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Guaranteeing the Code-Accepted Taret Collapse Mechanism of Steel Structure

Mateo Galvez

GUARANTEEING THE CODE-ACCEPTED

TARGET COLLAPSE MECHANISM

OF A STEEL STRUCTURE

by

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I would also like to thank my group members of my senior design project: Anthony Gogola, Norma Flores, Raphael Yepez, and Mhdameer Aldarkazanly.

May 03, 2022

ABSTRACT

GUARANTEEING THE CODE-ACCEPTED

TARGET COLLAPSE MECHANISM

OF A STEEL STRUCTURE

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The University of Texas at Arlington, 2022

Faculty Mentor: Juan Balderrama

Standard structural design practice requires designing for almost every possible load scenario, including high winds, strong earthquakes, and large magnitude live loads. However, it is impractical and expensive to design for every possible scenario The best engineers can do for these extreme load cases is design the failure mechanism to behave a specific and safe way so there is no sudden structural failure. This project designed the controlled and predictable failure mechanism for the 5-story apartment building that was designed by the senior design project group. This is not something that can be achieved using structural modeling software and a design short of this analysis is neither complete nor accurate. It was determined that several critical structural members were under designed because they were only designed via applied load software outputs while neglecting further analysis.

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CHAPTER 1

INTRODUCTION

1.1 Project Description

A series of apartment buildings will be constructed in Stephenville, Texas, and the resulting complex will be called Reined Rope Ranch Apartment Complex. Each apartment building consists of a 5-story apartment building atop an underground parking garage. This project focusses on the design of only one of these buildings because they are all identical.

The target tenants of this apartment complex are mostly the Tarleton student body, but it is also adaptable to operate as long term stay suites for people who may live in the area but are not students. The structure will be used to accommodate the Tarleton State University students as well as the growing population of Stephenville.

1.2 Senior Design Project

The senior design group was tasked with designing the structural components of a single five-story apartment building. This includes the floor diaphragm, beams, columns, vertical bracing, podium diaphragm, shear walls, retaining walls, and foundation.

1.2.1 Floor and Podium Diaphragm Design

The floor diaphragm is responsible for transferring wind loads from the exterior walls to the vertical bracing. The floor and podium diaphragms were designed as simply supported deep beams turned on their side with a distributed load applied to the top. Although they would qualify as deep based on ACI 318-19 [1], they were not analyzed using the strut and tie method. They were simplified and analyzed using standard beam methods to remain in the scope of an undergraduate civil engineer.

The area of flexural reinforcing steel required in the floor podiums was determined based on the table below.

Table 1.1: Required Floor Diaphragm Steel

Floor	F (kip)	Mu (kip-in)	Vu (Kip)	As
1st	30.8	7461.30	15.4	0.13
2nd	32.3	7824.68	16.15	0.14
3rd	33.97	8229.23	16.99	0.14
4th	34.8	8430.30	17.4	0.15
5th	45.37	10990.88	22.69	0.19
6th	34.03	8243.77	17.02	0.14

The required area of flexural reinforcement for the podium was determined to be .02 square inches.

1.2.2 Beam Design

There were many different beams designed for this project, but the one that affects this analysis was designed as a W12X16. This was designed per AISC-360-16 [2] for gravity loads. Non-composite action was assumed for the design process.

1.2.3 Vertical Brace Design

Vertical braces are responsible for decomposing the lateral forces into axial forces in the columns as a moment couple. For the chosen design of chevron bracing, the braces were designed to resist both compression and tension forces, so they need to be designed to prevent buckling. To do this, an HSS8X8X1/2 was chosen and was adequate to resist all applied lateral loads

1.2.4 Column Design

Columns are vertical members designed to transfer both gravity and lateral loads to the foundation. Similar to beam design, there were also many different columns that were designed as part of the senior design project. The columns that affect the scope of this analysis were designed per AISC 360-16 [2] to be W18X86. These are the columns that the lateral bracing will frame into. These columns will undergo more stress than just the lateral loads as they receive moment, compression, and tensile forces due to lateral loads.

1.2.5 Shear/Retaining Wall Design

Shear walls are responsible for transferring the load from the podium diaphragm to the foundation. For this project in particular, the walls designed to resist earth pressures and surcharge are the same walls resisting the lateral forces. In other words, the retaining walls are the acting shear walls. Because this is a very short shear wall based on dimension guidelines per ACI 318-19 [1], it will be very strong in-plane. So, the out-of-plane bending caused by earth pressure govern the reinforcement and thickness of the wall. It only had to be verified that the in-plane strength was adequate for the lateral loads based on the out-of-plane reinforcement and dimensions.

Per ACI 318-19 [1], the reinforcement required was about 2.27 mm^2/m of wall. This reinforcement resulted in an adequate design for in-plane bending as shown in the following interaction diagram of the shear wall for in plane bending.

