North Shore at Canton

Baltimore, MD

Beau Menard
Structural
Parfitt
4/05/06

Final Report

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Executive Summary

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Executive Summary:

North Shore at Canton is a 4 story town home and parking garage structure built on top of a pier in Baltimore harbor. The building is unique in the fact that it is built over the water. The first floor of the building is an enclosed parking level, from which the residence gain access to the town houses. The second, third, and fourth levels are comprised of the town house structure. The building is approximately 15,000 sq ft. per floor, with a total square footage of 60,000 sq ft.

Given that this building is built over water, the effects of moisture damage should be of great concern. Especially since the main lateral force resisting system is comprised of gypsum sheathed shear walls, and it is recommended to replace gypsum if it has sustained any water damage. Therefore a redesign of the structural system was done as a possible rectification to the problem.

Two systems were analyzed, a steel frame and a concrete frame. The structural systems were design to support both gravity and lateral loading. An analysis was also done to compare both the cost and construction durations of each proposed system.

It was found that the steel system was the best solution for this project. The system allows for the lightest members, while still staying within serviceability limitations. It was also found that the steel frame offered the most advantages in terms of cost and construction durations.

An analysis was also done to provide the parking level with a safe lighting environment. The design was based on both IES and ASHRAE standards.

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Introduction

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Introduction:

Project Overview:

This is the final report presented for the analysis of North Shore at Canton. The objective of this report is to gain knowledge and experience in practical engineering practices. The current building systems of the North Shore at Canton are described in a series of technical reports; there is also a building proposal, which describes some possible issue that might arise with the current structural system as well as some possible solutions to those problems. This report analyzes those solutions and presents the most efficient options.

Architectural Information:

North Shore at Canton is just one in a series of townhouses located on the 2300 block of Boston Street in Baltimore Harbor, however its specific location and unique setting place it apart from the rest. North Shore sit on top of a 50'x 266' pier in the center of the harbor. It offers private parking and a wooden pedestrian promenade. It contains three levels of living space above the parking level sitting on top of the pier. The third floor offers and exterior terrace offering great views of both the inner harbor and downtown Baltimore. The building is approximately 15,000 sq ft. per floor, with a total building area of 60,000 sq ft.

Current Structural System:

The framing for the garage level consists of structural steel columns and beams, with full moment connections at column interface. The columns are comprised of W12x96, and are all 9' in height. The columns are to be considered pinned at the foundation, and are connected to the pier through base plates, which range from 14"x14"x3/4" to 18"x18"x1-1/4". The beams range from W 14x22 to W 24x68, and have spans ranging from, 18' to 25'. The garage floor, floor system, is the top of the pier. A framing plan and typical weld section are located in Appendix A.

The first floor, floor system, is made up of 8" hollow core precast concrete planks, with 3" of rigid insulation topped with 2.5" of concrete. The hollow core precast planks have embedded steel plates that are welded to the steel beams.

The first, second, and third floors were framed out using light gauge metal stud shear walls, with gypsum used as the diaphragm. The interior walls use 4" studs while the exterior walls us 6" studs. There are also 3 inch hollow steel tubes used to support steel beams or wood PSL, which ever specified by the plan, that support joist spanning in the transverse direction. General shear wall data in given in appendix A.

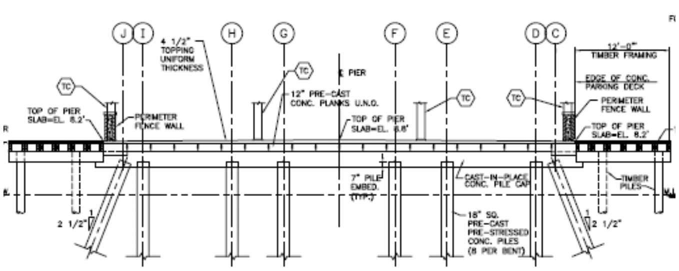
The floor systems of the remaining levels uses open web pre-engineered wood joists, which bear on the shear walls, exterior joist have top chord bearing while interior joists are bottom chord bearing. Typical joist layout is located in Appendix A.

Picture A: Floor plank assembly

Picture B: Parking/Pier level

Foundation:

The foundations of the structure are the pier bents, which are directly supported by pier piles. The pier bents/pile caps are 18” deep with steel rebar reinforcement. The pier bents are topped with 12” hollow core pre-cast planks, which have an additional 4.5” concrete topping. There are 6 vertical piles and 2 battered piles, one on each side, to support the parking level and town homes. Each pile is made up of 7000 psi concrete and 60 ksi reinforcing steel.



An examination of the loads of the structure, dead, live and lateral, was also done. The original design loads used were from BOCA 1996, while the loads in this analysis came from the ASCE 7-02. The dead loads were mainly comprised of the weight of the structure and the snow loads. The live loads that were uses were based on a residential structure. The lateral loads are made up from the wind and seismic loads.

Mechanical System:

The North Shore Pier units have two zone HVAC systems. The top floor is served by a 40,000 btu 90% natural gas furnace, and the bottom two floors are served by a 40,000 btu natural gas furnace. The attic system has a 1.5 ton thru wall condenser on the interior units and a 2 ton thru wall condenser on the exterior units. While the bottom two floors are served with a 2.5 ton thru wall condenser throughout the building. The thru wall condensers had to be installed on the garage level in closets that were constructed out of brick.

Lighting and Electrical Systems:

The electrical system consists of a low voltage 3 phase system, with the primary and secondary voltages being 220/210 from the panel. Since it is low voltage each unit uses only halogen lights approximately 90 watts each. The building also has security features hard wired into it, the fire detectors and alarms, and also a burglar alarm.

Fire Protection:

The units are protected by the gypsum sheathed walls that separate them. Since the walls are covered with 1/2" of gypsum board on each side, they provide more than adequate protection from unit to unit. Level to level protection is provided by the floor sheathing, and the ceiling elements, 1/2" gypsum ceiling panels, to provide a 2 hour fire rating.

Construction Process:

The previous wooden pier structure was demolished to make way for the new structure. Concrete piles were then driven, and the structural concrete was set in place as the base. Wooden piers and decking were then placed on top of the concrete piers. The steel frame for the parking level was erected, and the precast concrete planks were placed on top of the steel. Prefabricated wood and steel panels were delivered by barge and were erected by a barge mounted crane. The building was topped out and the MEP was installed as a design-build on site. Finishes were added later.

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Problem Statement

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Problem Statement:

“In general, gypsum board should not be exposed to elevated levels of moisture for extended periods. Examples of elevated levels of moisture include, but are not limited to, exposure to rain, condensation, water leakage, and standing water. Some board exposed to these conditions may not need to be replaced, depending upon the source of the moisture and the condition of the gypsum board being considered for replacement. However, IF THERE IS EVER A DOUBT ABOUT WHETHER TO KEEP OR REPLACE GYPSUM BOARD THAT HAS BEEN EXPOSED TO MOISTURE – REPLACE IT.”

This quote was taken directly from an article written by the Gypsum Association, in regards to moisture related problems with gypsum board. Since the structure of the top three floors are dependent on the stability of the gypsum sheathing, the effects of water damage should be of great consideration. There are also issues that arise from the buildings constant exposure to moisture, since the building is built over Baltimore harbor.

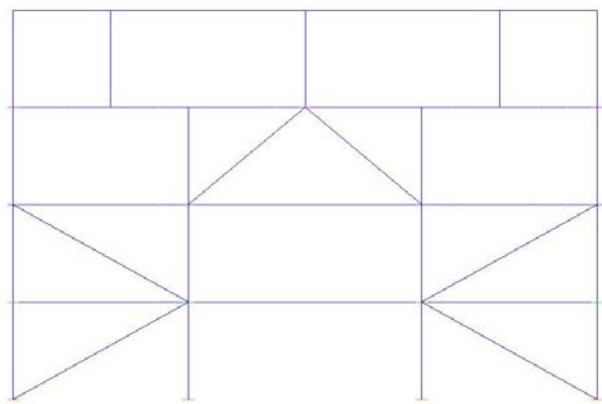
Example of moisture damage to gypsum.

There are also issues that arise in the parking level below the town homes. That area is basically the only public space in the building, and to my knowledge there were no specification made to assure the quality of lighting in that space.

Solution Overview:

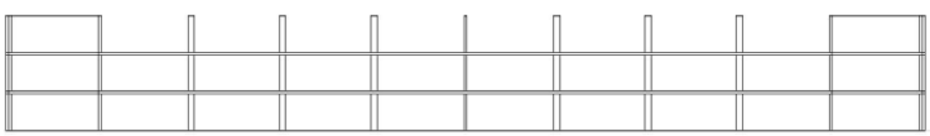
Structural Depth:

To help reduce the effect that moisture damage has on the existing structure, two alternative designs of the building superstructure will be proposed. The first structure will be comprised of a rigid steel frame; the second system will consist primarily of pre-cast concrete. Both systems will affect the cost and construction duration of the project.



Steel frame short span

The rigid steel frame will consist of four stories of steel columns and girders, , affixed to the pier bents. Each story height is approximately 10 feet. The floor system will be comprised of open web steel joists, maximum span of 25', topped with metal decking and light weight concrete. The frame will contain braced members along shared interior walls. The effects on the foundation will be addressed, given that the lateral loads will transfer differently than the original system. The design of the steel frame shall be in accordance with the AISC (LRFD) 3rd edition. Members will be analyzed by hand and checked against a computer model.



precast frame long span

The pre-cast system will consist of raising the pier bents to the first level of the town homes, an additional concrete slab will be poured on the first floor so the pier structure would utilize a double diaphragm system. The remaining three floors will consist of pre-cast concrete shear walls, the floor system will also consist of pre-cast concrete planks. The design of the concrete system shall be in accordance with the ACI 318-05. Members will be analyzed by hand and checked against a computer model.

Loads and load cases will be determined from ASCE 7-02. The IBC 2003 will also be referenced through out the design process.

Construction Management Breadth:

The overall project can be affected by dramatically changing the building systems. Altering the superstructure of the building greatly affects the overall project in both cost and construction durations. An analysis of cost and construction durations will be done for both structures presented, and a comparison will be done between the current system and the two proposed systems.

Cost and time duration estimates will be based on the data given in the RS Means manual.

Lighting Breadth:

As the parking level is the only public area of the building, it is only logical that the occupants would want a safe visual environment. To provide this an analysis was done based on IES and ASHRAE standards, to provide a safe lighting environment for the tenants of the building.

