_b M_n} + \frac{N_u}{\phi_c N_n}\right)^2 + \left(\frac{\Omega_v V_u}{\phi_v V_n}\right)^2} \leq 1.0$$

$Where$:

$$N_n = \sum t_n$$

$$V_n = \sum v_n$$

The tensile and shear capacity is the summation of all the connector capacities. Additionally, the topping reinforcement str is added to the shear capacity. The shear forces are taken by the connectors and topping reinforcement, while the moment is taken by the chord reinforcement. The nominal flexural moment capacity of the joint will depend on the diaphragm design option:

$\underline{For\ EDO:}$

$M_n = M_y$

$\underline{For\ BDO:}$

$$M_n = \frac{1 + \Omega_d}{2} M_y$$

$\underline{For\ RDO:}$

$M_n = \Omega_d M_y$

$Where:$

$M_y\ is\ the\ diaphragm\ joint\ yield\ moment$

$\Omega_d\ the\ flexural\ diaphragm\ overstrength\ factor$

M_y is allowed to be conservatively taken as:

$M_y = t_n{}^{chord} d'$

$Where:$

$t_n^{chord}\ is\ the\ chord\ yield\ force\ (A_s f_y)$

$d'\, is\ the\ depth\ between\ chords$

Ω_d is the ratio of the diaphragm's plastic moment capacity to the diaphragm's yield moment capacity (M_p/M_y). Ω_d may be conservatively taken as 1.0 for pretopped precast diaphragms and 1.25 for topped diaphragms (Fleischman, 2014). Reinforcement and connectors should be provided to adequately resist applied loads per the interaction equation above.

The strengths provided below are from Fleischman (2014) and are used to determine the diaphragm reinforcement design. Pre-approved connectors and reinforcements with their capacities can be found in Table 12 and Table 13. The capacities of these connectors are the result of the required testing per the ASCE 7-16 section 14.2.4.4 and ACI 550.4-18 and were obtained from the attached sources (Fleischman, 2014; Ren & Naito, 2013).

Table 12: Reinforcing Bar Properties, adapted from (Fleischman, 2014)

Reinforcing bars	Classification	k_t/A (k/in/in^2)	t_n/A (ksi)	δ_{ty} (in)	δ_{tu} (in)	k_v/A (k/in/in^2)	V_n/A (ksi)	δ_{vy} (in)
Dry Chord Gr. 60	LDE	1018	60	0.071	0.1	382	24.2	0.09
Dry chord w/ flat plate Gr. 60	MDE	1018	60	0.071	0.3	382	24.2	0.09
Pour strip chord Gr. 60	HDE	1234	60	0.057	0.7	382	24.2	0.9
Ductile Ladder Gr. 1018	HDE	1260	54	0.043	0.6	217	21.7	0.1
Standard ASTM A185 wwr	LDE	1414	65	0.035	0.1	709	39.7	0.56

Table 13: Connector Properties, adapted from (Fleischman, 2014)

Connectors	Classification	k_t (k/in)	t_n (k)	δ_{ty} (in)	δ_{tu} (in)	k_v (k/in)	V_n (k)	δ_{vy} (in)
JVI	HDE	55	3.1	0.066	0.6	226	18.1	0.082
Hairpin #4	HDE	209	9	0.043	0.6	181	18.1	0.1
Angled Bar #3	MDE	300	10.2	0.059	0.3	372	17.1	0.045

The provided capacities and reinforcement options are not required by code. Any reinforcement and connector with corresponding properties can be used as long as the elements satisfy the testing requirements and classifications per ASCE 7-16.

4.3 Diaphragm Connection and Reinforcement Design

For the following results, chord reinforcement, connector reinforcement, collector reinforcement, and topping reinforcement are classified as group #1 reinforcement. Reinforcement connecting the shear walls and beams to the diaphragm, as well as the beam collectors are classified as group #2 reinforcement. Figure 23 demonstrates the placement of the resultant reinforcement. Required shear reinforcement can be refined as the joints approach mid length, but the worst case required shear reinforcement is shown.

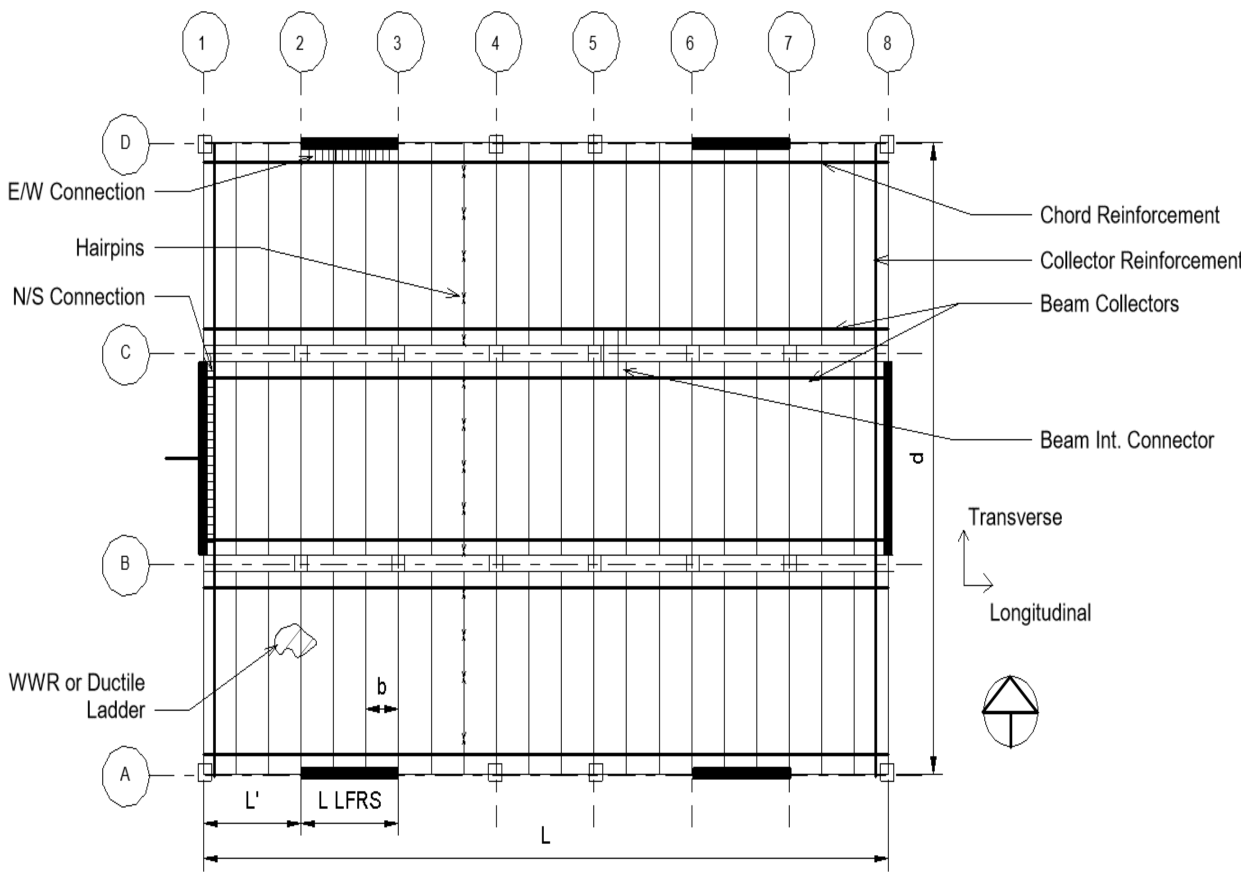

Figure 23: Reinforcement layout

4.3.1 SDC C Design

Table 14: SDC C Roof Diaphragm Group #1 Reinforcement

Roof											
	Chord		Hairpin		Topping Reinforcement					Collector	
Design Option	Size	#	#	s(ft)	Area (in^2/ft)	Pattern		Ductile Ladder	Type	Size	#
EDO	#7	6	40	3.7	0.029	10x	10	No	W2.9xW2.9	#10	9
BDO	#6	5	38	3.8	0.029	10x	10	No	W2.9xW2.9	#8	10
RDO	#5	5	33	4.4	0.029	6x	6	Yes	W2.9xW2.9	#7	9

Table 15: SDC C Roof Diaphragm Group #2 Reinforcement

Roof								
	N/S Connection		E/W Connection		Beam Int. Connection		Beam Collectors	
Design Option	Size	#	Size	#	Size	#	Size	#
EDO	#10	34	#10	17	#8	5	#6	2
BDO	#8	38	#8	19	#7	5	#5	2
RDO	#7	35	#7	18	#7	4	#5	2

4.3.2 SDC D Design

Table 16: SDC D Roof Group #1 Reinforcement

Roof											
	Chord		Hairpin		Topping Reinforcement					Collector	
Design Option	Size	#	#	s(ft)	Area (in^2/ft)	Pattern		Ductile Ladder	Type	Size	#
EDO	#8	7	35	4.2	0.029	6	6	No	W2.9xW2.9	#10	12
BDO	#7	5	30	4.9	0.029	6	6	No	W2.9xW2.9	#9	9
RDO	#5	6	40	3.7	0.029	6	6	Yes	W2.9xW2.9	#8	8

Table 17: SDC D Roof Group #2 Reinforcement

Roof								
	N/S Connection		E/W Connection		Beam Int. Connection		Beam Collectors	
Design Option	Size	#	Size	#	Size	#	Size	#
EDO	#10	45	#10	23	#8	7	#6	3
BDO	#9	35	#9	18	#7	6	#5	3
RDO	#8	32	#8	16	#6	5	#5	2

4.4 Discussion

The design option chosen for the procedure in the ASCE 7-16 will directly affect the diaphragm design force and the amount of reinforcement needed in the diaphragm. A higher seismic design category will result in a higher diaphragm design force and more required reinforcement since the response spectral acceleration values are higher. In addition to the higher spectral acceleration, the seismic demand level is affected by the SDC. With a higher SDC some of the design option forces are required to be increased, as observed with an EDO in the parametric study. It is also observed that the ASCE 7-10 diaphragm design forces are low compared to the design procedure in the ASCE 7-16 required for precast diaphragms.

As expected, an EDO produced the highest diaphragm design force. A BDO resulted in the moderate diaphragm design force, followed with the lowest by RDO. An EDO produced the highest diaphragm design force because the method targets an elastic diaphragm response during an MCE seismic event. To achieve an elastic response the diaphragm needs to be designed to a higher force. Since the design force is large, low deformability reinforcement is allowed to be used. However, since the study includes a topped precast diaphragm with HDE chord reinforcement, an EDO design is not economical. In contrast to an EDO, the RDO produced the lowest diaphragm design force. Due to the target inelastic behavior of the RDO during both the

DBE and MCE, the design force lower. To avoid a brittle shear failure and achieve the inelastic behavior of the diaphragm, a ductile ladder is needed in the RDO option. Since the ductile ladder has less shear capacity than the standard WWR, more slab reinforcement and shear reinforcement is required. On the other hand, the RDO option requires the least amount of chord and collector reinforcement due to the lower diaphragm design force. Since the ductile ladder reinforcement is often more difficult to install and requires more labor, this option is also not very economical. After viewing the results, the BDO appears to be the most economical solution for the parametric study. The BDO does not require as much chord reinforcement as the EDO and does not require as much shear reinforcement as the RDO.

The same trend was observed throughout all levels of the building. The EDO option produced the highest design force with the most reinforcement required in all categories except for a few locations of shear reinforcement required for the RDO. The BDO option produced a diaphragm design option with reasonable reinforcement requirements and does not require ductile ladder reinforcement.

4.4.1 Design Option Recommendation

It is determined that the most practical solution for the proposed parametric study would be the Basic Design Option. Though the BDO design requires larger and more reinforcement for the chords and the connections to the vertical elements of the LFRS than an RDO option, less shear reinforcement is required. The RDO option may save money and look economical due to the least amount of material, the labor cost and time to install a ductile ladder would be larger than the BDO option. From a constructability standpoint, extra time spent to install the shear reinforcement could mean increased cost. The increase in bar size and number for the other elements of the diaphragm would not make a large difference in the time of construction. It is recommended that the BDO be used for projects where the occupancy is non-life critical. The RDO option is recommended for projects not on a strict timeline or budget.

Additionally, the EDO option requires both a large amount of chord and collector reinforcement and shear reinforcement. This option is uneconomical unless designing for a life-critical facility or in a low seismic demand level. Since the diaphragm is to remain in the elastic range under an MCE event, it was expected that the amount of reinforcement would be the largest and therefore should be avoided unless budget allows.

Chapter 5: Conclusion

After investigating failed precast structures during the 1994 Northridge earthquake, insufficient precast diaphragms were found to be the cause of the major failures. Research projects were conducted to ensure diaphragm design forces were accurately represented in code provisions. After comparison, it was found that the previous code provisions were unconservative when determining diaphragm design forces. This is because previous code provisions assumed that only the fundamental mode attributed to the seismic diaphragm force along the height of the building. However, research results showed that diaphragm forces are not only dependent on the fundamental modal response, but higher modes as well. Another factor to the underestimated design force was the behavior of the precast diaphragm. Precast concrete diaphragms were assumed to remain elastic during a seismic event. Testing showed that in design performed in accordance with previous provisions, this was not the case. The inelastic behavior observed was the result of the lack of strength and excessive inelastic deformation which occurred between the panel joints. Therefore, changes were warranted and derived for the diaphragm force design provisions for the ASCE 7-16.

To account for the higher modal effects, new diaphragm provisions were produced. The new provisions are required for precast diaphragms. The provisions in ASCE 7-16 include a diaphragm acceleration coefficient, C_{px}. This coefficient encompasses the higher mode affects observed in horizontal floor acceleration. Since it was determined that the connections are critical in precast diaphragm performance, different design options requiring specific reinforcement ductility demands with different diaphragm behaviors were developed. The following design options were adopted into the ASCE 7-16: Elastic Design Option, Basic Design Option, and Reduced Design Option. With the addition of the design options, the diaphragm reduction factor is also introduced in the code. This factor corresponds to the different targeted behaviors of the design options listed above and will directly affect the diaphragm design force in the new provisions.

Furthermore, through the parametric study it was overserved that the diaphragm design forces in accordance with the ASCE 7-16 are larger than the those of ASCE 7-1. It was also observed that the different diaphragm design options will produce different design forces and connection and reinforcement requirements. The EDO resulted in the highest diaphragm design force with the most reinforcement required, the BDO resulted in a moderate design force and reinforcement required, followed by the RDO with the least design force. Though the RDO resulted in the lowest design force, a lot of ductile shear reinforcement is required.

The seismic design category also affected the diaphragm design force. When the seismic design category is increased from SDC C to SDC D, the seismic demand level may increase which could prohibit the use of certain design options. Moreover, an increase in the SDC would correspond to an increase in the spectral acceleration response and result in a higher diaphragm design force.

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Appendix A - Diaphragm Designs and Design Example

The following tables are the reinforcement design for levels 2-6 in the parametric study. A design example for all levels of the diaphragm in SDC C using a Basic Design Option can be found immediately after the tables.

