Hershey Academic Support Center
Hershey, PA

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Structural Option
Senior Thesis Spring 2006

Hershey Academic Support Center

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http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/sbj113/

Primary Project Team:

Owner/Developer: Penn State University *(Milton S. Hershey Medical Center and The Pennsylvania State University joint project)*
Construction Manager: Barclay White Skanska
~**Project Manager:** Jon Anthony
~**Project Engineer:** Jessica Kosoff
Architects: WTW Architects
Engineers:
~**Structural:** Whitney, Bailey, Cox & Magnani, LLP
~**Civil:** Rettew Associates, Inc. *(Civil Engineering and Landscape Architect)*
~**MEP:** Brinjac, Kambic & Associates
Consultants:
~**Lighting Design:** Brinjac, Kambic & Associates
~**Geotechnical Engineer:** Schnabel Engineering Assoc., Inc.

General Project Data:

Location and Site: Hershey, PA
Building Occupants: 680 people.
Size: 150,000 ft^2
Number of Stories Above Grade: 5 Stories
Total Height: 56'-0" to the 5th story.
Dates of Construction: March 1999—August 2000.
Overall Project Costs: $16,900,000

Architecture:

~Precast panel concrete and glass window façade

~Glass sheet at center of the building on both sides

~Membrane roofing system with rigid insulation and metal decking

Main lobby is encased by glass windows and extends through the middle of the building

Structural:

~Structural steel system with composite beam floor framing

~Galvanized steel metal decking between the beam and girder connections

~Typical beam size is W16x31, typical girder size is W21x50, and typical column size is a W14x120 & W14x176

~Identically spaced columns per wing in 28' to 33' bays

~Deep foundation system consisting of caissons and grade breams

~Moment connections as well as braced steel frames are used to resist wind force

Mechanical:

~Mechanical Penthouse located on the roof

~Four AHU fans at 42,500 CFM with 460V 3-phase power

~Ductwork system acts as an air plenum to the space

Electrical/Lighting:

~ Main system: 480Y/277 Volt, 3-phase, 4-wire setup

~ Auxiliary system: 208Y/120 Volt power transformers

~Main lighting is fluorescent luminares throughout

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Executive Summary

The Hershey Academic Support Center is part of the Hershey Medical Center complex and is owned by The Pennsylvania State University. Constructed from March 1999 to August 2000, The Penn State Geisinger Health System was designed as the primary occupant, but was dissolved before the building was occupied. Currently the building is used for auxiliary purposes of the Hershey Medical Center and accommodates 680 people. The building itself can be considered in two sections, an East and a West wing. The wings are structurally identical with the only difference between them found in the center section. The building footprint encompasses a total area of 150,000 square feet. The total height of the building over 5 stories is measured as 56'-0" with the height to top of the roof including the Mechanical Penthouse being 69'-0". The building consists of a conventional structural steel system with composite beam floor framing and a precast concrete and glass facade. Moment connections placed at the columns as well as braced steel frames help to resist the wind and lateral loads throughout the building.

A study was conducted to investigate why the lateral system in the building was supported by a composite floor system, moment frames, and braced framing on the roof. It was initially believed that the system was over-designed and that a considerable amount of money could be saved if less lateral resisting components were in place. To more accurately get connections information, the original assumption of fully rigid connections was thrown out and research was conducted to find the true rigidity of all the partially restrained moment connections. After the moment connections were designed for their true partial fixity, calculations were made to see if moment connections could be removed. The resulting change in moment could also alter the floor system since this building uses a special design known as Type 2 with Wind. The new system removed some of the top floor moment connections as well as reduced a few member sizes, but the total cost savings of about $16,000 was not worth the time needed to find the specific partial fixity values.

A construction management study was done to compare the differing costs of the 16 types of moment connections found in the Hershey Academic Support Center. Milton Steel Fabricators as well as RS Means were used to compare cost values and it was found that welded connections are considerably more expensive than bolted connections. Field welding in particular is the most costly option, so if cost is the major issue in the building, avoid welding as much as you can or stick with a braced frame system instead. Another advantage of bolts over welds is that they are easier to implement and therefore take less time than welds. Welds are particularly used when strength of connection is an important issue or if the connection could possibly fracture with bolts instead of yield.

The last study conducted was an architectural study focusing on the fire prevention of the building. It was noted that the building needed to have a 2 hour fire rating and when using Lightweight concrete, a two hour fire rated slab needs to be 3.5" thick. The original slab was only 2.5" thick and cementitious spray on fireproofing was added so that the building met up to code. The system was switched and the thicker slab ended up being around $70,000 cheaper overall. The added slab weight on the structure did not directly affect the column sizes, so the new system is a valid option for this structure.

Introduction

General Introduction

The Hershey Academic Support Center is part of the Hershey Medical Center complex located in Hershey, PA and is owned by The Pennsylvania State University. Constructed from March 1999 to August 2000, The Penn State Geisinger Health System was designed as the primary occupant, but was dissolved before the building was occupied. Currently the building is in an office setting and used for auxiliary purposes of the Hershey Medical Center accommodates a total of 680 people. The building itself can be considered in two sections, an East and a West wing. The wings are structurally identical with the only difference between them found in the center connecting section. The building footprint encompasses a total area of 150,000 square feet over 5 floors. The total height of the building over 5 stories is measured as 56'-0" with the height to top of the roof including the Mechanical Penthouse being 69'-0". The project was delivered as a Guaranteed Maximum Price totaled at $16,900,000.

Architecture

The Hershey Academic Support Center utilizes a postmodern look of concrete and glass. The "wings" of the building form a slight angle out from the center and are clad with a repeating window pattern laced with precast concrete panels. The center of the building has a canopy leading into the main lobby, which is encased by glass and extends across the entire first floor of the building. A sheet of glass is located from the top of the canopy to the top of the building. To break up the repeating window pattern, both sides of the building sport a concrete spike that juts from either side containing a stairwell. There is a membrane roofing system with rigid insulation and metal deck underneath. This overall style of architecture fits in with the rest of the Hershey Medical Center, which provides for a uniform look amongst the area.

<u>Primary Design Team</u>

Owner/Developer:
-Penn State University
-Hershey Medical Center
Architect:
-WTW Architects
Structural Engineer:
-Whitney, Bailey, Cox & Magnani, LLP
Civil Engineer:
-Rettew Associates, Inc.
Project Manager:
-Jon Anthony
Project Engineer:
-Jessica Kosoff
Construction Manager:
-Barclay White Skanska
MEP & Lighting Consultant:
-Brinjac, Kambic & Associates
Geotechnical Engineer:
-Schnabel Engineering Assoc., Inc.

Electrical/Lighting System

The Electrical system used in the building is 480Y/277 Volt, 3-phase, 4-wire setup. There is also 208Y/120 Volt power, which is used by some of the transformers on site as well as some of the boilers. All of the main lighting is done by fluorescent luminares with the only incandescent bulbs being used in the lobby towards the outside.

Mechanical System

The Hershey Academic Support Building has a mechanical penthouse located above the top floor of the building. This building is cooled by four AHUs that are split between the East and West wing. Each AHU has a fan power of 42,500 CFM and is powered by 460V 3-phase power. Air is delivered to the rooms via a ductwork system and the overall system acts as an air plenum to the building. The building also utilizes a boiler/chiller system to regulate the temperature throughout the building in the summer and winter months.

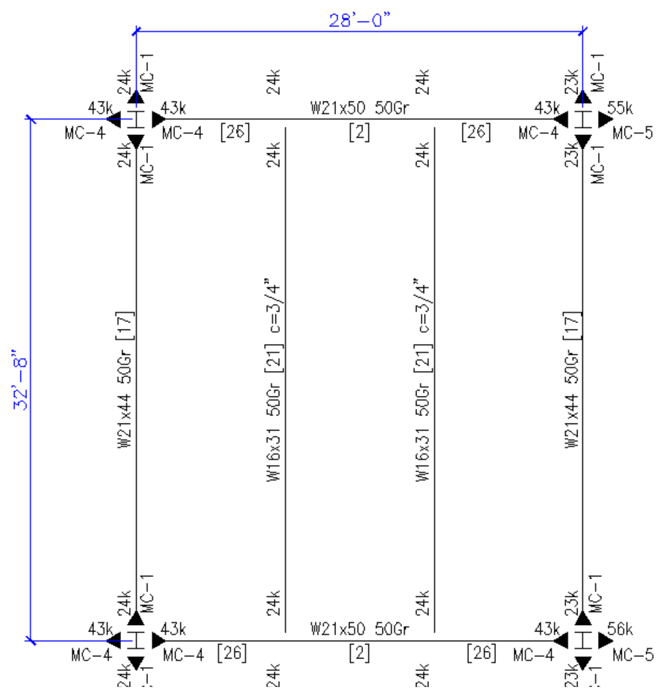
Transportation System:

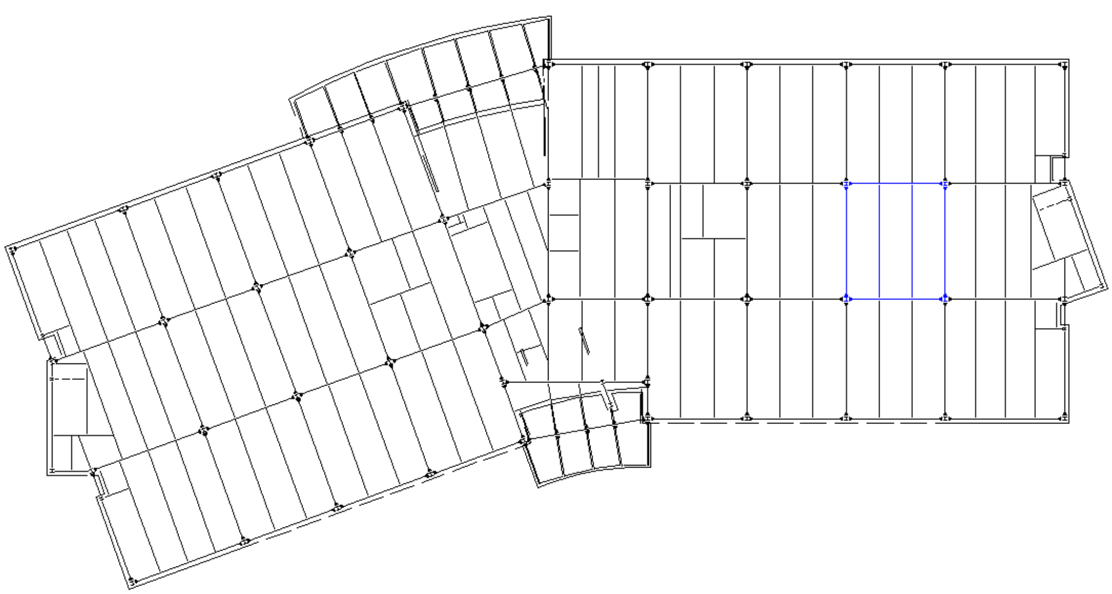
The Hershey Academic Support Building has two main entrances in the back and the front of the building as well as a loading dock on the right side. Each wing is serviced by its own set of stairs that jut out from the side of the building and are fire rated to code. There are two elevators and one stairwell located in the center of the building. The elevator services all 5 floors and the center stairwell provides access to the Mechanical room on the top of the building.

Structural System Information

Floor System

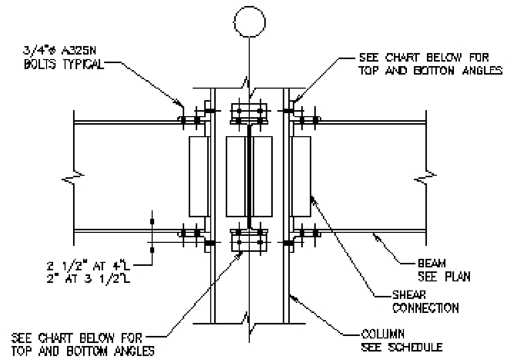
The floor system at the Hershey Academic Support Center utilizes a composite beam floor framing system with 3" 20 gage Vulcraft galvanized steel metal decking and 6x6 W1.4xW1.4 Welded Wire Fabric between the steel members and the concrete. The 2.5" Lightweight concrete along with the decking give an overall slab thickness of 5.5" and a total system depth at the girder of 26.5". To hold together the decking and concrete slab, 0.75" ø x 4.5" long headed steel studs were used. Shear connections are used between the beam flanges and columns to hold the gravity loads on the building. Each typical bay is 28' by 32'-8" and consists of W21x50 and W21x44 girders with W16x31 interior beams that have a ¾" camber. Material strength is given as 4000 psi for the concrete slab and Fy = 50 ksi ASTM A-572 steel in the beams and girders. The floor framing plan and a typical interior bay are shown below in blue.





Lateral System

The main lateral system for the Hershey Academic Support Center is varying moment connections located at almost every column with a total of 617 moment connections used in the building. These connections extend to all 5 floors of the buildings and brace the building in both the N-S and the E-W conditions. The top floor does not utilize moment connections in the E-W direction, but uses Cross Bracing to help prevent the lateral load instead due to the excess weight of the Mechanical Penthouse. Also, the floor system is of composite design which takes a small portion of the lateral load. There are 3 different moment connection types used but with size and bolt combinations, it comes to 16 total types. The three types of connections used are top & bottom angles, top & bottom plates, and top angles & bottom plates. These connections use different bolt numbers and sizes to add strength where needed and the most common connection used in a typical bay is a L6 x 4 x 7/8 x 0'7" steel angle with 4 bolts to a girder and 2 bolts to a column. A typical connection is shown.



WIND MOMENT CONNECTION–
TOP AND BOTTOM ANGLES
MC–1 THRU MC–10

SEE PLAN FOR LOCATIONS

DETAIL A
NOT TO SCALE

Foundation Design

The foundation for this structure is a deep foundation system consisting of caissons and grade beams. The concrete slab on grade is 4" thick and reinforced by WWF. Footings are placed under the columns and step footings were used at the corners of the building for extra support. All exterior footings must extend 3'-6" below the finished grade to protect from frost. Footings have been designed for a net soil bearing pressure of 6,500 psf. Geopiles could have been used in place of spread footings for the same criteria instead if desired.

Roof Design

This building utilizes an EPDM membrane roofing system with rigid insulation placed on a 3" lightweight concrete slab with 3" deep 20 gauge composite steel metal deck underneath. Girder size is increased slightly to W18x40 and W21x76 and the moment connections at the columns were increased in strength with more bolts. The Mechanical Penthouse is located on the roof and houses all the major mechanical components for the entire building.