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Structural Depth

Structural Depth:

Design Loads:

IBC 2003:

Live Load:

The floor live loads indicated by the IBC 2003, for a multifamily Residential, show a distributed floor pressure of 40 psf. The partition loading increased greatly, IBC 1607.5, since the live load does not exceed 80 psf, the partition load shall be at least a uniformly distributed dead load of 20 psf

Roof Live Load:

(IBC 1607.11)
A_t = 25'(span) * 2'(spacing) = 50 sqft < 200 sqft

$R_1 = 1$
F = 4 in rise per foot
$R_2 = 1$
$L_r = 20 * R_1 * R_2 = 20*1*1$
L_r = 20 psf

Snow Load:

Importance factor (I) = 1
Ground snow load (P_g) = 25 psf
Snow exposure factor (C_e) = 0.8(Category D fully exposed)\
Thermal factor (C_t) = 1
Flat roof Snow load (P_f) = I* P_g* C_t*C_e=1*25*0.9*1
P_f = 20 psf

Dead Load:

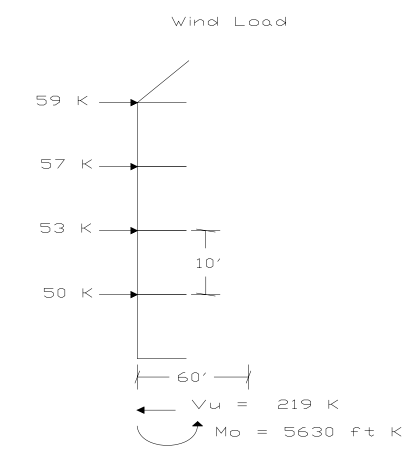
ASCE 7-02

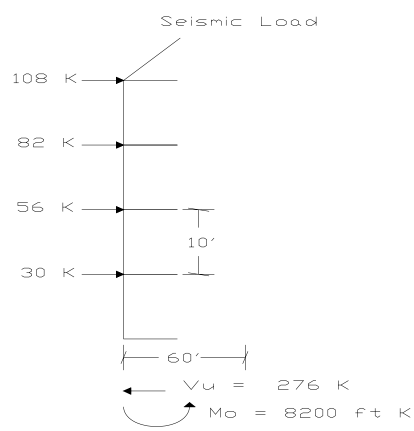
Precast hollow core concrete planks w/ 2” topping = 80 psf
Steel joist w/ metal decking & light weight concrete topping = 30 psf
Roof Trusses = 20psf
Misc. Roof = 15 psf
Structural Steel = (as noted on plans)

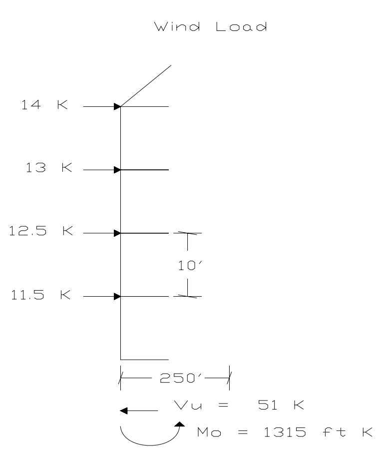
(assumed)
Mechanical = 10 psf
Electrical = 5 psf
Ceiling = 5 psf

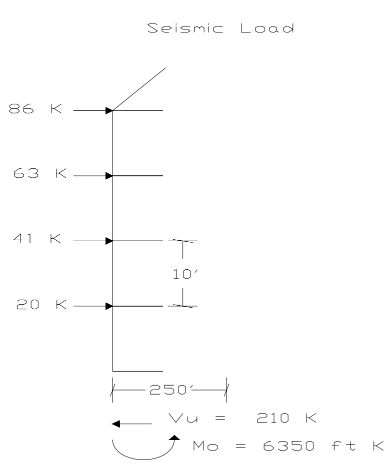
Lateral Loads:

Lateral load calculations are located in appendix A. The wind loads are applied to the steel frame and the seismic loads are applied to the pre-cast frame.









Structural Redesign:

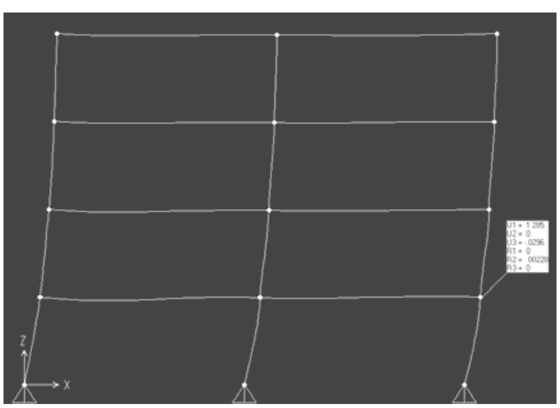
Steel frame

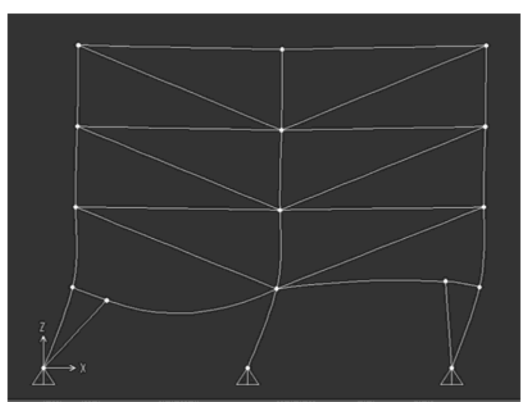
When this project started a comparison was going to be made between two types of steel frames, a moment frame and a braced frame. There would be five main frames used to resist the lateral forces that the building would be subjected to. The remaining six frames would be used to resist gravity loads and would consist of simple shear connections. All structural steel used shall be comprised of A992 grade. The structural analysis program SAP was used to check members used in each frame.

The controlling load case for the steel frames consists of:

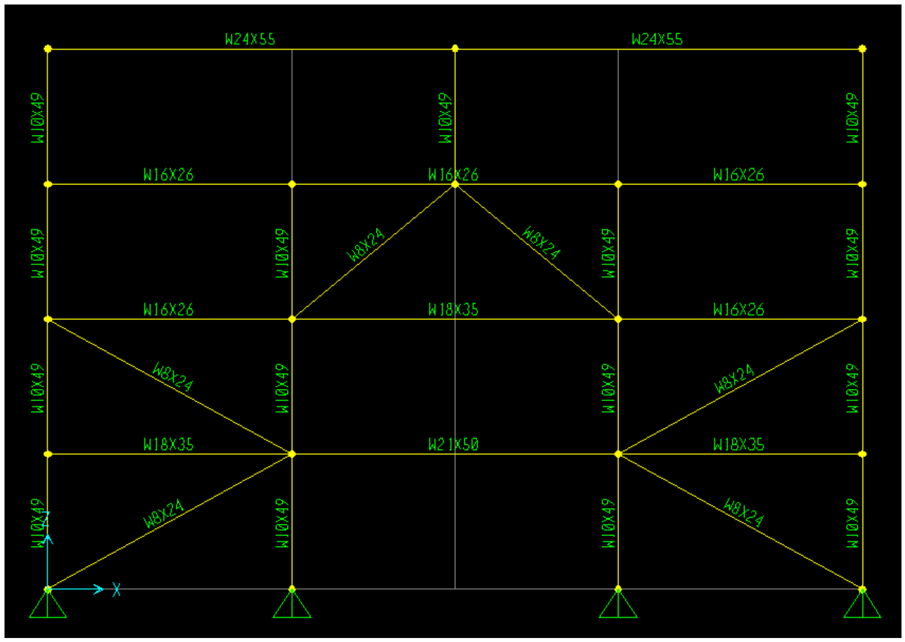
$$1.2\ D + 1.6\ W + 0.5\ L + 0.5\ S$$

The members for each frame were sized and a computer model was created. After applying the appropriate loads to the structure some serviceability issues started to arise, with both types of frames. Though the members could adequately support the gravity loads of the structure, when a lateral force was applied it was clear that the type of support modeled for the structure was not adequate to resist lateral deflections. The base of the columns were modeled as pin connections, this was done so as to transfer as little moment to the pier bents as possible. Since there were only 3 columns supporting the frame the initial lateral drift of the first story of the building was well over 1". This was well beyond the serviceability limit of the brick and glass façade of L/600, L = 120", 120"/600 = 0.2". A possible solution to this problem was to increase the size of the columns however the sizes of the columns were becoming extremely large and still not able to handle the serviceability limitation. Another solution to this problem was to apply braces to the exterior columns and girders. This did limit the deflection on the leeward side of the building, though it did not help resist against displacement on the windward side. It was then determined that a more efficient frame would need to be developed to limit lateral displacement.





In addition large initial deflections, I also noticed large amounts of force being placed on the pier bents in an area that is not reinforced for that type of loading. Therefore columns in the new frame will be placed in the same position as the existing columns. The exterior columns line up directly over the exterior pier piles, and the interior columns are spaced 18' on center from the exterior respectively, with a central spacing of 24'.

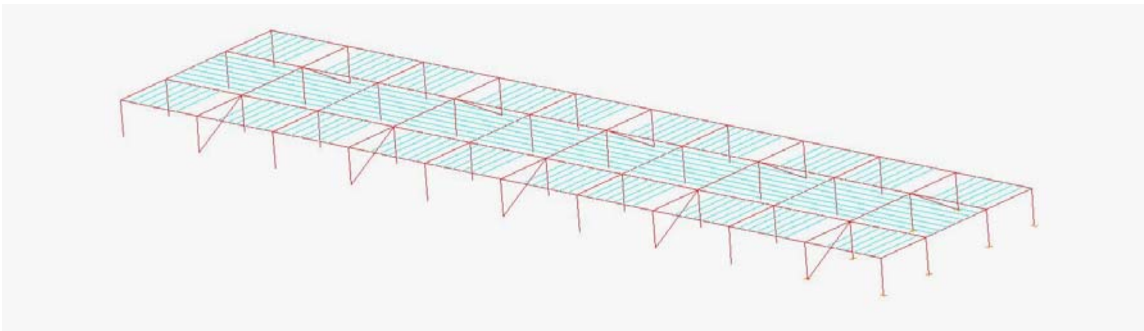


Member Sizing:

Trying to keep as many members as light, and typical, as possible I found that using a combination of the moment frame and braced frame resulted in the best solution. The combination of the two frame systems allowed for the use of numerous repetitive members, as well as reducing forces on members allowing for a smaller section, and therefore a lighter member. The braces help resist lateral displacement and allow the base connection to be modeled as pinned. Adding an internal column also helped reduce the amount of shear force the pier bents would be subjected to, in addition placing the columns in the position as the original design allows for gravity load transferred into the pier piles to be as originally designed for. Five of these combination frames used to resist the lateral loading of the building. Since there are eleven total frames, these will be spaced at every other frame. This allows the remaining six frames to be sized for gravity loads only.

First Floor Design:

Basic frame:



The floor joists used on this level were laid out based on stair well axis. Since the adjoining interior units share a stair well on the first level it was only logical to span the joist parallel to them. Also to help reduce moment placed on the column in a certain direction the interior joists span perpendicular to the exterior joists. This allows for the metal decking to be placed continuously along the floor, with only a few openings. All of the joists are spaced at three feet on center and the maximum span is 25'.

@ 3' spacing, and a span of 25'

K-series, 16k4

Total load on joist = 270 plf	Allowable total load = 313 plf
Live load on joist = 120 plf	Allowable live load = 195 plf

The joists are more than adequate for both the 18' span and the 25' span.

All columns used are W 10x49 this shape provides adequate axial support, it also has a fairly large section modulus along each axis which provides the moment resistance needed based on the joist layout. Bracing elements are used in each alternate frame to help resist lateral displacement. The braces used are W 8x24 and are only used on the exterior part of the frame. The girder used to support the stair wells are W 18x35 while the girders used to support the joists spanning 25' are W 21x50. Moment and shear diagrams are available in appendix A.

Short Frame:

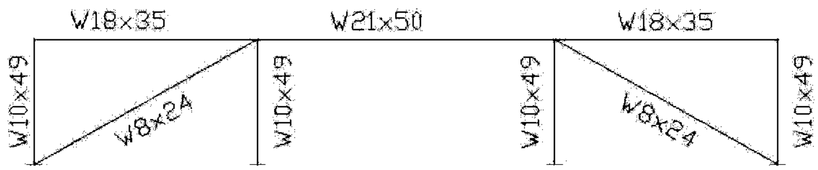


Table shows maximum design loads applied

Member	Pu (k)	Mux (k ft)	Muy (k ft)
W 10x49 (int)	260	43	20
W 10x49 (ext)	220	35	-
W 21x50	10	125	-
W 18x35	10	50	-
W 8x24	45	-	-

Though the loading on each member is not very large the members were assumed to be un-braced along the entire length, to give the most conservative value.

The floor joists are topped with metal decking and a light weight concrete. The joists are secured to the girders by tack welds, and are not considered a bracing member.