Table 18: SDC C Level 2 Diaphragm Group #1 Reinforcement

Level 2										
	Chord		Hairpin		Topping Reinforcement				Collector	
Design Option	Size	#	#	s(ft)	Area (in^2/ft)	Pattern		Type	Size	#
EDO	#6	5	15	9.7	0.029	10x	10	W2.9xW2.9	#8	7
BDO	#5	4	16	9.1	0.029	10x	10	W2.9xW2.9	#6	9
RDO	#4	4	17	8.6	0.029	8x	8	W2.9xW2.9	#6	7

Table 19: SDC C Level 2 Diaphragm Group #2 Reinforcement

Level 2								
	N/S Connection		E/W Connection		Beam Int. Connection		Beam Collectors	
Design Option	Size	#	Size	#	Size	#	Size	#
EDO	#8	28	#8	14	#7	4	#5	2
BDO	#6	35	#6	18	#7	3	#4	2
RDO	#6	25	#6	13	#5	4	#3	2

Table 20:SDC C Level 3 Diaphragm Group #1 Reinforcement

Level 3										
	Chord		Hairpin		Topping Reinforcement				Collector	
Design Option	Size	#	#	s(ft)	Area (in^2/ft)	Pattern		Type	Size	#
EDO	#6	5	15	9.7	0.029	10x	10	W2.9xW2.9	#8	7
BDO	#5	4	16	9.1	0.029	10x	10	W2.9xW2.9	#7	7
RDO	#4	4	17	8.6	0.029	8x	8	W2.9xW2.9	#6	7

Table 21: SDC C Level 3 Diaphragm Group #2 Reinforcement

Level 3								
	N/S Connection		E/W Connection		Beam Int. Connection		Beam Collectors	
Design Option	Size	#	Size	#	Size	#	Size	#
EDO	#8	28	#8	14	#7	4	#5	2
BDO	#7	26	#7	13	#7	3	#4	2
RDO	#6	25	#6	13	#5	4	#3	2

Table 22: SDC C Level 4 Diaphragm Group #1 Reinforcement

Level 4										
	Chord		Hairpin		Topping Reinforcement				Collector	
Design Option	Size	#	#	s(ft)	Area (in^2/ft)	Pattern		Type	Size	#
EDO	#6	5	15	9.7	0.029	10x	10	W2.9xW2.9	#8	7
BDO	#5	4	16	9.1	0.029	10x	10	W2.9xW2.9	#7	7
RDO	#4	4	17	8.6	0.029	8x	8	W2.9xW2.9	#6	7

Table 23: SDC C Level 4 Diaphragm Group #2 Reinforcement

Level 4								
	N/S Connection		E/W Connection		Beam Int. Connection		Beam Collectors	
Design Option	Size	#	Size	#	Size	#	Size	#
EDO	#8	28	#8	14	#8	3	#5	2
BDO	#7	26	#7	13	#7	3	#4	2
RDO	#6	25	#6	13	#5	4	#3	2

Table 24: SDC C Level 5 Diaphragm Group #1 Reinforcement

Level 5										
	Chord		Hairpin		Topping Reinforcement				Collector	
Design Option	Size	#	#	s(ft)	Area (in^2/ft)	Pattern		Type	Size	#
EDO	#6	5	15	9.7	0.029	10x	10	W2.9xW2.9	#9	6
BDO	#5	4	16	9.1	0.029	10x	10	W2.9xW2.9	#7	7
RDO	#4	4	17	8.6	0.029	8x	8	W2.9xW2.9	#7	5

Table 25: SDC C Level 5 Diaphragm Group #2 Reinforcement

Level 5								
	N/S Connection		E/W Connection		Beam Int. Connection		Beam Collectors	
Design Option	Size	#	Size	#	Size	#	Size	#
EDO	#9	22	#9	11	#7	4	#5	2
BDO	#7	26	#7	13	#7	3	#4	2
RDO	#7	19	#7	10	#5	4	#3	2

Table 26: SDC C Level 6 Diaphragm Group #1 Reinforcement

Level 6										
	Chord		Hairpin		Topping Reinforcement				Collector	
Design Option	Size	#	#	s(ft)	Area (in^2/ft)	Pattern		Type	Size	#
EDO	#6	5	20	7.3	0.029	10x	10	W2.9xW2.9	#9	7
BDO	#5	5	18	8.1	0.029	10x	10	W2.9xW2.9	#7	8
RDO	#4	5	20	7.3	0.029	8x	8	W2.9xW2.9	#7	6

Table 27: SDC C Level 6 Diaphragm Group #2 Reinforcement

Level 6								
	N/S Connection		E/W Connection		Beam Int. Connection		Beam Collectors	
Design Option	Size	#	Size	#	Size	#	Size	#
EDO	#9	25	#9	13	#8	3	#5	2
BDO	#7	30	#7	15	#7	3	#5	2
RDO	#9	25	#9	13	#8	3	#5	2

Table 28: SDC D Level 2 Diaphragm Group #1 Reinforcement

Level 2										
	Chord		Hairpin		Topping Reinforcement				Collector	
Design Option	Size	#	#	s(ft)	Area (in^2/ft)	Pattern		Type	Size	#
EDO	#7	5	18	8.1	0.029	8	8	W2.9xW2.9	#8	10
BDO	#5	5	20	7.3	0.029	10	10	W2.9xW2.9	#7	8
RDO	#4	5	13	11.2	0.029	6	6	W2.9xW2.9	#6	8

Table 29: SDC D Level 2 Diaphragm Group #2 Reinforcement

Level 2								
	N/S Connection		E/W Connection		Beam Int. Connection		Beam Collectors	
Design Option	Size	#	Size	#	Size	#	Size	#
EDO	#8	39	#8	20	#7	5	#5	2
BDO	#7	31	#7	16	#7	3	#4	2
RDO	#6	30	#6	15	#5	4	#4	2

Table 30: SDC D Level 3 Diaphragm Group #1 Reinforcement

Level 3										
	Chord		Hairpin		Topping Reinforcement				Collector	
Design Option	Size	#	#	s(ft)	Area (in^2/ft)	Pattern		Type	Size	#
EDO	#7	5	18	8.1	0.029	8	8	W2.9xW2.9	#8	10
BDO	#5	5	20	7.3	0.029	10	10	W2.9xW2.9	#7	8
RDO	#4	5	15	9.7	0.029	6	6	W2.9xW2.9	#6	8

Table 31: SDC D Level 3 Diaphragm Group #2 Reinforcement

Level 3								
	N/S Connection		E/W Connection		Beam Int. Connection		Beam Collectors	
Design Option	Size	#	Size	#	Size	#	Size	#
EDO	#8	38	#8	19	#7	5	#5	2
BDO	#7	30	#7	15	#7	3	#5	2
RDO	#6	29	#6	15	#5	4	#4	2

Table 32: SDC D Level 4 Diaphragm Group #1 Reinforcement

Level 4										
	Chord		Hairpin		Topping Reinforcement				Collector	
Design Option	Size	#	#	s(ft)	Area (in^2/ft)	Pattern		Type	Size	#
EDO	#7	5	18	8.1	0.029	8	8	W2.9xW2.9	#9	8
BDO	#5	5	20	7.3	0.029	10	10	W2.9xW2.9	#7	8
RDO	#4	5	15	9.7	0.029	6	6	W2.9xW2.9	#6	8

Table 33: SDC D Level 4 Diaphragm Group #2 Reinforcement

Level 4								
	N/S Connection		E/W Connection		Beam Int. Connection		Beam Collectors	
Design Option	Size	#	Size	#	Size	#	Size	#
EDO	#9	29	#9	15	#8	4	#5	2
BDO	#7	29	#7	15	#7	3	#5	2
RDO	#6	29	#6	15	#5	4	#4	2

Table 34: SDC D Level 5 Group #1 Reinforcement

	Chord		Hairpin		Topping Reinforcement				Collector	
Design Option	Size	#	#	s(ft)	Area (in^2/ft)	Pattern		Type	Size	#
EDO	#7	5	18	8.1	0.029	8	8	W2.9xW2.9	#9	7
BDO	#5	5	20	7.3	0.029	10	10	W2.9xW2.9	#7	8
RDO	#4	5	15	9.7	0.029	6	6	W2.9xW2.9	#7	6

Table 35: SDC D Level 5 Group #2 Reinforcement

Level 5								
	N/S Connection		E/W Connection		Beam Int. Connection		Beam Collectors	
Design Option	Size	#	Size	#	Size	#	Size	#
EDO	#9	28	#9	14	#7	5	#5	2
BDO	#7	29	#7	15	#7	3	#5	1
RDO	#7	21	#7	11	#5	4	#4	2

Table 36: SDC D Level 6 Group #1 Reinforcement

Level 6										
	Chord		Hairpin		Topping Reinforcement				Collector	
Design Option	Size	#	#	s(ft)	Area (in^2/ft)	Pattern		Type	Size	#
EDO	#7	5	20	7.3	0.029	8	8	W2.9xW2.9	#10	7
BDO	#5	5	21	7.0	0.029	10	10	W2.9xW2.9	#8	7
RDO	#4	5	15	9.7	0.029	6	6	W2.9xW2.9	#7	6

Table 37: SDC D Level 6 Group #2 Reinforcement

Level 6								
	N/S Connection		E/W Connection		Beam Int. Connection		Beam Collectors	
Design Option	Size	#	Size	#	Size	#	Size	#
EDO	#10	26	#10	13	#8	4	#5	2
BDO	#8	25	#8	13	#7	3	#5	2
RDO	#7	24	#7	12	#6	3	#4	2

Step	Calc				Ref
	Building Parameters:				
	Chattano0ga TN.	L:	210	ft	
		B:	150	ft	
		Level 2 height:	15	ft	
		Typical height:	15	ft	
		Roof height:	15	ft	
	Concrete Properties:				
		f'c:	3500	psi	
		λ:	1		
		fy:	60000	psi	
		Conc. Density:	145	pcf	
	Seismic Provisions:				ASCE7-16
		S_s:	0.467		
		S_1:	0.122		
		F_a:	1.426		T11.4-1
		F_v:	2.356		
		Risk Category:	II		
		I_e:	1		
		$S_{MS} = F_a\ S_s$:	0.6659		
		$S_{M1} = F_v\ S_1$:	0.2874		
		$S_{DS} = \frac{2}{3} S_{MS}$:	0.444		(11.4-3)
		$S_{D1} = \frac{2}{3} S_{M1}$:	0.1916		(11.4-4)
	Seismic Design Category:				
		SDC:	C		T11.6-1
		SDC:	C		T11.6-2
		SDC C governs			
	N/S and E/W Special RC Shear Wall	R=	6		T12.2-1
		$V = C_s W$			(12.8-1)
		$C_s = \frac{S_{DS}}{(\frac{R}{I_e})}$:	0.074		(12.8-2)
		$C_{smin} = 0.044 S_{DS} I_e > 0.01$:	0.0195		(12.8-5)
		Ct:	0.02		T12.8-2
		x:	0.75		
		h:	90	ft	
		$T_a = C_t {h_n}^x$:	0.5844	sec	(12.8-7)
		Assuming $T<T_L$			

Step	Calc	Ref
	$C_{smax} = \frac{S_{D1}}{T(\frac{R}{I_e})}$: 0.0546	
	Governing Cs: 0.0546	
	Floor DL: 100 psf	
	Roof DL: 100 psf	
	Cladding Gravity Load: 0 psf	
	L: 210 ft	
	B: 150 ft	
	Typical Story to Story height: 15 ft	
	2nd story height: 15 ft	
	Roof height: 15 ft	
	Number of typical levels: 5	
	W: 18900 kips	
	$V = C_sW$: 1032.86 kips	(12.8-11)

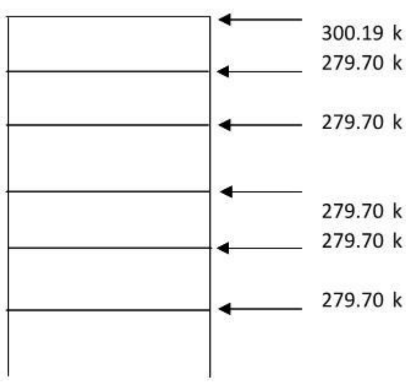
ASCE7-10 Forces

Level	h_x(ft)	W_x(k)	$W_xh_x^k$	C_{vx}	F_x(k)	F_{px}	V_x(k)
Roof	90	3150	342785.2	0.291	300.2	300.2	300.2
6th	75	3150	283465	0.240	248.2	279.7	548.4
5th	60	3150	224646.6	0.190	196.7	279.7	745.2
4th	45	3150	166451.9	0.141	145.8	279.7	890.9
3rd	30	3150	109085.3	0.092	95.5	279.7	986.5
2nd	15	3150	52970.35	0.045	46.4	279.7	1032.9
Σ		18900	1179404	1.0000			

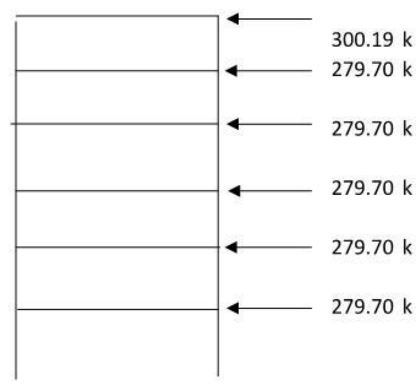
k= 1.042

Lateral Seismic Force:

$$C_{vx} = \frac{W_x {h_x}^k}{\sum W_i {h_i}^k}$$



Step	Calc		Ref
	Story Shears:		



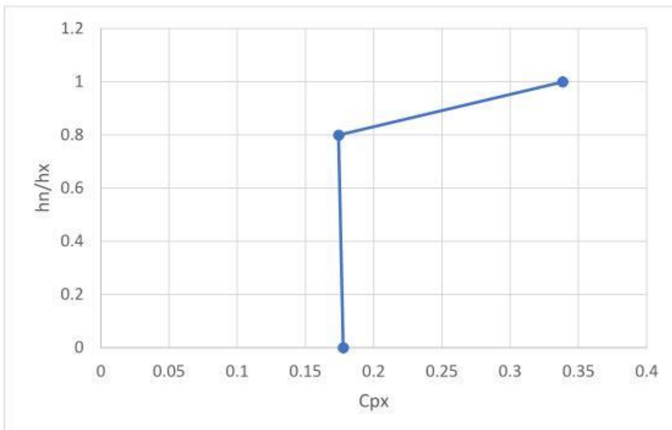
<u>Alternative Design Provisions 12.10.3:</u>

Step	Calc		Ref
	Level:	2nd	
	N:	6	
	hx:	15 ft	
	hn:	90 ft	
	z_s:	1.0	12.10.3.2
	Cs:	0.0546	
	Ω_O:	2.5	T12.2.1
	Wpx:	3150 k	
	$F_{px} = \frac{C_{px}}{R_s} W_{px}$		(12.10-4)
	$C_{p0} = 0.4 S_{DS} I =$	0.1776	(12.10-6)
	$\Gamma_{m1} = 1 + \frac{z_s}{2}\left(1 - \frac{1}{N}\right) =$	1.4167	(12.10-13)
	$\Gamma_{m2} = 0.9 z_s \left(1 - \frac{1}{N}\right)^2 =$	0.625	(12.10-14)
	Cpi is max of following:		
	$C_{pi} = 0.8 C_{p0} =$	0.1421	(12.10-8)
	$C_{pi} = 0.9 \Gamma_{m1} C_s \Omega_O =$	0.1742	(12.10-9)
	C_{pi}:	0.1742	
	Cs2 is min of the following:		
	$C_{s2} = (0.15N + 0.25) I_e S_{DS} =$	0.5106	(12.10-10)
	$C_{s2} = I_e S_{DS} =$	0.444	(12.10-11)
	$C_{s2} = \frac{I_e S_{D1}}{0.03(N-1)} =$	1.2775	(12.10-12a/b
	Cs2:	0.444	

Step	Calc	Ref
	$C_{pn} = \sqrt{(\Gamma_{m1}\Omega_o C_s)^2 + (\Gamma_{m2} C_{s2})^2} \geq C_{pi}$ 0.3383	(12.10-7)
	hx/hn: 0.1667	

Table for Graph:

hx/hn	C
0	0.17758
0.8	0.17419
1	0.33831

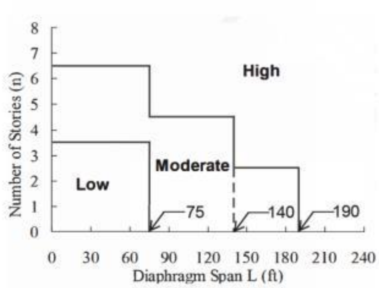


F12.10-2

<u>Determine Seismic Demand Level</u>

AR: 1.4

n: 6



Seismic Demand Level: Low

Design Option: BDO

Rs: 1

Step	Calc	Ref
	Cpx: 0.177	T12.10-1
	Wpx: 3150 k	
	$F_{px} = \frac{C_{px}}{R_s} W_{px} =$ 557.17 k	12.10-4
	$\Omega_v = 1.4R_s =$ 1.4	
	Alt provisions or normal? Alt	
	Fx to use: 557.17 k	

ASCE 7-10
Tabulated Values:

Level	Fx Norm
Roof	300.19
6th	279.70
5th	279.70
4th	279.70
3rd	279.70
2nd	279.70

Step	Calc	Ref

Building Parameters:

Design Option: BDO

Transverse is N/S — L= 210 ft

Longitudinal is E/W — d= 150 ft

L'= 30 ft

L_{LFRS}= 30 ft

b= 10 ft

L_{beam}= 30 ft

Diaphragm Internal Forces:

Longitudinal Loading:

Distributed Load:

$w = F_{px}/L$ = 2.65 klf

Boundary Reaction:

$R_{LFRS} = wL/2$ = 278.6 k

$q_{sw} = 0.05wL/L_{LFRS}$ = 0.929 klf

Transverse Loading:

Distributed Load:

$n = F_{px}/L$ = 2.65 klf

Boundary Reaction:

$q = nL/4L_{LFRS}$ = 4.64 klf

Primary Joint:

		Load Direction					
		Transverse			Longitudinal		
Joint	x(ft)	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
1	10	0	352.9	2653	26.5	0	0
2	20	0	315.7	5041	53.1	0	0
3	30	0	278.6	7164	79.6	0	0
4	40	0	241.4	7628	13.3	0	0
5	50	0	204.3	7827	-53.1	0	0
6	60	0	167.1	7761	-119.4	0	0
7	70	0	130.0	8822	-92.9	0	0
8	80	0	92.9	9618	-66.3	0	0
9	90	0	55.7	10148	-39.8	0	0
10	100	0	18.6	10414	-13.3	0	0