Problem Statement and Solution Method

Problem Statement

Upon reviewing the design of the Hershey Academic Support Center, it was apparent that the lateral system design was unique. There were a total of three systems in place that helped to resist lateral loads: composite flooring, braced frames, and moment frames. The composite flooring provided minimal lateral resistance support, mostly taking the excess moment from the moment connections while the braced frames were only used to support the Mechanical Penthouse. This leaves the 617 moment connections in the building to provide most of the lateral support. When conducting analysis of the moment frames under full restraint, it was found that the system was over designed and that the total number of connections could be reduced, saving cost. The problem with this data is that it's based off of the assumption that all the partially restrained connections were fully restrained, which isn't true. The question posed itself, if the moment connections were given their actual restraint values, would there still be savings like before? Another factor to consider is that this building is designed using the principals of "Type 2 with Wind Design," so changing the partial fixity will change the wind moments and possibly alter the floor system from it's current setup. These two defining factors, the number of connections and the size of the floor system, are the basis of this study in the hopes that either one or both can be reduced to save on overall building costs.

Lateral Analysis

To first make the change from fully restrained to partially restrained, the nature of partially fixed moment connections had to be reviewed. Research was conducted to determine the flexibility of partially restrained connections as well as methods to apply the partial fixity to loading on a structure. Specific moment values were calculated and applied to the structure depending on the location and type of connection. Checks for fracture versus yield were performed on the moment connection plates to make sure they would allow for flexibility before failing. After these calculations were performed, a 3D SAP2000 model was

created of the entire structure as well as individual framing sections. The moment percentages were entered into SAP with the current connection configuration and checks were made on the structure to ensure that the moment frames were still properly designed and to see if any changes could be made to save cost. Attempts to reduce the number of connections were performed and the results tallied.

The second system affected by the redistribution of moments according to the connection fixity is the Type 2 with Wind floor system. Since these members rely on the moment created by the wind for their design, the new calculated moment values would have to be checked against the current floor system to see if the design matches. All appropriate checks were made and the final system was found using the moment values and RAM Steel Software.

Construction Management Breadth

The Hershey Academic Support Center contains 16 different moment connections with three main types supported in the building. Each of the three main types of moment connection has a price associated with materials and installation. In an effort to reduce cost, the prices of the main moment connections were calculated and obtained from a steel fabricator to compare between the types. The lowest cost moment connection was then used as the base type and a system was made to replace the other connections as could be allowed by calculation. Time was also considered in the replacement with a scheduling comparison between all three types using RS Means.

Architectural Breadth

Another interesting system in the Hershey Academic Support Center is the Fire Prevention System. Going by the Pennsylvania Department of Labor and Industry's Fire and Panic Code, the building was designed with an extensive sprinkler system as well as the code required 2 hour fire rating between floors. The interesting thing about this design is that instead of making the Lightweight concrete composite slab 3.5" to meet the 2 hour fire code, they instead made the

slab 2.5” with ½” spray on cementitious fireproofing on the deck, beams, and girders. Cost could potentially be saved if the slab thickness was increased and also more options architecturally would be open because the beams and columns wouldn’t need the spray on fireproofing that resulting in some columns from being hidden from view.

Structural Background

Introduction

The Structural Depth analysis will look at many different things regarding the structure of the Hershey Academic Support Center. First, design criteria including some material strengths and code references will be presented as a guide to the work that follows. Next, existing conditions of both the gravity and lateral load will be presented to give a foundation to work from. Lateral systems will be next with more specifics and the actual calculation of partial fixity. After these values are confirmed, the floor system will be examined using the new loads in the Type 2 with Wind analysis. Lastly, conclusions will be made as to the success of the study and insights will be given as to why the data showed specific results.

Design Criteria

The main code used in the design of the Hershey Academic Support Building was the BOCA 1996 code, but for current design purposes and the purpose of computer analysis, ASCE 7-02 was used. The original building also used the 9^{th} Edition of the Allowable Stress Design for structural steel calculations, but AISC Load and Resistance Factor Design, 3^{rd} Edition was used for my calculations.

Another criterion given by the building designers was material strengths Concrete will be stone aggregate concrete with a minimum compressive strength of 4000 psi at 28 days. All Structural steel beams will be Fy = 50,000 psi as given by ASTM A-572 and all columns, angles, channels, and miscellaneous steel will be Fy = 36,000 psi as given by ASTM A-36. Welded connections shall be done with E70XX Electrodes with 3/16" minimum material and bolted connections will use ¾" ø ASTM A325N high strength bolts minimum. Lastly, all metal floor deck shall be 3" VLI – Galvanized 20 Gage composite decking and will be designed to resist a floor shear load of 2000 plf and a roof shear load of 3000 plf as well as uplift loads. All of these specifications were conformed to throughout the analysis.

Gravity loads used on the building are as follows:

Dead Loads

Total Roof Dead Load = 30 psf
Total Penthouse Dead Load = 125 psf
Total Office Dead Load = 70 psf

Live Loads

Roof = 30 psf + snow drifting
High Density File Storage = 200 psf, uniformly distributed
Main Floor = 100 psf (with corridors and partitions)
Mechanical Penthouse = 150 psf
Stairs = 100 psf
Total Snow Load = 21 psf

Lateral loading conditions that were used to check the structure:

~Case #1: 1.4D
~Case #2: 1.2D + 1.6L + 0.5S
~Case #3: 1.2D + 1.6S + 0.8W
~Case #4: 1.2D + 1.6W + 0.5L + 0.5S
~Case #5: 1.2D + 1.0E + 0.5L + 0.2S
~Case #6: 0.9D +1.6W
~Case #7: 0.9D + 1.0E

Existing Conditions

Presented below are some of the more important existing conditions of the Hershey Academic Support Center. Any other relevant conditions can be found in the appropriate Appendix.

Gravity Spot Check

A gravity load spot check was performed on the interior beams, a typical girder and a typical column to ensure stability. The results were:

Typical beam – øM_n = 274.78'k --> W18x40 with ø$_b M_p$ = 294'k
The original design was a W16x31 with ¾" of camber, which is why the designed beam is larger.

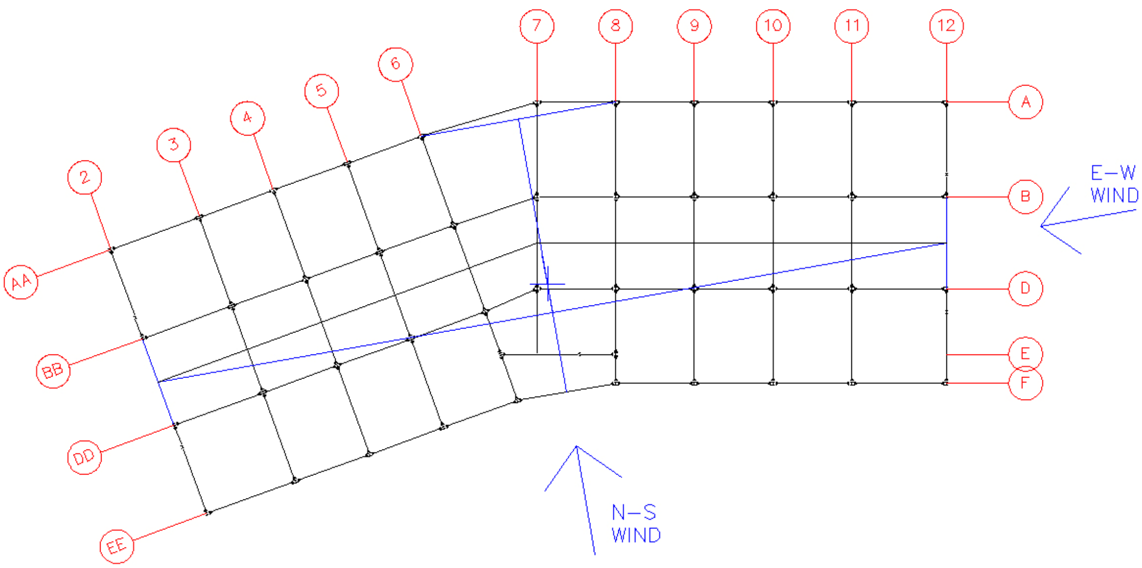
Typical Girder – øM_n = 313.60'k --> W21x44 with ø$_b M_p$ = 358'k
The original design was a W21x50 and since this is larger than the projected girder from wind moments, it passes shear checks.

Typical Column – P_{EFF} = 842.88k --> W14x90 with $\phi_b M_p$ = 969k

The original design was a W14x120 which can be attributed to the extra weight of the Mechanical Penthouse and possibly the wet weight of the composite slab.

Lateral Load Case Check

Using the 7 load cases above, loads were calculated and the controlling case was found to be Load Case #6: 0.9D+1.6W. This also led to the introduction of Type 2 with Wind Analysis that is explained a little later on.

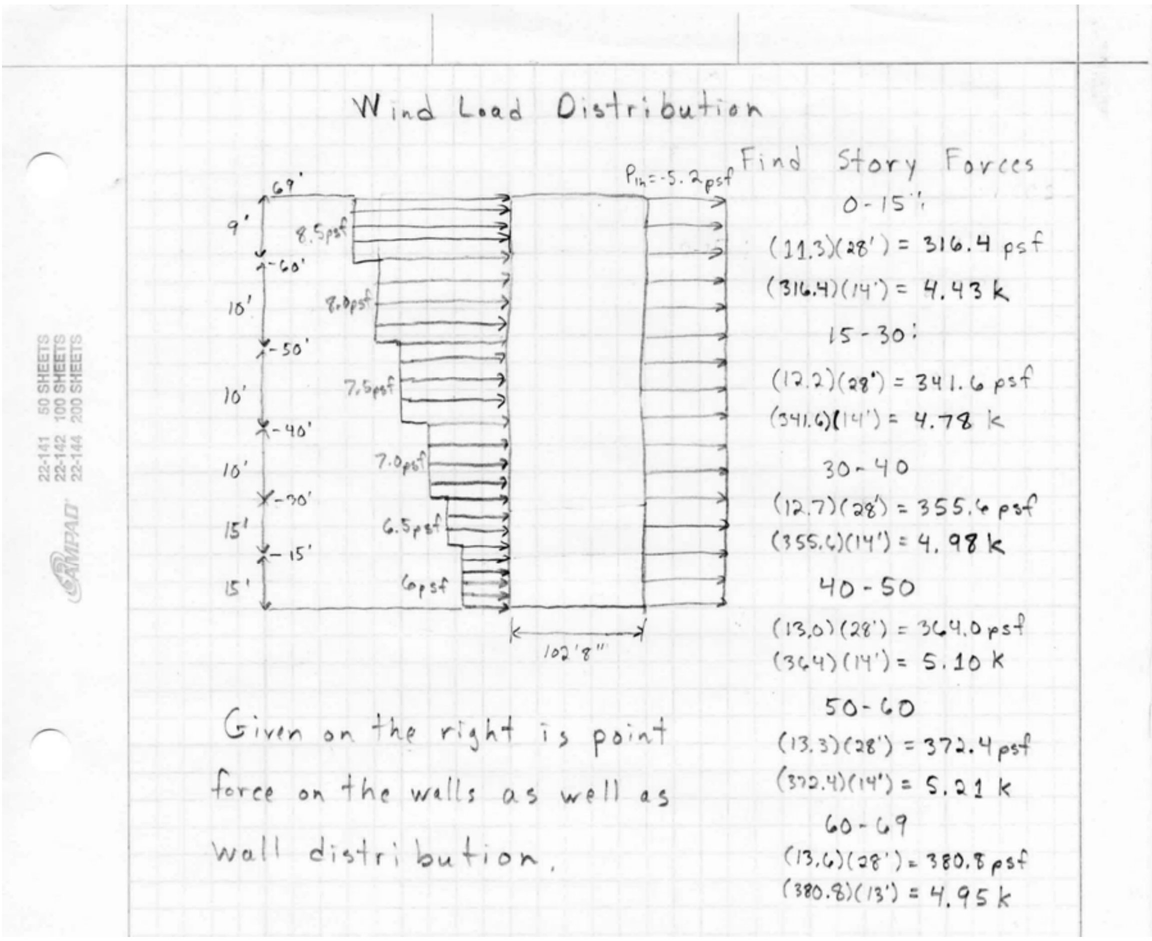


Wind Loads

z (ft)	K_z	q_z	(P_{wz}) N-S	(P_{lh}) N-S	(P_{tot}) N-S	(P_{wz}) E-W	(P_{lh}) E-W	(P_{tot}) E-W
0-15	0.85	9.06304	6.079937	-5.21265	11.29259	6.257873	-3.21912	9.476997
20	0.9	9.59616	6.437581	-5.21265	11.65023	6.625984	-3.21912	9.845107
25	0.94	10.02266	6.723695	-5.21265	11.93635	6.920472	-3.21912	10.1396
30	0.98	10.44915	7.00981	-5.21265	12.22246	7.21496	-3.21912	10.43408
40	1.04	11.0889	7.438982	-5.21265	12.65163	7.656692	-3.21912	10.87582
50	1.09	11.62202	7.796626	-5.21265	13.00928	8.024802	-3.21912	11.24393
60	1.13	12.04851	8.08274	-5.21265	13.29539	8.319291	-3.21912	11.53841
70	1.17	12.47501	8.368855	-5.21265	13.58151	8.613779	-3.21912	11.8329

	N-S	E-W
Story Shear @ 0	21.21098	6.811023
Story Shear @ 1	43.07454	13.87904
Story Shear @ 2	46.18385	15.10357
Story Shear @ 3	48.39108	15.97283
Story Shear @ 4	50.09928	16.64556
Story Shear @ 5	35.30126	10.81774

The charts shown above summarize the results found from my wind calculation analysis. Shown below is the wind loading for a typical building wall as well as story forces. Specific calculations of wind forces are located in the Appendix as well as the calculation of Seismic forces.



Story Deflection Check

Story Deflection for the assumed fully restrained moment connections was calculated by SAP2000 which was used to analyze each moment frame individually in the building. Using a 1k force at the top of the each frame structure, story deflections were found and then converted into stiffness values by the equation Stiffness (K) = 1/deflection (Δ). When combined, these stiffnesses

give the load distribution for the moment frame, the floor, and the total section as well. The values obtained for a typical frame in each section are listed below. Detailed calculations can be found in the appendix.

Deflection Calculation H/400: ((69')*(12in/ft))/400 = 2.07in
East Section Frame #12: Story Drift = 2.02in < 2.07in ALLOW
West Section Frame #2: Story Drift = 1.91in < 2.07in ALLOW
Center Section Frame #D: Story Drift = 1.83in < 2.07in ALLOW

Spot Checks, Overturning, and Strength checks were all also calculated as well and all of them passed (Detailed Calculations in the Appendix).