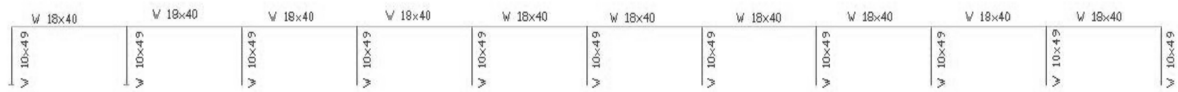
Steel Deck manual:

Deck: 1.5"x6"
Fy = 33 ksi
f'c = 3 ksi
Weight Concrete = 115 pcf

- **19 Gauge Steel Decking**
- **Max unshored span**: 9.05' (3 span)
- **Uniform Live Load Capacity**: 155 psf (no studs, service load)
- **Slab Depth**: 4"
- **Weight**: 29 psf
- **Connection**: 1/8" thick 1" long fillet weld on each side

Long frame:

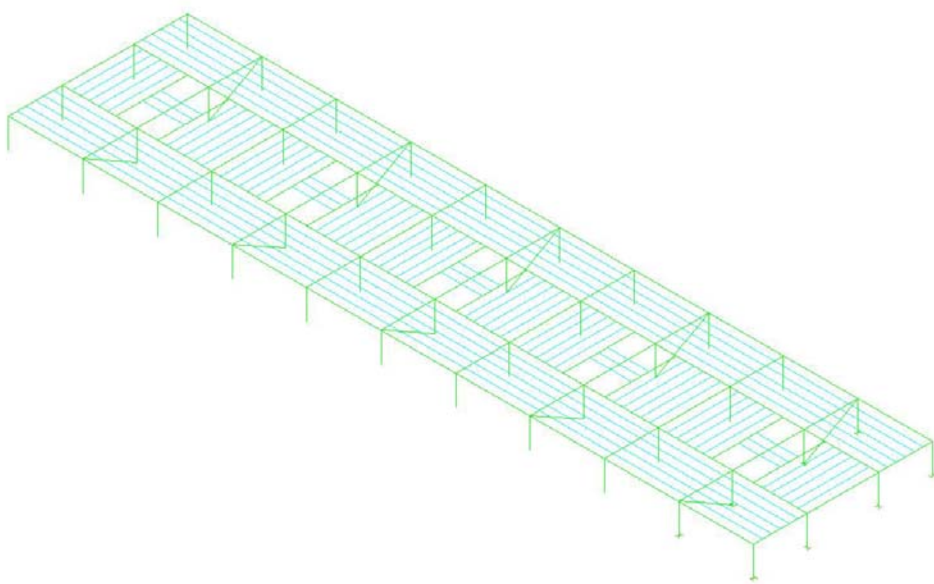
The frame in the long span direction of the building is to be considered to have full moment connections at the beam to column interfaces. The columns are oriented accordingly, so as to resist the moments transferred to them. The bases of the columns are modeled as pinned, so as to transfer as little moment to the pier bents as possible.



The column design has already been done in the short frame span, though they did need to be checked for stability. The beams did need to be sized, and it was found that a W 18x40 could adequately support the design loads, including lateral forces. Moment and shear diagrams are located in appendix A.

<u>Second Floor Design:</u>

Basic frame:



The floor joists used on this level were laid out based on stair well axis. Since the adjoining units share a stair well on the second level it was only logical to span the joist parallel to them. Also to help reduce moment placed on the column in a certain direction the interior joists span perpendicular to the exterior joists. This allows for the metal decking to be placed continuously along the floor, with only a few openings. All of the joists are spaced at three feet on center and the maximum span is 25'.

@ 3' spacing, and a span of 25'

K-series, 16k4

Total load on joist = 270 plf   Allowable total load = 313 plf
Live load on joist = 120 plf   Allowable live load = 195 plf

The joists are more than adequate for both the 24' span and the 25' span.

Short Frame:

All columns used are W 10x49 this shape provides adequate axial support, it also has a fairly large section modulus along each axis which provides the moment resistance needed based on the joist layout. Bracing elements are used in each alternate frame to help resist lateral displacement. The braces used are W 8x24 and are only used on the exterior part of the frame. The girder used to support the stair wells are W 18x35 while the girders used to support the joists spanning 25' are W 16x26. Moment and shear diagrams are available in appendix A.

Table shows maximum design values

Member	Pu (k)	Mux (k ft)	Muy (k ft)
W 10x49 (int)	118	45	20
W 10x49 (ext)	80	32	25
W 16x26	20	65	-
W 18x35	20	80	-
W 8x24	25	-	-

Though the loading on each member is not very large the members were assumed to be un-braced along the entire length, to give the most conservative value.

The floor joists are topped with metal decking and a light weight concrete. The joists are secured to the girders by tack welds, and are not considered a bracing member.

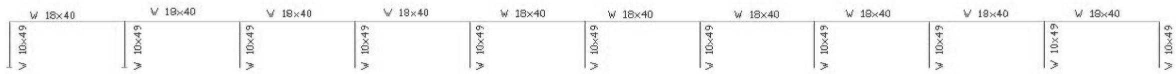
<u>Steel Deck manual:</u>

Deck: 1.5"x6"
Fy = 33 ksi
f'c = 3 ksi
Weight Concrete = 115 pcf

- **19 Gauge Steel Decking**
- **Max unshored span**: 9.05' (3 span)
- **Uniform Live Load Capacity**: 155 psf (no studs, service load)
- **Slab Depth**: 4"
- **Weight**: 29 psf
- **Connection**: 1/8" thick 1" long fillet weld on each side

Long frame:

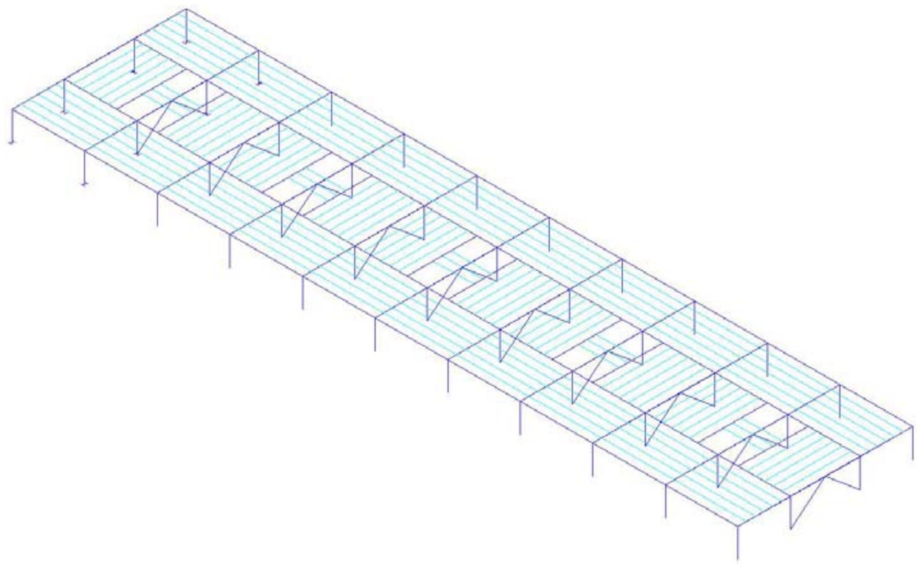
The frame in the long span direction of the building is to be considered to have full moment connections at the beam to column interfaces. The columns are oriented accordingly, so as to resist the moments transferred to them. The bases of the columns are modeled as pinned, so as to transfer as little moment to the pier bents as possible.



The column design has already been done in the short frame span, though they did need to be checked for stability. The beams did need to be sized, and it was found that a W 18x40 could adequately support the design loads, including lateral forces. Moment and shear diagrams are located in appendix A.

Third Floor Design:

Basic frame:



The floor joists used on this level were laid out based on stair well axis. Since the adjoining units share a stair well on the second level it was only logical to span the joist parallel to them. Also to help reduce moment placed on the column in a certain direction the interior joists span perpendicular to the exterior joists. This allows for the metal decking to be placed continuously along the floor, with only a few openings. All of the joists are spaced at three feet on center and the maximum span is 25'.

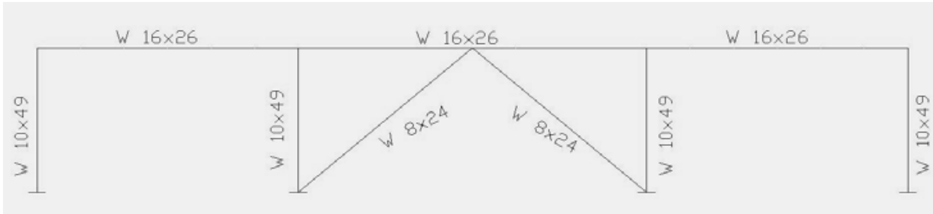
@ 3' spacing, and a span of 25'

K-series, 16k4

Total load on joist = 270 plf   Allowable total load = 313 plf
Live load on joist = 120 plf   Allowable live load = 195 plf

The joists are more than adequate for both the 24' span and the 25' span.

Short Frame:



All columns used are W 10x49 this shape provides adequate axial support, it also has a fairly large section modulus along each axis which provides the moment resistance needed based on the joist layout. Bracing elements are used in each alternate frame to help resist lateral displacement. The braces used are W 8x24 and are only used on the exterior part of the frame. The girder used to support the stair wells are W 18x35 while the girders used to support the joists spanning 25' are W 16x26. Moment and shear diagrams are available in appendix A.

Table shows maximum design values

Member	Pu (k)	Mux (k ft)	Muy (k ft)
W 10x49 (int)	35	52	20
W 10x49 (ext)	45	35	15
W 16x26(ext)	15	65	-
W 16x26(int)	10	46	-
W 8x24	65	-	-

Though the loading on each member is not very large the members were assumed to be un-braced along the entire length, to give the most conservative value.

The floor joists are topped with metal decking and a light weight concrete. The joists are secured to the girders by tack welds, and are not considered a bracing member.

<u>Steel Deck manual:</u>

Deck: 1.5"x6"
Fy = 33 ksi
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- **19 Gauge Steel Decking**
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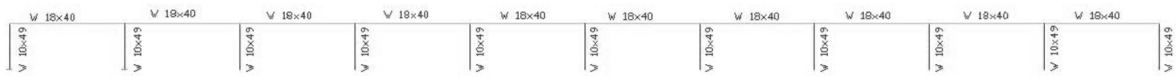
- **Uniform Live Load Capacity**: 155 psf (no studs, service load)
- **Slab Depth**: 4"
- **Weight**: 29 psf
- **Connection**: 1/8" thick 1" long fillet weld on each side

Long frame:

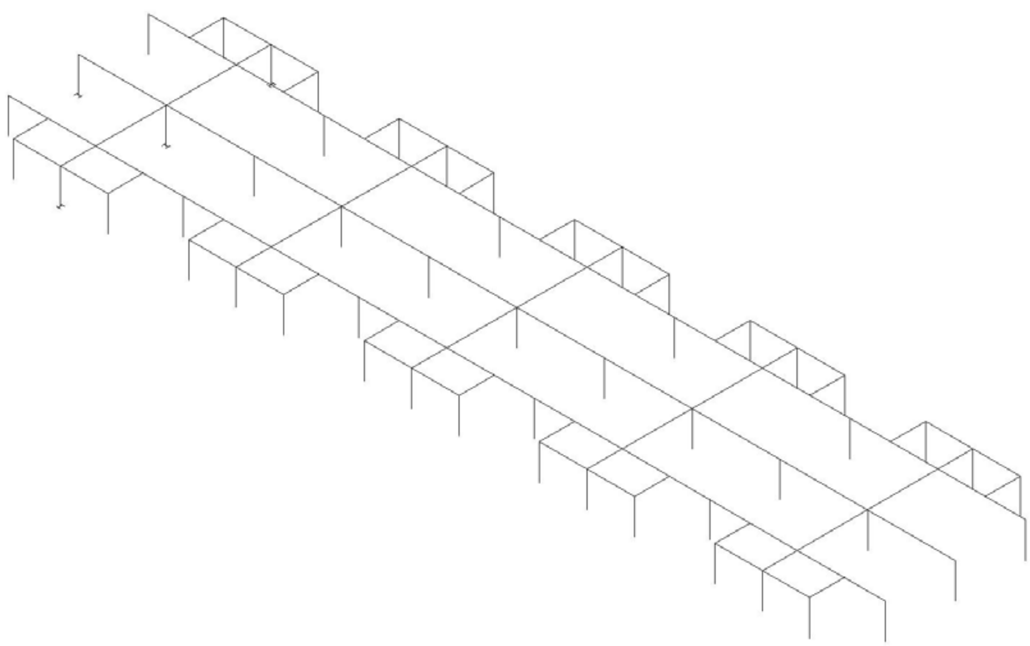
The frame in the long span direction of the building is to be considered to have full moment connections at the beam to column interfaces. The columns are oriented accordingly, so as to resist the moments transferred to them. The bases of the columns are modeled as pinned, so as to transfer as little moment to the pier bents as possible.



The column design has already been done in the short frame span, though they did need to be checked for stability. The beams did need to be sized, and it was found that a W 18x40 could adequately support the design loads, including lateral forces. Moment and shear diagrams are located in appendix A.