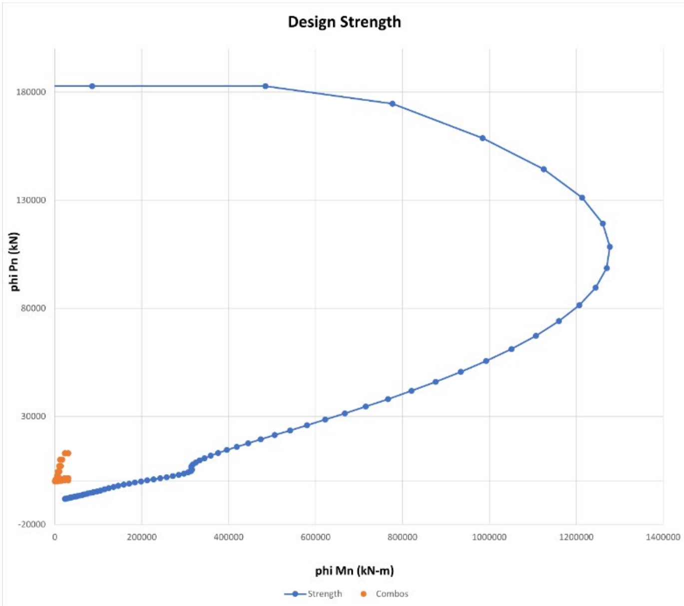

Figure 1.1: Shear Wall Interaction Diagram

The shear resistance of the wall was also checked and verified for adequacy of its resisting strength of in-plane shear.

1.2.6 Foundation Design

There was a combination of different foundation designs for this project, including concrete podiums and strip footings. The foundation redesign was out of the scope of this analysis because the additional forces will change the loads on the foundation, which at minimum would change foundation depth and dimensions but could lead to a complete redesign of all footings.

1.3 Analysis Theory

The design approach to the code-accepted target collapse mechanism is to have plastic deformation of shear links before the plastic deformation of a more critical member, such as column. If a column were to undergo plastic deformation, sudden and catastrophic failure is imminent. Although engineers do not typically design for this type of failure under normal loading conditions, this approach ensures that when extreme loading cases do occur, the structure will fail in a safe and controlled manner.

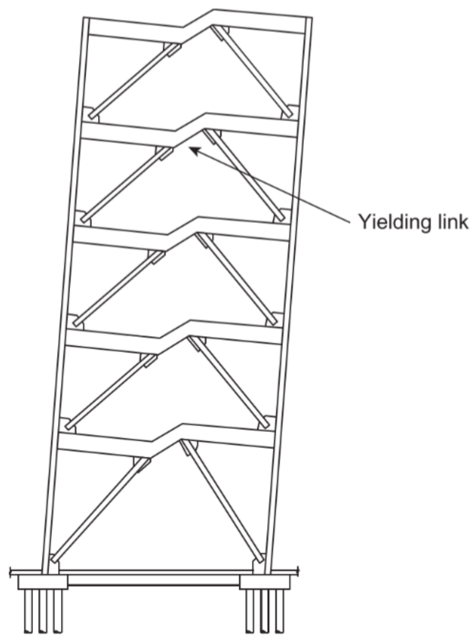

Figure 1.2: Target Collapse Mechanism

To ensure that the link is the only inelastic mechanism in an eccentrically braced frame, all components must be stronger than the member designed to fail. In this case, the beam that the vertical braces are framed into will be the failing member. Following the load path of lateral forces, all members must be stronger than the beam.

The strength of the beam is not defined in the typical manner that engineers use in design. Instead, it is defined by its Adjusted Link Shear Strength, or ALSS. The derivation for the ALSS is shown in Chapter 2. The ALSS will act as a compressive and tensile force on the vertical bracing and will replace the applied lateral loads. This replacement of applied loads means the structural model can no longer be used to determine the internal forces in the members. Analysis must be done manually with the use of hand calculations and calculation software such as Microsoft Excel.

CHAPTER 2

LITERATURE REVIEW

2.1 Behavior of Steel

As seen in Figure 2.1 provided from Steel Design by Advanced Analysis [3], steel behaves differently under different stresses.

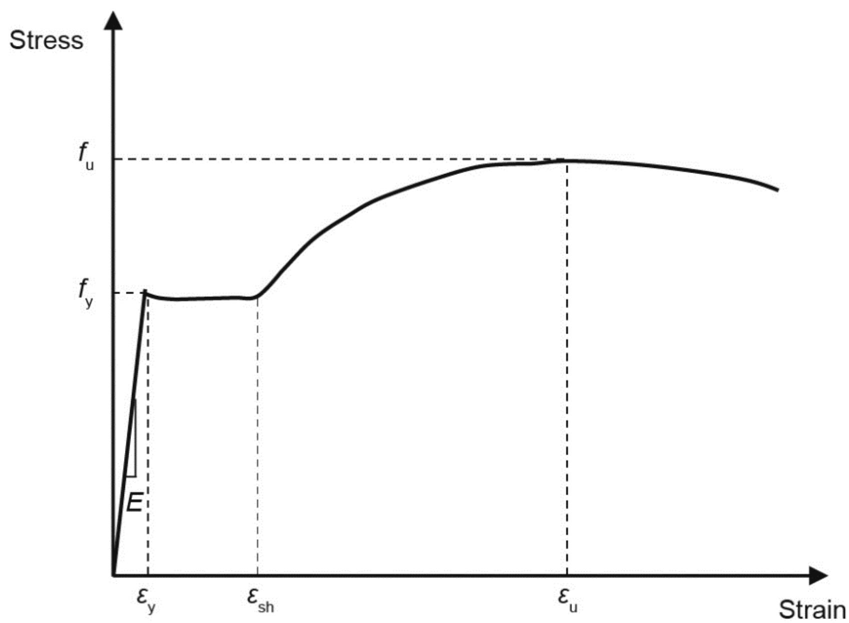

Figure 2.1: Typical Structural Steel Stress-Strain Diagram

The first portion is the elastic deformation stage, where engineers design structures to behave. After the f_y limit has been reached, engineers consider the member failing because it will then undergo plastic deformation and not return to its original shape. However, this is not the full strength of the material and designing members to resist the force induced by this strength would be under designing as described in [4] and [5].