Structural Depth

Lateral System - Background Information

Connections in buildings have always been an important issue to consider when going through the design process. The two main types of connections used are fully restrained and partially restrained connections. Fully restrained connections are designed to not allow any rotation at the connection and therefore preventing any moment transfer. A partially restrained connection is a connection that will allow the ends of a beam to rotate slightly to help transfer some of the lateral moment loading. The connection must be designed to flex far enough to allow rotation before the connection fractures.

The graph shown here is and example of End Moments versus Rotation for different types of connections. Curve one represents a fully flexible connection which yields at low moment allowing the connection to rotate. This type of curve is usually attained from top angle or top plate connections. The second curve is the semi-rigid or partially restrained connection. This connection has a varying level of rigidity depending on the type of connection in place and specifically is based off of the slope of the initial stiffness. Connections in this category can include top and bottom angles, top and bottom plates, as well as a combination of the two. Curve 3 represents a fully rigid connection as there is almost no rotation with the introduction of moment. These connections are usually associated with short stiff plates used at the columns.

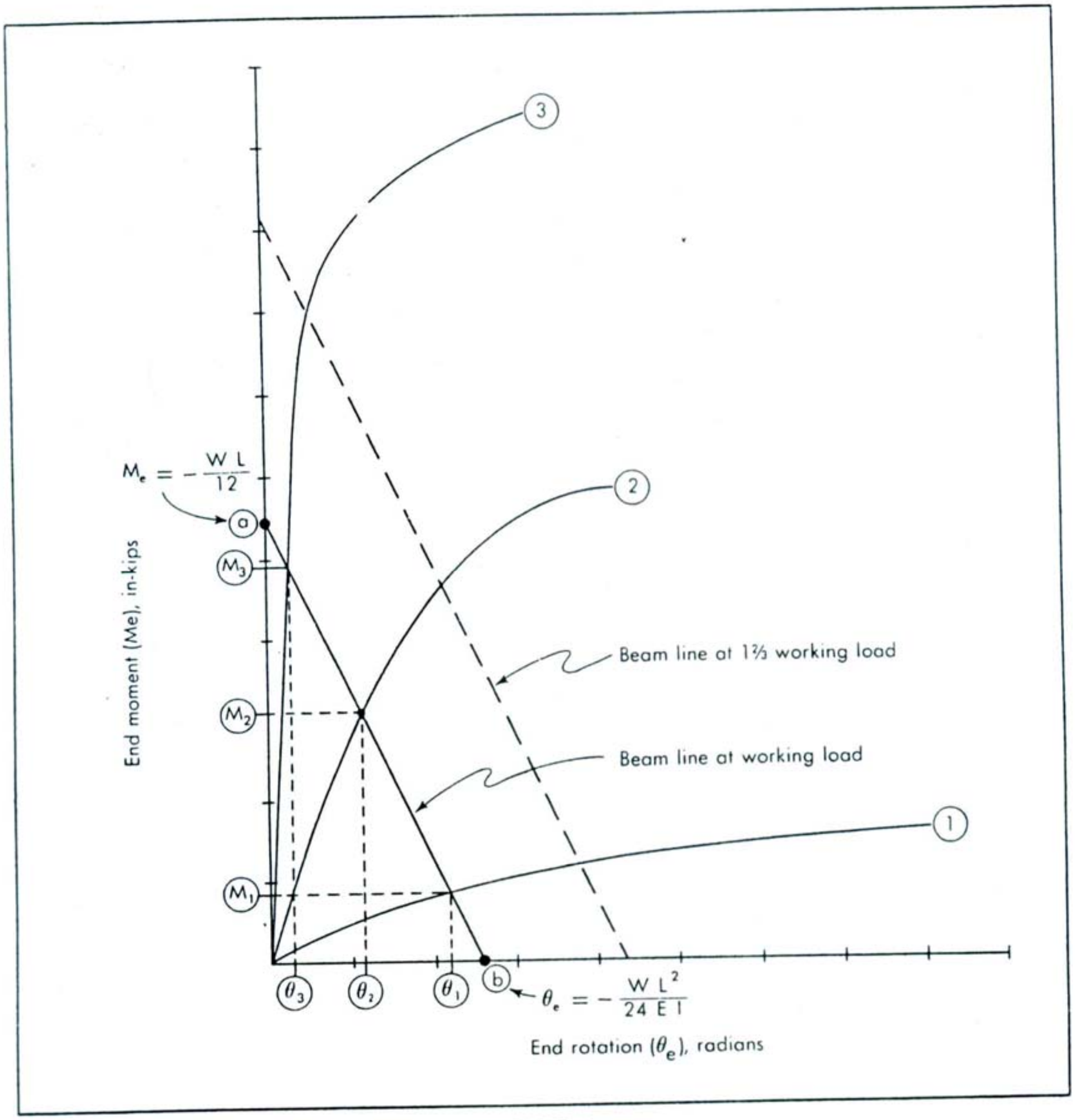


FIGURE 2

While partially restrained moment connections are not often used en masse in lateral design, some firms such as Stanley D. Lindsey and Associates Ltd. have shown that buildings which utilize PR connections can result in very economical designs. Fabrication designs are not complicated and most welding is eliminated as the connections are simple in design. While this is mostly true for the Hershey Academic Support Center, not all welding is avoided in the use of PR

connections in the building and with a total of 16 different specifications; the simple design becomes slightly more complex.

To model the partial fixity of a moment connection, there are two defining equations which can be used to find this value. From a paper by John Christopher and Reidar Bjorhovde on Semi-Rigid Frame design, the equations are given as:

$$M = \frac{R_{ki}\phi}{\left[1 + \left(\frac{\phi}{\phi_0}\right)^n\right]^{1/n}}$$

where
R_{ki} = initial stiffness factor
n = shape factor
φ_o = reference plastic rotation, calculated as $\varphi_o = M_u/R_{ki}$
M_u = ultimate moment capacity of the connection
and

$$\alpha_i = \frac{EI}{R_{ki}d}$$

where
E = modulus of elasticity
I = moment of inertia of the beam
α_i = non-dimensional characteristic length factor
d = beam depth

These two equations were used to compare the fixity of the different types of moment connections.

Another method of comparison that was used to determine the moments transferred through the partially restrained moment connections is from the Blodgett, Lincoln Arc Welding Foundation as seen on the next page below. Each different type of connection has its own moment equation to describe the behavior of the moment across the end. It is important to note that the connections listed are shown as welds but that the angled connection with bolts performs similarly to one with welded ends, so the values shown are comparable to the connections found in the Hershey Academic Support Center.

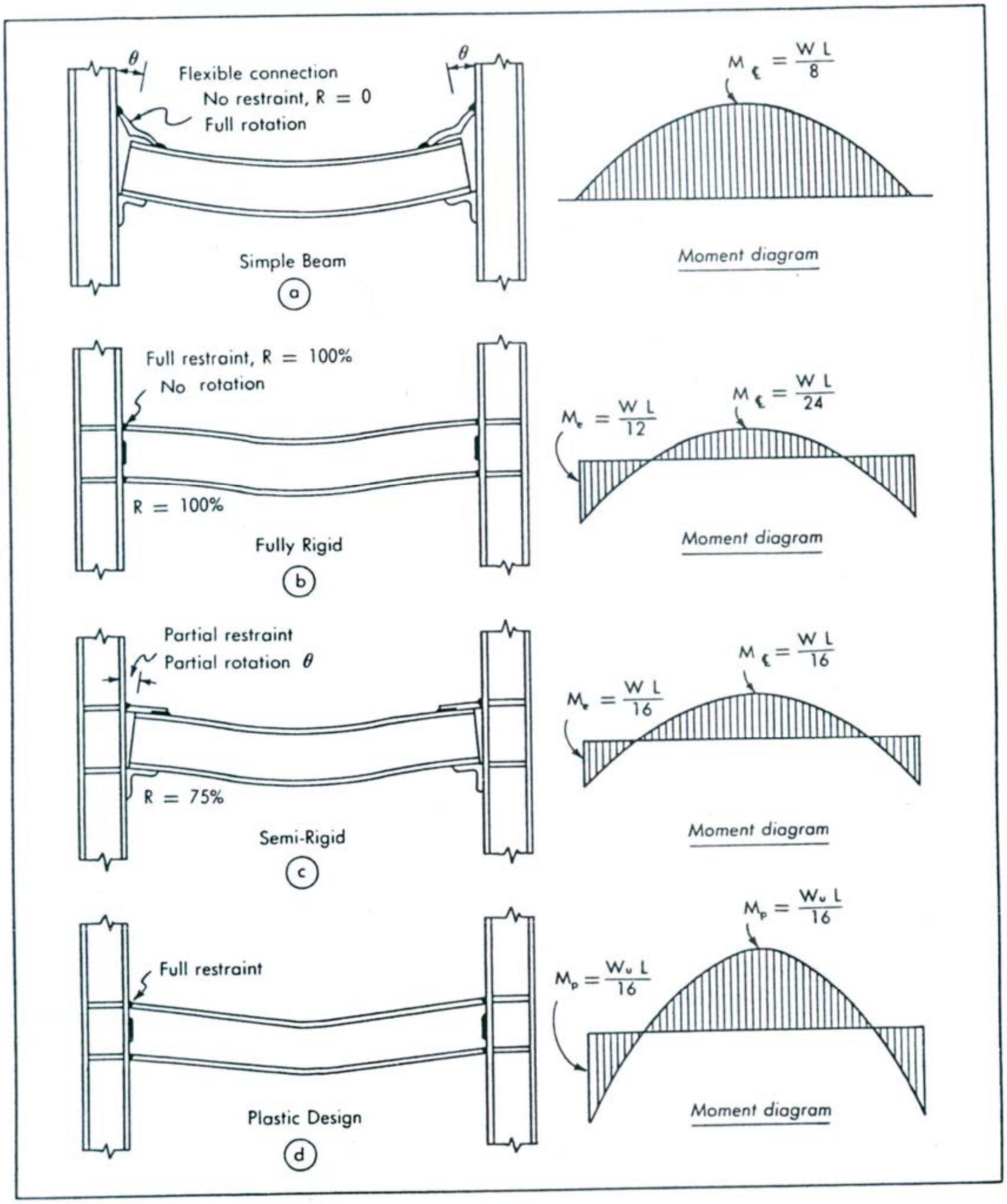


FIGURE 1

Lateral System – Calculations

The first calculation was to check to make sure the moment connections in the building would yield before fracturing or weld rupturing. If any connections were to fracture or rupture before reaching their yield strength then no moment could be transferred across the connection. Due to the nature of semi-rigid

connections, it is important that the connections will yield. The equations used were:

Fracture = $\phi F_u A_n = (0.75)*(58ksi)*A_n = 43.5ksi*(A_n)$

Rupture = $\phi F_n A_w = (0.75)*(0.6)*(70)*A_w = 31.5*A_w$

Yield = $\phi FyAg = (0.9)*(36ksi)*A_g = 32.4ksi*(A_g)$

where An is the net area of fracture, A_w is the weld area and Ag is the gross area of the connection.

MC-1 & MC-2: $A_n = 6.48in^2$ and $A_g = 7.98in^2$
Fracture = 281.88k, Yield = 258.55k, Fracture > Yield ALLOW

MC-3: $A_n = 3.25\ in^2$ and $A_g = 4in^2$
Fracture = 150.08k, Yield = 129.6k, Fracture > Yield ALLOW

MC-4, MC-5, & MC-7: $A_n = 5.44in^2$ and $A_g = 6.94in^2$
Fracture = 236.64k, Yield = 224.86k, Fracture > Yield ALLOW

MC-6: $A_n = 3.86in^2$ and $A_g = 4.61in^2$
Fracture = 167.91k, Yield = 149.04k, Fracture > Yield ALLOW

MC-8 & MC-10: $A_n = 2.5in^2$ and $A_g = 3.25in^2$
Fracture = 108.75k, Yield = 105.3k, Fracture > Yield ALLOW

MC-9: $A_n = 2.88in^2$ and $A_g = 3.63in^2$
Fracture = 125.28k, Yield = 117.61k, Fracture > Yield ALLOW

MC-11 Top: $A_w = 4in^2$ and $A_g = 2.5in^2$
Fracture = 126k, Yield = 81k, Rupture > Yield ALLOW

MC-11 Bottom: $A_w = 10in^2$ and $A_g = 3in^2$
Fracture = 315k, Yield = 97.2k, Rupture > Yield ALLOW

MC-12 Top: $A_w = 6in^2$ and $A_g = 3.5in^2$
Fracture = 189k, Yield = 113.4k, Rupture > Yield ALLOW

MC-12 Bottom: $A_w = 14in^2$ and $A_g = 4.5in^2$
Fracture = 441k, Yield = 145.8k, Rupture > Yield ALLOW

MC-13 Top: $A_w = 6in^2$ and $A_g = 2.5in^2$
Fracture = 189k, Yield = 81k, Rupture > Yield ALLOW

MC-13 Bottom: $A_w = 12in^2$ and $A_g = 3.75in^2$
Fracture = 378k, Yield = 121.5k, Rupture > Yield ALLOW

MC-14 & MC-16 Top: $A_w = 5in^2$ and $A_g = 1.5in^2$
Fracture = 157.5k, Yield = 48.6k, Rupture > Yield ALLOW

MC-15 Top: $A_w = 6in^2$ and $Ag = 1.88in^2$
Fracture = 189k, Yield = 60.91k, Rupture > Yield ALLOW

Both sets of calculations passed for all connections so it is safe to assume the connections will transfer moment.

With 617 total connections in the building, some assumptions were made due to the similar nature between beam sizes in the effort to save time. When calculating the individual connection stiffnesses, each angled connection was taken in conjunction with the beam it was most commonly found on and this was assumed to be the average for that connection. The initial stiffness becomes the initial slope for the connection's Moment vs. Rotation graph and can be checked accordingly. For the plates, the stiffness was calculated using a reference graph from W. McGuire on Steel Structures. To test the validity of the graphs with my connections data, the calculated angle connections were compared with the data on the graph using relative area as a basis for comparison. The values came out very similar which can be seen in the graph below.