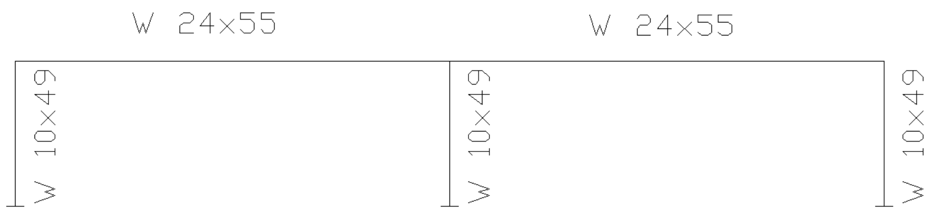
Fourth Floor Design:

Basic frame:



The frame was laid out in a particular order, so as to utilize the existing roof structure. The interior long frame span would support the center ridge of the roof and would support a majority of the roof load. The five central short frame spans would support the transverse ridge lines and they too would carry most of the load in that direction.

Short Frame:



All columns used are W 10x49 this shape provides adequate axial support, it also has a fairly large section modulus along each axis which provides the moment resistance needed based on the roof layout. The central column bears directly on the beam below, however that is supported by the two bracing members, the columns are considered pinned at the base so as not to apply any types of torsion effects to the flange of the beam below. The roof girders consist of W 24x55 this shape more than adequately supports the roof structure and resist the lateral forces placed on them.

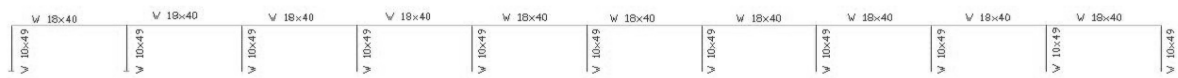
Table shows maximum design values

Member	Pu (k)	Mux (k ft)	Muy (k ft)
W 10x49 (int)	50	52	30
W 10x49 (ext)	18	35	30
W 24x55	15	152	-

Though the loading on each member is not very large the members were assumed to be un-braced along the entire length, to give the most conservative value.

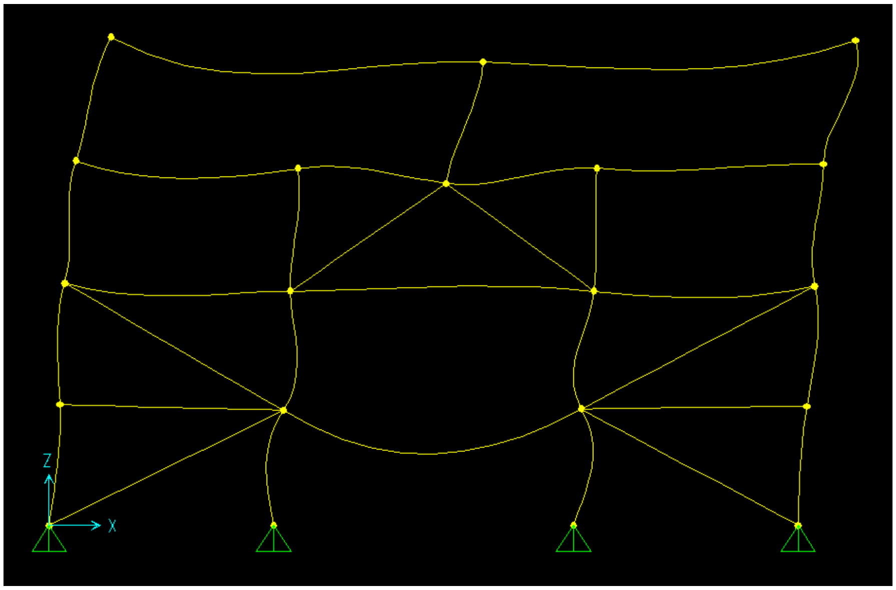
Long frame:

The frame in the long span direction of the building is to be considered to have full moment connections at the beam to column interfaces. The columns are oriented accordingly, so as to resist the moments transferred to them. The bases of the columns are modeled as pinned, so as to transfer as little moment to the pier bents as possible.



The column design has already been done in the short frame span, though they did need to be checked for stability. The beams did need to be sized, and it was found that a W 18x40 could adequately support the design loads, including lateral forces. Moment and shear diagrams are located in appendix A.

Lateral Displacement:

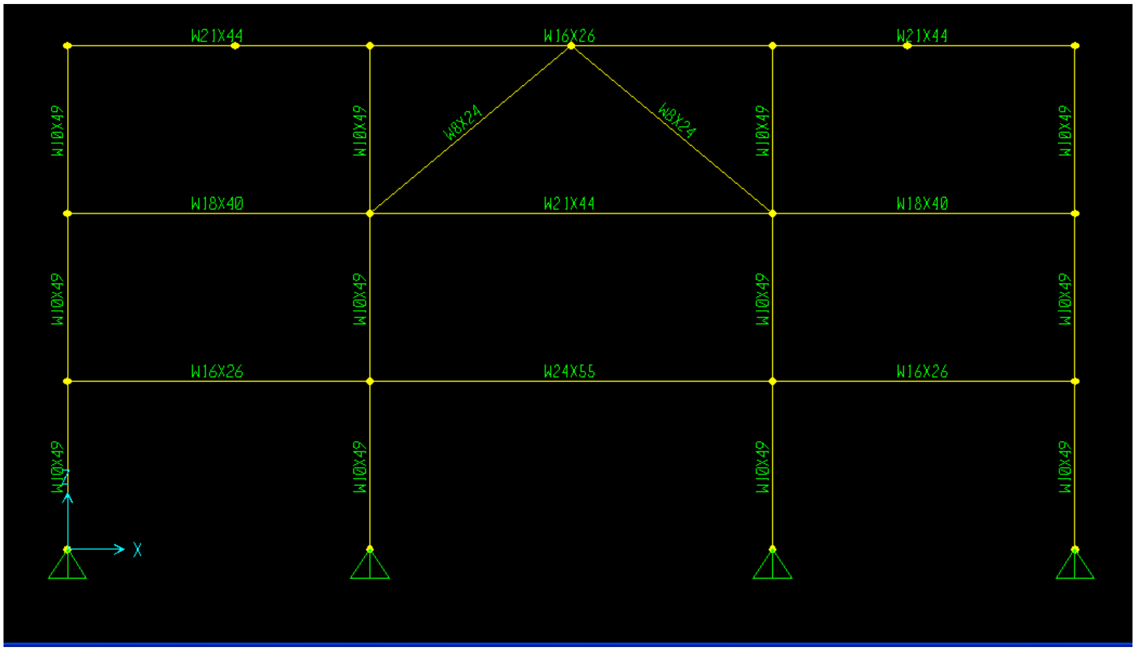


Story	**Total Lateral Displacement** (in)
1	.041
2	.079
3	.121
4	.276

The frame provides more than sufficient resistance to lateral displacements. The serviceability displacement requirement for the façade was L/600, and it is clear that the frame easily meets the recommended design displacement. Inter-story drift is not an issue either as it meets the displacement requirements.

Lateral displacement in the long frame span direction was found to be minimal. All connections in the long span direction are to be type two with wind, so as to only transfer partial moments into the columns. The top floor uses one central frame while the remaining floor use four frames. Using all four frames as a lateral force resisting system allows for light members to be used, and less loads applied to the foundation.

Gravity Frame:



The design of this frame was based solely on gravity loads, and it was found that the load case; 1.2 D + 1.6 L, was the controlling load case. All girders are assumed to be connected to the columns with only shear connects, so as to limit moments distributed to the columns. Loading diagrams, as well as shear and moment diagrams are located in appendix A.

Foundation Issues:

Since the columns of the redesigned frame are placed in the same position as the current system, no new issues arise from the distribution of gravity loads. The only change that has occurred is an increase in the axial load from the column that transfers into the pier bents as shear, the current design shows loads for internal columns to be about 190 k, while exterior column had loads of around 170 kips; the loads from the redesign only increased to 220 k and 200 kips respectively. It should be noted that he pier piles are made from 7000 psi concrete and have full steel reinforcement and have capacity of 135 tons in compression, and 30 tons in tension. Issues could arise from the lateral forces that are placed on the structure, and then get transferred into the batter piles on the edge of the piers, since there are eleven batter piles on each side of the pier there is more than enough capacity to resist these forces in tension or compression. Therefore there is no need to make any major changes to the foundation of this structure.

Steel Frame Conclusion:

Originally two types of steel frames were going to be compared, a moment frame and a braced frame. After analysis on both types of frames it was concluded that not one type of frame would be sufficient to resist the loads applied to the structure. It was then determined that a combination between a moment and braced frame would be the best solution. The combination frame adequately supported the loads applied to it while allowing for lighter members than would be required by either separate frame. One possible down fall to this type of system are the various types of connections that would be required. The different types of connections would require some factor assembly as well as some specialty work on site, in particular field welds. The connections could also increase the price of the project, and depending on the crew assembling the project could cause some possible delays in construction time.

The frame proved more than adequate in resisting lateral loads while staying with in the serviceability limitations. The maximum building drift was only 0.27 inches while the maximum inter-story drift was only 0.15 inches. Also it was found that the foundation was more than adequate in resisting the base shear applied to them, through the distribution of the load through the batter piles.

The foundation also proved more than adequate in supporting the redesigned structure. The only issue that caused concern, from the redesign, was the distribution of the lateral forces into the pier piles. The batter piles extending diagonally off of each side of the pier were found to be more than adequate in resisting those types of loads.

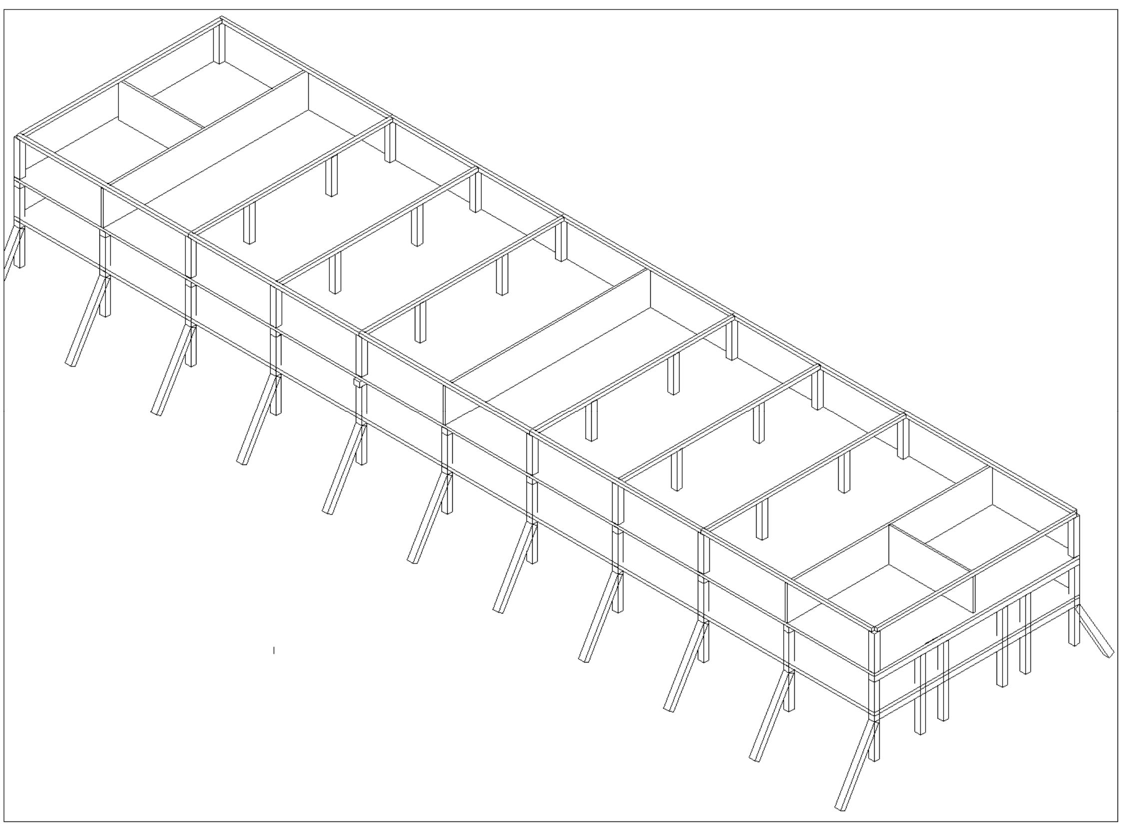
One issue that needs to be mentioned is that of moisture damage, since this structure is susceptible to high moisture content, it is only logical to try to prevent as much moisture damage as possible. One possible solution to this is to paint the exposed steel members; this would provide a fairly cost effective solution, though it would need to be evaluated annually and refurbished as required.