2.2 Adjusted Shear Strength

It is explained in Ductile Design of Steel Structures [4] that the shear strength must be adjusted for material overstrength and strain hardening. Strain hardening occurs when a material is subjected to excessive stress and surpasses the plastic deformation of that material. This is illustrated in Figure 2.1. Because the shear link will be subjected to extreme loads per the design of the failure mechanism, this needs to be accounted for in the link strength calculation. The material overstrength considers the inaccuracies of the steel production process. Engineers must design members assuming the true strength is less than the actual strength due to any inconsistencies or deformities in the manufacturing process. The adjusted link shear strength must assume the steel is at its true potential strength to be accurate with the determination of the true limit of plastic deformation. The preceding variables can be stated quantitatively with the following equation from Ductile Design of Steel Structures [4].

$$V_{link} = 1.25 R_y F_y \, (0.6 A_{lw})$$

Where R_y is the material overstrength, F_y is the yield strength of steel, and A_{lw} is the area of the web, excluding flanges.

Chapter F of AISC [5] has a similar equation for the ALSS where $V_{link} = 1.25 R_y F_y V_n$. Because V_n is a known value and can be found in AISC 360-16 [2] for any standard member, this equation will be used to determine the ALSS. It is explained in Seismic Design Practice for Eccentrically Braced Frames [6] that excess link capacity is not a good thing as it will lead to a further overdesigned structure to ensure the link is still the failing member. Seismic Design Practice For Eccentrically Braced Frames [6] also notes that built-up sections may be preferred as they can more precisely optimize the link.

2.3 Distributor

As stated in Seismic Design of Reinforced Concrete Buildings [7], " tension and compression elements called collectors are required to collect this shear". This element is responsible for transferring the forces from the bracing to the diaphragm. The design of the element ensures the proper transfer of forces from vertical bracing to the horizontal diaphragm.

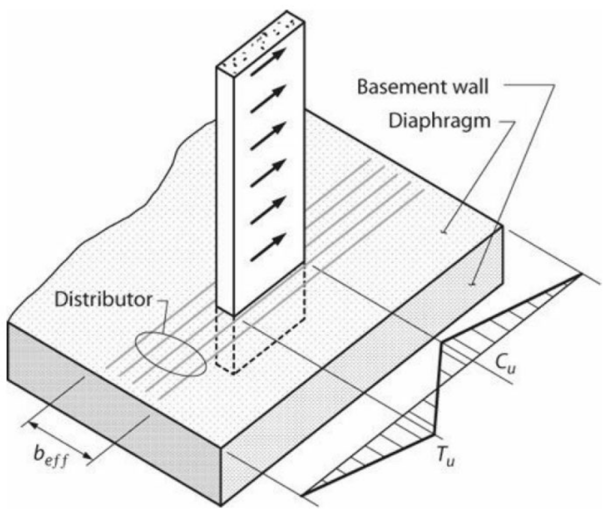

Figure 2.2: Distributor Illustration

2.4 Diaphragm

Also shown in Seismic Design of Reinforced Concrete Buildings [7] is a plan view of a typical floor slab that is subjected to lateral loads as a uniform line load. This diagram helps illustrate why the floor diaphragm can be idealized as a simply supported deep beam.

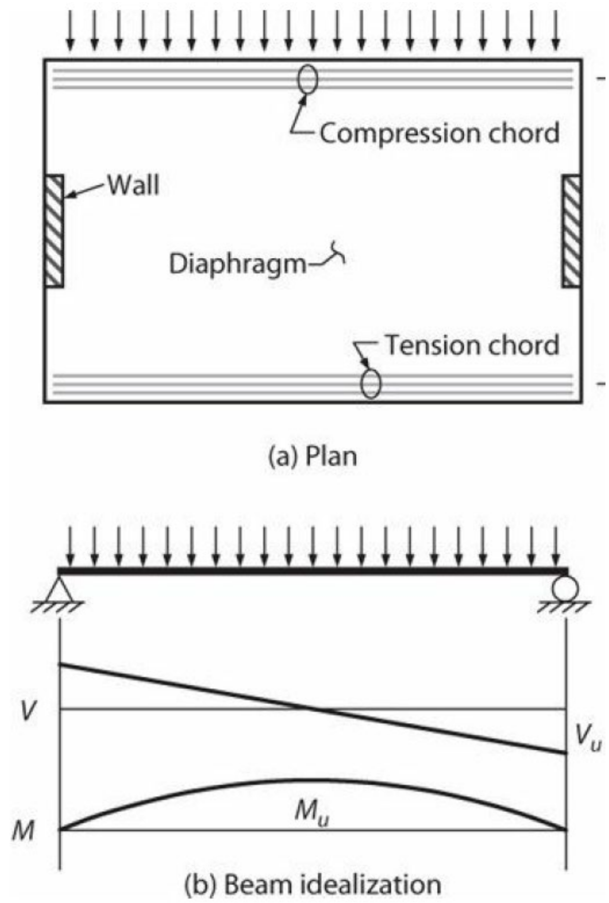

Figure 2.3: Diaphragm Idealization

CHAPTER 3

DESIGN CALCULATIONS

3.1 Adjusted Link Shear Strength

The adjusted link shear strength was determined using the following equation.