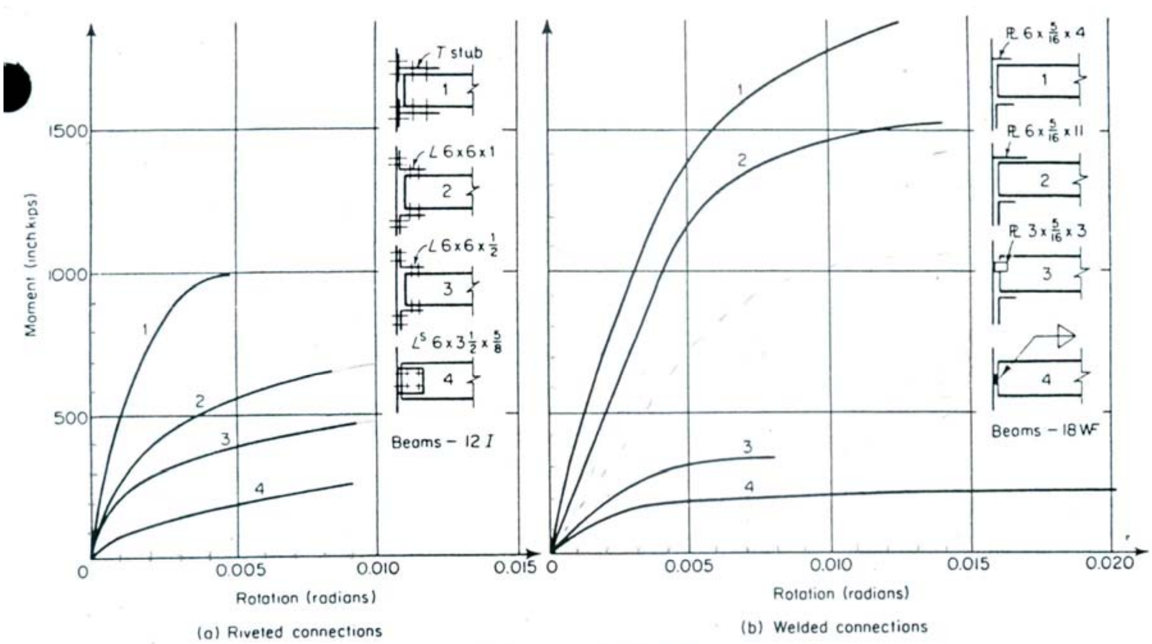
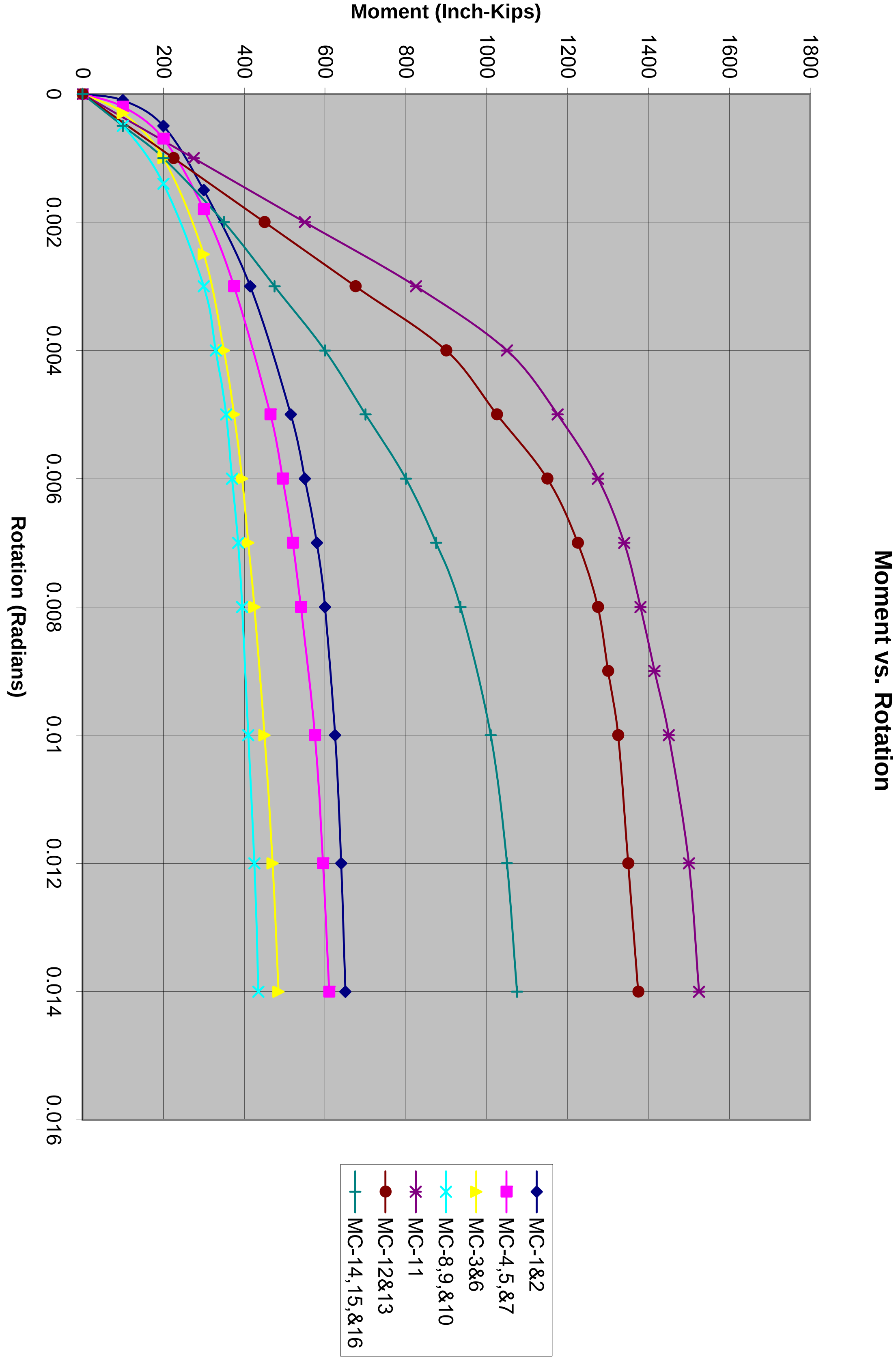


Fig. 6.67. Sample M-θ diagrams.

Steel Structures, W. McGuire, Prentice-Hall 1968.

Connection Designation	Connection Type	Connection Size	Relative Stiffnesses (R_{ki})
MC-1	Top and Bottom Angles	L6 X 4 X 7/8 X 0'-7"	**101,549**
MC-2	Top and Bottom Angles	L6 X 4 X 7/8 X 0'-6"	**97,589**
MC-3	Top and Bottom Angles	L3-1/2 X 3-1/2 X 5/8 X 0'-6 1/2"	**79,203**
MC-4	Top and Bottom Angles	L6 X 4 X 3/4 X 0'-7"	**87,551**
MC-5	Top and Bottom Angles	L6 X 4 X 3/4 X 0'-8"	**88,380**
MC-6	Top and Bottom Angles	L4 X 4 X 5/8 X 0'-10"	**79,417**
MC-7	Top and Bottom Angles	L6 X 4 X 3/4 X 0'-9"	**92,323**
MC-8	Top and Bottom Angles	L3-1/2 X 3-1/2 X 1/2 X 0'-6 1/2"	**68,596**
MC-9	Top and Bottom Angles	L3-1/2 X 3-1/2 X 9/16 X 0'-5"	**68,830**
MC-10	Top and Bottom Angles	L3-1/2 X 3-1/2 X 1/2 X 0'-10"	**73,001**
MC-11	Top Plate	PL4 X 5/8 X 1'-2"	262,300
	Bottom Plate	PL8 X 3/8 X 2'-0	241,000
	Equivalent Stiffness		**251,650**
MC-12	Top Plate	PL7 X 1/2 X 1'-8"	248,100
	Bottom Plate	PL12 X 3/8 X 2'-10"	212,700
	Equivalent Stiffness		**230,400**
MC-13	Top Plate	PL8 X 3/8 X 1'-8"	236,600
	Bottom Plate	P12 X 5/16 X 2'-8"	214,000
	Equivalent Stiffness		**225,300**
MC-14	Top Plate	PL4 X 3/8 X 1'-6"	256,000
	Bottom Angle	L3-1/2 X 3-1/2 X 1/2 X 0'-6 1/2"	68,596
	Equivalent Stiffness		**162,298**
MC-15	Top Plate	PL5 X 3/8 X 1'10"	238,700
	Bottom Angle	L3-1/2 X 3-1/2 X 5/8 X 0'-6 1/2"	79,203
	Equivalent Stiffness		**158,952**
MC-16	Top Plate	PL4 X 3/8 X 1'-6"	256,000
	Bottom Angle	L3-1/2 X 3-1/2 X 1/2 X 0'-10"	73,001
	Equivalent Stiffness		**164,501**

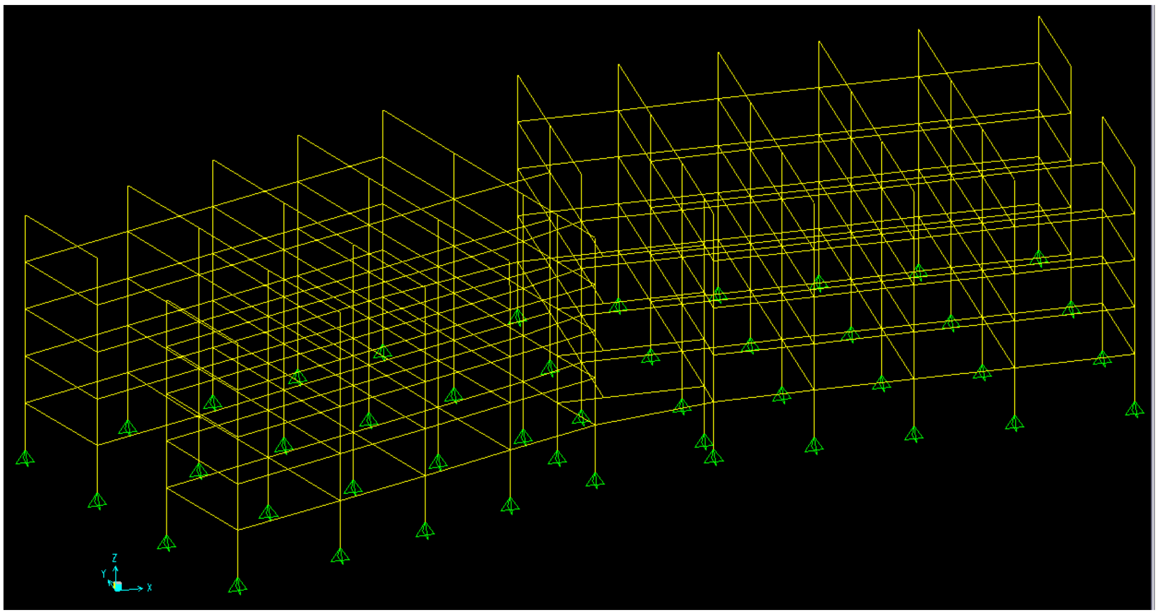

Moment vs. Rotation
Moment (Inch-Kips)
Rotation (Radians)
0
200
400
600
800
1000
1200
1400
1600
1800
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
MC-1&2
MC-4,5,&7
MC-3&6
MC-8,9,&10
MC-11
MC-12&13
MC-14,15,&16

The data above shows that in terms of flexibility, angled connections allow the most rotation for the same amount of moment as the other two connection types. For simplicity sake, connections that had R_{ki} values within 5% are shown as the same curve above, though in reality the curves would be slightly different. Every connection has a unique Moment-rotation curve, but it's interesting to note that at low moments all of these connections behave alike. At about 200"k of moment, the connections branch off depending on their type. As a general rule with angles, the thicker the angle is, the less rotation it allows. Oppositely, plates function in a different manner where that the smaller the plate used, the stiffer it is and the less rotation it allows. For connections with both angles and plates, the two separate values were found and an average was taken to find stiffness over the whole connection.

Using the graph above and the R_{ki} values obtained from previous calculations, the restraint value 'R' can be calculated as a percent of moment transferred for each moment connection. Most partially restrained connections fall between R = 90% and R = 20% for their restraint value, which proved true with the connections in my building. The highest restraint value was from the top and bottom plate connections at 85% whereas the lowest value was the top and bottom angles with 23%. The calculated values are shown below.

Moment Connection	Restraint Value (R)
MC-1	34%
MC-2	33%
MC-3	27%
MC-4	30%
MC-5	30%
MC-6	27%
MC-7	31%
MC-8	23%
MC-9	23%
MC-10	25%
MC-11	85%
MC-12	78%
MC-13	76%
MC-14	55%
MC-15	54%
MC-16	56%

With the restraint percentages, a 3D SAP2000 model can be created and used to test the story deflection of the entire structure. While there is no officially set criteria for story deflection, H/400 will be used to test and see if the structure meets the deflection requirements. SAP2000 models of each individual frame were also created to test and see if any moment connections can be removed and have the structure still meet the deflection requirements, possibly saving time and money. The full lateral model is shown below.

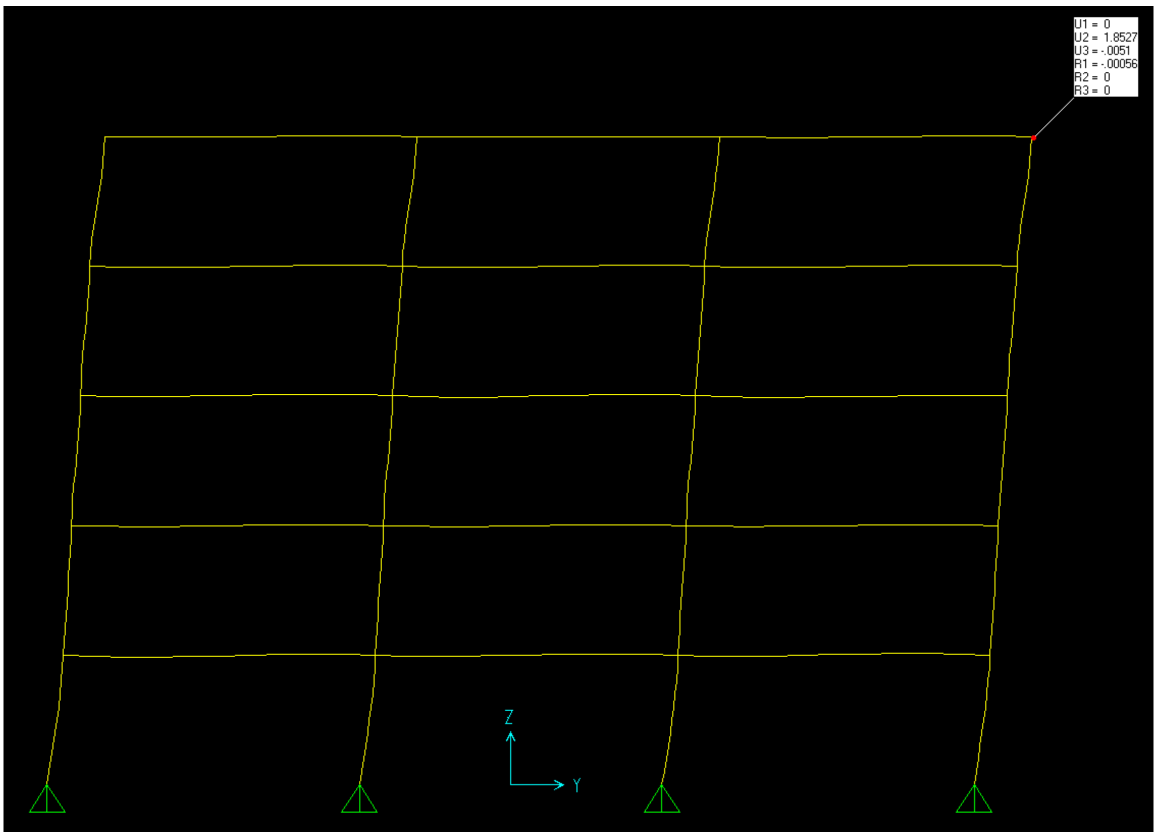


Deflections for the entire structure were calculated and three frames were picked for a typical frame in the East, West, and Center section. Results were:

Deflection Calculation H/400: ((69')*(12in/ft))/400 = 2.07in
East Section Frame #12: Story Drift = 1.53in < 2.07in ALLOW
West Section Frame #2: Story Drift = 1.47in < 2.07in ALLOW
Center Section Frame #D: Story Drift = 1.87in < 2.07in ALLOW

All sections passed with the partial fixity in place which shows a good design. This data also shows that partially restrained connections allow more deflection than fully restrained connections when compared with my initial fully restrained data. The original data only analyzed one frame at a time whereas the new data was taken with the entire lateral system supporting itself and yet the deflections were very similar.