Structural Redesign:

Concrete Option:

An alternative system was selected to compare against the steel frame chosen. Based on constructability and location it was determined that a pre-cast concrete frame would give a good comparative study. Some variations on the foundation do need to be noted. The current foundation system of North Shore at Canton, are concrete piers topped with 12”plank with an additional 4.5” concrete topping, creating a single diaphragm to resist the loads applied. For the purposes of this alternative system, the foundation piles will be raised to support the first floor of the town homes. While keeping the original plank system on the grage level, an additional plank system will be applied at that level so as to create a duel diaphragm system. A pre-cast frame system, which consists of bearing/shear walls, column, and girders would sit on top of the upper plank system, and would support the three upper levels. A small perimeter beam would span the exterior to help support the façade. All pre-cast concrete used shall be 4000 psi, unless otherwise noted, and all steel reinforcement shall be grade 60.

Basic Frame:



Pier level and first floor frame of the town homes.

Floor Structure:

To keep consistent with idea behind this frame, pre-cast hollow core planks with a concrete topping will be used as the floor system. The parking level, which is the original foundation level will remain the same with 12” planks and a 4.5” topping. The remaing floors will be supported with 8” planks with a 2” concrete topping. All size selections were made using span tables from Nitterhouse Concrete Products.

8”x4’ SpanDeck –U.L.-J917

Plank span = 25’

Superimposed Load:

DL = 50 psf
LL = 40 psf

Total Load = 90 psf

Strand Pattern:

Flexure 4 - 1/2" Φ allowable load = 110 psf

Typical Beam:

Typical beams will be sized for both the interior beams and the perimeter beams. The beams and the columns will be sized for gravity loads only and will not be considered part of the lateral force resisting system. The beams are to bear directly on the columns and will be considered to be pinned at their supports. Typical beams will be continuous over multiple spans and have a maximum length of 60’. It was found that minimal beam sizes with only minimal reinforcement were able to support the loads applied to them. Though some beams did not meet ductility requirements, the appropriate safety factors were applied and the beams were still adequate to resist the loads. It was also found that minimal shear and torsional reinforcement was necessary, at a minimum spacing of d/2. Having continuous beams also helps with constructability issues as the crane would only need to make one pick as apposed to multiple picks made for separate beams. Interior beams are to be considered a three span beam, with the exterior spans being 18’ and the interior being 24’. The perimeter and roof beams are to be considered 2 spans, with equal span lengths of 25’. Shear and Moment diagrams are available in appendix A.

Typical Beam	Size	Top Reinforment	Bottom Reinforcement	Shear and Torsional Reinforcement
Interior	14”x12”	4 # 9 bars	3 # 9 bars	# 3 bars spacing = 7
Perimeter	12”x10”	2 # 5 bars	2 # 5 bars	# 3 bars spacing = 7
Roof	16”x12”	4 # 9 bars	4 # 9 bars	# 4 bars spacing = 7

Typical Columns:

All columns are to be considered approximately 8.5’ in height, this assumption is based on the assumption of the beams being continuous on each level, and therefore the columns would have to be separate from story to story. The columns will be connected to the beams with only minimum fixity so as to transfer as little moment as possible. Columns were sized based on gravity loads, and beam bearing requirements were also considered.

Column Level	Axial Load (kips)	Column Size	Reinforcement $\rho = 0.03$	Interaction Value
1	86	12”x12”	2 #7 bars / face	0.23
2	172	12”x12”	2 #7 bars / face	0.46
3	258	14”x14”	3 #7 bars / face	0.51

Columns were checked against applied moments, though there is no moment transferred from the beam, the beam support is at an eccentricity from the columns centroidal axis. The interior beams would have no moment designed for them as they directly support the beam, however the exterior beams will have some eccentricities applied to them, though they receive a reduced load when compared to the interior column, and therefore the moment received is not extensively large. The reinforcement used proved more than adequate, when applied to the interaction diagram. Column calculations and interaction solutions are available upon request.

Bearing/Shear Walls:

The bearing/shear walls were designed using the full span of 60’. In actuality they will be constructed in pieces and assembled in the field, connected either by plates welded together or by inserting rebar and grouting the exposed area. Similar to the columns the bearing/shear walls are not continuous from story to story, though it should be noted that they would be connected from level to level, by inserting rebar and grouting the exposed area, to ensure that the lateral loads transfer through the structure as they were designed for. Only three walls will be used in the short span direction, so as to reduce the weight of the structure, and to maximize wall effectiveness. The first wall will be placed at the center of the building, and the other two will be spaced at 100’ on each side respectively. The capacity of the wall in shear is more than adequate to resist the loads placed on the wall, even torsional effects, and it was found that only a minimal amount of reinforcing steel, $\rho = 0.0025$, and minimum spacing of 18” was required in both the vertical and horizontal spans. The same is true for the transverse shear walls, though there would be only one of them on each end of the building, they only required minimal reinforcement and spacing, and since they lie on the axis of rigidity they would receive no torsional effects. Lateral drift was calculated and found that there was a maximum story drift of 0.02 in, and a total building drift of 0.092 in.

Foundation Issues:

Foundation concerns do arise in this redesign option. Since the frame has been changed to concrete, the weight of the structure dramatically increased from the first option, though the piles can adequately support the compressive load. Also extending the pier up an additional level would increase the complexity of the pile connections to the parking level diaphragm. An additional pier bent would need to be designed so it would be able to support the loads from the parking deck. The lateral load transfer would also be affected, and would need to travel through the vertical piles as shear before it would reach the batter piles at the water level. I attempted to contact the pier engineers Whitney, Bailey, Cox, and Magnani, to try and determine what other issues might arise from raising the pier level up, and one of the main issues that arose was the connectivity between the shear walls and the piers. Logically only minimal connections would be used so as to transfer the loads as tension and compression at the ends of the walls.

Concrete Conclusion:

The concrete frame proved more than sufficient in resisting the loads applied to it. It was found that a load case of, 1.2 D + 1.0 E + L + 0.2 S, was the controlling load case. One downfall that they frame has is the inherent self weight, which actually affects the type of lateral load applied to the structure. It should be noted that the frame elements were more than satisfactory in resisting the lateral loads regardless of the type. However The system puts extra stresses into the foundation due to the extra weight of the structure, and the transfer of lateral loads in pier piles. Connection types are also of concern, since moisture damage is inevitable, the steel reinforcing members would have to be protected either by galvanization or other means. Also connection would probably have to be inserted reinforcing bars, grouted in place, instead of field welds, so as not to expose the metal to the elements. Water damage in the concrete is also an issue; however there are add mixtures available to place in the concrete mixture to help reduce the amount of water that would absorb into the material.

In terms of constructability, this frame should have no problems. As the whole super structure is made of pre-cast concrete the only task would be to pick the members of the barges and place them. The durations of the project would depend more on the crew placing the members, however the structural elements are identical from floor to floor so once the initial problems are worked out the installation of the rest of the members should not be a problem. I attempted to contact Nitterhouse, to determine the feasibility of my design, however I have not yet received a response to my questions.

Structural Conclusion:

When this project first started three structural frames were going to be analyzed. Two steel frames, a moment frame and a braced frame; as well as a pre-cast concrete frame. After some analysis it was determined that the individual steel frames were insufficient due to serviceability issues. It was determined that a combination between the two types of steel frames would be the best option for that type of material. As for the concrete frame, the goal was to try to keep the frame as light as possible, so the shear walls were placed specifically so as to resist the lateral load applied. The rest of the concrete frame consisted of pre-cast columns and girders, which support the pre-cast floor planks and concrete topping. The roof system for each frame option will not change; frame elements were placed in specific locations so as not to disrupt the façade or the roof.

The connection types for each frame could cause some issues to arise. The frame used in the steel design consisted of various types of connections that vary from full moment connections to diagonal bracing elements, which could cause some constructability, and possible procurement delays. The connections used for the pre-cast frame could create some problems due to the exposure to high moisture content. Reinforcing steel members would either have to be galvanized or grouted, when placed in other pre-cast elements.

Moisture damage is a concern for both systems, as this building is exposed to high levels of moisture. The exposed steel members could be painted, however the paint would need to be checked and touched up as necessary. The pre-cast frame would have issues with both the damage to the concrete, and the damage to the reinforcing steel. Possible solutions are to galvanize or grout the steel members so as not to expose them to moisture; another is to place certain admixtures into the concrete mix, to reduce the amount of moisture absorbed by the concrete members.

Fire protection was not an issue for either type of frames. The joist spacing and concrete topping are well within the guidelines set for a two hour fire rating, with the addition of a gypsum ceiling membrane in place. The pre-cast concrete system is naturally fire resistant and is well within code specifications. A two hour fire rating can be achieved from bay to bay, by the gypsum board used in the shared unit walls.

The foundation was affected by both systems; however more issues arose with the concrete frame than with the steel one. The steel frame only slightly increased the shear loads that the pier bent would have to resist. It also changed the way the lateral loads are distributed to the piles, since the frame is full height the lateral load transfer directly into the piles, however it was determined that the batter piles are more than adequate to handle the load applied. The concrete frame had more dramatic effects on the foundation. The concrete increased the dead load of the structure and adds compressive stress on the pier piles. Also the transfer of lateral loads would change; the loads would transfer through the vertical piles as shear before they would get to the batter piles at the garage level. The implementation of the concrete frame could create the need for a possible redesign of the pier piles.

In conclusion both frames are sufficient redesigns of the superstructure of the building. Each structural system has inherent advantages over the other. Based on structural efficiency, it is hard to decide between either of the frame options; therefore a decision can not be made until a further analysis, that includes cost and construction duration, is made.

North Shore at Canton

Baltimore, MD

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Final Report

Construction Management Breadth

Cost Analysis:

A cost analysis based on the total cost values, including operation and production, given in the RS Means manual was done, to compare the cost of the steel frame and the pre-cast concrete frame against the original cost of the town homes. Cost analysis includes structural elements of each frame system, as well as each respective floor system. It should be noted that the cost estimates did not include the cost of connections required for each system, and the total cost of the structure is subject to change based on the connections used.

The cost of the original systems was based on an estimate given to me from the engineers at WCMB, on the town home structural elements, the steel stud walls sheathed in gypsum, as well as the floor and roof systems. However the cost estimate given did not include the cost of the steel columns or the cost of the hollow core planks, which support the first floor of the town homes. An estimate was done on the columns and planks and concrete topping, and was then added to the cost of the town homes. It was found that the cost of the original structure was about \$ 448,600.00.

The cost of the steel structure was based on the columns, girders and bracing elements used. Also the cost of the floor joists, metal decking, and concrete topping were taken into consideration. The estimated cost of the steel frame came out to roughly \$ 650,000.00.

The cost of the pre-cast frame was based on the columns, girders, shear walls, and the floor system used. The cost does not include the cost required to alter the pier foundation of the building. The total cost of the pre-cast concrete frame came out to be roughly \$ 766,000.00.

Overall the original system was the cheapest system to implement, which is fairly reasonable considering the structural members consisted of gypsum board and steel studs.

The steel frame only increased the cost from the original by roughly $ 200,000, while the pre-cast frame increased the rough estimate by $ 310,000. Though these estimates did not consider the specific cost of the connections between members, I do believe they fairly represent the general cost of the structure. Member quantities and cost take offs are located in appendix A.

Construction Durations:

The construction durations estimated for this analysis were done using information given by the RS Means manuals. A duration estimate was done for a typical bay of each redesign option, and then was extrapolated to determine the construction duration of the entire structure.

The original construction of the town homes lasted approximately 5 months. The project started in late January of 2002, and was completed near the middle of July of 2002.