$$V_{link} = 1.25R_yF_yV_n$$

Structural steel with a yield strength of 50 ksi was used. The area of the web of the chosen W12X16 is 2.64 square inches and the overstrength adjustment is 10%. This leads to the following shear link strength calculation.

$$V_{link} = 1.25R_yF_y(V_n) = 1.25(1.1)(50)(132) = 181\ kip$$

The applied shear due to gravity and lateral loads was approximately 90 kip which means the ALSS is about 2x more than the applied loads; therefore, all members, including the bracing, columns, podium distributor, podium diaphragm, and shear walls, must be designed for approximately twice the load than the model showed to ensure the target failure mechanism.

Since a full structural analysis can never practically be done by hand, the overstrength factor was used to adjust the output loads from ETABS software to account for the adjusted link shear strength. To do this the applied load was multiplied by 2.1.

<u>3.2 Design of Components</u>

3.2.1 Vertical Braces

The first component that needs to be designed in the load path is the vertical chevron braces. These were originally designed using the axial-moment interaction strength to ensure the brace could resist the combination of applied stresses using chapter H of [2] and will be designed using the same methods for this further analysis. This is a series of calculations (computed using Microsoft Excel) because each LFRD load combination has a unique set of moment and axial forces. This must be done for every brace on the structure leading to hundreds of calculations. The strength of an HSS10X10X5/8 was determined as follows:

The critical buckling stress is determined by

$$F_{cr} = (F_y).658^{\frac{F_y}{F_e}} = (50)(.658)^{\frac{50\ ksi}{41.76\ ksi}} = 30\ ksi$$

The compression strength can then be determined by

$$\phi p_n = \phi F_{cr} A_g = (.9)(30)(21) = 572\ kip$$

Note that this brace member was designed not to buckle under compression to ensure the load is split evenly between the compression and the tension brace.

The flexural strength can be determined by

$$\phi M_n = F_y Z_{weak} = (50)(73.2) = 3{,}660\ kip - ft$$

Where z is the plastic section modulus about the governing axis. The selected member has a square section, therefore does not have a strong and weak direction. This means that $M_{n_{strong}} = M_{n_{weak}}$

The adequacy of the section is then determined by the combination of the ratios of the applied stress to the strength of the section.

$$\frac{p_u}{\phi P_n} + \frac{M_{u_2}}{\phi M_{n_{weak}}} + \frac{M_{u_3}}{\phi M_{n_{strong}}} \leq 1$$

The full series of calculations must be computed in Excel because $P_u, M_{u_2}, and\ M_{u_3}$ are different for every load combination and brace on the structure. A snapshot of the Excel document can be found in Appendix A.

3.2.2 Columns

The internal forces in the columns were determined using geometry and truss analysis.

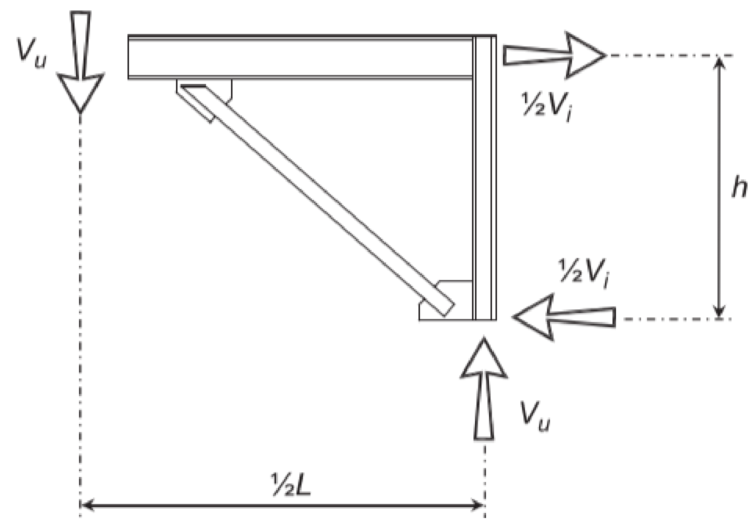

Figure 3.1: Section of Chevron Brace

The axial force in the column decomposes to be equal to the adjusted link shear strength. However, a worst-case scenario needs to be assumed and this is the case of simultaneous shear link capacities. This means the column at the bottom floor must resist the compression and/or tension loads of all the failing shear links at the same time.

The governing LRFD load combination for the columns that includes the ALSS is 1.2D+L+W. This is because the senior design group determined wind governs the lateral loads in this geographical area based on the procedure in ASCE 7-16 [8]. The following table shows the calculation process of the total axial loads on the columns and the required column to resist the stress.