Since each section passed, all three moment frames had two connections removed from the roof section to see if deflection would still pass if fewer connections were in place. The connections from the roof were the chose to be removed because they represent the smallest angles and plates involved in the lateral system due to the braced frames supporting the other direction. Two connections were removed instead of just one to keep the frame symmetric and t he wind loads balanced. The East Section is shown below with the Center and West Sections summarized as well.

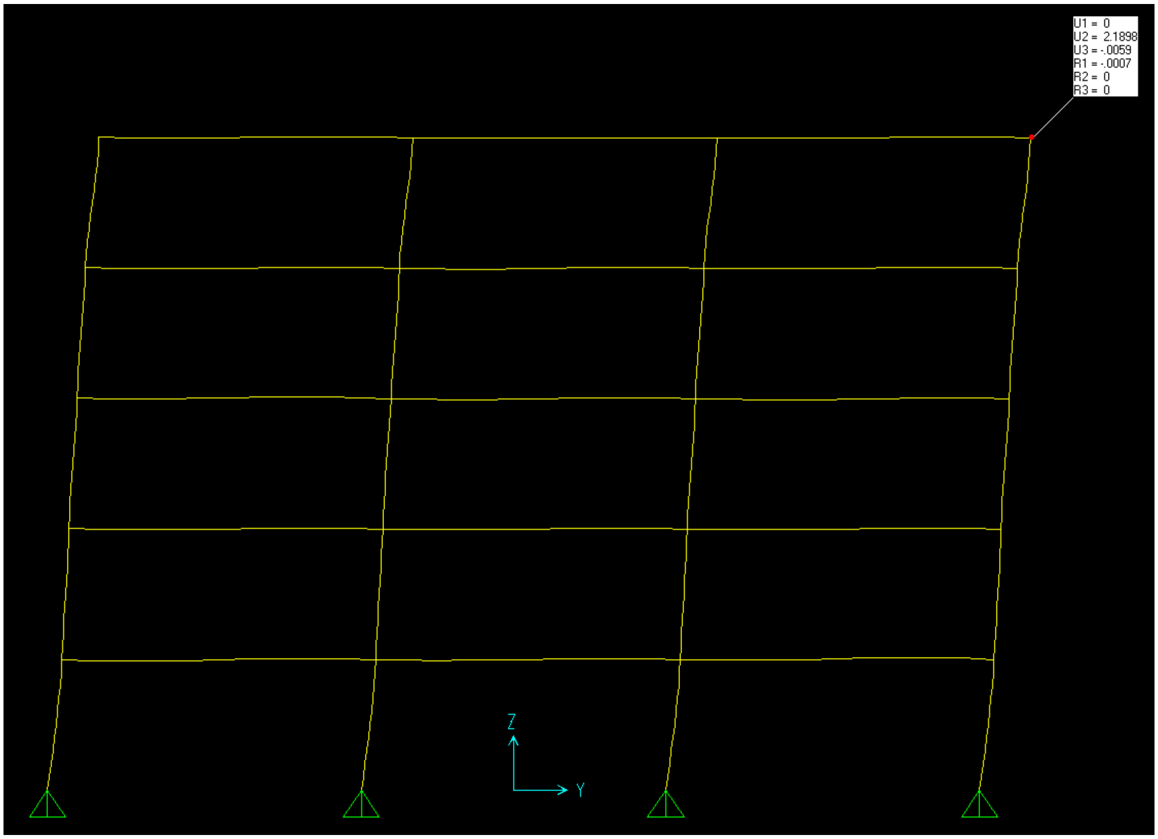


The new results after removing two moment connections:

Deflection Calculation H/400: ((69')*(12in/ft))/400 = 2.07in
East Section Frame #12: Story Drift = 1.85in < 2.07in ALLOW
West Section Frame #2: Story Drift = 1.76in < 2.07in ALLOW
Center Section Frame #D: Story Drift = 2.23in < 2.07in FAIL

Upon removal of two moment connections, the story drift increased in all three sections with the Center section going over the allotted H/400 level. The

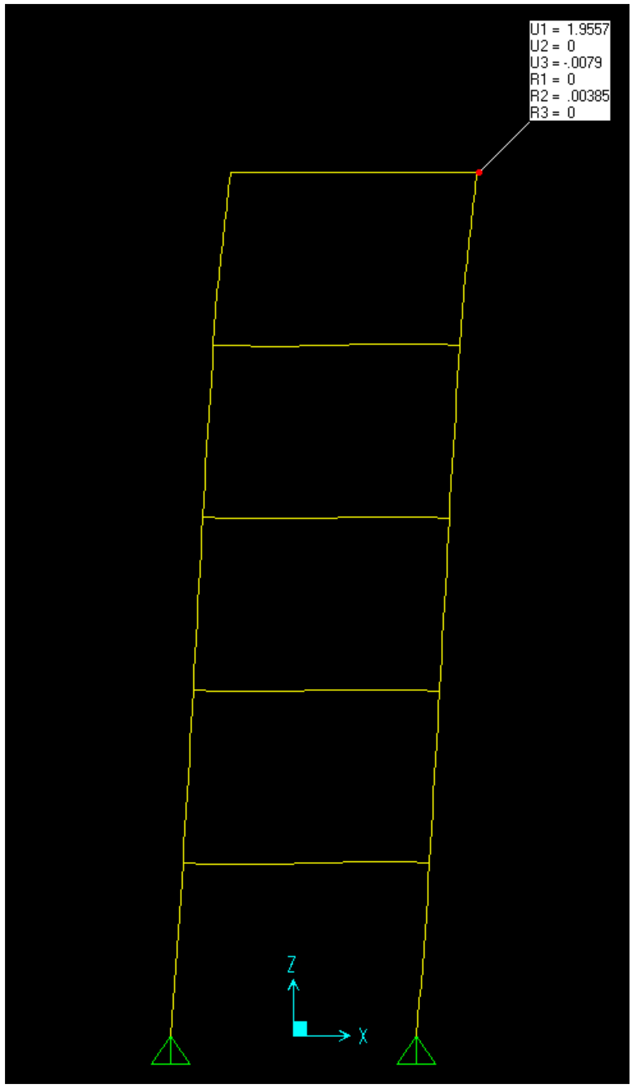
next test was to remove all the roof connections in the East and West Section to see if the deflection checks would still pass. The West Section is shown below:



The new results after removing the roof moment connections for East & West:

Deflection Calculation H/400: ((69')*(12in/ft))/400 = 2.07in
East Section Frame #12: Story Drift = 2.28in < 2.07in FAIL
West Section Frame #2: Story Drift = 2.19in < 2.07in FAIL

Removing all the connections was too much as the deflection of the side sections didn't meet the H/400 requirement. One last trial was conducted where the moment connections were removed from every other frame on both the East and West Section with the Center section left as designed. The Center section is shown below as well as all three results.



The final results after removing every other frame:

Deflection Calculation H/400: ((69')*(12in/ft))/400 = 2.07in
East Section Frame #12: Story Drift = 2.03in < 2.07in ALLOW
West Section Frame #2: Story Drift = 1.94in < 2.07in ALLOW
Center Section Frame #D: Story Drift = 1.96in < 2.07in ALLOW

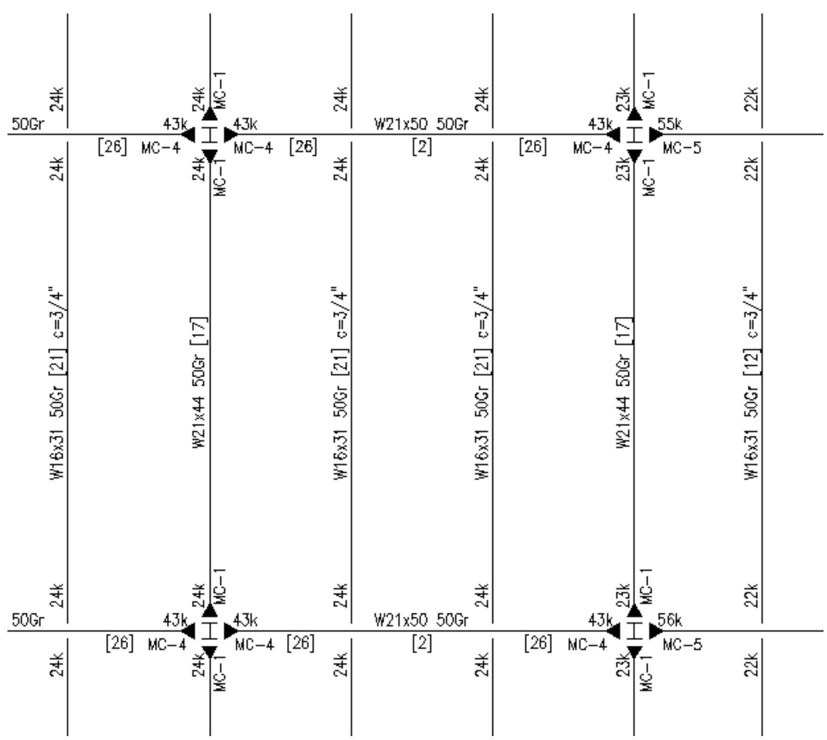
The new system passes the deflection check showing that it is possible to remove some of the smaller moment connections and still have the system work. The total savings of removing 24 total moment connections is valued at approximately $4,000 using cost data from the Milton Steel Corporation.

Floor System - Background Information

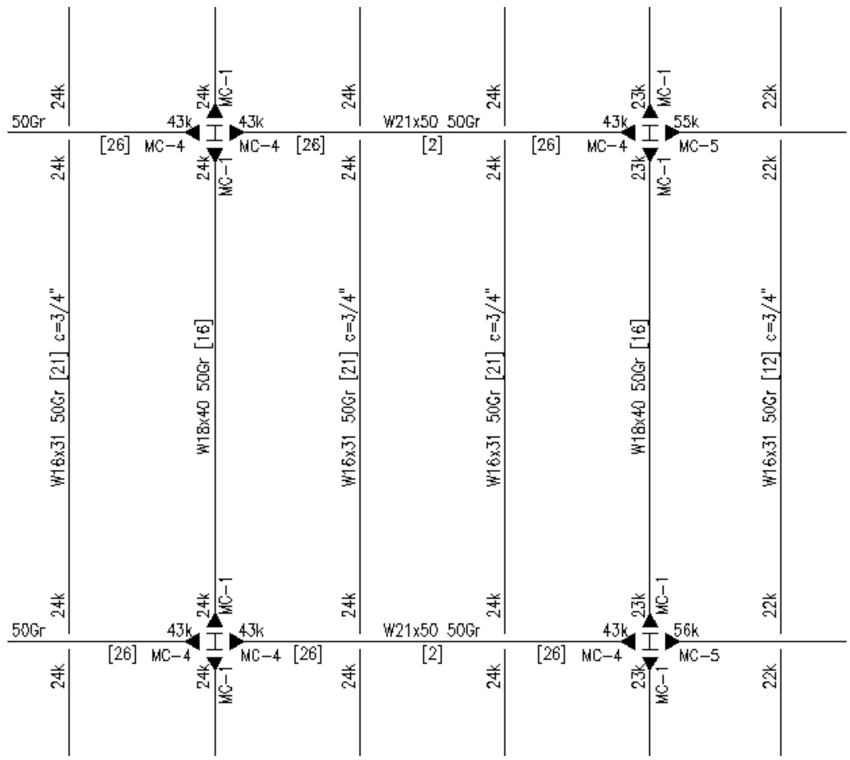
The Hershey Academic Support Center uses a special type of floor design known as “Type 2 with Wind”. The basic principal for Type 2 with Wind design is to take the negative moment value from the wind force and use this when designing the lateral force member. Members located within the moment frames have a laterally based design while interior beams use the standard gravity load design to choose member sizes. This method ensures that the lateral force will be adequately resisted within the structure, but can often result in varied member types throughout the building. Another factor attributed from Type 2 with Wind design is that shear studs are used to help adjust the balance between the positive moment in the center of a normal gravity load distribution and the negative moment located at the ends. This creates an issue where economy must be considered to pick a member that has an optimum girder size to shear stud ratio. Since the lateral system now uses the partially restrained connections, new moments needed to be calculated and the floor members checked.