Note: Other side is symmetric and shear overstrength is applied

Internal Beam Joint:

Load Direction					
Transverse			Longitudinal		
N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
0	24.76	0	0	0	0

Step	Calc	Ref
	Other Joints:	
	Diaphragm to Transverse Wall Joint	

Load Direction	Transverse			Longitudinal		
	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
	0	278.6	0	0	0	0

Diaphragm to Longitudinal Wall Joint

Load Direction	Transverse			Longitudinal		
	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
	0	0.0	0	0	139.3	0

Note: small contribution form qsw can be ignored since it will not govern

Reinforcement Selection:

Chord Reinf: Pour strip chord Gr. 60
Shear Reinf 1: Standard ASTM A185 wwr
Shear Reinf 2: Hairpin #4
Collector Reinf: Pour strip chord Gr. 60
Internal Beam Joint: Pour strip chord Gr. 60
Internal Beam Joint Collector: Pour strip chord Gr. 60

Determine Reinforcement Strength:

Topped or Untopped: Topped

$$V_n = \sum v_n$$

$$N_n = \sum t_n$$

For EDO: $M_n = M_y$ 10862.4 k-ft

For BDO: $M_n = \frac{1+\Omega_d}{2} M_y$ 12220 k-ft

For RDO: $M_n = \Omega_d M_y$ 13578 k-ft

M_y: 10862.4 k-ft
d': 146 ft
Ω_d: 1.25
My for design option: 12220.2 k-ft

Step	Calc					Ref
	Chord Reinf.					
			Size:	#5		
	k_t/A (k/in/in^2)	1234	#bars:	4		
	t_n/A (ksi)	60	As	1.24	in^2	
	k_v/A (k/in/in2)	382	d_b	0.625	in^2	
	t_v/A (ksi)	24.2	fy	60	ksi	
	Connector and Collector Reinf:					
	k_t/A (k/in/in^2)	1234	Size:	#6		
	t_n/A (ksi)	60	#bars:	1		
	k_v/A (k/in/in2)	382	As	0.44	in^2	
	V_n/A (ksi)	24.2	d_b	0.75	in^2	
	Shear Reinf 1					
	k_t/A (k/in/in^2)	1414	WWR:	W2.9xW2.9		ACI 318-19
	t_n/A (ksi)	65	10x 10			App B
	k_v/A (k/in/in2)	709				
	V_n/A (ksi)	39.7	Area:	0.029	in^2	
	Shear Reinf 2					
	k_t (k/in)	209				
	t_n (k)	9				
	k_v(k/in)	181				
	v_n(k)	18.1				
	Internal Beam Joint:					
	k_t/A (k/in/in^2)	1234	Size:	#7		
	t_n/A (ksi)	60	#bars:	1		
	k_v/A (k/in/in2)	382	As	0.6	in^2	
	V_n/A (ksi)	24.2	d_b	0.875	in^2	
	Internal Beam Collector:					
	k_t/A (k/in/in^2)	1234	Size:	#4		
	t_n/A (ksi)	60	#bars:	1		
	k_v/A (k/in/in2)	382	As	0.2	in^2	
	V_n/A (ksi)	24.2	d_b	0.5	in^2	

Step	Calc								Ref
	Joint	Chord		Hairpin		W2.9xW2.9		M-N-V	
	#	Size	#	#	s(ft)	Area(in^2)	10x"	N/S	E/W
	1	#5	4	16	9.13	0.029	10	0.907	0.202
	2	#5	4	16	9.13	0.029	10	0.906	0.404
	3	#5	4	16	9.13	0.029	10	0.949	0.606
	4	#5	4	16	9.13	0.029	10	0.916	0.101
	5	#5	4	16	9.13	0.029	10	0.873	0.404
	6	#5	4	16	9.13	0.029	10	0.818	0.909
	7	#5	4	16	9.13	0.029	10	0.864	0.707
	8	#5	4	16	9.13	0.029	10	0.904	0.505
	9	#5	4	16	9.13	0.029	10	0.933	0.303
	10	#5	4	16	9.13	0.029	10	0.948	0.101

Ω_v: 1.4

ϕ_b: 0.9

ϕ_c: 0.9

ϕv: 0.75

Interaction Equation:

$$\sqrt{(\frac{|M_u|}{\phi_b M_n}+\frac{N_u}{\phi_c N_n})^2+(\frac{\Omega_v V_u}{\phi_v V_n})^2} \leq 1.0$$

Diaphragm to LFRS:

Anchorage	L(ft)	V_u (k)	N_u (k)	M_u (k-ft)	v_n (k)	t_n (k)
N/S Frame	50	278.6	0	0	10.648	26.4
E/W Frame	30	139.3	0	0	10.648	26.4

N/S Frame Required: 35.0

E/W Frame Required: 18.0

Provide 35 #6 bars for N/S Frame

Provide 18 #6 bars for E/W Frame

Collector Design(same demand as anchorage):

Note: per ASCE 7-16 Collector forces are multiplied by 1.5

$A_s=1.5V_u/\phi fy$: 3.869 in^2

Provide 9 #6bars at face of each exterior wall

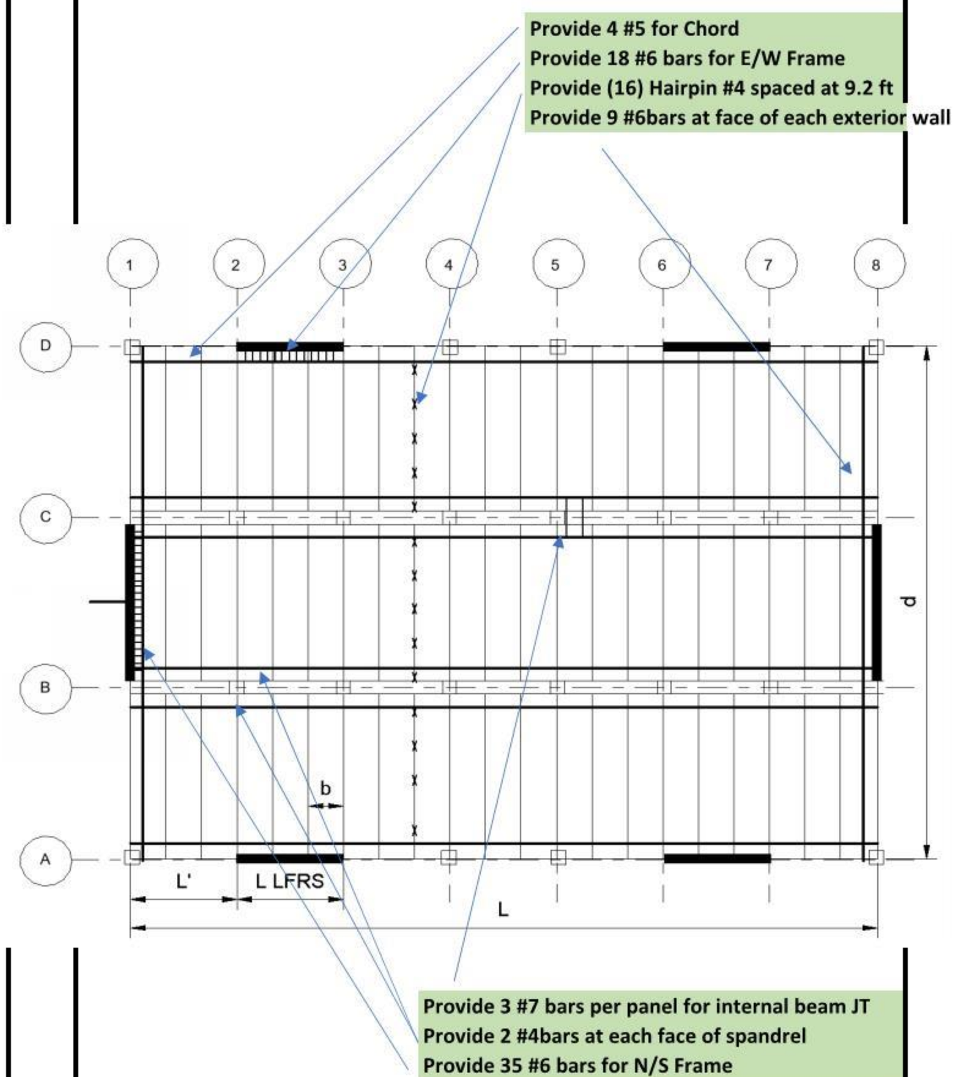
Internal Beam Joint:

Anchorage	N(k)	V(k)	M(k-ft)	v_n (k)	t_n (k)
Int. Joint	0	24.76	0	14.52	36
Collector:	0	24.76	0	4.84	12
				Required:	3

Provide 3 #7 bars per panel for internal beam JT

Provide 2 #4bars at each face of spandrel

Step	Calc	Ref



Step	Calc	Ref
	Check for Drift check:	

Design Option	Cases Requiring Drift Check		
	n≤3	3<n≤5	n>5
EDO	-	-	AR>3.5
BDO	AR>3.8	AR>3.5	AR>3.2
RDO	AR>3.6	AR>3.4	AR>3.1

AR: 1.4

Check: **Not Required**

Step	Calc			Ref
	Building Parameters:			
	Chattano0ga TN.	L:	210 ft	
		B:	150 ft	
		Level 2 height:	15 ft	
		Typical height:	15 ft	
		Roof height:	15 ft	
	Concrete Properties:			
		f'c:	3500 psi	
		λ:	1	
		fy:	60000 psi	
		Conc. Density:	145 pcf	
	Seismic Provisions:			ASCE7-16
		S_s:	0.467	
		S_1:	0.122	
		F_a:	1.426	T11.4-1
		F_v:	2.356	
		Risk Category:	II	
		I_e:	1	
		$S_{MS} = F_a\ S_s$:	0.6659	
		$S_{M1} = F_v\ S_1$:	0.2874	
		$S_{DS} = \frac{2}{3} S_{MS}$:	0.444	(11.4-3)
		$S_{D1} = \frac{2}{3} S_{M1}$:	0.1916	(11.4-4)
	Seismic Design Category:			
		SDC:	C	T11.6-1
		SDC:	C	T11.6-2
		SDC C governs		
	N/S and E/W Special RC Shear Wall	R=	6	T12.2-1
		$V = C_s W$		(12.8-1)
		$C_s = \frac{S_{DS}}{(\frac{R}{I_e})}$:	0.074	(12.8-2)
		$C_{smin} = 0.044 S_{DS} I_e > 0.01$:	0.0195	(12.8-5)
		Ct:	0.02	T12.8-2
		x:	0.75	
		h:	90 ft	
		$T_a = C_t h_n{}^x$:	0.5844 sec	(12.8-7)
		Assuming $T<T_L$		

Step	Calc	Ref

$$C_{smax} = \frac{S_{D1}}{T(\frac{R}{I_e})}: \ 0.0546$$

Governing Cs:	0.0546	
Floor DL:	100	psf
Roof DL:	100	psf
Cladding Gravity Load:	0	psf
L:	210	ft
B:	150	ft
Typical Story to Story height:	15	ft
2nd story height:	15	ft
Roof height:	15	ft
Number of typical levels:	5	
W:	18900	kips
$V = C_s W$:	1032.86	kips

(12.8-11)

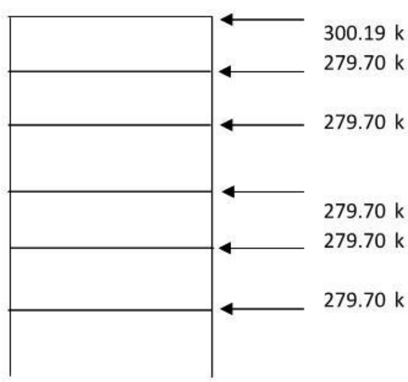
ASCE7-10 Forces

Level	h_x(ft)	W_x(k)	$W_x h_x^k$	C_{vx}	F_x(k)	F_{px}	V_x(k)
Roof	90	3150	342785.2	0.291	300.2	300.194	300.2
6th	75	3150	283465	0.240	248.2	279.696	548.4
5th	60	3150	224646.6	0.190	196.7	279.696	745.2
4th	45	3150	166451.9	0.141	145.8	279.696	890.9
3rd	30	3150	109085.3	0.092	95.5	279.696	986.5
2nd	15	3150	52970.35	0.045	46.4	279.696	1032.9
Σ		18900	1179404	1.0000			

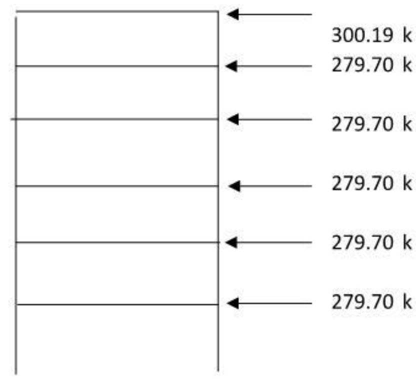
k= 1.042

Lateral Seismic Force:

$$C_{vx} = \frac{W_x h_x^{\ k}}{\sum W_i h_i^{\ k}}$$



Step	Calc	Ref
	Story Shears:	

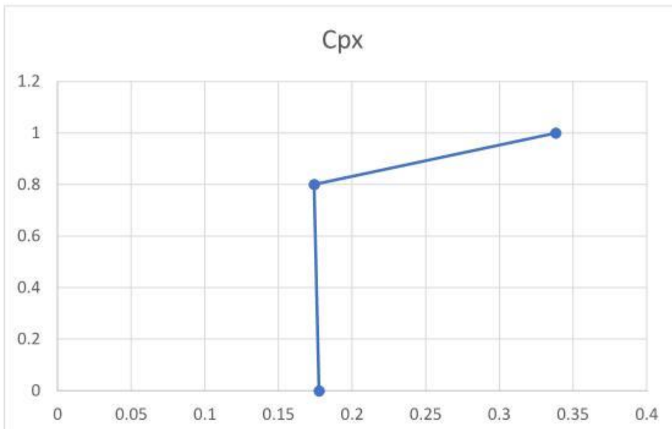


Step	Calc		Ref
	<u>Alternative Design Provisions 12.10.3:</u>		
	Level:	3rd	
	N:	6	
	hx:	30 ft	
	hn:	90 ft	
	z_s:	1.0	12.10.3.2
	Cs:	0.0546	
	Ω_O:	2.5	T12.2.1
	Wpx:	3150 k	
	$F_{px} = \frac{C_{px}}{R_s} W_{px}$		(12.10-4)
	$C_{p0} = 0.4 S_{DS} I =$	0.1776	(12.10-6)
	$\Gamma_{m1} = 1 + \frac{z_s}{2}\left(1 - \frac{1}{N}\right) =$	1.4167	(12.10-13)
	$\Gamma_{m2} = 0.9 z_s \left(1 - \frac{1}{N}\right)^2 =$	0.625	(12.10-14)
	Cpi is max of following:		
	$C_{pi} = 0.8 C_{p0} =$	0.1421	(12.10-8)
	$C_{pi} = 0.9 \Gamma_{m1} C_s \Omega_O =$	0.1742	(12.10-9)
	C_{pi}:	0.1742	
	Cs2 is min of the following:		
	$C_{s2} = (0.15N + 0.25) I_e S_{DS} =$	0.5106	(12.10-10)
	$C_{s2} = I_e S_{DS} =$	0.444	(12.10-11)
	$C_{s2} = \frac{I_e S_{D1}}{0.03(N-1)} =$	1.2775	(12.10-12a/b
	Cs2:	0.444	

Step	Calc	Ref
	$C_{pn} = \sqrt{(\Gamma_{m1}\Omega_o C_s)^2 + (\Gamma_{m2} C_{s2})^2} \geq C_{pi}$ 0.3383	(12.10-7)
	hx/hn: 0.3333	

Table for Graph:

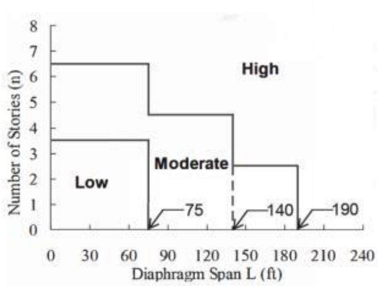
hx/hn	C
0	0.17758
0.8	0.17419
1	0.33831



F12.10-2

Determine Seismic Demand Level

AR: 1.4
n: 6



Seismic Demand Level: Low
Design Option: BDO
Rs: 1

Step	Calc	Ref
	Cpx: 0.1762	T12.10-1
	Wpx: 3150 k	
	$F_{px} = \frac{C_{px}}{R_s} W_{px} =$ 554.94 k	12.10-4
	$\Omega_v = 1.4R_s =$ 1.4	
	Alt provisions or normal? Alt	
	Fx to use: 554.94 k	

ASCE 7-10
Tabulated Values:

Level	Fx Norm
Roof	300.19
6th	279.70
5th	279.70
4th	279.70
3rd	279.70
2nd	279.70

Step	Calc	Ref

Building Parameters:

Design Option: BDO

Transverse is N/S — $L=$ 210 ft

Longitudinal is E/W — $d=$ 150 ft

$L'=$ 30 ft

$L_{LFRS}=$ 30 ft

$b=$ 10 ft

$L_{beam}=$ 30 ft

Diaphragm Internal Forces:

Longitudinal Loading:

Distributed Load:

$w = F_{px}/L =$ 2.64 klf

Boundary Reaction:

$R_{LFRS} = wL/2 =$ 277.5 k

$q_{sw} = 0.05wL/L_{LFRS} =$ 0.925 klf

Transverse Loading:

Distributed Load:

$n = F_{px}/L =$ 2.64 klf

Boundary Reaction:

$q = nL/4L_{LFRS} =$ 4.62 klf

Primary Joint:

		Load Direction					
		Transverse			Longitudinal		
Joint	x(ft)	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
1	10	0	351.5	2643	26.4	0	0
2	20	0	314.5	5021	52.9	0	0
3	30	0	277.5	7135	79.3	0	0
4	40	0	240.5	7597	13.2	0	0
5	50	0	203.5	7796	-52.9	0	0
6	60	0	166.5	7730	-118.9	0	0
7	70	0	129.5	8787	-92.5	0	0
8	80	0	92.5	9579	-66.1	0	0
9	90	0	55.5	10108	-39.6	0	0
10	100	0	18.5	10372	-13.2	0	0

Note: Other side is symmetric and shear overstrength is applied

Internal Beam Joint:

Load Direction					
Transverse			Longitudinal		
N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
0	24.66	0	0	0	0

Step	Calc	Ref
	Other Joints:	
	Diaphragm to Transverse Wall Joint	

	Load Direction					
	Transverse			Longitudinal		
	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
	0	277.5	0	0	0	0

Diaphragm to Longitudinal Wall Joint

	Load Direction					
	Transverse			Longitudinal		
	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
	0	0.0	0	0	138.7	0

Note: small contribution form qsw can be ignored since it will not govern

Reinforcement Selection:

Chord Reinf: Pour strip chord Gr. 60

Shear Reinf 1: Standard ASTM A185 wwr

Shear Reinf 2: Hairpin #4

Collector Reinf: Pour strip chord Gr. 60

Internal Beam Joint: Pour strip chord Gr. 60

Internal Beam Joint Collector: Pour strip chord Gr. 60

Determine Reinforcement Strength:

Topped or Untopped: Topped

$$V_n = \sum v_n$$

$$N_n = \sum t_n$$

For EDO: $M_n = M_y$ 10862.4 k-ft

For BDO: $M_n = \frac{1+\Omega_d}{2} M_y$ 12220 k-ft

For RDO: $M_n = \Omega_d M_y$ 13578 k-ft

M_y: 10862.4 k-ft

d': 146 ft

Ω_d: 1.25

My for design option: 12220.2 k-ft

Step	Calc				Ref
	Chord Reinf.				
			Size:	#5	
	k_t/A (k/in/in^2)	1234	#bars:	4	
	t_n/A (ksi)	60	As	1.24 in^2	
	k_v/A (k/in/in2)	382	d_b	0.625 in^2	
	t_v/A (ksi)	24.2	fy	60 ksi	
	Connector and Collector Reinf:				
	k_t/A (k/in/in^2)	1234	Size:	#7	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.6 in^2	
	V_n/A (ksi)	24.2	d_b	0.875 in^2	
	Shear Reinf 1				
	k_t/A (k/in/in^2)	1414	WWR:	W2.9xW2.9	ACI 318-19
	t_n/A (ksi)	65		10x 10	App B
	k_v/A (k/in/in2)	709			
	V_n/A (ksi)	39.7	Area:	0.029 in^2	
	Shear Reinf 2				
	k_t (k/in)	209			
	t_n (k)	9			
	k_v(k/in)	181			
	v_n(k)	18.1			
	Internal Beam Joint:				
	k_t/A (k/in/in^2)	1234	Size:	#7	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.6 in^2	
	V_n/A (ksi)	24.2	d_b	0.875 in^2	
	Internal Beam Collector:				
	k_t/A (k/in/in^2)	1234	Size:	#4	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.2 in^2	
	V_n/A (ksi)	24.2	d_b	0.5 in^2	

Step	Calc	Ref

Joint	Chord		Hairpin		W2.9xW2.9		M-N-V	
#	Size	#	#	s(ft)	Area(in^2)	10x"	N/S	E/W
1	#5	4	16	9.13	0.029	10	0.903	0.201
2	#5	4	16	9.13	0.029	10	0.903	0.403
3	#5	4	16	9.13	0.029	10	0.945	0.604
4	#5	4	16	9.13	0.029	10	0.912	0.101
5	#5	4	16	9.13	0.029	10	0.87	0.403
6	#5	4	16	9.13	0.029	10	0.815	0.906
7	#5	4	16	9.13	0.029	10	0.861	0.704
8	#5	4	16	9.13	0.029	10	0.901	0.503
9	#5	4	16	9.13	0.029	10	0.929	0.302
10	#5	4	16	9.13	0.029	10	0.944	0.101

Ω_v: 1.4

ϕ_b: 0.9

ϕ_c: 0.9

ϕv: 0.75

Interaction Equation:

$$\sqrt{\left(\frac{|M_u|}{\phi_b M_n}+\frac{N_u}{\phi_c N_n}\right)^2+\left(\frac{\Omega_v V_u}{\phi_v V_n}\right)^2}\leq 1.0$$

Diaphragm to LFRS:

Anchorage	L(ft)	V_u (k)	N_u (k)	M_u (k-ft)	v_n (k)	t_n (k)
N/S Frame	50	277.5	0	0	14.52	36
E/W Frame	30	138.7	0	0	14.52	36

N/S Frame Required: 26.0

E/W Frame Required: 13.0

Provide 26 #7 bars for N/S Frame

Provide 13 #7 bars for E/W Frame

Collector Design(same demand as anchorage):

Note: per ASCE 7-16 Collector forces are multiplied by 1.5

$A_s=1.5V_u/\phi fy$: 3.854 in^2

Provide 7 #7bars at face of each exterior wall

Internal Beam Joint:

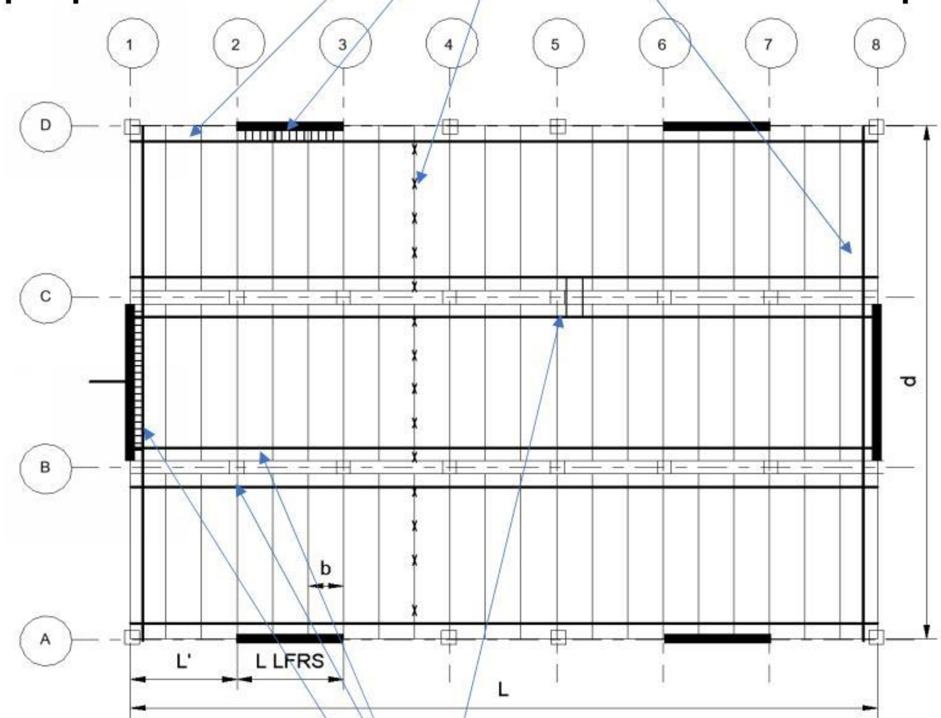
Anchorage	N(k)	V(k)	M(k-ft)	v_n (k)	t_n (k)
Int. Joint	0	24.66	0	14.52	36
Collector:	0	24.66	0	4.84	12

Required: 3

Provide 3 #7 bars per panel for internal beam JT

Provide 2 #4bars at each face of spandrel

Step	Calc	Ref
	Provide 4 #5 for Chord Provide 13 #7 bars for E/W Frame Provide (16) Hairpin #4 spaced at 9.2 ft Provide 7 #7bars at face of each exterior wall	



Provide 3 #7 bars per panel for internal beam JT
Provide 2 #4bars at each face of spandrel
Provide 26 #7 bars for N/S Frame

Step	Calc	Ref
	Check for Drift check:	

Design Option	Cases Requiring Drift Check		
	n≤3	3<n≤5	n>5
EDO	-	-	AR>3.5
BDO	AR>3.8	AR>3.5	AR>3.2
RDO	AR>3.6	AR>3.4	AR>3.1

AR: 1.4

Check: **Not Required**

Step	Calc			Ref
	Building Parameters:			
	Chattano0ga TN.	L:	210 ft	
		B:	150 ft	
		Level 2 height:	15 ft	
		Typical height:	15 ft	
		Roof height:	15 ft	
	Concrete Properties:			
		f'c:	3500 psi	
		λ:	1	
		fy:	60000 psi	
		Conc. Density:	145 pcf	
	Seismic Provisions:			ASCE7-16
		S_s:	0.467	
		S_1:	0.122	
		F_a:	1.426	T11.4-1
		F_v:	2.356	
		Risk Category:	II	
		I_e:	1	
		$S_{MS} = F_a\ S_s$:	0.6659	
		$S_{M1} = F_v\ S_1$:	0.2874	
		$S_{DS} = \frac{2}{3} S_{MS}$:	0.444	(11.4-3)
		$S_{D1} = \frac{2}{3} S_{M1}$:	0.1916	(11.4-4)
	Seismic Design Category:			
		SDC:	C	T11.6-1
		SDC:	C	T11.6-2
		SDC C governs		
	N/S and E/W Special RC Shear Wall	R=	6	T12.2-1
		$V = C_s W$		(12.8-1)
		$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$:	0.074	(12.8-2)
		$C_{smin} = 0.044 S_{DS} I_e > 0.01$:	0.0195	(12.8-5)
		Ct:	0.02	T12.8-2
		x:	0.75	
		h:	90 ft	
		$\boldsymbol{T_a = C_t {h_n}^x}$:	0.5844 sec	(12.8-7)
		Assuming $T<T_L$		

Step	Calc	Ref

$$C_{smax} = \frac{S_{D1}}{T(\frac{R}{I_e})}: \; 0.0546$$

Governing Cs: 0.0546
Floor DL: 100 psf
Roof DL: 100 psf
Cladding Gravity Load: 0 psf
L: 210 ft
B: 150 ft
Typical Story to Story height: 15 ft
2nd story height: 15 ft
Roof height: 15 ft
Number of typical levels: 5
W: 18900 kips
$V = C_s W$: 1032.86 kips (12.8-11)

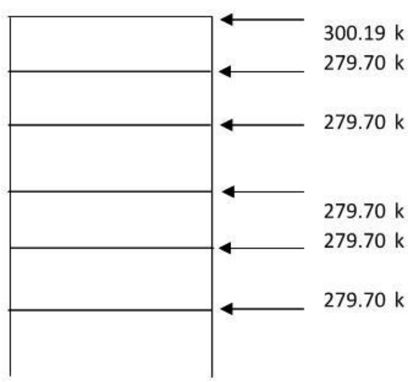
ASCE7-10 Forces

Level	h_x(ft)	W_x(k)	$W_x h_x^k$	C_{vx}	F_x(k)	F_{px}	V_x(k)
Roof	90	3150	342785.2	0.291	300.2	300.194	300.2
6th	75	3150	283465	0.240	248.2	279.696	548.4
5th	60	3150	224646.6	0.190	196.7	279.696	745.2
4th	45	3150	166451.9	0.141	145.8	279.696	890.9
3rd	30	3150	109085.3	0.092	95.5	279.696	986.5
2nd	15	3150	52970.35	0.045	46.4	279.696	1032.9
Σ		18900	1179404	1.0000			

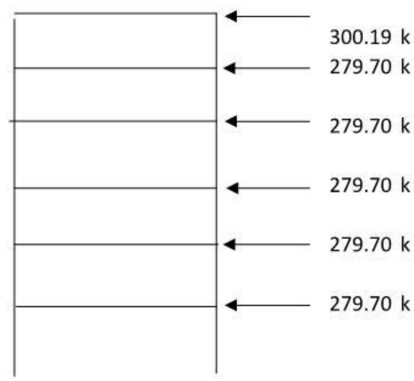
k= 1.042

Lateral Seismic Force:

$$C_{vx} = \frac{W_x {h_x}^k}{\sum W_i {h_i}^k}$$



Step	Calc	Ref
	Story Shears:	

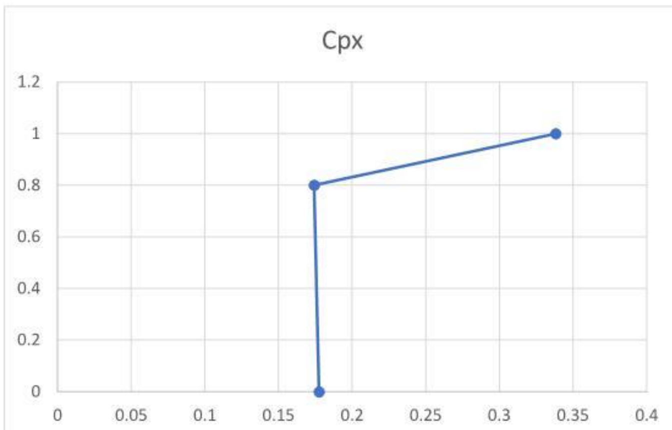


Step	Calc	Ref
	<u>Alternative Design Provisions 12.10.3:</u>	
	Level: 4th	
	N: 6	
	hx: 45 ft	
	hn: 90 ft	
	z_s: 1.0	12.10.3.2
	Cs: 0.0546	
	Ω_O: 2.5	T12.2.1
	Wpx: 3150 k	
	$F_{px} = \frac{C_{px}}{R_s} W_{px}$	(12.10-4)
	$C_{p0} = 0.4 S_{DS} I =$ 0.1776	(12.10-6)
	$\Gamma_{m1} = 1 + \frac{z_s}{2}\left(1 - \frac{1}{N}\right) =$ 1.4167	(12.10-13)
	$\Gamma_{m2} = 0.9 z_s \left(1 - \frac{1}{N}\right)^2 =$ 0.625	(12.10-14)
	Cpi is max of following:	
	$C_{pi} = 0.8 C_{p0} =$ 0.1421	(12.10-8)
	$C_{pi} = 0.9 \Gamma_{m1} C_s \Omega_O =$ 0.1742	(12.10-9)
	C_{pi}: 0.1742	
	Cs2 is min of the following:	
	$C_{s2} = (0.15N + 0.25) I_e S_{DS} =$ 0.5106	(12.10-10)
	$C_{s2} = I_e S_{DS} =$ 0.444	(12.10-11)
	$C_{s2} = \frac{I_e S_{D1}}{0.03(N-1)} =$ 1.2775	(12.10-12a/b
	Cs2: 0.444	

Step	Calc	Ref
	$C_{pn} = \sqrt{(\Gamma_{m1}\Omega_o C_s)^2 + (\Gamma_{m2} C_{s2})^2} \geq C_{pi}$ 0.3383	(12.10-7)
	hx/hn: 0.5	

Table for Graph:

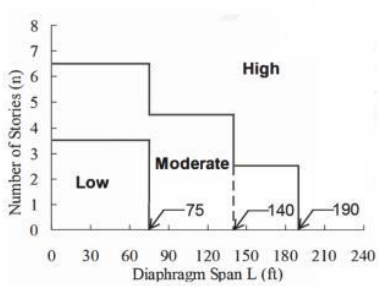
hx/hn	C
0	0.17758
0.8	0.17419
1	0.33831



F12.10-2

Determine Seismic Demand Level

AR: 1.4
n: 6



Seismic Demand Level: Low
Design Option: BDO
Rs: 1

Step	Calc	Ref
	Cpx: 0.1755	T12.10-1
	Wpx: 3150 k	
	$F_{px} = \frac{C_{px}}{R_s} W_{px} =$ 552.71 k	12.10-4
	$\Omega_v = 1.4R_s =$ 1.4	
	Alt provisions or normal? Alt	
	Fx to use: 552.71 k	

ASCE 7-10

Tabulated Values:

Level	Fx Norm
Roof	300.19
6th	279.70
5th	279.70
4th	279.70
3rd	279.70
2nd	279.70

Step	Calc	Ref

Building Parameters:

	Design Option:	BDO
Transverse is N/S	L=	210 ft
Longitudinal is E/W	d=	150 ft
	L'=	30 ft
	L_{LFRS}=	30 ft
	b=	10 ft
	L_{beam}=	30 ft

Diaphragm Internal Forces:

Longitudinal Loading:

Distributed Load:

$w = F_{px}/L$ = 2.63 klf

Boundary Reaction:

$R_{LFRS} = wL/2$ = 276.4 k

$q_{sw} = 0.05wL/L_{LFRS}$ = 0.921 klf

Transverse Loading:

Distributed Load:

$n = F_{px}/L$ = 2.63 klf

Boundary Reaction:

$q = nL/4L_{LFRS}$ = 4.61 klf

Primary Joint:

		Load Direction					
		Transverse			Longitudinal		
Joint	x(ft)	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
1	10	0	350.1	2632	26.3	0	0
2	20	0	313.2	5001	52.6	0	0
3	30	0	276.4	7106	79.0	0	0
4	40	0	239.5	7567	13.2	0	0
5	50	0	202.7	7764	-52.6	0	0
6	60	0	165.8	7699	-118.4	0	0
7	70	0	129.0	8751	-92.1	0	0
8	80	0	92.1	9541	-65.8	0	0
9	90	0	55.3	10067	-39.5	0	0
10	100	0	18.4	10330	-13.2	0	0

Note: Other side is symmetric and shear overstrength is applied

Internal Beam Joint:

Load Direction					
Transverse			Longitudinal		
N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
0	24.57	0	0	0	0

Step	Calc	Ref
	Other Joints:	
	Diaphragm to Transverse Wall Joint	

	Transverse			Longitudinal		
Load Direction	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
	0	276.4	0	0	0	0

Diaphragm to Longitudinal Wall Joint

	Transverse			Longitudinal		
Load Direction	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
	0	0.0	0	0	138.2	0

Note: small contribution form qsw can be ignored since it will not govern

Reinforcement Selection:

Chord Reinf: Pour strip chord Gr. 60
Shear Reinf 1: Standard ASTM A185 wwr
Shear Reinf 2: Hairpin #4
Collector Reinf: Pour strip chord Gr. 60
Internal Beam Joint: Pour strip chord Gr. 60
Internal Beam Joint Collector: Pour strip chord Gr. 60

Determine Reinforcement Strength:

Topped or Untopped: Topped

$$V_n = \sum v_n$$

$$N_n = \sum t_n$$

For EDO: $M_n = M_y$ 10862.4 k-ft

For BDO: $M_n = \frac{1+\Omega_d}{2} M_y$ 12220 k-ft

For RDO: $M_n = \Omega_d M_y$ 13578 k-ft

M_y: 10862.4 k-ft
d': 146 ft
Ω_d: 1.25
My for design option: 12220.2 k-ft

Step	Calc				Ref
	Chord Reinf.				
			Size:	#5	
	k_t/A (k/in/in^2)	1234	#bars:	4	
	t_n/A (ksi)	60	As	1.24 in^2	
	k_v/A (k/in/in2)	382	d_b	0.625 in^2	
	t_v/A (ksi)	24.2	fy	60 ksi	
	Connector and Collector Reinf:				
	k_t/A (k/in/in^2)	1234	Size:	#7	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.6 in^2	
	V_n/A (ksi)	24.2	d_b	0.875 in^2	
	Shear Reinf 1				
	k_t/A (k/in/in^2)	1414	WWR:	W2.9xW2.9	ACI 318-19
	t_n/A (ksi)	65	10x 10		App B
	k_v/A (k/in/in2)	709			
	V_n/A (ksi)	39.7	Area:	0.029 in^2	
	Shear Reinf 2				
	k_t (k/in)	209			
	t_n (k)	9			
	k_v(k/in)	181			
	v_n(k)	18.1			
	Internal Beam Joint:				
	k_t/A (k/in/in^2)	1234	Size:	#7	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.6 in^2	
	V_n/A (ksi)	24.2	d_b	0.875 in^2	
	Internal Beam Collector:				
	k_t/A (k/in/in^2)	1234	Size:	#4	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.2 in^2	
	V_n/A (ksi)	24.2	d_b	0.5 in^2	

Step	Calc								Ref
	Joint	Chord		Hairpin		W2.9xW2.9		M-N-V	
	#	Size	#	#	s(ft)	Area(in^2)	10x"	N/S	E/W
	1	#5	4	16	9.13	0.029	10	0.9	0.200
	2	#5	4	16	9.13	0.029	10	0.899	0.401
	3	#5	4	16	9.13	0.029	10	0.941	0.601
	4	#5	4	16	9.13	0.029	10	0.908	0.100
	5	#5	4	16	9.13	0.029	10	0.866	0.401
	6	#5	4	16	9.13	0.029	10	0.812	0.902
	7	#5	4	16	9.13	0.029	10	0.857	0.702
	8	#5	4	16	9.13	0.029	10	0.897	0.501
	9	#5	4	16	9.13	0.029	10	0.926	0.301
	10	#5	4	16	9.13	0.029	10	0.94	0.100

Ω_v: 1.4

ϕ_b: 0.9

ϕ_c: 0.9

ϕv: 0.75

Interaction Equation:

$$\sqrt{(\frac{|M_u|}{\phi_b M_n}+\frac{N_u}{\phi_c N_n})^2+(\frac{\Omega_v V_u}{\phi_v V_n})^2} \leq 1.0$$

Diaphragm to LFRS:

Anchorage	L(ft)	V_u (k)	N_u (k)	M_u (k-ft)	v_n (k)	t_n (k)
N/S Frame	50	276.4	0	0	14.52	36
E/W Frame	30	138.2	0	0	14.52	36

N/S Frame Required: 26.0

E/W Frame Required: 13.0

Provide 26 #7 bars for N/S Frame

Provide 13 #7 bars for E/W Frame

Collector Design(same demand as anchorage):

Note: per ASCE 7-16 Collector forces are multiplied by 1.5

A_s=1.5V_u/ϕfy: 3.838 in^2

Provide 7 #7bars at face of each exterior wall

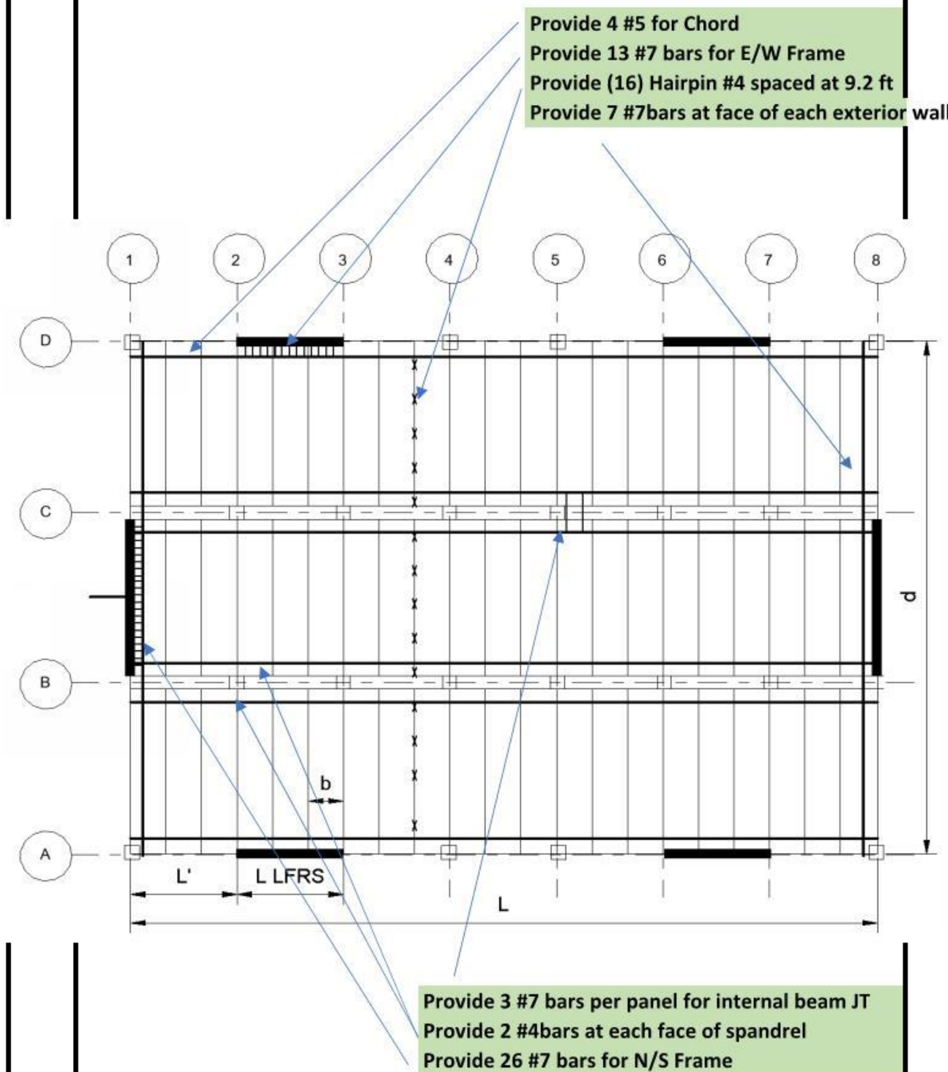
Internal Beam Joint:

Anchorage	N(k)	V(k)	M(k-ft)	v_n (k)	t_n (k)
Int. Joint	0	24.57	0	14.52	36
Collector:	0	24.57	0	4.84	12
				Required:	3

Provide 3 #7 bars per panel for internal beam JT

Provide 2 #4bars at each face of spandrel

Step	Calc	Ref



Step	Calc	Ref
	Check for Drift check:	

Design Option	Cases Requiring Drift Check		
	n≤3	3<n≤5	n>5
EDO	-	-	AR>3.5
BDO	AR>3.8	AR>3.5	AR>3.2
RDO	AR>3.6	AR>3.4	AR>3.1

AR: 1.4

Check: **Not Required**

Step	Calc			Ref
	Building Parameters:			
	Chattano0ga TN.	L:	210 ft	
		B:	150 ft	
		Level 2 height:	15 ft	
		Typical height:	15 ft	
		Roof height:	15 ft	
	Concrete Properties:			
		f'c:	3500 psi	
		λ:	1	
		fy:	60000 psi	
		Conc. Density:	145 pcf	
	Seismic Provisions:			ASCE7-16
		S_s:	0.467	
		S_1:	0.122	
		F_a:	1.426	T11.4-1
		F_v:	2.356	
		Risk Category:	II	
		I_e:	1	
		$S_{MS} = F_a\ S_s$:	0.6659	
		$S_{M1} = F_v\ S_1$:	0.2874	
		$S_{DS} = \frac{2}{3} S_{MS}$:	0.444	(11.4-3)
		$S_{D1} = \frac{2}{3} S_{M1}$:	0.1916	(11.4-4)
	Seismic Design Category:			
		SDC:	C	T11.6-1
		SDC:	C	T11.6-2
		SDC C governs		
	N/S and E/W Special RC Shear Wall	R=	6	T12.2-1
		$V = C_s W$		(12.8-1)
		$C_s = \frac{S_{DS}}{(\frac{R}{I_e})}$:	0.074	(12.8-2)
		$C_{smin} = 0.044 S_{DS} I_e > 0.01$:	0.0195	(12.8-5)
		Ct:	0.02	T12.8-2
		x:	0.75	
		h:	90 ft	
		$T_a = C_t {h_n}^x$:	0.5844 sec	(12.8-7)
		Assuming $T<T_L$		

Step	Calc	Ref

$C_{smax} = \frac{S_{D1}}{T(\frac{R}{I_e})}$: 0.0546

Governing Cs: 0.0546

Floor DL: 100 psf

Roof DL: 100 psf

Cladding Gravity Load: 0 psf

L: 210 ft

B: 150 ft

Typical Story to Story height: 15 ft

2nd story height: 15 ft

Roof height: 15 ft

Number of typical levels: 5

W: 18900 kips

$V = C_s W$: 1032.86 kips (12.8-11)

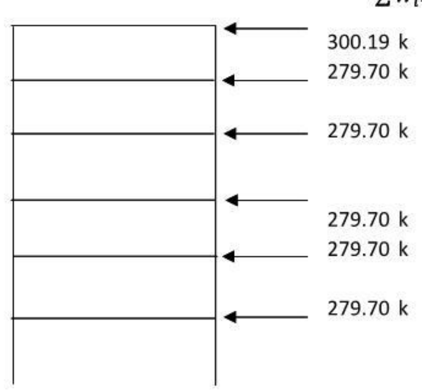
ASCE7-10 Forces

Level	h_x(ft)	W_x(k)	$W_x h_x^k$	C_{vx}	F_x(k)	F_{px}	V_x(k)
Roof	90	3150	342785.2	0.291	300.2	300.194	300.2
6th	75	3150	283465	0.240	248.2	279.696	548.4
5th	60	3150	224646.6	0.190	196.7	279.696	745.2
4th	45	3150	166451.9	0.141	145.8	279.696	890.9
3rd	30	3150	109085.3	0.092	95.5	279.696	986.5
2nd	15	3150	52970.35	0.045	46.4	279.696	1032.9
Σ		18900	1179404	1.0000			

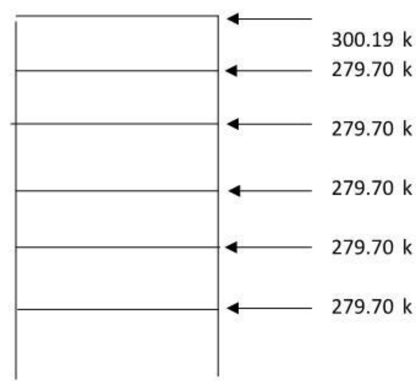
k= 1.042

Lateral Seismic Force:

$$C_{vx} = \frac{W_x {h_x}^k}{\sum W_i {h_i}^k}$$



Step	Calc	Ref
	Story Shears:	



Alternative Design Provisions 12.10.3:

Level:	5th	
N:	6	
hx:	60 ft	
hn:	90 ft	
z_s:	1.0	12.10.3.2
Cs:	0.0546	
Ω_O:	2.5	T12.2.1
Wpx:	3150 k	
$F_{px} = \frac{C_{px}}{R_s} W_{px}$		(12.10-4)
$C_{p0} = 0.4 S_{DS} I =$	0.1776	(12.10-6)
$\Gamma_{m1} = 1 + \frac{z_s}{2}\left(1 - \frac{1}{N}\right) =$	1.4167	(12.10-13)
$\Gamma_{m2} = 0.9 z_s \left(1 - \frac{1}{N}\right)^2 =$	0.625	(12.10-14)

Cpi is max of following:

$C_{pi} = 0.8 C_{p0} =$	0.1421	(12.10-8)
$C_{pi} = 0.9 \Gamma_{m1} C_s \Omega_O =$	0.1742	(12.10-9)
C_{pi}:	0.1742	

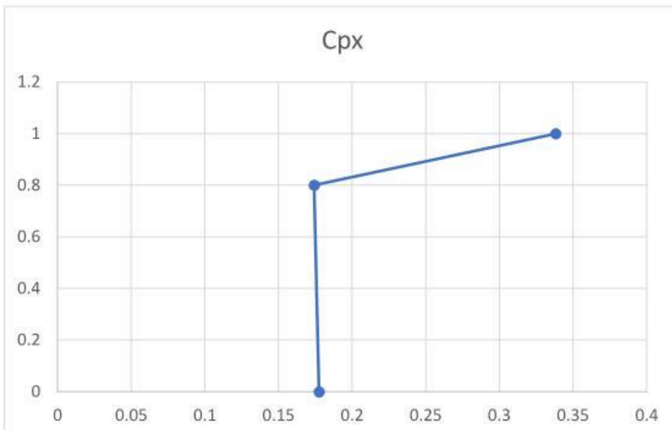
Cs2 is min of the following:

$C_{s2} = (0.15N + 0.25) I_e S_{DS} =$	0.5106	(12.10-10)
$C_{s2} = I_e S_{DS} =$	0.444	(12.10-11)
$C_{s2} = \frac{I_e S_{D1}}{0.03(N-1)} =$	1.2775	(12.10-12a/b
Cs2:	0.444	

Step	Calc	Ref
	$C_{pn} = \sqrt{(\Gamma_{m1}\Omega_o C_s)^2 + (\Gamma_{m2} C_{s2})^2} \geq C_{pi}$ 0.3383	(12.10-7)
	hx/hn: 0.6667	

Table for Graph:

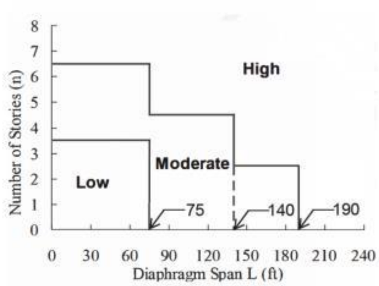
hx/hn	C
0	0.17758
0.8	0.17419
1	0.33831



F12.10-2

Determine Seismic Demand Level

AR: 1.4
n: 6



Seismic Demand Level: Low
Design Option: BDO
Rs: 1

Step	Calc		Ref
	Cpx:	0.1748	T12.10-1
	Wpx:	3150 k	
	$F_{px} = \frac{C_{px}}{R_s} W_{px} =$	550.49 k	12.10-4
	$\Omega_v = 1.4 R_s =$	1.4	
	Alt provisions or normal?	Alt	
	Fx to use:	550.49 k	

ASCE 7-10
Tabulated Values:

Level	Fx Norm
Roof	300.19
6th	279.70
5th	279.70
4th	279.70
3rd	279.70
2nd	279.70

Step	Calc	Ref

Building Parameters:

Design Option: BDO

Transverse is N/S — L= 210 ft

Longitudinal is E/W — d= 150 ft

L'= 30 ft

L_{LFRS}= 30 ft

b= 10 ft

L_{beam}= 30 ft

Diaphragm Internal Forces:

Longitudinal Loading:

Distributed Load:

$w = F_{px}/L =$ 2.62 klf

Boundary Reaction:

$R_{LFRS} = wL/2 =$ 275.2 k

$q_{sw} = 0.05wL/L_{LFRS} =$ 0.917 klf

Transverse Loading:

Distributed Load:

$n = F_{px}/L =$ 2.62 klf

Boundary Reaction:

$q = nL/4L_{LFRS} =$ 4.59 klf

Primary Joint:

		Load Direction					
		Transverse			Longitudinal		
Joint	x(ft)	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
1	10	0	348.6	2621	26.2	0	0
2	20	0	311.9	4981	52.4	0	0
3	30	0	275.2	7078	78.6	0	0
4	40	0	238.5	7536	13.1	0	0
5	50	0	201.8	7733	-52.4	0	0
6	60	0	165.1	7668	-118.0	0	0
7	70	0	128.4	8716	-91.7	0	0
8	80	0	91.7	9502	-65.5	0	0
9	90	0	55.0	10027	-39.3	0	0
10	100	0	18.3	10289	-13.1	0	0

Note: Other side is symmetric and shear overstrength is applied

Internal Beam Joint:

Load Direction					
Transverse			Longitudinal		
N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
0	24.47	0	0	0	0

Step	Calc	Ref
	Other Joints:	

Diaphragm to Transverse Wall Joint

	Load Direction				
Transverse			Longitudinal		
N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
0	275.2	0	0	0	0

Diaphragm to Longitudinal Wall Joint

	Load Direction				
Transverse			Longitudinal		
N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
0	0.0	0	0	137.6	0

Note: small contribution form qsw can be ignored since it will not govern

Reinforcement Selection:

Chord Reinf: Pour strip chord Gr. 60
Shear Reinf 1: Standard ASTM A185 wwr
Shear Reinf 2: Hairpin #4
Collector Reinf: Pour strip chord Gr. 60
Internal Beam Joint: Pour strip chord Gr. 60
Internal Beam Joint Collector: Pour strip chord Gr. 60

Determine Reinforcement Strength:

Topped or Untopped: Topped

$$V_n = \sum v_n$$

$$N_n = \sum t_n$$

For EDO: $M_n = M_y$ 10862.4 k-ft

For BDO: $M_n = \frac{1+\Omega_d}{2} M_y$ 12220 k-ft

For RDO: $M_n = \Omega_d M_y$ 13578 k-ft

M_y: 10862.4 k-ft
d': 146 ft
Ω_d: 1.25
My for design option: 12220.2 k-ft

Step	Calc				Ref
	Chord Reinf.				
			Size:	#5	
	k_t/A (k/in/in^2)	1234	#bars:	4	
	t_n/A (ksi)	60	As	1.24 in^2	
	k_v/A (k/in/in2)	382	d_b	0.625 in^2	
	t_v/A (ksi)	24.2	fy	60 ksi	
	Connector and Collector Reinf:				
	k_t/A (k/in/in^2)	1234	Size:	#7	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.6 in^2	
	V_n/A (ksi)	24.2	d_b	0.875 in^2	
	Shear Reinf 1				
	k_t/A (k/in/in^2)	1414	WWR:	W2.9xW2.9	ACI 318-19
	t_n/A (ksi)	65	10x 10		App B
	k_v/A (k/in/in2)	709			
	V_n/A (ksi)	39.7	Area:	0.029 in^2	
	Shear Reinf 2				
	k_t (k/in)	209			
	t_n (k)	9			
	k_v(k/in)	181			
	v_n(k)	18.1			
	Internal Beam Joint:				
	k_t/A (k/in/in^2)	1234	Size:	#7	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.6 in^2	
	V_n/A (ksi)	24.2	d_b	0.875 in^2	
	Internal Beam Collector:				
	k_t/A (k/in/in^2)	1234	Size:	#5	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.31 in^2	
	V_n/A (ksi)	24.2	d_b	0.625 in^2	

Step	Calc									Ref
	Joint	Chord		Hairpin		W2.9xW2.9		M-N-V		
	#	Size	#	#	s(ft)	Area(in^2)	10x"	N/S	E/W	
	1	#5	4	16	9.13	0.029	10	0.896	0.200	
	2	#5	4	16	9.13	0.029	10	0.896	0.399	
	3	#5	4	16	9.13	0.029	10	0.938	0.599	
	4	#5	4	16	9.13	0.029	10	0.905	0.100	
	5	#5	4	16	9.13	0.029	10	0.863	0.399	
	6	#5	4	16	9.13	0.029	10	0.808	0.898	
	7	#5	4	16	9.13	0.029	10	0.854	0.699	
	8	#5	4	16	9.13	0.029	10	0.893	0.499	
	9	#5	4	16	9.13	0.029	10	0.922	0.299	
	10	#5	4	16	9.13	0.029	10	0.937	0.100	

Ω_v: 1.4

ϕ_b: 0.9

ϕ_c: 0.9

ϕv: 0.75

Interaction Equation:

$$\sqrt{\left(\frac{|M_u|}{\phi_b M_n}+\frac{N_u}{\phi_c N_n}\right)^2+\left(\frac{\Omega_v V_u}{\phi_v V_n}\right)^2} \leq 1.0$$

Diaphragm to LFRS:

Anchorage	L(ft)	V_u (k)	N_u (k)	M_u (k-ft)	v_n (k)	t_n (k)
N/S Frame	50	275.2	0	0	14.52	36
E/W Frame	30	137.6	0	0	14.52	36

N/S Frame Required: 26.0

E/W Frame Required: 13.0

Provide 26 #7 bars for N/S Frame

Provide 13 #7 bars for E/W Frame

Collector Design(same demand as anchorage):

Note: per ASCE 7-16 Collector forces are multiplied by 1.5

A_s=1.5V_u/ϕfy: 3.823 in^2

Provide 7 #7bars at face of each exterior wall

Internal Beam Joint:

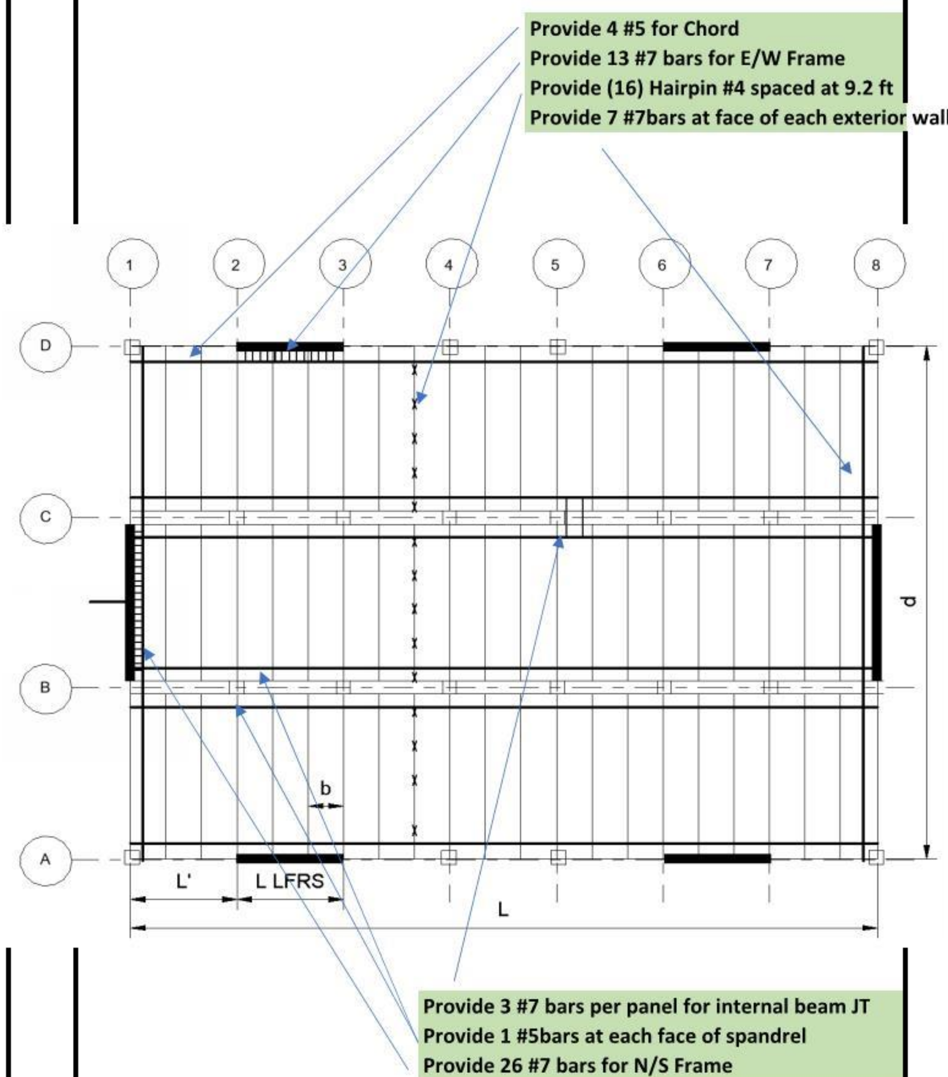
Anchorage	N(k)	V(k)	M(k-ft)	v_n (k)	t_n (k)
Int. Joint	0	24.47	0	14.52	36
Collector:	0	24.47	0	7.502	18.6

Required: 3

Provide 3 #7 bars per panel for internal beam JT

Provide 1 #5bars at each face of spandrel

Step	Calc	Ref



Step	Calc	Ref
	Check for Drift check:	

Design Option	Cases Requiring Drift Check		
	n≤3	3<n≤5	n>5
EDO	-	-	AR>3.5
BDO	AR>3.8	AR>3.5	AR>3.2
RDO	AR>3.6	AR>3.4	AR>3.1

AR: 1.4

Check: **Not Required**

Step	Calc			Ref
	Building Parameters:			
	Chattano0ga TN.	L:	210 ft	
		B:	150 ft	
		Level 2 height:	15 ft	
		Typical height:	15 ft	
		Roof height:	15 ft	
	Concrete Properties:			
		f'c:	3500 psi	
		λ:	1	
		fy:	60000 psi	
		Conc. Density:	145 pcf	
	Seismic Provisions:			ASCE7-16
		S_s:	0.467	
		S_1:	0.122	
		F_a:	1.426	T11.4-1
		F_v:	2.356	
		Risk Category:	II	
		I_e:	1	
		$S_{MS} = F_a\ S_s$:	0.6659	
		$S_{M1} = F_v\ S_1$:	0.2874	
		$S_{DS} = \frac{2}{3} S_{MS}$:	0.444	(11.4-3)
		$S_{D1} = \frac{2}{3} S_{M1}$:	0.1916	(11.4-4)
	Seismic Design Category:			
		SDC:	C	T11.6-1
		SDC:	C	T11.6-2
		SDC C governs		
	N/S and E/W Special RC Shear Wall	R=	6	T12.2-1
		$V = C_s W$		(12.8-1)
		$C_s = \frac{S_{DS}}{(\frac{R}{I_e})}$:	0.074	(12.8-2)
		$C_{smin} = 0.044 S_{DS} I_e > 0.01$:	0.0195	(12.8-5)
		Ct:	0.02	T12.8-2
		x:	0.75	
		h:	90 ft	
		$\boldsymbol{T_a = C_t h_n{}^x}$:	0.5844 sec	(12.8-7)
		Assuming $T<T_L$		

Step	Calc	Ref

$$C_{smax} = \frac{S_{D1}}{T(\frac{R}{I_e})}: \ 0.0546$$

Governing Cs: 0.0546

Floor DL: 100 psf

Roof DL: 100 psf

Cladding Gravity Load: 0 psf

L: 210 ft

B: 150 ft

Typical Story to Story height: 15 ft

2nd story height: 15 ft

Roof height: 15 ft

Number of typical levels: 5

W: 18900 kips

$V = C_s W$: 1032.86 kips (12.8-11)

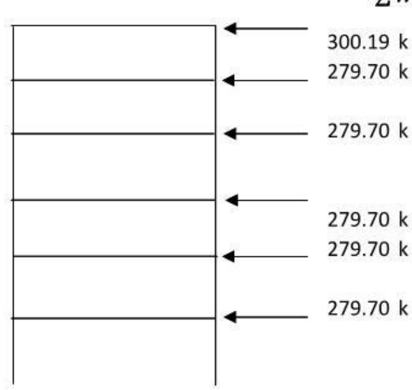
ASCE7-10 Forces

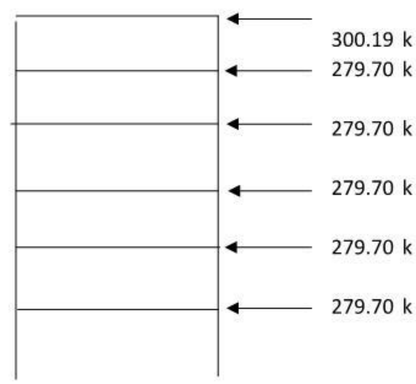
Level	h_x(ft)	W_x(k)	$W_x h_x^k$	C_{vx}	F_x(k)	F_{px}	V_x(k)	
Roof	90	3150	342785.2	0.291	300.2	300.194	300.2	0.0952997
6th	75	3150	283465	0.240	248.2	279.696	548.4	0.0887923
5th	60	3150	224646.6	0.190	196.7	279.696	745.2	0.0887923
4th	45	3150	166451.9	0.141	145.8	279.696	890.9	0.0887923
3rd	30	3150	109085.3	0.092	95.5	279.696	986.5	0.0887923
2nd	15	3150	52970.35	0.045	46.4	279.696	1032.9	0.0887923
Σ		18900	1179404	1.0000				

k= 1.042

Lateral Seismic Force:

$$C_{vx} = \frac{W_x {h_x}^k}{\sum W_i {h_i}^k}$$

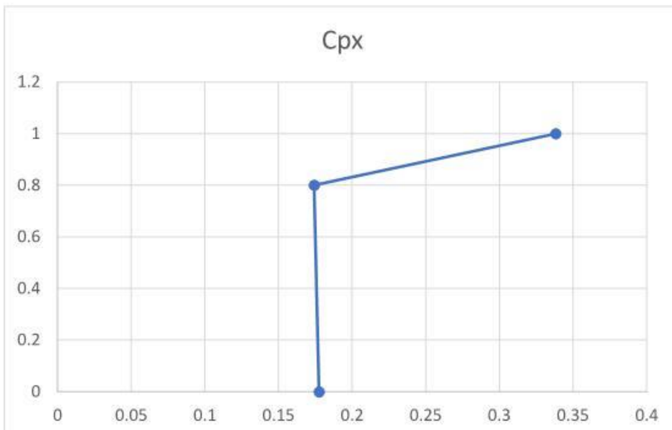


Step	Calc		Ref
	Story Shears:		
			
	<u>Alternative Design Provisions 12.10.3:</u>		
	Level:	6th	
	N:	6	
	hx:	75 ft	
	hn:	90 ft	
	z_s:	1.0	12.10.3.2
	Cs:	0.0546	
	Ω_O:	2.5	T12.2.1
	Wpx:	3150 k	
	$F_{px} = \frac{C_{px}}{R_s} W_{px}$		(12.10-4)
	$C_{p0} = 0.4 S_{DS} I =$	0.1776	(12.10-6)
	$\Gamma_{m1} = 1 + \frac{z_s}{2}\left(1 - \frac{1}{N}\right) =$	1.4167	(12.10-13)
	$\Gamma_{m2} = 0.9 z_s \left(1 - \frac{1}{N}\right)^2 =$	0.625	(12.10-14)
	Cpi is max of following:		
	$C_{pi} = 0.8 C_{p0} =$	0.1421	(12.10-8)
	$C_{pi} = 0.9 \Gamma_{m1} C_s \Omega_O =$	0.1742	(12.10-9)
	C_{pi}:	0.1742	
	Cs2 is min of the following:		
	$C_{s2} = (0.15N + 0.25) I_e S_{DS} =$	0.5106	(12.10-10)
	$C_{s2} = I_e S_{DS} =$	0.444	(12.10-11)
	$C_{s2} = \frac{I_e S_{D1}}{0.03(N-1)} =$	1.2775	(12.10-12a/b
	Cs2:	0.444	