Table 3.1: Column Selections Per ALSS

	Gravity Loads (1.2D+L)		ALSS	Total (Gravity+ ALSS)		Total (kip)		L		Selections	
Floor	Single (kN)	Double (kN)	kN	Single (kN)	Double (kN)	Single	Double	m	ft	Single	Double
5	181	150		181	150	40.69	33.72	3.6	11.81	W8X31	W8X31
4	378	312	181.50	559.5	493.5	125.78	110.94	3.6	11.81	W8X31	W8X31
3	577	475	363.00	940	838	211.32	188.39	3.6	11.81	W8X31	W8X31
2	774	638	544.50	1318.5	1182.5	296.41	265.84	3.6	11.81	W8X35	W8X31
1	972	802	726.00	1698	1528	381.73	343.51	3.6	11.81	w10X45	W10X39
0	1271	1075	907.50	2178.50	1982.50	489.75	445.68	3.6	11.81	w12X53	W10X49

It can be seen in the table that the heaviest beam sections required are W12X53 while the columns sections required for the previous analysis is W18X86. This is because the live load is large enough that the reduced live load factor subtracts more axial load than the ALSS analysis adds. Therefore, the columns are already adequate for the ALSS.

3.2.3 Distributors

The distributors in the podium were simplified to be designed as short columns. This is a valid simplification because the axial forces applied to this strip are far greater than the applied moment. It is short because it does not have to be designed for buckling because there are shear studs connecting it to a steal beam below the distributors.

Both tensile and compressive forces are applied to the distributors as the lateral forces push and pull the structure, as shown in Figure 3.2. Since concrete cannot resist any tensile loads, enough steel must be in the column to resist the horizonal tensile force.

Compression steel was not considered, so the area of concrete was determined by the area required to resist all compressive forces.

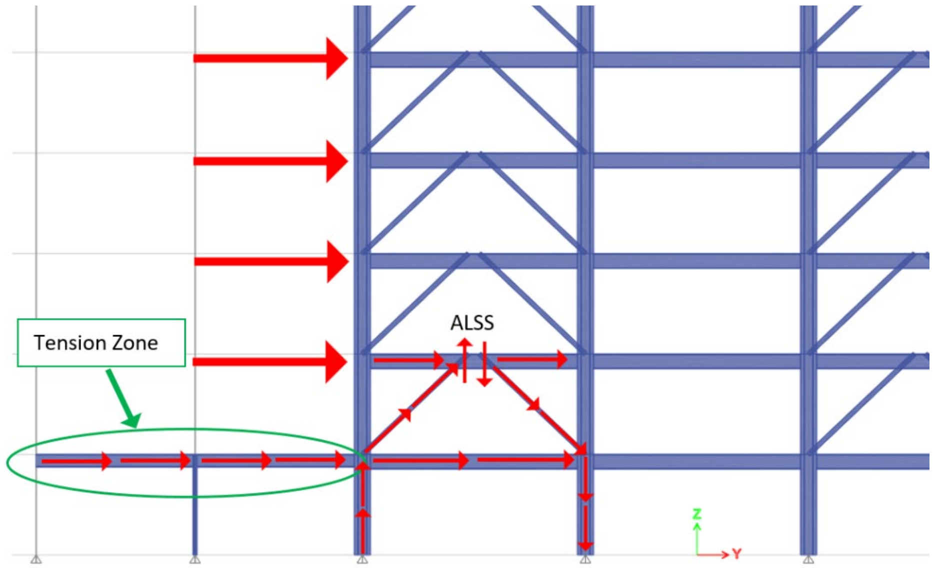

Figure 3.2: Lateral Force Load Path

The tensile strength of reinforcing steel can be determined by the following equation:

$$\phi P_u = \phi A_s F_y$$

When the required strength is known, A_s can be solved.

$$A_s = \frac{P_u}{F_y} = \frac{163{,}450}{60{,}000} = 2.724\ in^2$$

With a chosen bar of #8, the number of bars can be found.

$$\#\ of\ bars = \frac{A_s}{A_b} = \frac{2.724}{.79} \approx 5.56\ bars$$→ Use 6 bars

Six bars of #8 steel is required to resist the tensile stresses in the distributor.

Next, the area of concrete required to resist compression must be determined. With a given total depth of slab of seven inches, the width of the concrete strip required to produce the area needed can be found. It was assumed one inch cover on top and bottom.

$$W = \frac{P_u}{\phi k(.85) f_c' d} = \frac{163{,}450}{(.9)(.8)(.85)(3000)(7-2)} = 21.4" \approx 22"$$

The width of the strip was determined to be 22 inches.

Minimum tie spacing was used for a highly ductile column. The following cross section detail of the distributor is shown in Figure 3.3.

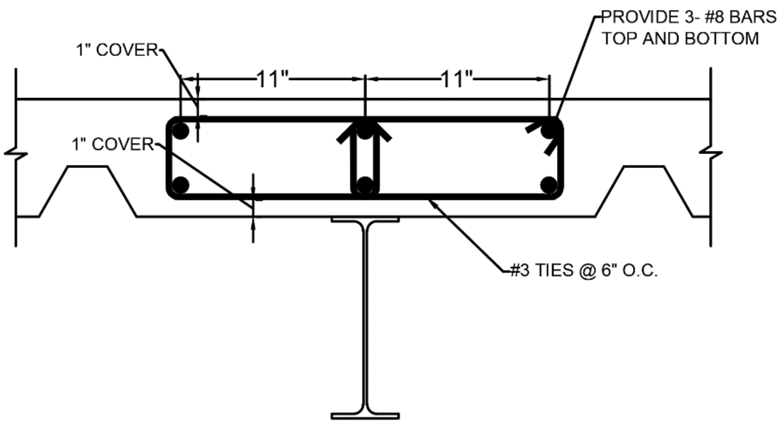

Figure 3.3: Distributor Reinforcement Detail

3.2.4 Pedestal Diaphragm

Similar to the floor diaphragms, a pedestal diaphragm is analyzed as a deep beam with the standard beam analysis, not the strut and tie model.