Floor System - Calculations

To assist with the design of new floor members, a RAM Steel Model was created for each floor to see if the new moments would affect the member design. An example floor section from the East Wing is shown under the old moment system:



The new loading data from SAP2000 was entered into RAM and the new floor plans were compared to the old. Most members stayed the same as before but a few changes were noted as shown below:

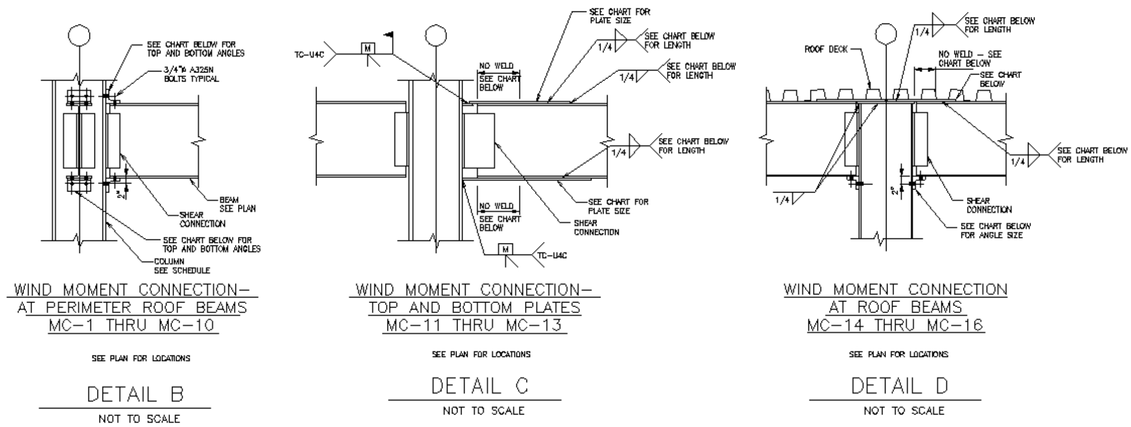


The North-South spanning members changed from a W21x44 with 17 shear studs to a W18x40 with 16 shear studs on the first and second floors and changed from a W18x40 with 16 shear studs to a W18x35 with 12 shear studs on the third and forth floors. The top floor experienced no change in member sizes which is most likely due to the removal of some of the moment connections. Both East and West sections experienced this change with a total of 40 W21x44s changing into W18x40s and 40 W18x40s changing into 18x35s. This totaled up to 6.2 tons of steel between all the members and RMS estimates steel prices at about $2,000 per ton of steel, so the total savings was approximately $12,320.

Construction Management Breadth

Introduction

One important aspect in any building is cost and the Hershey Academic Support Center is no different. With a total of 617 moment connections containing 2,329 bolts and 318.6 linear feet of weld, the lateral system poses a significant portion to the overall building cost. Each of the 16 different connections has a unique cost associated with the material and labor. Shown below are the three main connection types: top and bottom angles, top and bottom plates, and top plate and bottom angle.



Calculations

Chris Holcombe of Milton Steel was consulted as to what price their company would charge for three example connections. The cost includes both the fabrication and the labor to install the connection. RS Means was also consulted to determine each angle's price, but the values were significantly lower than the fabricators so they were not used. The three connections and their associated price are listed below:

MC-5
Connection Type #1: Top and Bottom Angles Common Example:
L6 X 4 X 3/4 X 0'-8"
4 bolts to the beam
2 bolts to the column
Priced at $160 per connection

MC-12
Connection Type #2: Top and Bottom Plates Common Example:
Top Plate - 7 X 1/2 X 1'-8"
Bottom Plate - 12 X 3/8 X 2'-10"
Weld across end of plate - Top: 8", Bottom: -
Weld along plate per side - Top: 6", Bottom: 14"
Weld to column - TC-U4C for both
No weld length - Top: 1'-0", Bottom: 1'6"
Priced at $328 per connection

MC-14
Connection Type #3: Top Plate and Bottom Angle Common Example:
Top Plate - 4 X 3/8 X 1'-6"
Weld along plate per side - 5"
Weld to column - 5"
No weld length - 6"
Bottom Angle - L3 1/2 X 3 1/2 X 1/2 X 0'-6 1/2"
2 bolts to the beam
Priced at $145 per connection

From these prices, the other moment connections were priced. The angled connections were priced based on the size of the angle used and the number of bolts in contained relative to MC-5. Labor costs were decreased slightly for connections having less than 6 bolts. From Means, the breakdown of expenses by percentage for an angle connection is approximately 25% Material Cost and 75% Labor Cost. Extrapolation values are shown below:

MC-1 = $165
MC-2 = $165
MC-3 = $103
MC-4 = $160
MC-5 = **$160**
MC-6 = $107
MC-7 = $160
MC-8 = $99
MC-9 = $101
MC-10 = $99

The plated connections were priced based on the size of the plate used and the length of weld contained relative to MC-12. Labor costs were altered on a percentage basis from the originally priced connection. From Means, the

breakdown of expenses by percentage for an plate connection is approximately 25% Material Cost and 75% Labor Cost. Extrapolation values are shown below:

MC-11 = $268
MC-12 = **$328**
MC-13 = $352

The last connection is a combination of the two methods and is priced in the same manner:

MC-14 = **$145**
MC-15 = $157
MC-16 = $145

The above data clearly shows that welded connections are significantly more expensive than bolted ones, mostly due to the labor involved. RS Means also gives data that welded connections can take up to twice as long to complete when compared to bolted connections. The output comparison was 105 high strength bolts per day versus only 50 linear feet of weld per day. The average angled connection has 4-6 bolts total and the average welded connection has 3-4 linear feet of weld necessary.

Architecture Breadth

Introduction

One feature present in all buildings as defined by code is a fire prevention system. The Hershey Academic Support Center in accordance with the Pennsylvania Department of Labor and Industry's Fire and Panic Code designed the building to have a standard 2 hour fire rating throughout the building. Aside from the standard pull box switches, a hydraulic sprinkler system fire suppressant is in place throughout the building. Smoke detectors are placed in all major areas including hallways, elevator shafts, and ducts. The pipes that relegate water throughout the building are located in the stairwells that have a 3-hour fire rating. This type of fire prevention is standard practice, but there is one interesting detail. For a composite lightweight concrete slab, a 2 hour fire rating can be obtained by having a 3.5" thick slab. Instead of doing this, the Hershey Academic Support Center has a 2.5" thick slab with cementitious spray on fireproofing on all of the columns, beams, and decking. For this study, the cementitious fireproofing will be removed in all locations, but the stairwells and the concrete slab will be increased to 3.5" to meet the 2 hour fire rating requirement. The extra weight of concrete will be checked to make sure the system still works.

Calculations

Pricing calculations are shown below:

Cost Calcs

Concrete

Each floor is 30,000 ft^2 of area

Thickness needs 1" for 2-hour fire rating

RS Means lists the price of Concrete at $91.00/$yd^3$

Total price increase = $2500\,ft^3 \left(\frac{1yd^3}{27ft^3}\right)(\$91.00/yd^3)(5\text{ floors}) = \$42,130$

Assume 10% material waste → $42,130 (1.1) = $46,343

Assume 10% extra labor + equipment costs → $46,343 (1.1) = $50,977

Cementitious Fireproofing 1" thick for 2 hour fire-rating

30,000 ft^2 of deck area per floor → 150,000 ft^2

TYPICAL

W14x120 225 total columns @ 101 ft^2 per floor per column → 113,625 ft^2

W21x50 620 total beams @ 156 ft^2 per floor per beam → 483,600 ft^2

All cost data taken from RS Means

Materials (Add 10% overspray cost)

Deck: $(150,000ft^2)(0.08\bar{3}ft)(\$0.64/ft^3) = \$8,000(1.1) = \$8,800$

Columns: $(113,625ft^2)(0.08\bar{3}ft)(\$0.48/ft^3) = \$4,545(1.1) = \$5,000$

Beams: $(483,600ft^2)(0.08\bar{3}ft)(\$0.43/ft^3) = \$17,329(1.1) = \$19,062$

Labor + Equipment

Deck: $(150,000ft^2)(0.08\bar{3}ft)(\$0.65/ft^3) = \$8,125$

Columns: $(113,625ft^2)(0.08\bar{3}ft)(\$0.75/ft^3) = \$7,102$

Beams: $(483,600ft^2)(0.08\bar{3}ft)(\$0.55/ft^3) = \$22,165$

Individual Totals

Deck = $16,925   Columns = $12,102   Beams = $41,227

Grand Total = $70,254

The new system was found to be cheaper and saved $70,254 over the spray on fireproofing system. Eliminating the spray on fireproofing also gives more options architecturally since beams and columns do not necessarily have to be covered since cementitious fireproofing is not aesthetically pleasing. In doing a weight comparison between the systems, cementitious fireproofing was found to be half as heavy as concrete with ½" of lightweight concrete equal to 1" of fireproofing. The spray on fire-proofing in the Hershey Academic Support Center is ½" so ¾" of concrete weight must be accounted for in the system. Strength calculations for the columns were performed and are shown below. The first column was chosen at connection B between the East section and the Center section. The second column was selected at connection D between the West section and the Center section. To compare the values, the equation $Pu/b + Mu/m < 1$ was used. Table 6-2 from the Steel Manual was used to obtain the b and m values for each column.

Section B:
W14x193, $Pu/b + Mu/m = (196.37)/(0.47) + (387.34)/(0.668) = 0.998 < 1$
ALLOW

Section D:
W14x175, $Pu/b + Mu/m = (203.08)/0.516) + (403.23)/(0.741) = 0.938 < 1$
ALLOW

Both columns pass the strength check, so the new design is feasible.

Conclusions

Partially restrained moment connections can be a useful way to resist lateral loads in a building. While slightly more expensive than braced framing, moment connections allow you more space architecturally and can be more easily used to resist problem spots of lateral loads.

The Hershey Academic Support Center utilizes a well designed lateral support system. When partial fixity calculations were applied, only a small portion of the building changed. Some of the top floor moment connections were able to be removed, but only at a cost savings of about $4000. Also, the new moment values changed some of the floor members that were designed using "Type 2 with Wind" principals. The total savings of the steel totaled to $12,320 bring the total money saved at $16,320. For a project nearly $17,000,000 in total budgeted money, this savings is very minimal. While the new system did save money overall, the amount of time required to find the specific fixity of each connection and apply it to the structure is not worth the money saved. If cost was an issue in the building, it would be more economical to use braced frames to resist the lateral loads and work around them architecturally.

For Construction Management, it was found that welded connections should be avoided when compared to bolted connections whenever possible. Bolted connections cost about half as much as welded connections and they were quicker to place as well. The use of plates over angles is usually due to the need for some extra strength against gravity loads on the building such as the Mechanical Penthouse on this building. If bolted plates would be used instead of welded ones, plate fracture must always be checked against the yield value of the plate to ensure that it can take moment. Another solution if possible can be to do the welding in-shop as that saves a considerable amount of money over welding in the field.

Architecturally speaking, it seems that adding an extra inch of concrete saves a considerable $70,000 over cementitious fireproofing. The extra weight of concrete did not prove to be an issue with the columns or the structure when added. One reason why spray on fireproofing could have been selected over a

thicker slab is the time needed to complete each task. While the duration of both is not very far off, each of these tasks is completed during a separate step of construction, so it is possible that spray on fireproofing would be more time effective. Another reason spray on fireproofing could have been chosen is because the wet weight of the extra concrete could have caused problems in the structure depending on how the concrete was added. Both systems effectively meet the first protection code and both have their advantages.

All in all, the newly designed system didn't turn out to be quite as advantageous as planned, but much was learned from the overall design. My final recommendation for the building is to keep the original design and make better use of the time it would take to fully design each connection in the building.

Acknowledgements

First and foremost I would like to extend a big thank you to everyone who has helped me out over the last five years and helped me to achieve my goal of becoming a Structural Engineer.

Special Thanks goes to:

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~The AE Faculty & Staff for providing me with a top-notch education and a great college learning experience

~My dearest friends who I'll not soon forget…you guys are the reason I made it this far and I have always been able to count on you for anything

~Last but not least, My Family for always believing in me and for giving me every opportunity possible to succeed

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AE 439 & AE 534 Class Materials provided by Dr. Linda Hanagan

Blodgett, Lincoln Arc Welding Foundation. *Beam-to-Column Connections*

John Christopher & Reidar Bjorhovde. *Semi-Rigid Frame Design Methods for Practicing Engineers*

Construction Management Design:

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Architectural Design:

Pennsylvania Department of Labor and Industry's Fire and Panic Code

Appendix A – Wind & Seismic
Wind Loading Calculations

	Hershey 5	Assumptions and Information
(K_{zt}) Topographic Factor	1	Code 6.5.7.2, Figure 6-4, $K_{zt} = (1+(k_1)*(k_2)*(k_3))^2$
(K_d) Directional Wind Factor	0.85	Code 6.5.5.4, Table 6-4
(V) Basic Wind Speed	70	Given by Structural Notes
(I) Importance Factor	1	Code 6.5.5, Table 6-1
(C_t) Peroid Parameter	0.02	Code 9.5.3.2, Table 9.5.5.3.2
(h) Building Height in Feet	69	Height to the 5th Story
(f) Frequency in Hz	2.08849378	Code 9.5.3.2, Table 9.5.5.3.2, f = 1/((C_t)*((h)^0.75))
Exposure Category C	α	Given by Structural Notes
(α)	9.5	Code 6.5, Table 6-2
(zg (ft))	900	Code 6.5, Table 6-2
(^a)	2/19	Code 6.5, Table 6-2
(^b)	1	Code 6.5, Table 6-2
(α bar)	1/6	Code 6.5, Table 6-2
(b bar)	0.65	Code 6.5, Table 6-2
(c) alsdrj	0.2	Code 6.5, Table 6-2
(L (ft))	500	Code 6.5, Table 6-2
(Є bar)	1/5	Code 6.5, Table 6-2
(z min)	15	Code 6.5, Table 6-2
Rigid Structures N-S		
*Exposure C, Table 6-2		
(g_q) Gust Coefficient	3.4	Code 6.5.8.2, Equation 6-8
(g_v) Gust Coefficient	3.4	Code 6.5.8.2, Equation 6-8
(z bar) Wind Coefficient	41.4	Code 6.5.8.2, Table 6-2, z bar = 0.6(h)
(L_z) Turbulence Scale Factor	523.199457	L_z = L*((z bar)/33)^(Є bar)
(I_z) Turbulence Intensity	0.19258196	Code 6.5.8.1, Equation 6-5, Iz = (c)*(33/(z bar))^(1/6)
(B) Perpendicular to Wind	268.33	Code 6.3, Given in Plan
(L) Parallel to Wind	102.67	Code 6.3, Given in Plan
(Q) Background Response	0.82260391	Code 6.5.8.1, Equation 6-6, Q = SQRT(1/(1+(0.63*((B+h)/L_z)^0.63)))
(G) Gust Factor	0.83856209	Code 6.5.8.2, Equation 6-8, G = 0.925*((1+(1.7*(g_q)*(I_z)*(Q)))/(1+(1.7*(g_v)*(I_z))))
Rigid Structures E-W		
(B)	102.67	
(Q)	0.8729702	
(G)	0.86310353	
Flexible Structures N-S		
*Exposure B, Table 6-2		
(g_q)	3.4	