Step	Calc	Ref
	$C_{pn} = \sqrt{(\Gamma_{m1}\Omega_o C_s)^2 + (\Gamma_{m2} C_{s2})^2} \geq C_{pi}$ 0.3383	(12.10-7)
	hx/hn: 0.8333	

Table for Graph:

hx/hn	C
0	0.17758
0.8	0.17419
1	0.33831

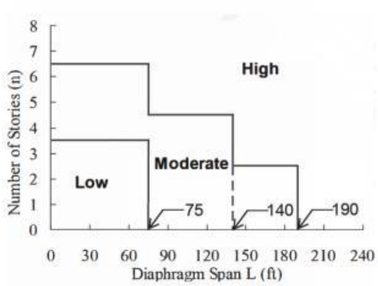


F12.10-2

Determine Seismic Demand Level

AR: 1.4

n: 6



Seismic Demand Level: Low

Design Option: BDO

Rs: 1

Step	Calc		Ref
	Cpx:	0.2015	T12.10-1
	Wpx:	3150 k	
	$F_{px} = \frac{C_{px}}{R_s} W_{px} =$	634.87 k	12.10-4
	$\Omega_v = 1.4R_s =$	1.4	
	Alt provisions or normal?	Alt	
	Fx to use:	634.87 k	

ASCE 7-10
Tabulated Values:

Level	Fx Norm
Roof	300.19
6th	279.70
5th	279.70
4th	279.70
3rd	279.70
2nd	279.70

Step	Calc	Ref

Building Parameters:

Design Option: BDO

Transverse is N/S — $L=$ 210 ft

Longitudinal is E/W — $d=$ 150 ft

$L'=$ 30 ft

$L_{LFRS}=$ 30 ft

$b=$ 10 ft

$L_{beam}=$ 30 ft

Diaphragm Internal Forces:

Longitudinal Loading:

Distributed Load:

$w = F_{px}/L =$ 3.02 klf

Boundary Reaction:

$R_{LFRS} = wL/2 =$ 317.4 k

$q_{sw} = 0.05wL/L_{LFRS} =$ 1.058 klf

Transverse Loading:

Distributed Load:

$n = F_{px}/L =$ 3.02 klf

Boundary Reaction:

$q = nL/4L_{LFRS} =$ 5.29 klf

Primary Joint:

		Load Direction					
		Transverse			Longitudinal		
Joint	x(ft)	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
1	10	0	402.1	3023	30.2	0	0
2	20	0	359.8	5744	60.5	0	0
3	30	0	317.4	8163	90.7	0	0
4	40	0	275.1	8692	15.1	0	0
5	50	0	232.8	8918	-60.5	0	0
6	60	0	190.5	8843	-136.0	0	0
7	70	0	148.1	10052	-105.8	0	0
8	80	0	105.8	10959	-75.6	0	0
9	90	0	63.5	11564	-45.3	0	0
10	100	0	21.2	11866	-15.1	0	0

Note: Other side is symmetric and shear overstrength is applied

Internal Beam Joint:

Load Direction					
Transverse			Longitudinal		
N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
0	28.22	0	0	0	0

Step	Calc	Ref
	Other Joints:	
	Diaphragm to Transverse Wall Joint	

	Transverse			Longitudinal		
Load Direction	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
	0	317.4	0	0	0	0

Diaphragm to Longitudinal Wall Joint

	Transverse			Longitudinal		
Load Direction	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
	0	0.0	0	0	158.7	0

Note: small contribution form qsw can be ignored since it will not govern

Reinforcement Selection:

Chord Reinf: Pour strip chord Gr. 60
Shear Reinf 1: Standard ASTM A185 wwr
Shear Reinf 2: Hairpin #4
Collector Reinf: Pour strip chord Gr. 60
Internal Beam Joint: Pour strip chord Gr. 60
Internal Beam Joint Collector: Pour strip chord Gr. 60

Determine Reinforcement Strength:

Topped or Untopped: Topped

$$V_n = \sum v_n$$

$$N_n = \sum t_n$$

For EDO: $M_n = M_y$ 13578 k-ft

For BDO: $M_n = \frac{1+\Omega_d}{2} M_y$ 15275 k-ft

For RDO: $M_n = \Omega_d M_y$ 16972.5 k-ft

M_y: 13578 k-ft
d': 146 ft
Ω_d: 1.25
My for design option: 15275.3 k-ft

Step	Calc				Ref
	Chord Reinf.				
			Size:	#5	
	k_t/A (k/in/in^2)	1234	#bars:	5	
	t_n/A (ksi)	60	As	1.55 in^2	
	k_v/A (k/in/in2)	382	d_b	0.625 in^2	
	t_v/A (ksi)	24.2	fy	60 ksi	
	Connector and Collector Reinf:				
	k_t/A (k/in/in^2)	1234	Size:	#7	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.6 in^2	
	V_n/A (ksi)	24.2	d_b	0.875 in^2	
	Shear Reinf 1				
	k_t/A (k/in/in^2)	1414	WWR:	W2.9xW2.9	ACI 318-19
	t_n/A (ksi)	65		10x 10	App B
	k_v/A (k/in/in2)	709			
	V_n/A (ksi)	39.7	Area:	0.029 in^2	
	Shear Reinf 2				
	k_t (k/in)	209			
	t_n (k)	9			
	k_v(k/in)	181			
	v_n(k)	18.1			
	Internal Beam Joint:				
	k_t/A (k/in/in^2)	1234	Size:	#7	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.6 in^2	
	V_n/A (ksi)	24.2	d_b	0.875 in^2	
	Internal Beam Collector:				
	k_t/A (k/in/in^2)	1234	Size:	#5	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.31 in^2	
	V_n/A (ksi)	24.2	d_b	0.625 in^2	

Step	Calc	Ref

Joint	Chord		Hairpin		W2.9xW2.9		M-N-V	
#	Size	#	#	s(ft)	Area(in^2)	10x"	N/S	E/W
1	#5	5	18	8.11	0.029	10	0.959	0.205
2	#5	5	18	8.11	0.029	10	0.934	0.410
3	#5	5	18	8.11	0.029	10	0.946	0.615
4	#5	5	18	8.11	0.029	10	0.899	0.102
5	#5	5	18	8.11	0.029	10	0.844	0.410
6	#5	5	18	8.11	0.029	10	0.78	0.922
7	#5	5	18	8.11	0.029	10	0.808	0.717
8	#5	5	18	8.11	0.029	10	0.834	0.512
9	#5	5	18	8.11	0.029	10	0.854	0.307
10	#5	5	18	8.11	0.029	10	0.865	0.102

Ω_v: 1.4

ϕ_b: 0.9

ϕ_c: 0.9

ϕv: 0.75

Interaction Equation:

$$\sqrt{\left(\frac{|M_u|}{\phi_b M_n}+\frac{N_u}{\phi_c N_n}\right)^2+\left(\frac{\Omega_v V_u}{\phi_v V_n}\right)^2} \leq 1.0$$

Diaphragm to LFRS:

Anchorage	L(ft)	V_u (k)	N_u (k)	M_u (k-ft)	v_n (k)	t_n (k)
N/S Frame	50	317.4	0	0	14.52	36
E/W Frame	30	158.7	0	0	14.52	36

N/S Frame Required: 30.0

E/W Frame Required: 15.0

Provide 30 #7 bars for N/S Frame

Provide 15 #7 bars for E/W Frame

Collector Design(same demand as anchorage):

Note: per ASCE 7-16 Collector forces are multiplied by 1.5

$A_s=1.5V_u/\phi fy$: 4.409 in^2

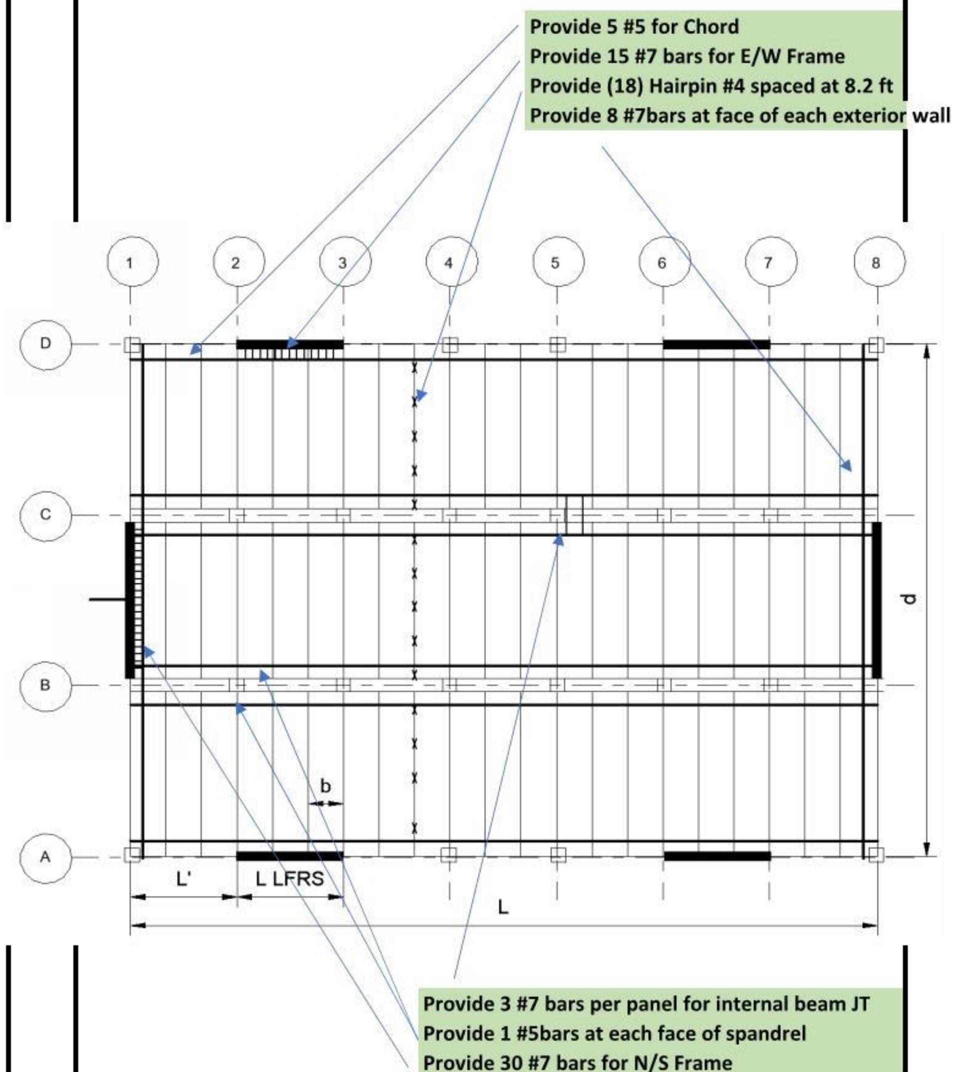
Provide 8 #7bars at face of each exterior wall

Internal Beam Joint:

Anchorage	N(k)	V(k)	M(k-ft)	v_n (k)	t_n (k)
Int. Joint	0	28.22	0	14.52	36
Collector:	0	28.22	0	7.502	18.6
				Required:	3

Provide 3 #7 bars per panel for internal beam JT

Provide 1 #5bars at each face of spandrel

Step	Calc	Ref
		

Step	Calc	Ref
	Check for Drift check:	

Design Option	Cases Requiring Drift Check		
	n≤3	3<n≤5	n>5
EDO	-	-	AR>3.5
BDO	AR>3.8	AR>3.5	AR>3.2
RDO	AR>3.6	AR>3.4	AR>3.1

AR: 1.4

Check: **Not Required**

Step	Calc			Ref
	Building Parameters:			
	Chattano0ga TN.	L:	210 ft	
		B:	150 ft	
		Level 2 height:	15 ft	
		Typical height:	15 ft	
		Roof height:	15 ft	
	Concrete Properties:			
		f'c:	3500 psi	
		λ:	1	
		fy:	60000 psi	
		Conc. Density:	145 pcf	
	Seismic Provisions:			ASCE7-16
		S_s:	0.467	
		S_1:	0.122	
		F_a:	1.426	T11.4-1
		F_v:	2.356	
		Risk Category:	II	
		I_e:	1	
		$S_{MS} = F_a\ S_s$:	0.6659	
		$S_{M1} = F_v\ S_1$:	0.2874	
		$S_{DS} = \frac{2}{3} S_{MS}$:	0.444	(11.4-3)
		$S_{D1} = \frac{2}{3} S_{M1}$:	0.1916	(11.4-4)
	Seismic Design Category:			
		SDC:	C	T11.6-1
		SDC:	C	T11.6-2
		SDC C governs		
	N/S and E/W Special RC Shear Wall	R=	6	T12.2-1
		$V = C_s W$		(12.8-1)
		$C_s = \frac{S_{DS}}{(\frac{R}{I_e})}$:	0.074	(12.8-2)
		$C_{smin} = 0.044 S_{DS} I_e > 0.01$:	0.0195	(12.8-5)
		Ct:	0.02	T12.8-2
		x:	0.75	
		h:	90 ft	
		$T_a = C_t {h_n}^x$:	0.5844 sec	(12.8-7)
		Assuming $T<T_L$		

Step	Calc	Ref

$$C_{smax} = \frac{S_{D1}}{T(\frac{R}{I_e})}: \ 0.0546$$

Governing Cs: 0.0546
Floor DL: 100 psf
Roof DL: 100 psf
Cladding Gravity Load: 0 psf
L: 210 ft
B: 150 ft
Typical Story to Story height: 15 ft
2nd story height: 15 ft
Roof height: 15 ft
Number of typical levels: 5
W: 18900 kips
$V = C_s W$: 1032.86 kips (12.8-11)

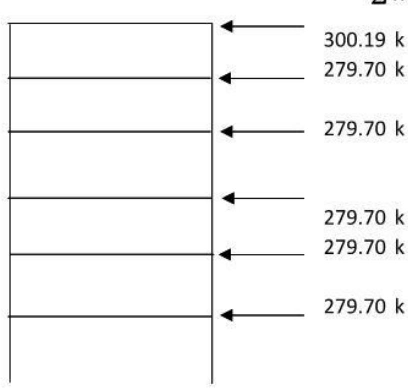
ASCE7-10 Forces

Level	h_x(ft)	W_x(k)	$W_x h_x^k$	C_{vx}	F_x(k)	F_{px}	V_x(k)
Roof	90	3150	342785.2	0.291	300.2	300.194	300.2
6th	75	3150	283465	0.240	248.2	279.696	548.4
5th	60	3150	224646.6	0.190	196.7	279.696	745.2
4th	45	3150	166451.9	0.141	145.8	279.696	890.9
3rd	30	3150	109085.3	0.092	95.5	279.696	986.5
2nd	15	3150	52970.35	0.045	46.4	279.696	1032.9
Σ		18900	1179404	1.0000			

k= 1.042

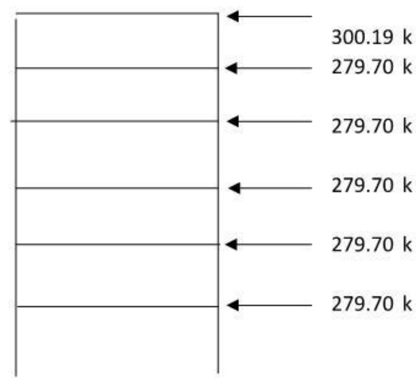
Lateral Seismic Force:

$$C_{vx} = \frac{W_x {h_x}^k}{\sum W_i {h_i}^k}$$



Step	Calc	Ref

Story Shears:



Alternative Design Provisions 12.10.3:

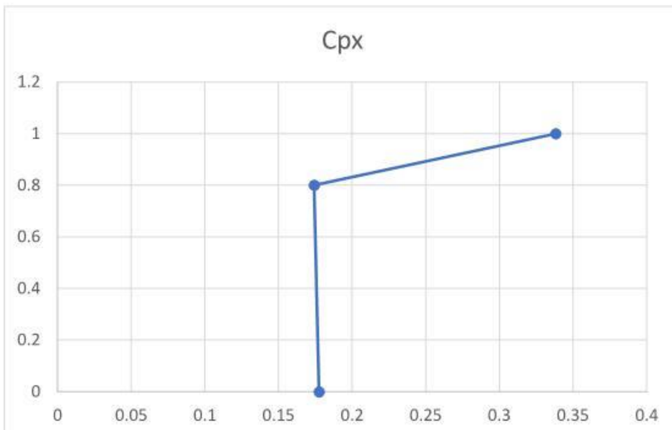
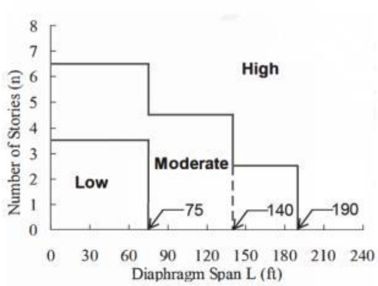
		Ref
Level:	Roof	
N:	6	
hx:	90 ft	
hn:	90 ft	
z_s:	1.0	12.10.3.2
Cs:	0.0546	
Ω_O:	2.5	T12.2.1
Wpx:	3150 k	
$F_{px} = \frac{C_{px}}{R_s} W_{px}$		(12.10-4)
$C_{p0} = 0.4 S_{DS} I =$	0.1776	(12.10-6)
$\Gamma_{m1} = 1 + \frac{z_s}{2}\left(1 - \frac{1}{N}\right) =$	1.4167	(12.10-13)
$\Gamma_{m2} = 0.9 z_s \left(1 - \frac{1}{N}\right)^2 =$	0.625	(12.10-14)

Cpi is max of following:

		Ref
$C_{pi} = 0.8 C_{p0} =$	0.1421	(12.10-8)
$C_{pi} = 0.9 \Gamma_{m1} C_s \Omega_O =$	0.1742	(12.10-9)
C_{pi}:	0.1742	

Cs2 is min of the following:

		Ref
$C_{s2} = (0.15N + 0.25) I_e S_{DS} =$	0.5106	(12.10-10)
$C_{s2} = I_e S_{DS} =$	0.444	(12.10-11)
$C_{s2} = \frac{I_e S_{D1}}{0.03(N-1)} =$	1.2775	(12.10-12a/b
Cs2:	0.444	

Step	Calc	Ref
	$C_{pn} = \sqrt{(\Gamma_{m1}\Omega_o C_s)^2 + (\Gamma_{m2}C_{s2})^2} \geq C_{pi}$ 0.3383	(12.10-7)
	hx/hn: 1	
	Table for Graph:	
	hx/hn C	
	0 0.17758	
	0.8 0.17419	
	1 0.33831	
		F12.10-2
	Determine Seismic Demand Level AR: 1.4	
	n: 6	
		
	Seismic Demand Level: Low	
	Design Option: BDO	
	Rs: 1	

Step	Calc	Ref
	Cpx: 0.3383	T12.10-1
	Wpx: 3150 k	
	$F_{px} = \frac{C_{px}}{R_s} W_{px} =$ 1065.68 k	12.10-4
	$\Omega_v = 1.4R_s =$ 1.4	
	Alt provisions or normal? Alt	
	Fx to use: 1065.7 k	

ASCE 7-10
Tabulated Values:

Level	Fx Norm
Roof	300.19
6th	279.70
5th	279.70
4th	279.70
3rd	279.70
2nd	279.70

Step	Calc	Ref

Building Parameters:

Design Option: RDO

Transverse is N/S — L= 210 ft

Longitudinal is E/W — d= 150 ft

L'= 30 ft

L_{LFRS}= 30 ft

b= 10 ft

L_{beam}= 30 ft

Diaphragm Internal Forces:

Longitudinal Loading:

Distributed Load:

$w = F_{px}/L =$ 5.07 klf

Boundary Reaction:

$R_{LFRS} = wL/2 =$ 532.8 k

$q_{sw} = 0.05wL/L_{LFRS} =$ 1.776 klf

Transverse Loading:

Distributed Load:

$n = F_{px}/L =$ 5.07 klf

Boundary Reaction:

$q = nL/4L_{LFRS} =$ 8.88 klf

Primary Joint:

		Load Direction					
		Transverse			Longitudinal		
Joint	x(ft)	N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
1	10	0	674.9	5075	50.7	0	0
2	20	0	603.9	9642	101.5	0	0
3	30	0	532.8	13702	152.2	0	0
4	40	0	461.8	14590	25.4	0	0
5	50	0	390.7	14970	-101.5	0	0
6	60	0	319.7	14843	-228.4	0	0
7	70	0	248.7	16873	-177.6	0	0
8	80	0	177.6	18396	-126.9	0	0
9	90	0	106.6	19411	-76.1	0	0
10	100	0	35.5	19918	-25.4	0	0

Note: Other side is symmetric and shear overstrength is applied

Internal Beam Joint:

Load Direction					
Transverse			Longitudinal		
N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
0	47.36	0	0	0	0

Step	Calc	Ref
	Other Joints:	

Diaphragm to Transverse Wall Joint

	Load Direction				
Transverse			Longitudinal		
N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
0	532.8	0	0	0	0

Diaphragm to Longitudinal Wall Joint

	Load Direction				
Transverse			Longitudinal		
N(k)	V(k)	M(k-ft)	N(k)	V(k)	M(k-ft)
0	0.0	0	0	266.4	0

Note: small contribution form qsw can be ignored since it will not govern

Reinforcement Selection:

Chord Reinf: Pour strip chord Gr. 60
Shear Reinf 1: Standard ASTM A185 wwr
Shear Reinf 2: Hairpin #4
Collector Reinf: Pour strip chord Gr. 60
Internal Beam Joint: Pour strip chord Gr. 60
Internal Beam Joint Collector: Pour strip chord Gr. 60

Determine Reinforcement Strength:

Topped or Untopped: Topped

$$V_n = \sum v_n$$
$$N_n = \sum t_n$$

For EDO: $M_n = M_y$ 19272 k-ft

For BDO: $M_n = \frac{1+\Omega_d}{2} M_y$ 21681 k-ft

For RDO: $M_n = \Omega_d M_y$ 24090 k-ft

M_y: 19272 k-ft
d': 146 ft
Ω_d: 1.25
My for design option: 24090 k-ft

Step	Calc				Ref
	Chord Reinf.				
			Size:	#6	
	k_t/A (k/in/in^2)	1234	#bars:	5	
	t_n/A (ksi)	60	As	2.2 in^2	
	k_v/A (k/in/in2)	382	d_b	0.75 in^2	
	t_v/A (ksi)	24.2	fy	60 ksi	
	Connector and Collector Reinf:				
	k_t/A (k/in/in^2)	1234	Size:	#8	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.79 in^2	
	V_n/A (ksi)	24.2	d_b	1 in^2	
	Shear Reinf 1				
	k_t/A (k/in/in^2)	1414	WWR:	W2.9xW2.9	ACI 318-19
	t_n/A (ksi)	65	10x 10		App B
	k_v/A (k/in/in2)	709			
	V_n/A (ksi)	39.7	Area:	0.029 in^2	
	Shear Reinf 2				
	k_t (k/in)	209			
	t_n (k)	9			
	k_v(k/in)	181			
	v_n(k)	18.1			
	Internal Beam Joint:				
	k_t/A (k/in/in^2)	1234	Size:	#7	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.6 in^2	
	V_n/A (ksi)	24.2	d_b	0.875 in^2	
	Internal Beam Collector:				
	k_t/A (k/in/in^2)	1234	Size:	#5	
	t_n/A (ksi)	60	#bars:	1	
	k_v/A (k/in/in2)	382	As	0.31 in^2	
	V_n/A (ksi)	24.2	d_b	0.625 in^2	

Step	Calc								Ref
	Joint	Chord		Hairpin		W2.9xW2.9			M-N-V
	#	Size	#	#	s(ft)	Area(in^2)	10x"	N/S	E/W
	1	#6	5	38	3.84	0.029	10	0.989	0.164
	2	#6	5	38	3.84	0.029	10	0.968	0.328
	3	#6	5	38	3.84	0.029	10	0.987	0.492
	4	#6	5	38	3.84	0.029	10	0.941	0.082
	5	#6	5	38	3.84	0.029	10	0.887	0.328
	6	#6	5	38	3.84	0.029	10	0.822	0.738
	7	#6	5	38	3.84	0.029	10	0.855	0.574
	8	#6	5	38	3.84	0.029	10	0.885	0.410
	9	#6	5	38	3.84	0.029	10	0.908	0.246
	10	#6	5	38	3.84	0.029	10	0.92	0.082

Ω_v: 1.4

ϕ_b: 0.9

ϕ_c: 0.9

ϕv: 0.75

Interaction Equation:

$$\sqrt{(\frac{|M_u|}{\phi_b M_n}+\frac{N_u}{\phi_c N_n})^2+(\frac{\Omega_v V_u}{\phi_v V_n})^2} \leq 1.0$$

Diaphragm to LFRS:

Anchorage	L(ft)	V_u (k)	N_u (k)	M_u (k-ft)	v_n (k)	t_n (k)
N/S Frame	50	532.8	0	0	19.118	47.4
E/W Frame	30	266.4	0	0	19.118	47.4

N/S Frame Required: 38.0

E/W Frame Required: 19.0

Provide 38 #8 bars for N/S Frame

Provide 19 #8 bars for E/W Frame

Collector Design(same demand as anchorage):

Note: per ASCE 7-16 Collector forces are multiplied by 1.5

A_s=1.5V_u/ϕfy: 7.401 in^2

Provide 10 #8bars at face of each exterior wall

Internal Beam Joint:

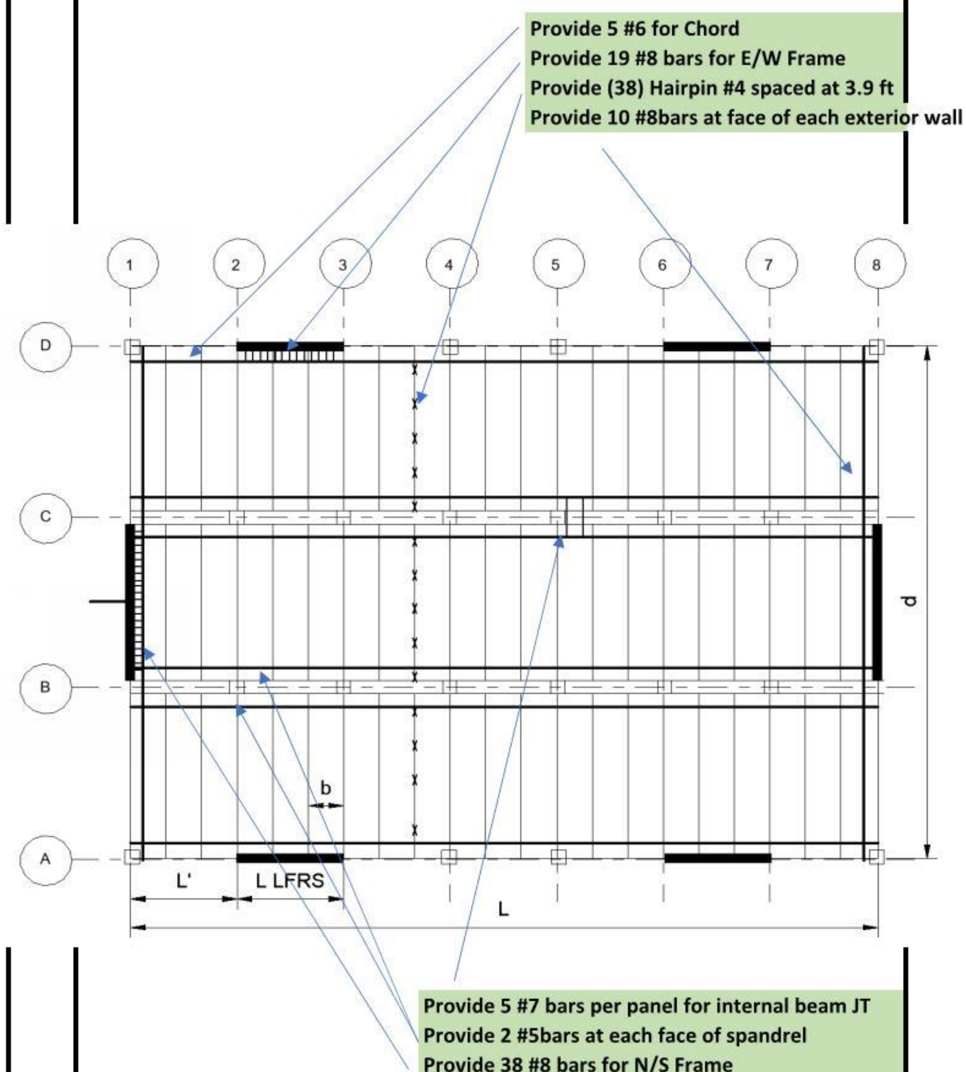
Anchorage	N(k)	V(k)	M(k-ft)	v_n (k)	t_n (k)
Int. Joint	0	47.36	0	14.52	36
Collector:	0	47.36	0	7.502	18.6

Required: 5

Provide 5 #7 bars per panel for internal beam JT

Provide 2 #5bars at each face of spandrel

Step	Calc	Ref



Step	Calc	Ref
	Check for Drift check:	

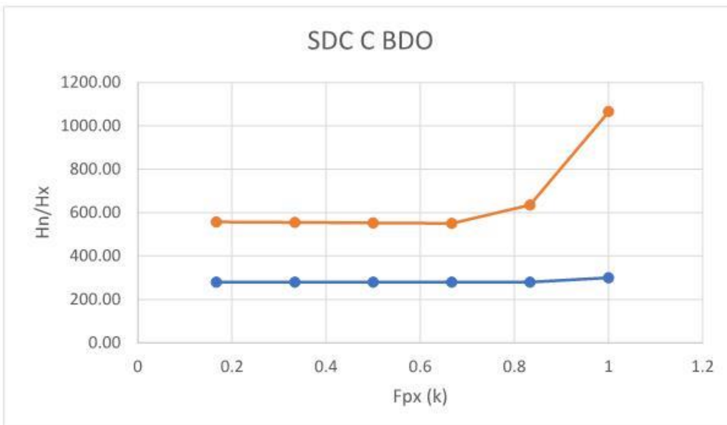
Design Option	Cases Requiring Drift Check		
	n≤3	3<n≤5	n>5
EDO	-	-	AR>3.5
BDO	AR>3.8	AR>3.5	AR>3.2
RDO	AR>3.6	AR>3.4	AR>3.1

AR: 1.4

Check: **Not Required**

Step	Calc	Ref

	hn/hx	Weight (k)	Cpx/Rs	Force (k)	ASCE7-10	Fi/Wi
Level 2	0.1666667	3150	0.177	557.1656453	279.70	0.089
Level 3	0.3333333	3150	0.176	554.9400106	279.70	0.089
Level 4	0.5	3150	0.175	552.7143759	279.70	0.089
Level 5	0.6666667	3150	0.175	550.4887411	279.70	0.089
Level 6	0.8333333	3150	0.202	634.8695824	279.70	0.089
Roof	1	3150	0.338	1065.676328	300.19	0.095



Level	Chord		Hairpin		Topping Reinf				Collector	
	Size	#	#	s(ft)	Area (in^2/ft)	Pattern		Type	Size	#
Level 2	#5	4	16	9.1	0.029	10	10	2.9xW2	#6	9
Level 3	#5	4	16	9.1	0.029	10	10	2.9xW2	#7	7
Level 4	#5	4	16	9.1	0.029	10	10	2.9xW2	#7	7
Level 5	#5	4	16	9.1	0.029	10	10	2.9xW2	#7	7
Level 6	#5	5	18	8.1	0.029	10	10	2.9xW2	#7	8
Roof	#6	5	38	3.8	0.029	10	10	2.9xW2	#8	10

Level	N/S Connection		E/W Connection		Beam Int. Connection		Beam Collectors	
	Size	#	Size	#	Size	#	Size	#
Level 2	#6	35	#6	18	#7	3	#4	2
Level 3	#7	26	#7	13	#7	3	#4	2
Level 4	#7	26	#7	13	#7	3	#4	2
Level 5	#7	26	#7	13	#7	3	#5	1
Level 6	#7	30	#7	15	#7	3	#5	2
Roof	#8	38	#8	19	#7	5	#5	2

Appendix B - Permissions

FB

Fleischman, Robert B - (rfleisch) <rfleisch@arizona.edu>
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From Chapter 14:
Figure 14.2-1 Diaphragm Seismic Demand Level
Figure C14.2-1 Diaphragm Dimensions
Figure C14.2.2 Diaphragm Shear Amplification Factor Results from NTHA at MCE: (a) BDO; (b) RDO

From Chapter 12:
Figure C12.10-3 Comparison of Factors (gamma) Obtained from Analytical Models and Actual Structures with Those Predicted by Eqs. (12.10-13) and (12.10-14)
Figure C12.10-5 Comparison of Measured Floor Accelerations and Accelerations Predicted by Eq (12.10-4) for a Five-Story Special Moment-Resisting Frame Building

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