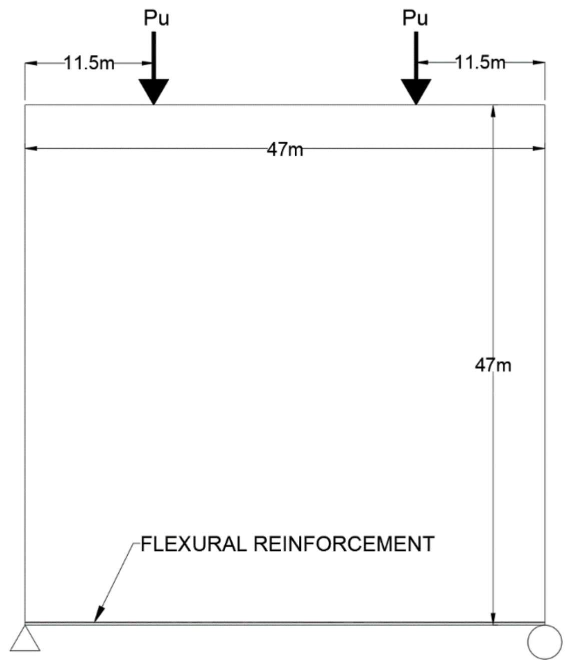

Figure 3.4: Podium Diaphragm Idealized as Beam

Four #5 bars were chosen as the reinforcement to remain consistent with previous floor diaphragms. The resisting moment must be greater than the applied moment. The moment applied by the vertical bracing is calculated by the following.

$M_u = P_n L = (163\ kip)(462\ in) = 75{,}500\ kip - in$

The flexural strength of the deep concrete beam

$$\phi M_n = \phi A_s F_y \left(d - \frac{a}{2}\right) = (.9)(1.24)(60{,}000)\left(1820 - \frac{7.29}{2}\right) = 101.35x10^6\ kip - in$$

Where a is given by

$$a = \frac{A_s F_y}{.85 b f_c'} = \frac{(.31)(60000)}{(.85)(4)(3000)} = 7.29"$$

The variable "b" represents the base of the beam, which is thickness of the slab above the steel deck. The variable "d" represents the depth of the beam, which is the full length of the slab. The flexural strength of the diaphragm is many times larger than the applied moment, so 4-#5 bars work. This same reinforcement was used to reinforce the other sides of the diaphragm. Figure 3.5 shows the reinforcement detail.

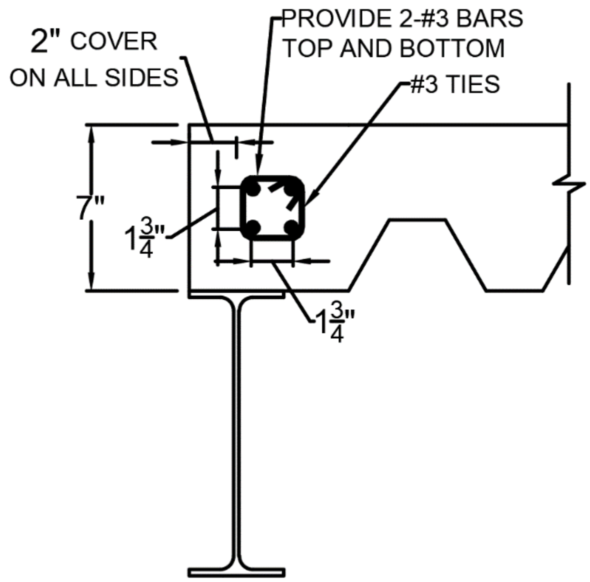

Figure 3.5: Diaphragm Reinforcement Detail

The reinforcement did not have to change for the podium diaphragm.

3.2.5 Shear Wall

The same design approach for the previous shear wall calculation was used for this analysis. The interaction diagram for the shear wall is the same because neither the dimensions nor the reinforcement of the wall changed. The applied loads were increased by a factor of 2x to account for the adjusted link shear strength. The points plot in the interaction diagram as shown in Figure 3.6.