(g_v)	3.4	
(z bar)	41.4	
(L_z)	523.199457	
(I_z)	0.19258196	
(B) Perpendicular to Wind	268.33	
(L) Parallel to Wind	102.67	
(Q)	0.82260391	
(β) Damping Ratio	0.05	Code 6.3, Section 9
(n_1) Natural Frequency	2.08849378	Code 6.5.8.2
(V_z) Mean Hourly Wind Speed	69.3038272	Code 6.5.8.2, Equation 6-14, V_z = ((b bar)*(B40/33)^(α bar)*(V)*(88/60)
(η_h) R_l Coefficient	9.5649541	Code 6.5.8.2, Equation 6-13, η_h = 4.6*(n_1)*(h)/(V_z)
(η_B) R_l Coefficient	37.1965816	Code 6.5.8.2, Equation 6-13, η_B = 4.6*(n_1)*(B)/(V_z)
(η_L) R_l Coefficient	47.6475145	Code 6.5.8.2, Equation 6-13, η_L = 4.6*(n_1)*(L)/(V_z)
(R_h) R_l Coefficient	0.09908316	Code 6.5.8.2, Equation 6-13, R_h = (1/η_h)-(1/(2*(η_h^2)))*(1-(2.718281828^(-2*η_h)))
(R_B) R_l Coefficient	0.02652281	Code 6.5.8.2, Equation 6-13, R_h = (1/η_B)-(1/(2*(η_B^2)))*(1-(2.718281828^(-2*η_B)))
(R_L) R_l Coefficient	0.02076722	Code 6.5.8.2, Equation 6-13, R_h = (1/η_L)-(1/(2*(η_L^2)))*(1-(2.718281828^(-2*η_L)))
(N_1) Reduced Frequency	15.7667889	Code 6.5.8.2, Equation 6-12, N_1 = (n_1*L_z)/V_z
(R_n) Resonance Coefficient	0.0241168	Code 6.5.8.2, Equation 6-11, R_n = (7.47*N_1)/((1+(10.3*N_1))^(5/3))
(R) Resonance Response Factor	0.02615683	Code 6.8.5.2, Equation 6-10, R = (1/β)*R_n*R_h*R_B*(0.53+(0.47*R_L)))
(g_R) Gust Coefficient	4.36152676	Equation 6-9, g_R = (SQRT((2*(LN(3600*n_1))))+(0.577/(SQRT((2*LN(3600*n_1))))))
(G_f) Gust Factor	0.8388954	Equation 6-8, G_f = 0.925*((1+(1.7*I_z*(SQRT(((g_q)^2)*((Q)^2)+((g_R)^2)*((R)^2)))))/(1+(1.7*g_v*I_z
Flexible Structures E-W		
*Exposure B, Table 6-2		
(B) Perpendicular to Wind	102.67	
(L) Parallel to Wind	268.33	
(Q)	0.8729702	
(η_B)	14.2323744	
(η_L)	124.527686	
(R_B)	0.06779395	
(R_L)	0.0079981	
(R)	0.04158559	
(G_f)	0.863897	
(Cp) Windward	0.8	Code 6.5.11.2, Figure 6-6
(Cp) Leeward N-S	-0.5	Code 6.5.11.2, Figure 6-6, L/B
(Cp) Leeward E-W	-0.3	Code 6.5.11.2, Figure 6-6, L/B
(q_z)*K_z Velocity Pressure	10.6624	Code 6.5.10, Equation 6-15, (qz)*Kz = 0.00256*K_{zt}*K_d*(V^2)*I
(q_h) Velocity Pressure at z	12.4323584	Code 6.5.12.2, Table 6-3, q_h = ((h-C131)/(C132-C131))*(A132-A131)*((qz)*Kz)+(((qz)*Kz)*A
(P_{wz})*q_z N-S	0.69048283	(Pwz)*qz = (Cp Windward)*G
(P_{wz})*q_z E-W	0.67111632	(Pwz)*qz = (Cp Windward)*G_f
Leeward Wind Pressure		

(P_{lh}) N-S	-5.2126522	$P_{lh} = q_h$*(Cp Leeward N-S)*G
(P_{lh}) E-W	- 3.21912374	$P_{lh} = q_h$*(Cp Leeward E-W)*G_f
Windward Pressure N-S		
(P_{wz}) 0-15	6.07993738	
(P_{wz}) 20	6.43758076	
(P_{wz}) 25	6.72369546	
(P_{wz}) 30	7.00981016	
(P_{wz}) 40	7.43898221	
(P_{wz}) 50	7.79662559	
(P_{wz}) 60	8.08274029	
(P_{wz}) 70	8.36885499	
Windward Pressure E-W		
(P_{wz}) 0-15	6.25787348	
(P_{wz}) 20	6.62598369	
(P_{wz}) 25	6.92047185	
(P_{wz}) 30	7.21496002	
(P_{wz}) 40	7.65669226	
(P_{wz}) 50	8.02480247	
(P_{wz}) 60	8.31929063	
(P_{wz}) 70	8.6137788	
K_z	q_z	z (ft)
0.85	9.06304	0-15
0.9	9.59616	20
0.94	10.022656	25
0.98	10.449152	30
1.04	11.088896	40
1.09	11.622016	50
1.13	12.048512	60
1.17	12.475008	70
Total Pressure N-S		$P = P_{wz} + P_{lh}$
(P_{tot}) 0-15	11.2925896	
(P_{tot}) 20	11.650233	
(P_{tot}) 25	11.9363477	
(P_{tot}) 30	12.2224624	
(P_{tot}) 40	12.6516344	
(P_{tot}) 50	13.0092778	
(P_{tot}) 60	13.2953925	
(P_{tot}) 70	13.5815072	

Total Pressure E-W

$P = P_{wz} + P_{lh}$

(P_{tot}) 0-15	9.47699723
(P_{tot}) 20	9.84510743
(P_{tot}) 25	10.1395956
(P_{tot}) 30	10.4340838
(P_{tot}) 40	10.875816
(P_{tot}) 50	11.2439262
(P_{tot}) 60	11.5384144
(P_{tot}) 70	11.8329025

Leeward Shear N-S

(B) Perpendicular to Wind	268.33
Shear @ Ground	-9790.97675
Shear @ Floors	-19581.9535
Shear @ Roof	-9091.62127

Leeward Shear E-W

(B) Perpendicular to Wind	102.67
Shear @ Ground	-2313.55204
Shear @ Floors	-4627.10408
Shear @ Roof	-2148.29833

Winward Shear N-S

(B) Perpendicular to Wind	268.33
Shear @ 0	11420.0072
Shear @ 1	23492.5862
Shear @ 2	26601.8991
Shear @ 3	28809.1274
Shear @ 4	30517.3301
Shear @ 5	15719.304

Windward Shear E-W

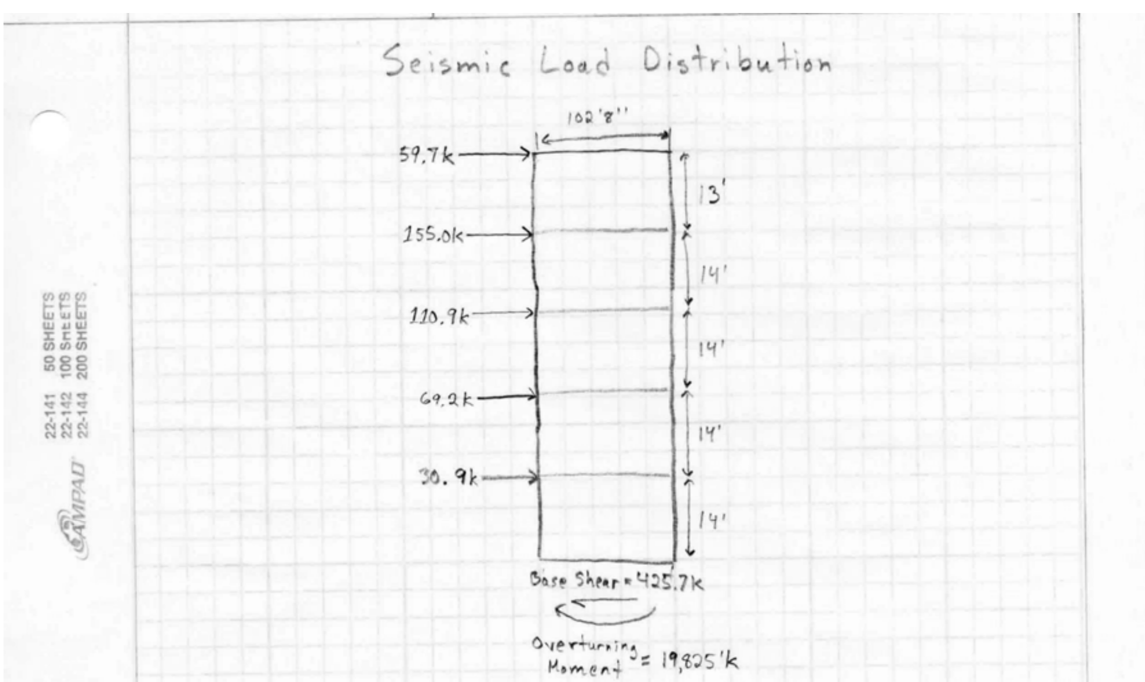
(B)Perpendicular to Wind	102.67
Shear @ 0	4497.47109
Shear @ 1	9251.94054
Shear @ 2	10476.4621
Shear @ 3	11345.7212
Shear @ 4	12018.4522
Shear @ 5	6190.63668

Seismic Loads

Hershey 5 Vertical Distribution N-S					
Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x
1	2195.5809	14	47409.8578	0.072521	30.8688
2	2195.5809	28	106249.519	0.162527	69.1796
3	2195.5809	42	170346.004	0.260573	110.9131
4	2195.5809	56	238114.2	0.364236	155.0374
5	662.50927	69	91616.8137	0.140143	59.65217
Value Sum			653736.394	1	425.6511
Base Shear					425.6511
Overturning Moment					19825.64

Hershey 5 Vertical Distribution E-W					
Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x
1	2195.5809	14	47409.8578	0.072521	30.8688
2	2195.5809	28	106249.519	0.162527	69.1796
3	2195.5809	42	170346.004	0.260573	110.9131
4	2195.5809	56	238114.2	0.364236	155.0374
5	662.50927	69	91616.8137	0.140143	59.65217
Value Sum			653736.394	1	425.6511
Base Shear					425.6511
Overturning Moment					19825.64

The charts shown above summarize the results found from my seismic calculation analysis. Shown below is the seismic loading for a typical building as depicted by story forces. Specific calculations of seismic forces are located in the Appendix.



Appendix B – Existing Conditions

AE Senior Thesis | Spot Check | Technical Assignment 1

First Solve the loading for a typical Office Floor:

Dead Load = 70 psf (See Loading for specifics)

Live Load = 100 psf (Main Floor)

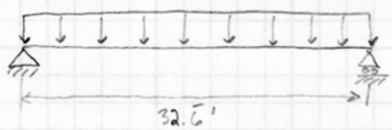
Use Live Load Reduction → $L = L_o(0.25 + \frac{15}{\sqrt{A_I}})$ where

$A_{TOT} = (9.\bar{3}')(32.\bar{6}') = 304.\bar{8}\ ft^2$, $A_I = 2A_T = 2(304.\bar{8}) = 609.\bar{7}\ ft^2$

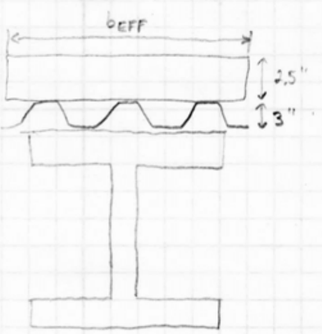
$L = (100)(0.25 + \frac{15}{\sqrt{609.\bar{7}}}) = 85.74\ psf$

Using Load Factors: $1.2DL + 1.6LL = 1.2(70) + 1.6(85.74) = 221.84\ psf$

$P_u = 221.84\ psf$, $\omega_u = (9.\bar{3}')(221.84) = 2.06\ klf$



Typical Beam Calculation



Lightweight (115 pcf)

Given: $f'_c = 4 ksi$ & $f_y = 50 ksi$

From Above, $\omega_u = 2.06\ klf$

$M_u = \frac{\omega_u l^2}{8} = \frac{(2.06)(32.\bar{6}')^2}{8} = 274.78\ 'k$

Assume $a = 1"$, $b_{EFF} = \begin{cases} l_n = 112" \\ \frac{(32.\bar{6})}{4}(12) = \boxed{98"} \end{cases}$

$Y_2 = 5.5" - \frac{a}{2} = 5"$

Use LRFD Table 5-14, Try W18×40 where $\phi_b M_p = 294\ 'k$

PNA @ 7 for $Y_2 = 5"$ → 400 so $\phi_b M_p = 400\ 'k$ & $\Sigma Q_n = 148\ k$

$\Sigma Q_n = 0.85 f'_c b a \rightarrow a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{148}{0.85(4)(98)} = 0.444$

$Y_2 = 5.5'' - \frac{0.444}{2} = 5.28'' \rightarrow \phi_b M_N = 403'k$ by interpolation

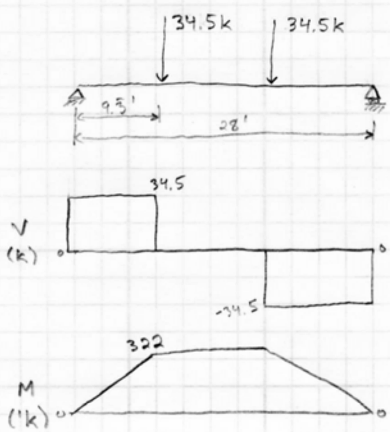
From the Structural notes, Minimum stud capacity is 9.7k

$\frac{\Sigma Q_n}{\text{Stud capacity}} = \frac{148}{9.7} = 15.26 \rightarrow$ Use 32 shear studs

Overall Design W18×40 with 32 shear studs

The beam specified in the drawings is W16×31. I believe my beam is larger because an older Code was used for factoring and there is a perscribed camber of $c = \frac{3}{4}''$ for the given design.

Typical Girder Calculation



$P = \frac{w_u l}{2} = \frac{(2.06)(32.\bar{6})}{2} = 33.6k$

$M_u = (33.6)(9.\bar{3}) = 313.6'k$

Assume $a = 1''$, $b_{EFF} = \begin{cases} l_n = 112'' \\ \frac{(28)}{4}(12) = \boxed{84''} \end{cases}$

$Y_2 = 5.5'' - \frac{a}{2} = 5''$, LRFD Table 5-14

Try W21×44 where $\phi_b M_p = 358'k$

$A_s = 13.0\ in^2$, $\Sigma Q_n = A_s f_y = (50)(13) = 650k$

Find $a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{650}{0.85(4)(84)} = 2.28$, $Y_2 = 5.5'' - \frac{2.28}{2} = 4.36''$

For PNA @ 7 + $Y_2 = 4.36''$ → $\phi_b M_p = 481'k$ by interpolation

For shear studs : $\Sigma Q_n = 163k$ & capacity is given as 33k

$\frac{\Sigma Q_n}{\text{stud capacity}} = \frac{163}{33} = 4.9$ → Use 10 shear studs

Overall Design W21x44 with 10 studs

The girder specified in the drawings is given as W21x50. The design I proposed is close in size and the slight variance can be attributed from the beam load being slightly different.