The duration estimates used for the steel frame were based on a typical bay, which consisted of 8 typical columns, 4 exterior girders, 2 interior girders, and 2 bracing members. It was found that construction duration for a typical frame was about 3.5 days per bay. Since there are approximately 40 typical bays in the structure, it was determine that the construction of the steel frame would take approximately 4 months to complete.

The duration estimates used for the pre-cast concrete frame were based on a typical bay, which consisted of a 8 columns, 2 interior girders, 2 perimeter beams and a full length shear wall. It was found that construction duration for a typical pre-cast frame was about 15 days. Since there are approximately 12 typical bays in the structure, it was determined that the construction of the pre-cast concrete frame would take about 6 months to complete.

The durations for each system were roughly the same. Construction time estimates are highly subject-able and are easily altered. There are a lot of factors that affect construction durations, including material procurement, types of connections, crew sizes and equipment used. The values estimated for this report were based on construction time only, with a single crew. Adding multiple crews, which would be a more realistic situation, would decrease construction time for either frame.

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Lighting Analysis:

An analysis was performed at the parking level of the structure, to assure the quality of lighting. As the parking level is the only public area of the building, a sense of security is needed. Designing the lighting system seemed to be a reasonable solution to the problem. The design of the lighting system was based on both the IES and ASHRAE standards.

Based on the IES code it was found that the average horizontal illuminance for a parking garage, in both the day and night, was 5 foot candles, on the pavement. It was also found that the power density limitations, set by the ASHRAE standards, worked out to be 0.3 watts/ft^2.

The lighting design program Luxicon was used to determine the luminaries used and how they will be laid out. However I was unable to come up with a satisfactory design. Though the illuminance levels and power density limits could be met, the luminaries did not meet the spacing requirements. A reasonable design could not be made with out an in-depth analysis.

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Final Report

Recommendations

<u>Final Conclusion:</u>

Designing the structure of a building is an enduring process. It is not always easy to come up with the best answer. Trying to keep the solution as easy as possible is the first step, however easy is not always best. At the beginning of this project a simple solution was tried and rejected as it was unable to constrict to certain guidelines. As a result a more efficient system needed to be developed. There are a lot of factors that come into play when designing a buildings structure, including size, location, and use. Two types of frames were considered for the redesign of this project, a steel frame and a pre-cast concrete frame; both were more than sufficient in resisting the loads applied to them. However structural efficiency was not the only consideration in choosing the best option. When both types of frames were further analyzed it was clear that the steel frame was the most advantageous. Allowing for lightweight members, while still being able to resist the load, was a major factor in the final decision. The pre-cast frame, though it could easily resisted the loads applied, the inherent self-weight of the structure was its biggest shortcoming. The pre-cast frame also affected the way the lateral loads transfer through the pier, which would require an in-depth analysis of the pier structure to determine if it was still structurally stable. The steel frame, though it did slightly increase the loads on the pier, did not affect the pier so as to cause the need for an additional analysis. Project cost and construction durations were also factors in determining that the steel frame would be the best solution for this problem. While the values calculated are based on assumptions and subject to change, steel had the clear advantage. The resulting steel frame was approximately $ 100,000.00 cheaper than the pre-cast frame, it was also determined that the steel frame had shorter construction durations than the pre-cast frame. Both frames are clearly valid solutions to the problem, however due to some minor differences it was clear that for this particular situation a steel frame was the best option.

North Shore at Canton

Baltimore, MD

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References

American Institute of Steel Construction. LRFD Manual of Steel Construction, Second Edition. Chicago: AISC, 1994.

Geschwinder, Louis F. West, Harry H. Fundamentals of Structural Analysis New York: John Wiley & Sons, Inc., 2002

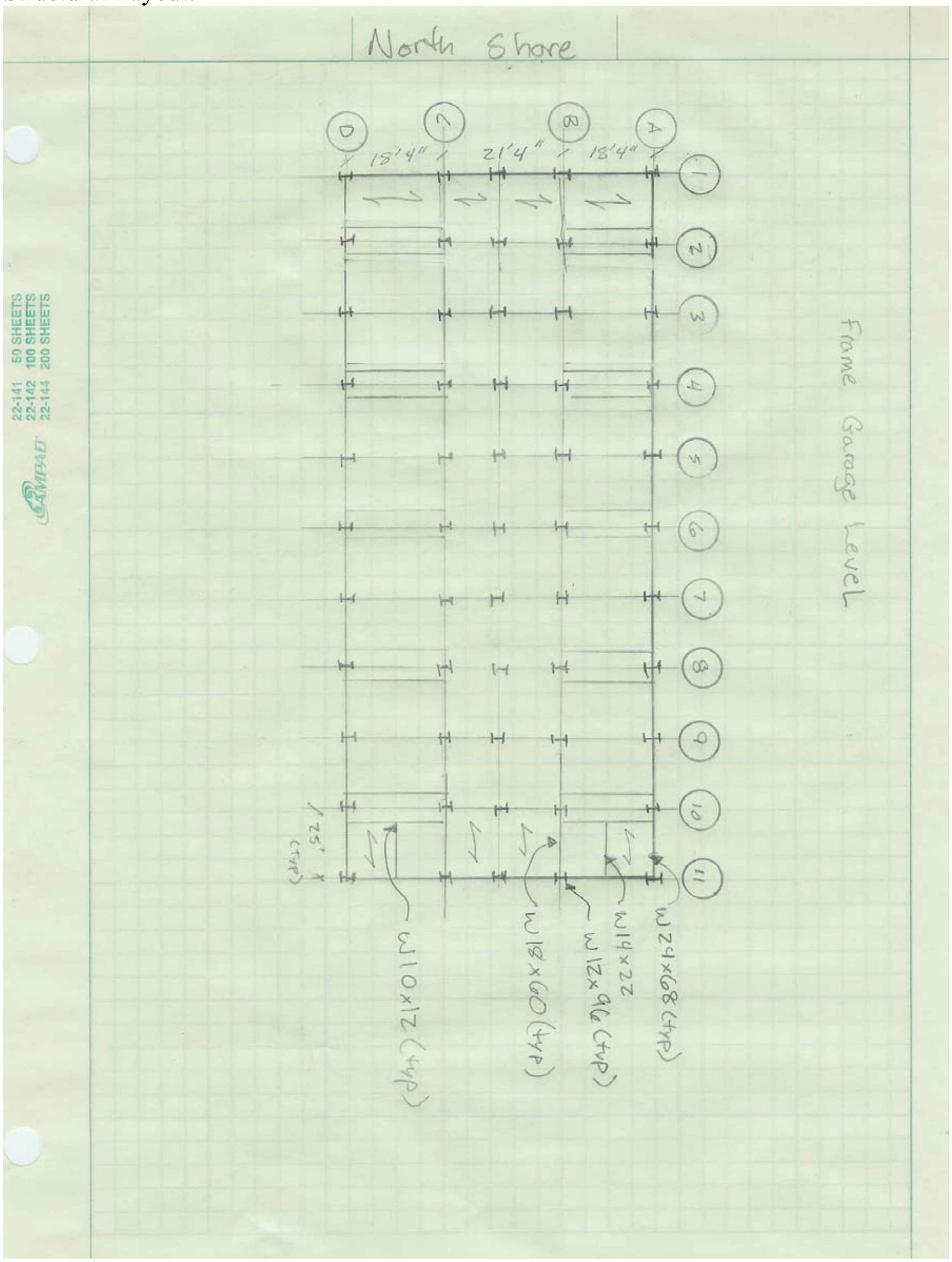
RS Means. Building Construction Cost Data, 2005. RS Means, 2004.

2003 International Building Code. Country Club Hills, IL. Copyright 2002.

Appendix A

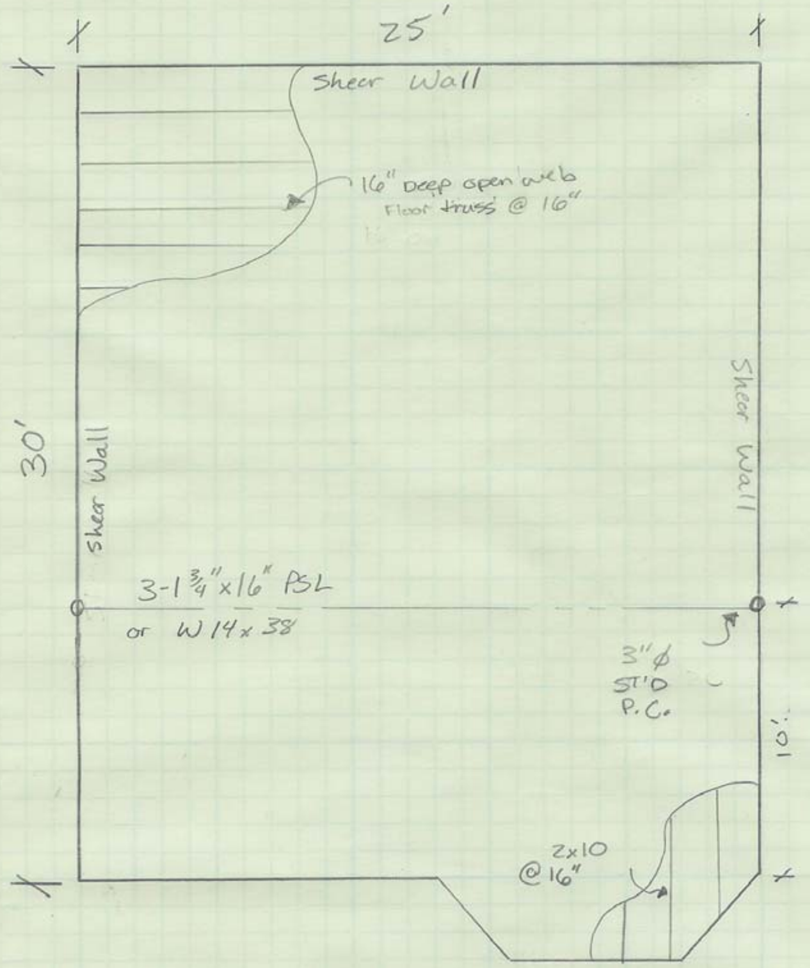
Current Structural System Details

Structural Layout:



North Shore

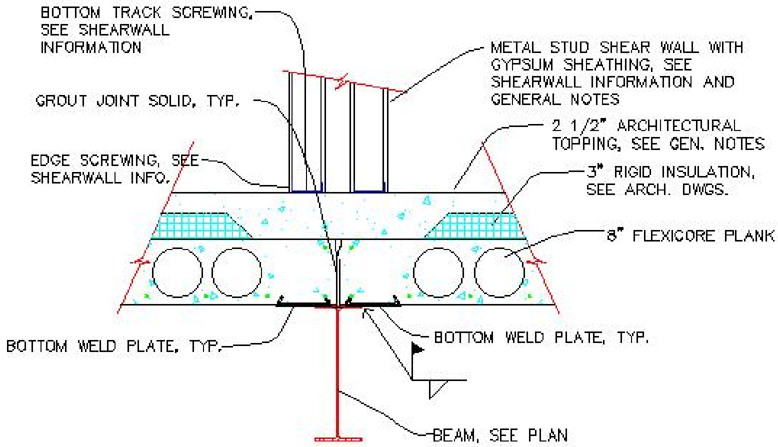
Typical Bay



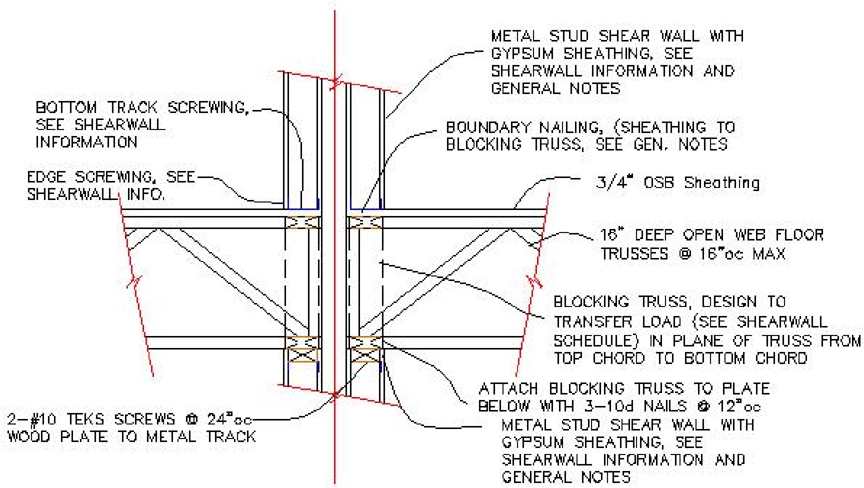
Typical Shear Wall

Sheathing	Studs	Chords	Sheathing Attachment	Chord anchor
1/2" Gypsum (Both sides)	JW16 @16" O.C.	2-JW16	#6 TEKS Screws @ 6" O.C. (edge) @ 12" OC (field)	2-CS22 Simpson Straps