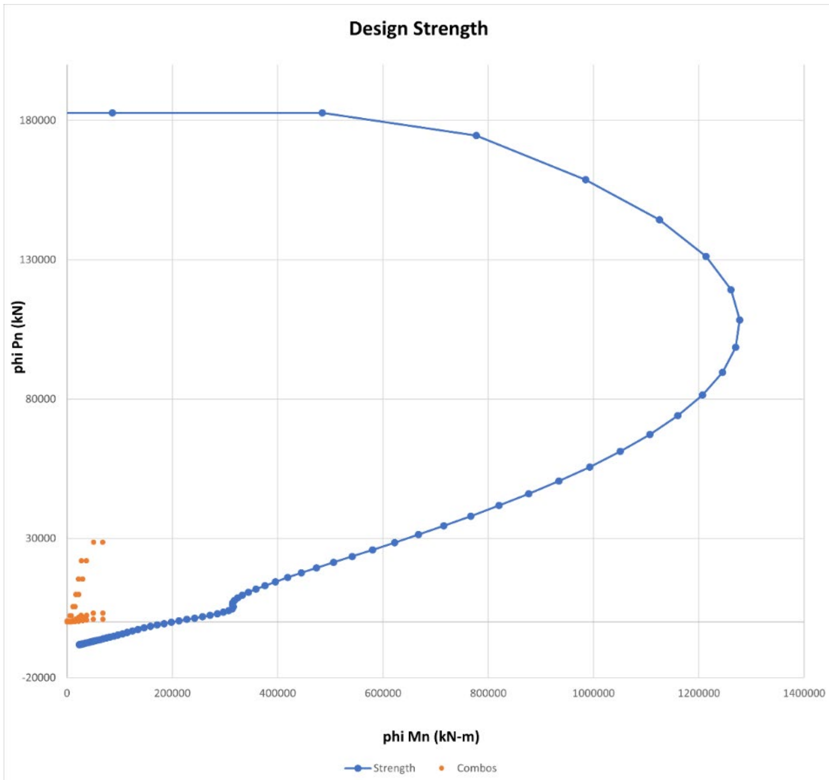

Figure 3.6: Shear Wall Interaction Diagram

Although the plotted applied loads are closer to the edge of the interaction diagram, they all still lie within the design strength of the wall.

The in-plane shear strength of the wall was determined using shear strength of beam analysis and the size effect factor was determined to be zero. The shear strength of the wall can be taken as the greater of the equations in Table 3.2 per the ACI's Building Code Requirements for Structureal Concrete [1].

$\left(0.17\lambda\sqrt{f_c'}+\dfrac{N_u}{6A_g}\right)b_w d$
$\left(0.66\lambda(\rho_w)^{\frac{1}{3}}\sqrt{f_c'}+\dfrac{N_u}{6A_g}\right)b_w d$

Table 3.2: Shear Strength per ACI 318

The first equation yields a higher value in this case, so it will be used to determine the shear strength of the wall. The shear strength calculation is as follows.

$$\phi V_n = \left((0.17)\lambda\sqrt{f_c'}\right) b_w d = (0.17)(1)\sqrt{3000}(.3)(37.6) = 33.5 \text{ MN}$$

The total applied base shear on one wall due to the adjusted link shear strength is just under 2 MN. This shows just how strong the shear wall is when designed for out-of-plane bending. Because of their strength, the shear walls do not need to be made thicker or contain more reinforcement.

CHAPTER 4

DISCUSSION AND RECOMMENDATIONS

4.1 Design Summary and Recommendations

After determining the adjusted link shear strength of the W12X16 beam link, the strength was used to analyze the rest of the structure and ensure that the only component to reach plastic deformation would be the shear link. This leads to a controlled, predictable, and safe structural failure. The strength of the vertical bracing, columns, podium distributor, podium diaphragm, and shear walls were all checked for adequacy.

Of these elements that must resist lateral loads, only the vertical braces and the podium collectors had to be changed; the dimensions of all the other elements can remain unchanged and with the same reinforcement.

It is recommended that the vertical braces be increased to HSS10X10X5/8 and the podium collector to have a width of 22" with 6-#8 bars for reinforcement. The increased strength of these two elements guarantees the code-accepted failure mechanism of the steel structure.

4.2 Further Study and Analysis

4.2.1 Assumptions

A more in-depth and accurate analysis can be conducted if certain assumptions were not made. The assumption that the loads can be multiplied by the adjusted shear strength is not fully accurate. With different loads come different moments and shear stresses that cannot be accurately quantified by doubling the axial load due to the adjusted link shear strength. It was also assumed that the podium diaphragm can be modeled as simply supported. In reality, nothing is fully simply supported and additional stresses will form throughout the diaphragm than what was analyzed. It was also assumed that the flexural analysis was accurate when the strut and tie model would be the correct approach. However, the strut and tie model is a graduate-level subject and beyond the scope of undergraduate coursework. It was also assumed that the shear strength of the wall could be analyzed using a beam shear strength analysis. Although close, a shear wall of this magnitude would most likely need to be analyzed using more advanced models.

4.2.2 Further Analysis

For this project, analysis was stopped at the shear walls. This procedure would normally have to continue through foundation analysis and design. A design without this last step would be incomplete because if the foundation is not checked, it could fail before the shear links and lead to catastrophic structural failure. Because the foundation is not as overdesigned as other components of the structure, the foundation will have to be modified to account for the loads caused by the adjusted link shear strength.