Typical Column Design

Dead Load = $70psf + \overset{\text{self weight}}{5psf} = 75psf$ (Office Design)

Live Load = 100psf (Main Floor)

Use Live Load Reduction → $L = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right)$ where

Column Tributary Area = $(28')(32.\bar{6}') = 914.\bar{6}\ ft^2$

$A_T = (4\ \text{floors})(914.\bar{6}) = 3658.\bar{6}\ ft^2$, $A_I = 4A_T = 4(3658.\bar{6}) = 14,634.\bar{6}\ ft^2$

Reduction Factor $= \left(0.25 + \frac{15}{\sqrt{14,634.\bar{6}}}\right) = 0.374 \ngtr 0.4$ for multiple story buildings, $L = (100)(0.4) = 40psf$, Factor the loading

$1.2DL + 1.6LL = 1.2(75) + 1.6(40) = 154psf$, $P_u = 154psf$,

$P_{Floor} = (154)(3658.\bar{6}) = 563.43K$, $\omega_u = 2.06 + \underset{\text{self weight}}{(0.05)}(1.2) = 2.12\ klf$

$M_u = \frac{\omega l^2}{12} = \frac{(2.12)(28)^2}{12} = 138.51'k$, Calculate for the wall

Appendix C – Lateral Calculations

Total Stiffness per Floor

Moment Frames	Floor 5	Floor 4	Floor 3	Floor 2	Floor 1	Total Stiffness
East N-S #7	7.199424046	8.849557522	11.14827202	15.03759	22.98851	65.22335332
East N-S #8	8.183306056	10.55966209	13.96648045	20.04008	32.05128	84.8008108
East N-S #9	6.752194463	9.00090009	11.9760479	16.33987	25.5102	69.57921582
East N-S #10	6.285355123	8.680555556	11.31221719	15.74803	24.39024	66.41640327
East N-S #11	6.422607579	8.960573477	11.69590643	16.33987	25.5102	68.92916085
East N-S #12	5.93824228	7.930214116	10.18329939	14.12429	21.05263	59.22868115
East E-W #A	11.09877913	16.55629139	21.59827214	30.39514	51.54639	131.1948712
East E-W #B	14.81481481	22.83105023	29.58579882	41.49378	69.44444	178.1698842
East E-W #D	18.24817518	30.03003003	39.37007874	54.64481	89.28571	231.578807
East E-W #F	8.34028357	12.61034048	16.61129568	23.20186	39.0625	99.82627588
West N-S #2	5.93824228	7.936507937	10.18329939	14.12429	21.05263	59.23497497
West N-S #3	6.422607579	8.960573477	11.69590643	16.33987	25.5102	68.92916085
West N-S #4	6.285355123	8.680555556	11.31221719	15.74803	24.39024	66.41640327
West N-S #5	6.711409396	9.033423668	11.77856302	16.44737	25.70694	69.67770537
West N-S #6	11.72332943	15.12859304	18.97533207	25.64103	38.02281	109.4910939
West E-W #AA	11.24859393	15.38461538	20.40816327	30.03003	51.81347	128.8848741
West E-W #BB	16.97792869	23.58490566	30.48780488	42.37288	70.92199	184.3455064
West E-W #DD	16.36661211	22.88329519	29.3255132	40.48583	67.56757	176.628818
West E-W #FF	10.03009027	13.24503311	1.689189189	23.36449	39.84064	88.169436
Center E-W #A	37.03703704	0	0	0	0	37.03703704
Center E-W #B	6.426735219	9.813542689	12.93661061	18.05054	30.30303	77.53046033
Center E-W #D	3.579098067	4.995004995	6.618133686	9.451796	16.89189	41.53592448
Center E-W #E	3.785011355	5.224660397	6.108735492	7.385524	11.7096	34.21353349
Center E-W #F	23.4741784	0	0	0	0	23.4741784
Total Stiffness Per Floor	259.2894111	280.8798861	348.9671372	506.807	824.5731	2220.51657

Direct Shear

Direct Shear	Stiffness	Relative Stiffness	Max Story Shear	Direct Shear
East N-S #7	65.22335	0.029373054	425.65	12.5026405
East N-S #8	84.80081	0.038189677	425.65	16.2554361
East N-S #9	69.57922	0.031334698	425.65	13.3376141
East N-S #10	66.4164	0.029910339	425.65	12.7313358
East N-S #11	68.92916	0.031041948	425.65	13.2130053
East N-S #12	59.22868	0.026673379	425.65	11.353524
East E-W #A	131.1949	0.059083041	425.65	25.1486963
East E-W #B	178.1699	0.080238034	425.65	34.1533192
East E-W #D	231.5788	0.104290511	425.65	44.3912559
East E-W #F	99.82628	0.04495633	425.65	19.1356619
West N-S #2	59.23497	0.026676214	425.65	11.3547304
West N-S #3	68.92916	0.031041948	425.65	13.2130053
West N-S #4	66.4164	0.029910339	425.65	12.7313358
West N-S #5	69.67771	0.031379052	425.65	13.3564936
West N-S #6	109.4911	0.049308839	425.65	20.9883073
West E-W #AA	128.8849	0.058042744	425.65	24.7058938
West E-W #BB	184.3455	0.083019199	425.65	35.3371219
West E-W #DD	176.6288	0.079544022	425.65	33.8579128
West E-W #FF	88.16944	0.039706723	425.65	16.9011666
Center E-W #A	37.03704	0.016679469	425.65	7.09961593
Center E-W #B	77.53046	0.034915506	425.65	14.8617853
Center E-W #D	41.53592	0.018705523	425.65	7.962006
Center E-W #E	34.21353	0.015407916	425.65	6.55837958
Center E-W #F	23.47418	0.010571494	425.65	4.49975658

Torsional Shear

Torsion	k	x (ft)	kx^2	$kx/\Sigma kx^2$
East N-S #7	65.22335332	20.41	27169.97	9.36421E-05
East N-S #8	84.8008108	28.31	67964.13	0.000168875
East N-S #9	69.57921582	54.24	204700.5	0.000265475
East N-S #10	66.41640327	81.55	441695.8	0.000381
East N-S #11	68.92916085	109.22	822256.5	0.000529578
East N-S #12	59.22868115	142.65	1205246	0.000594332
East E-W #A	131.1948712	92.55	1123750	0.000854119
East E-W #B	178.1698842	73.11	952330.9	0.000916297
East E-W #D	231.578807	66.25	1016414	0.00107922
East E-W #F	99.82627588	87.67	767267.6	0.000615632
West N-S #2	59.23497497	142.65	1205374	0.000594395
West N-S #3	68.92916085	109.22	822256.5	0.000529578
West N-S #4	66.41640327	81.55	441695.8	0.000381
West N-S #5	69.67770537	54.24	204990.2	0.000265851
West N-S #6	109.4910939	28.31	87752.31	0.000218044
West E-W #AA	128.8848741	103.04	1368402	0.000934185
West E-W #BB	184.3455064	86	1363419	0.001115209
West E-W #DD	176.628818	80.24	1137217	0.00099696
West E-W #FF	88.169436	86.67	662301.4	0.000537542
Center E-W #A	37.03703704	63.41	148919.6	0.000165203
Center E-W #B	77.53046033	31.87	78747.45	0.000173812
Center E-W #D	41.53592448	14.21	8387.104	4.15186E-05
Center E-W #E	34.21353349	25.31	21917.06	6.09137E-05
Center E-W #F	23.4741784	39.02	35740.85	6.44322E-05

MC- 1		**MC- 2**	
General Properties		General Properties	
Elastic Modulus(E) ksi	29000	Elastic Modulus(E) ksi	29000
Yield Stress (Fv) ksi	36	Yield Stress (Fv) ksi	36
Bolt Properties	3/4"ø A325N	Bolt Properties	3/4"ø A325N
Bolt Diameter(d_b) in	0.750	Bolt Diameter(d_b) in	0.750
Nut Width(w) in	1.125	Nut Width(w) in	1.125
Connection Type	MC-1	Connection Type	MC-2
Top Angle	L6 X 4 X 7/8 X 0'-7"	Top Angle	L6 X 4 X 7/8 X 0'-6"
Leg Thickness(t) in	0.875	Leg Thickness(t) in	0.875
Leg Length(l) in	7.000	Leg Length(l) in	6.000
Beam Properties	W21X44	Beam Properties	W18X40
Beam Length (L) ft	28.00	Beam Length (L) ft	32.67
Beam Depth (d_t) in	20.70	Beam Depth (d_t) in	17.90
Moment of Inertia(I) in^4	843.00	Moment of Inertia(I) in^4	612.00
Connection Properties		Connection Properties	
Length Factor (α_i)	11.63	Length Factor (α_i)	10.16
Initial Connection Stiffness(R_{ki})	101,549	Initial Conn. Stiffness(R_{ki})	97,589

MC- 3		**MC- 4**	
General Properties		General Properties	
Elastic Modulus(E) ksi	29000	Elastic Modulus(E) ksi	29000
Yield Stress (Fv) ksi	36	Yield Stress (Fv) ksi	36
Bolt Properties	3/4"ø A325N	Bolt Properties	3/4"ø A325N
Bolt Diameter(d_b) in	0.750	Bolt Diameter(d_b) in	0.750
Nut Width(w) in	1.125	Nut Width(w) in	1.125
Connection Type	MC-3	Connection Type	MC-4
Top Angle	L3-1/2 X 3-1/2 X 5/8 X 0'-6 1/2"	Top Angle	L6 X 4 X 3/4 X 0'-7"
Leg Thickness(t) in	0.625	Leg Thickness(t) in	0.750
Leg Length(l) in	7.000	Leg Length(l) in	7.000
Beam Properties	W18X35	Beam Properties	W21X50
Beam Length (L) ft	28.00	Beam Length (L) ft	28.00
Beam Depth (d_t) in	17.70	Beam Depth (d_t) in	20.80
Moment of Inertia(I) in^4	510.00	Moment of Inertia(I) in^4	984.00
Connection Properties		Connection Properties	
Length Factor (α_i)	10.55	Length Factor (α_i)	15.67
Initial Connection Stiffness(R_{ki})	79,203	Initial Conn. Stiffness(R_{ki})	87,551

MC- 5		MC- 6	
General Properties		General Properties	
Elastic Modulus(E) ksi	29000	Elastic Modulus(E) ksi	29000
Yield Stress (Fv) ksi	36	Yield Stress (Fv) ksi	36
Bolt Properties	3/4"ø A325N	Bolt Properties	3/4"ø A325N
Bolt Diameter(db) in	0.750	Bolt Diameter(db) in	0.750
Nut Width(w) in	1.125	Nut Width(w) in	1.125
Connection Type	MC-5	Connection Type	MC-6
Top Angle	L6 X 4 X 3/4 X 0'-8"	Top Angle	L4 X 4 X 5/8 X 0'-10"
Leg Thickness(t) in	0.750	Leg Thickness(t) in	0.625
Leg Length(l) in	8.000	Leg Length(l) in	10.000
Beam Properties	W24X55	Beam Properties	W27X84
Beam Length (L) ft	33.67	Beam Length (L) ft	40.90
Beam Depth (dt) in	23.60	Beam Depth (dt) in	24.10
Moment of Inertia(I) in4	1350.00	Moment of Inertia(I) in4	2370.00
Connection Properties		Connection Properties	
Length Factor (αi)	18.77	Length Factor (αi)	35.91
Initial Connection Stiffness(Rki)	88,380	Initial Connection Stiffness(Rki)	79,417
MC- 7		MC- 8	
General Properties		General Properties	
Elastic Modulus(E) ksi	29000	Elastic Modulus(E) ksi	29000
Yield Stress (Fv) ksi	36	Yield Stress (Fv) ksi	36
Bolt Properties	3/4"ø A325N	Bolt Properties	3/4"ø A325N
Bolt Diameter(db) in	0.750	Bolt Diameter(db) in	0.750
Nut Width(w) in	1.125	Nut Width(w) in	1.125
Connection Type	MC-7	Connection Type	MC-8
Top Angle	L6 X 4 X 3/4 X 0'-9"	Top Angle	L3-1/2 X 3-1/2 X 1/2 X 0'-6 1/2"
Leg Thickness(t) in	0.750	Leg Thickness(t) in	0.500
Leg Length(l) in	9.000	Leg Length(l) in	6.500
Beam Properties	W24X76	Beam Properties	W21X50

Beam Length (L) ft	31.28	Beam Length (L) ft	28.14
Beam Depth (dt) in	23.90	Beam Depth (dt) in	20.80
Moment of Inertia(I) in4	2100.00	Moment of Inertia(I) in4	984.00
Connection Properties		Connection Properties	
Length Factor (αi)	27.60	Length Factor (αi)	20.00
Initial Connection Stiffness(Rki)	92,323	Initial Connection Stiffness(Rki)	68,596

General Properties			General Properties		
Elastic Modulus(E) ksi		29000	Elastic Modulus(E) ksi		29000
Yield Stress (Fv) ksi		36	Yield Stress (Fv) ksi		36
Bolt Properties	3/4"ø A325N		Bolt Properties	3/4"ø A325N	
Bolt Diameter(d_b) in		0.750	Bolt Diameter(d_b) in		0.750
Nut Width(w) in		1.125	Nut Width(w) in		1.125
Connection Type	MC-9		Connection Type	MC-10	
Top Angle	L3-1/2 X 3-1/2 X 9/16 X 0'-5"		Top Angle	L3-1/2 X 3-1/2 X 1/2 X 0'-10"	
Leg Thickness(t) in		0.563	Leg Thickness(t) in		0.500
Leg Length(l) in		5.000	Leg Length(l) in		10.000
Beam Properties	W14X22		Beam Properties	W33X118	
Beam Length (L) ft		12.76	Beam Length (L) ft		33.67
Beam Depth (d_t) in		13.70	Beam Depth (d_t) in		32.90
Moment of Inertia(I) in^4		199.00	Moment of Inertia(I) in^4		5900.00
Connection Properties			Connection Properties		
Length Factor (α_i)		6.12	Length Factor (α_i)		71.24
Initial Connection Stiffness(R_{ki})		68,830	Initial Connection Stiffness(R_{ki})		73,001

Appendix D - Miscellaneous

SAP, RAM, & Excel Calculations available upon request