North Shore at Canton

Baltimore, MD

Final Report

Structural Depth

Beau Menard
Structural
Parfitt
4/05/06

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:

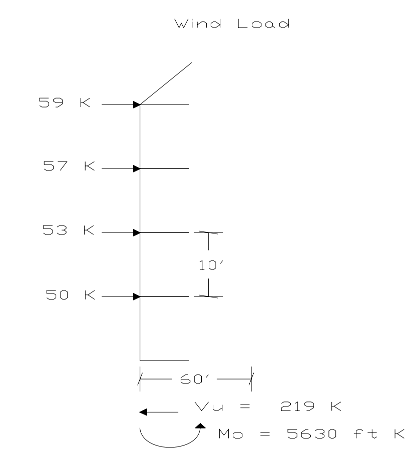
ASCE 7-02

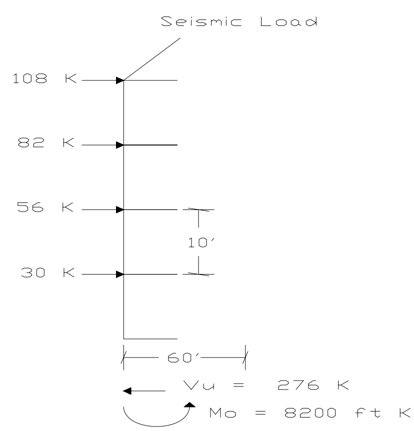
Precast hollow core concrete planks w/ 2" topping = 80 psf
Steel joist w/ metal decking & light weight concrete topping = 30 psf
Roof Trusses = 20psf
Misc. Roof = 15 psf
Structural Steel = (as noted on plans)

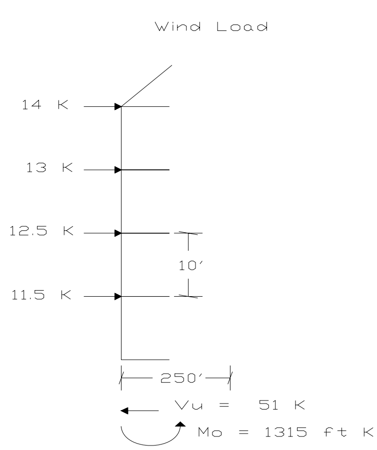
(assumed)
Mechanical = 10 psf
Electrical = 5 psf
Ceiling = 5 psf

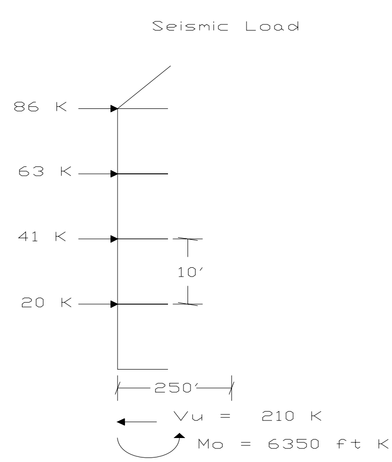
Lateral Loads:

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:

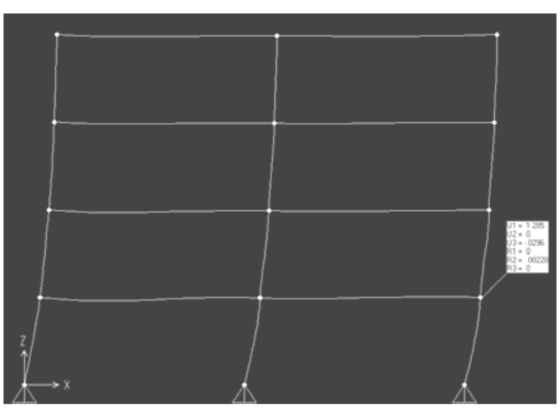
Steel frame

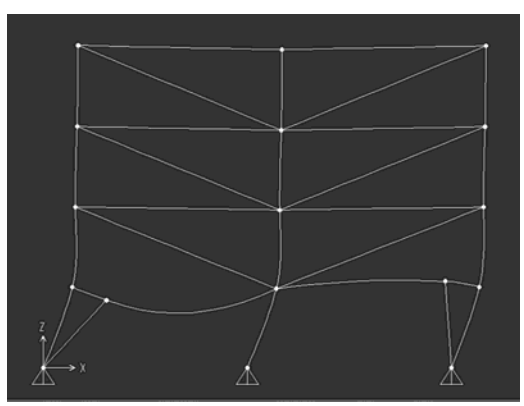
When this project started a comparison was going to be made between two types of steel frames, a moment frame and a braced frame. There would be five main frames used to resist the lateral forces that the building would be subjected to. The remaining six frames would be used to resist gravity loads and 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:

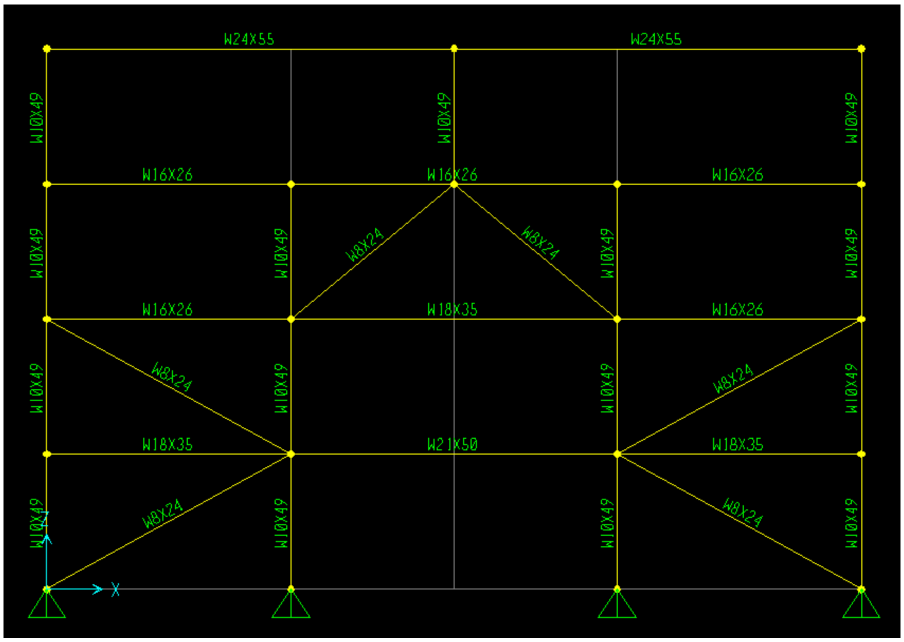
$$1.2\ D + 1.6\ W + 0.5\ L + 0.5\ S$$

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'.

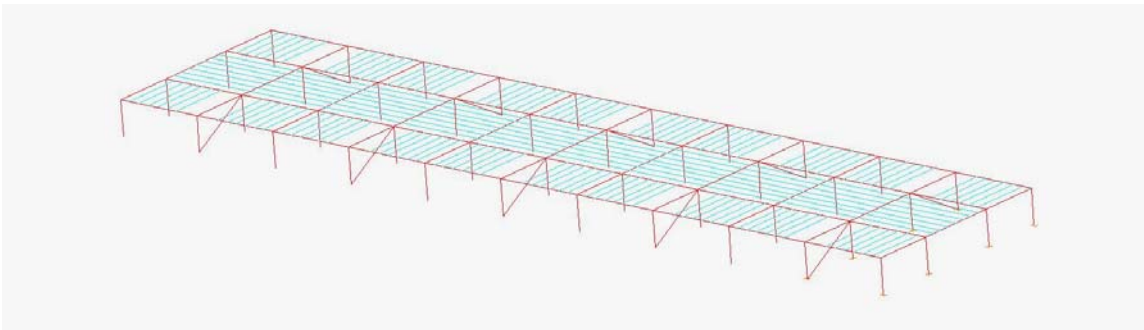


Member Sizing:

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:

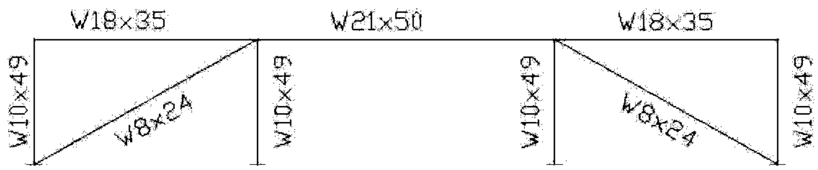


Table shows maximum design loads applied

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.

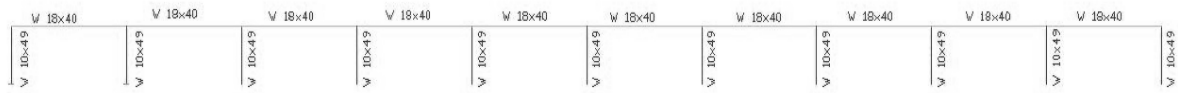
Steel Deck manual:

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:

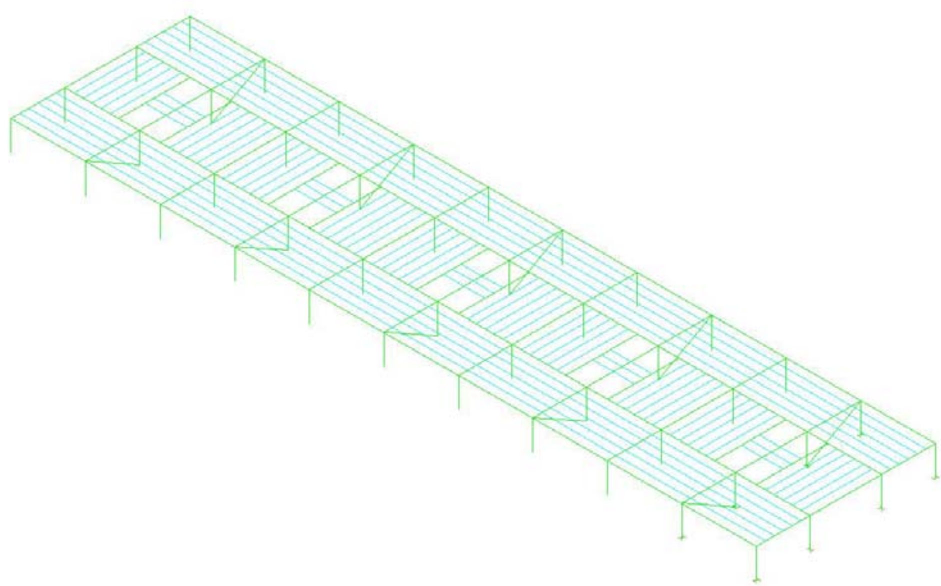
The frame in the long span direction of the building is to be considered to have full moment connections at the beam to column interfaces. The columns are oriented accordingly, so as to resist the moments transferred to them. The bases of the columns are modeled as pinned, so as to transfer as little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

Second 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'.