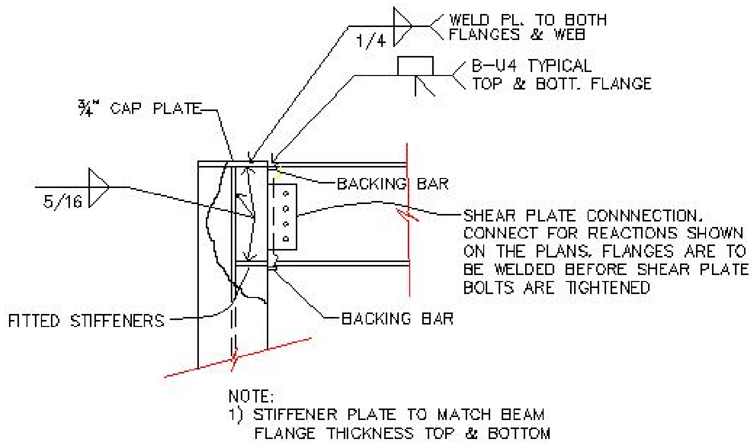
Shear truss design load ≃ 140 plf



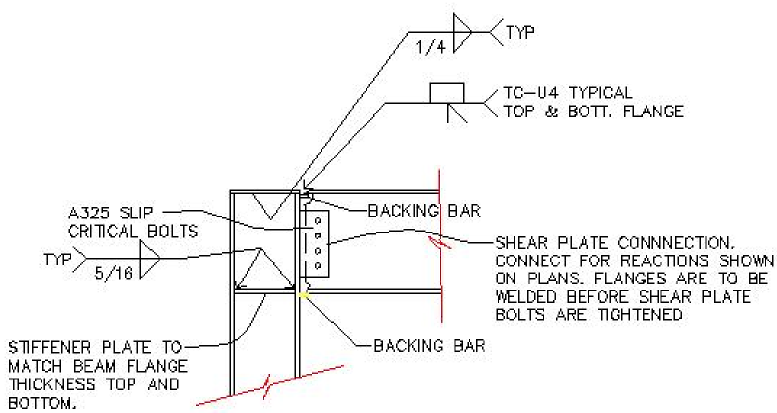
1st Floor
Floor Structure



2nd, 3rd, and 4th Floor
Floor Structure

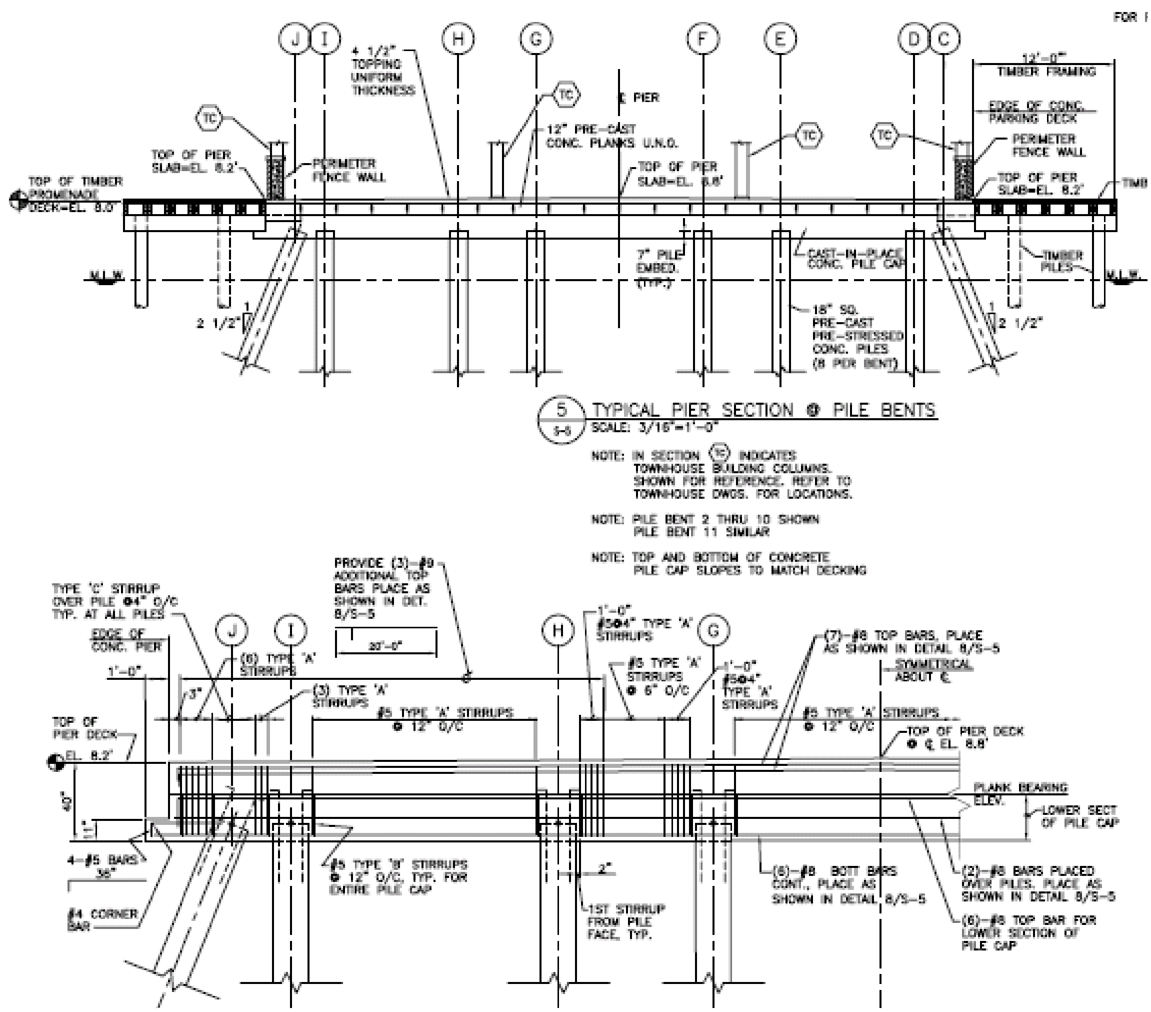


Typical Moment Connection 1st floor frame.
Perpendicular to column Web



Typical Moment Connection 1st floor frame.
Parallel to column Web

Pier structure, typical concrete bent.



Seismic Loading:

PROJECT TITLE :	North Shore at Canton	Date Designed
CLIENT :	Architectural Engineering Department	Date Checked
TITLE:	EQUIVALENT LATERAL FORCE METHOD	Page:
Designed By :	Beau Menard	
Checked By :		

REFERENCE	CALCULATION					OUTPUT
	1. Introduction					
ASCE 7-02	These calculation sheets serve to determine the Seismic Design Category and calculate Seismic Design Storey shear using the **Equivalent Lateral Force Procedure** as outlined in Section 9 of ASCE 7-02 "*Minimum Design Loads for Buildings and Other Structures*".					
	2. Seismic Design Parameters					
	Building Location :			**Scranton, Pennsylvania**		
	Number of Stories :	N		4		
	Inter-story Height	h_s		12 ft		
	Building Height :	h_n		48 ft		
Table 9.1.3 & Table 1.1	Seismic Use Group :	I		I	(Office)	
	Occupany Importance Factor :			1.00		
Table 9.4.1.2.1	Site Classification :			D	(Assumed stiff soil)	
Figure 9.4.1.1a	0.2s Acceleration :	S_S		0.18 g-s		
Figure 9.4.1.1b	1s Acceleration :	S_1		0.06 g-s		
Table 9.4.1.2.4a	Site Class Factor :	F_a		1.20		
Table 9.4.1.2.4b	Site Class Factor :	F_v		1.70		
	Adjusted Accelerations :	S_{MS}	$= F_a S_S$	0.216 g-s		
		S_{M1}	$= F_v S_1$	0.102 g-s		
	Design Spectral Response Accelerations :	S_{DS}	$= (2/3)S_{MS}$	0.144 g-s		S_{DS} = **0.144** g-s
		S_{D1}	$= (2/3)S_{M1}$	0.068 g-s		S_{D1} = **0.068** g-s
Table 9.4.2.1a & Table 9.4.2.1b	Seismic Design Category :			C	Equivalent Lateral Load Method can be used	Seismic Design Cateorgy is **C**
	3. Equivalent Lateral Force Procedure (9.5.3)					
	a. Seismic Base Shear Coefficient (9.5.3.2)					
	i. N-S Direction				Assumed light framed walls shear panels	
Table 9.5.2.2	Response Modification Factor :	R_{N-S}		2		R_{N-S} = **2**
Equation 9.5.3.2.1-1	Seismic Response Coefficient :	$C_{s, N-S}$	$= S_{DS}/(R_{N-S}/I)$	**0.072**		
Table 9.5.5.3.2		$C_{T, N-S}$		0.02		
Table 9.5.5.3.2		x		0.75		
	Approximate Period of Structure :	T_{N-S}	$= C_{T, N-S}h_n^x$	0.36		T_{N-S} = **0.36** s
	but Seismic Response Coefficient need not be					
Equation 9.5.3.2.1-2	greater than	$C_{S\ max, N-S}$	$S_{D1}/T(R_{N-S}/I)$	**0.093**		
Equation 9.5.3.2.1-3	and	$C_{S\ min}$	$= 0.044IS_{DS}$	**0.0063**		
	Therefore, the Seismic Response Coefficient ($C_{s, N-S}$) used is			**0.072**		$C_{s, N-S}$ = **0.093**
	ii. E-W Direction				Assumed light framed walls shear panels	
Table 9.5.2.2	Response Modification Factor :	R_{E-W}		2		R_{E-W} = **2**
Equation 9.5.3.2.1-1	Seismic Response Coefficient :	$C_{s, E-W}$	$= S_{DS}/(R_{E-W}/I)$	**0.072**		
Table 9.5.5.3.2		$C_{T, E-W}$		0.028		
Table 9.5.5.3.2		x		0.80		
	Approximate Period of Structure :	T_{E-W}	$= C_{T, E-W}h_n^x$	0.62		T_{E-W} = **0.62** s
	but Seismic Response Coefficient need not be					
Equation 9.5.3.2.1-2	greater than	$C_{S\ max, E-W}$	$S_{D1}/T(R_{E-W}/I)$	**0.055**		
Equation 9.5.3.2.1-3	and	$C_{S\ min}$	$= 0.044IS_{DS}$	**0.0063**		
	Therefore, the Seismic Response Coefficient ($C_{s, E-W}$) used is			**0.055**		$C_{s, E-W}$ = **0.055**
Table 2.2-1 *"Basic Loading Criteria"* of Design Report	**b. Loading Characteristics**					
	i. Roof :					
	Dead					
	Membrane			1.0 psf		
	Rigid Insulation			2.0 psf		
	Metal Roof Deck			2.0 psf		
	Roof Framing			20.0 psf		
	Drywall ceilding 0.5"			5.0 psf		
	M&E Services			5.0 psf		
	TOTAL q_{roof} :			**35** psf of roof area		q_{roof} = **35** psf
	ii. All other Floors :					
	Dead					
	Flooring			1.0 psf		
	Open Web Floor Joists			16.0 psf		
	3/2" OSB Sheathing			4.0 psf		
	Structural Steel Studs w/ 1/2" Gyp.Sheathing			10.0 psf		
	0.5" Drywall Ceiling			5.0 psf		
	M&E Services			5.0 psf		
	Live					
As required in 9.5.3.2	Moveable Partition			20.0 psf		
	TOTAL q_{floor} :			**61.0** psf of floor area		q_{floor} = **61.0** psf
	iii. Perimeter Wall:					
	Dead					
	Brick Curtain Wall, q_{wall} :			**10.0** psf		q_{wall} = **10.0** psf
	iv. Snow Load:					
	Snow					
	Snow, q_{snow} :			**20.0** psf		q_{snow} = **20.0** psf