APPENDIX A

VERTICAL BRACING SPREADSHEET

Section Properties

Area, Ag	21	in2
Istrong	304	in4
Iweak	304	in4
rstrong	3.804758925	in
rweak	3.804758925	in
Kstrong	1	
Kweak	1	
Lstrong	315	in
Lweak	315	in
KL/r strong	82.79105358	
KL/R weak	82.79105358	
E	29000	ksi
Fy	50.00	ksi
4.71*sqrt(E/Fy)	113.43	
Fe(weak)	41.76	ksi
Fcr	30.29101155	ksi
Phi	0.9	
Phi*Pn = Phi * I	572.5001183	kip
Zstrong (assum	73.2	in3
Zweak (assume	73.2	in3
Mn about stror	3660	kip in
Mn about weal	3660	kip in
Phi Mn about s	3294	kip in
Phi Mn about v	3294	kip in

Design Data

Output Case	P	M2	M3	Pu/(Phi*Pn)	Mu2/(Phi*Mnwe	Mu3/(Phi*Mnstro	D:C (H1.1)	Acceptance
UDStlS1	-60.95439	0.55356	-2.94126	0.0532	0.00017	0.0009	0.0543	OK
UDStlS1	-57.87957	0.20244	1.50465	0.0505	0.00006	0.0005	0.0511	OK
UDStlS1	-54.80496	-0.14868	-2.54394	0.0479	0.00005	0.0008	0.0487	OK
UDStlS2	-52.27152	0.47481	-2.52105	0.0457	0.00014	0.0008	0.0466	OK
UDStlS2	-49.63602	0.17367	1.28982	0.0434	0.00005	0.0004	0.0438	OK
UDStlS2	-47.00052	-0.12747	-2.18043	0.0410	0.00004	0.0007	0.0417	OK
UDStlS3	-396.00141	0.47229	-0.6657	0.6917	0.00013	0.0002	0.6920	OK
UDStlS3	-393.36591	0.17283	0.4662	0.6871	0.00005	0.0001	0.6873	OK
UDStlS3	-390.73041	-0.12663	-5.6826	0.6825	0.00003	0.0015	0.6841	OK
UDStlS4	291.47706	0.47712	-4.3764	0.5091	0.00013	0.0012	0.5104	OK
UDStlS4	294.11256	0.1743	2.11323	0.5137	0.00005	0.0006	0.5144	OK
UDStlS4	296.74806	-0.12831	1.32174	0.5183	0.00003	0.0004	0.5187	OK
UDStlS5	-52.57413	0.83664	-2.51937	0.0459	0.00025	0.0008	0.0469	OK
UDStlS5	-49.93863	0.30597	1.28898	0.0436	0.00009	0.0004	0.0441	OK
UDStlS5	-47.30313	-0.2247	-2.18358	0.0413	0.00007	0.0007	0.0420	OK
UDStlS6	-51.95043	0.11256	-2.52273	0.0454	0.00003	0.0008	0.0462	OK
UDStlS6	-49.31493	0.04116	1.29045	0.0431	0.00001	0.0004	0.0435	OK
UDStlS6	-46.67943	-0.03024	-2.17728	0.0408	0.00001	0.0007	0.0414	OK
UDStlS7	-86.76402	0.46473	-2.3373	0.0758	0.00014	0.0007	0.0766	OK
UDStlS7	-84.12831	0.16359	1.2075	0.0735	0.00005	0.0004	0.0739	OK
UDStlS7	-81.49281	-0.13755	-2.5284	0.0712	0.00004	0.0008	0.0720	OK
UDStlS8	-17.76054	0.48468	-2.7048	0.0155	0.00015	0.0008	0.0165	OK
UDStlS8	-15.12504	0.18354	1.37193	0.0132	0.00006	0.0004	0.0137	OK
UDStlS8	-12.48954	-0.11739	-1.83246	0.0109	0.00004	0.0006	0.0115	OK
UDStlS9	-86.76402	0.46473	-2.3373	0.0758	0.00014	0.0007	0.0766	OK
UDStlS9	-84.12831	0.16359	1.2075	0.0735	0.00005	0.0004	0.0739	OK
UDStlS9	-81.49281	-0.13755	-2.5284	0.0712	0.00004	0.0008	0.0720	OK
UDStlS10	-17.76054	0.48468	-2.7048	0.0155	0.00015	0.0008	0.0165	OK
UDStlS10	-15.12504	0.18354	1.37193	0.0132	0.00006	0.0004	0.0137	OK
UDStlS10	-12.48954	-0.11739	-1.83246	0.0109	0.00004	0.0006	0.0115	OK
UDStlS11	-396.00141	0.47229	-0.6657	0.6917	0.00013	0.0002	0.6920	OK
UDStlS11	-393.36591	0.17283	0.4662	0.6871	0.00005	0.0001	0.6873	OK
UDStlS11	-390.73041	-0.12663	-5.6826	0.6825	0.00003	0.0015	0.6841	OK
UDStlS12	291.47706	0.47712	-4.3764	0.5091	0.00013	0.0012	0.5104	OK
UDStlS12	294.11256	0.1743	2.11323	0.5137	0.00005	0.0006	0.5144	OK
UDStlS12	296.74806	-0.12831	1.32174	0.5183	0.00003	0.0004	0.5187	OK
UDStlS13	-52.57413	0.83664	-2.51937	0.0459	0.00025	0.0008	0.0469	OK

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BIOGRAPHICAL INFORMATION

Mateo Galvez is completing an Honors Bachelor of Science in Civil Engineering in May 2022 and going straight into graduate school at UT Arlington. He plans on earning his master's degree in Structures and Applied Mechanics in May 2023. He begins full time employment after graduation at Falkofske Engineering, where he has been interning since 2020. Although a few challenging years lie ahead with school and licensing examinations, Mateo looks forward to facing the challenge head-on and doing his best to excel in all he does.