Seismic Results:

	PROJECT TITLE : North Shore at Canton	Date Designed
	CLIENT : Architectural Engineering Department	Date Checked
	TITLE: EQUIVALENT LATERAL FORCE METHOD	Page:
	Designed By : Beau Menard	
	Checked By :	
REFERENCE	CALCULATION	OUTPUT

Building Width : W 60.0 ft
Building Length : L 250.0 ft
Gross Roof or Floor Area: $A = W \times L =$ 15,000.0 sq. ft

Total weight of roof, $w_{roof} = A \times (q_{roof} + q_{snow}) + 2(W + L)0.5h_x q_{wall}$ = 862 kips
Total weight of each floor, $w_{per\ floor} = A \times q_{floor} + 2(W + L)h_x q_{wall}$ = 989 kips
$w_{floors} = (N-1)w_{per\ floor}$ 2,968 kips

Total Building Weight, $W = w_{roof} + w_{floors}$ = 3,830 kips — OUTPUT: W = 3,830 kips

Equation 9.5.3.2-1 — Hence Seismic Base Shear, $V_{N\text{-}S} = C_{s,N\text{-}S}W$ = 276 kips — OUTPUT: $V_{N\text{-}S}$ = 276 kips
Hence Seismic Base Shear, $V_{E\text{-}W} = C_{s,E\text{-}W}W$ = 210 kips — OUTPUT: $V_{E\text{-}W}$ = 210 kips

c. Vertical Distribution of Seismic Forces (9.5.3.4)

The distribution of lateral forces over the height of the building is shown in Table 1 and 2 below.

Equation 9.5.3.4-2 — Exponent $k_{N\text{-}S} = 1 + (T_{N\text{-}S} - 0.5)/(2.5 - 0.5)$ = 0.932

Table 1 : Vertical Distribution of Seismic Forces (N-S)

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
	862	0	-	0.000	-		-
	989	0	-	0.000	-	-	-
	989	48	36,551	0.390	108	-	5,164
4	989	36	27,952	0.298	82	108	2,962
3	989	24	19,153	0.204	56	190	1,353
2	989	12	10,036	0.107	30	246	355
Ground 1						276	
	Σ = 5809		Σ = 93691	Σ = 1.000	Σ = 276		Σ = 9834

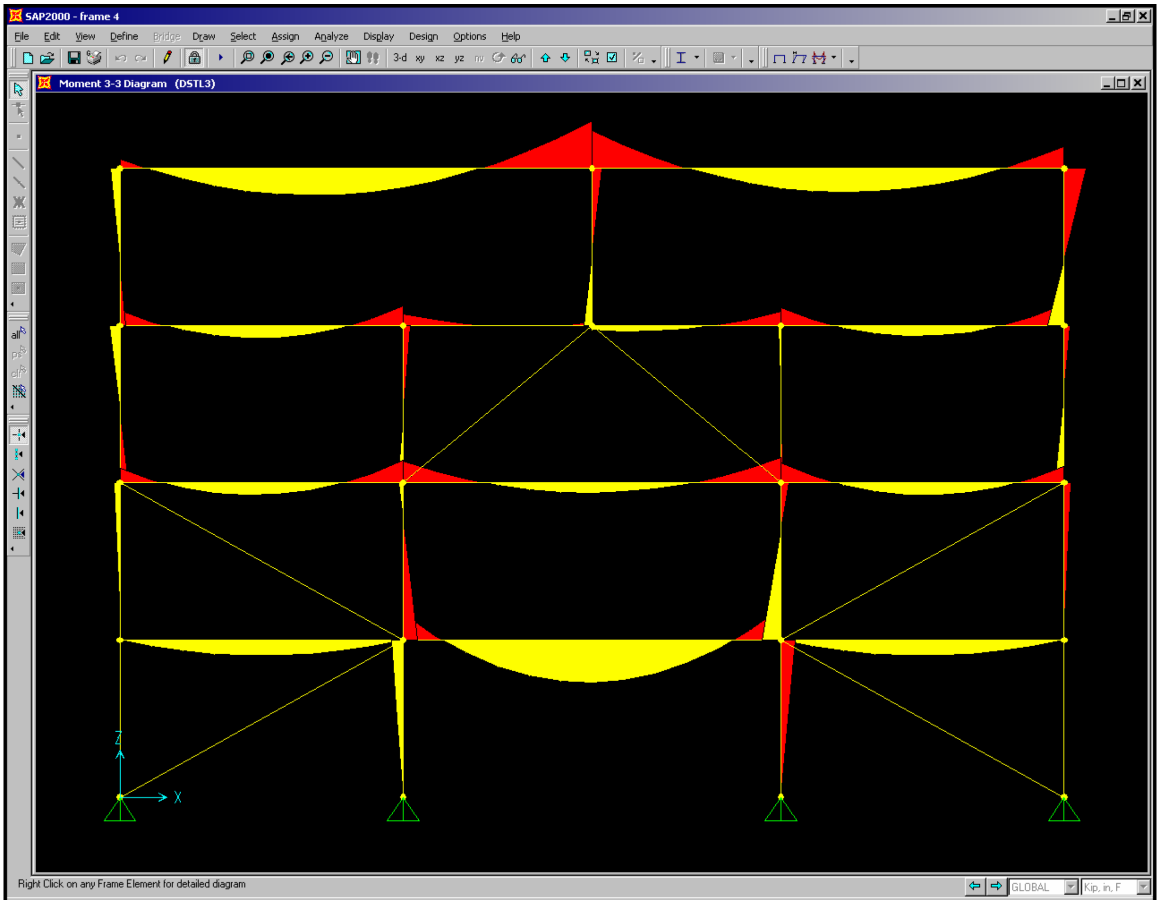
Exponent $k_{N\text{-}S} = 1 + (T_{N\text{-}S} - 0.5)/(2.5 - 0.5)$ = 1.060

Table 2 : Vertical Distribution of Seismic Forces (E-W)

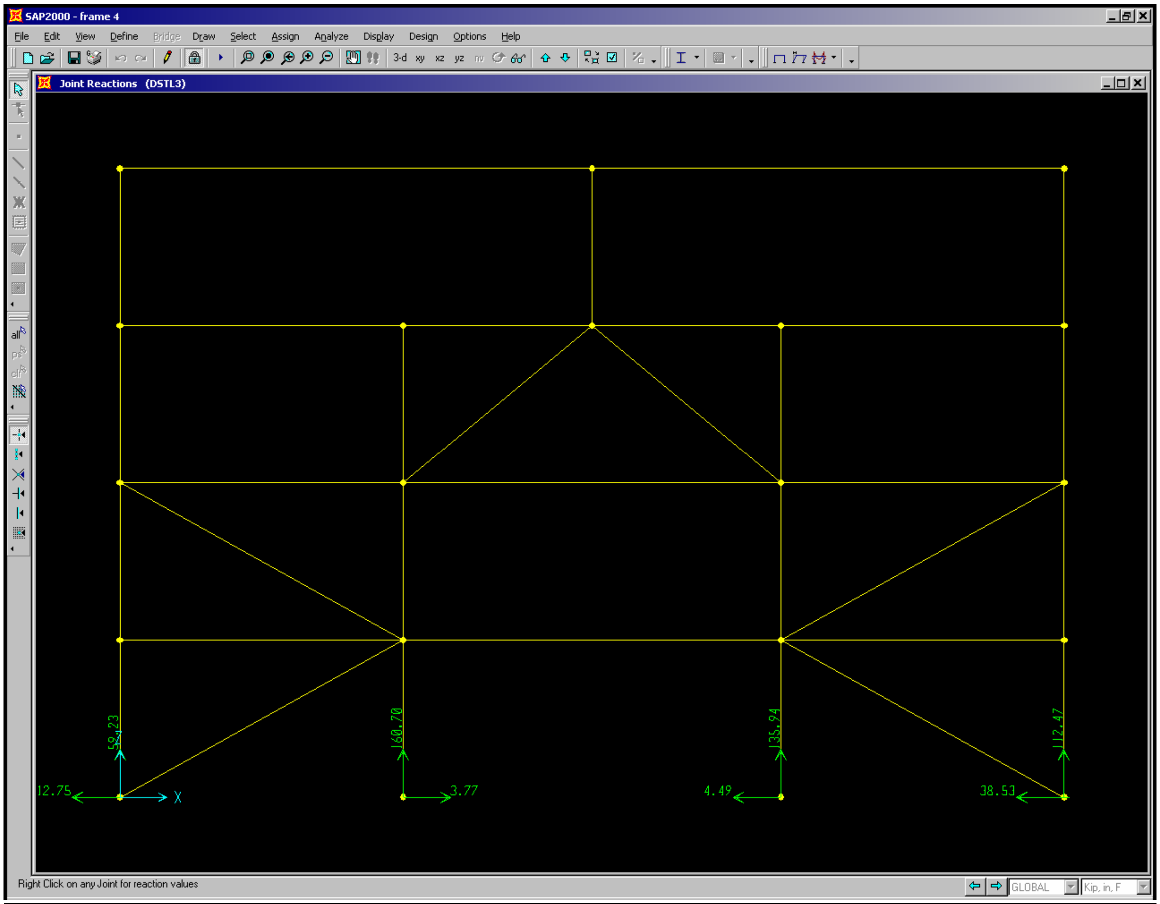
where $C_{vx} = w_x h_x^k / \Sigma_{all\ levels}(w_x h_x^k)$
$F_x = C_{vx}V$

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
	862	0	-	0.000	-		-
	989	0	-	0.000	-	-	-
	989	48	59,869	0.409	86	-	4,123
4	989	36	44,135	0.301	63	86	2,279
3	989	24	28,718	0.196	41	149	989
2	989	12	13,776	0.094	20	190	237
Ground 1						210	
	Σ = 5809		Σ = 146498	Σ = 1.000	Σ = 210		Σ = 7628

Steel Frame Moment Diagrams



Steel Frame Support Reactions



Material estimates

Structural Steel Members estimate

Member	Quantity	Length	Cost	Total Cost
		(ft)	($/ft)	($)
W 10x49	132	10	66	87120
W 16x26	32	18	35.5	20448
	11	24	35.5	9372
W 18x40	160	25	53.5	214000
W 18x35	5	24	47.5	5700
	10	18	47.5	8550
W 8x24	38	10	37	14060
	22	15.5	37	12617
	20	20.5	37	15170
W 21x44	12	18	50.38	10882.08
	6	24	50.38	7254.72
W 21x50	5	24	56.88	6825.6
W 24x55	10	30	61.71	18513
	6	24	61.71	8886.24
			Total =	439398.6

Metal Decking estimate

Member	Quantity	Area	Cost	Total Cost
		(ft^2)	($/ft^2)	($)
19 gauge Steel Decking		44000	1.64	72160

Steel Joist estimate

Member	Quantity	Length	Weight	Total Weight
		(ft)	(lbs/ft)	(lbs)
K-serie Joist	120	18	7	15120
	170	25	7	29750
	50	24	7	8400

Total Weight	26.635	(tons)
Cost	1775	($/ton)
Total cost	47277.125	($)

Concrete

Member	Quantity	Area	Cost	Total Cost
		(ft^2)	($/ft^2)	($)
Precast Walls	9	540	13.05	63423
8"	6	225	13.05	17617.5

Total Cost 81040.5

Member	Quantity	Area	Cost	Total Cost
		(ft^2)	($/ft^2)	($)
Precast Planks		44000	8.4	369600

Member	Quantity	Length	Cost	Total Cost
		(ft)	($/ft)	($)
Precast Column	95	9	102	87210

Member	Quantity	Length	Cost	Total Cost
Precast Beam		(ft)	($/Unit)	($)
Interior	24	60	3150	75600
Perimeter	20	50	3150	63000

Total Cost 138600