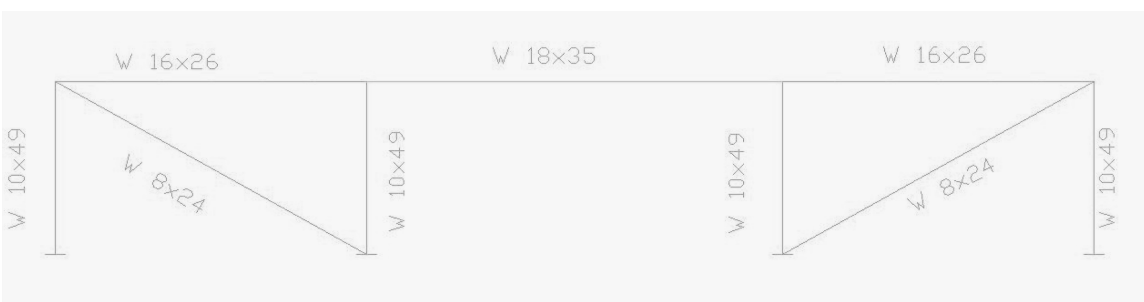
@ 3' spacing, and a span of 25'

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)	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.

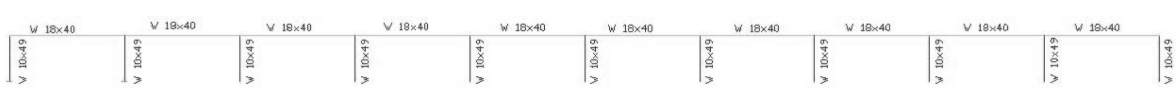
<u>Steel Deck manual:</u>

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:

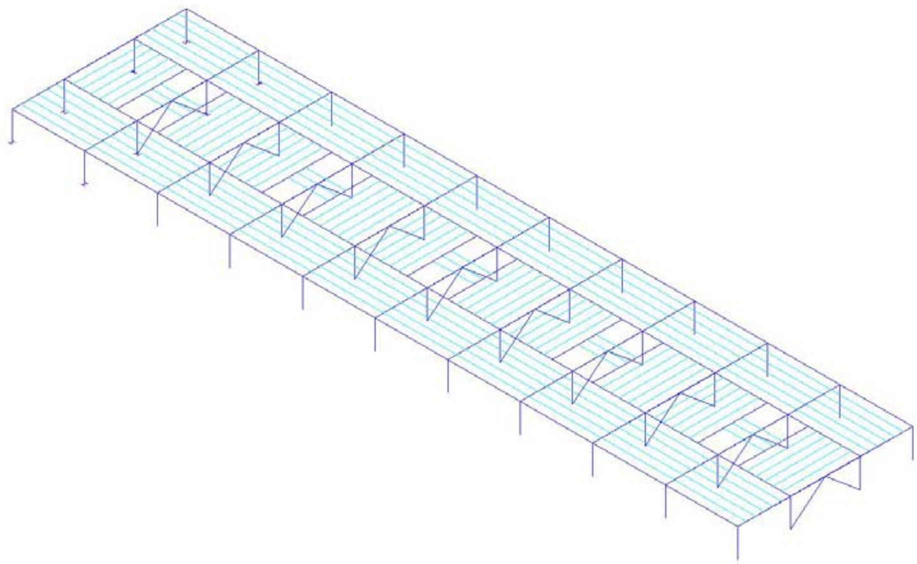
The frame in the long span direction of the building is to be considered to have full moment connections at the beam to column interfaces. The columns are oriented accordingly, so as to resist the moments transferred to them. The bases of the columns are modeled as pinned, so as to transfer as little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

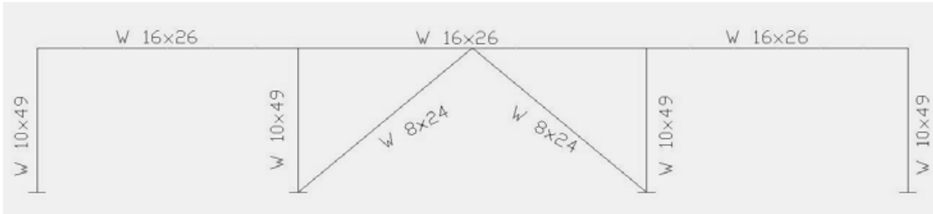
@ 3' spacing, and a span of 25'

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.

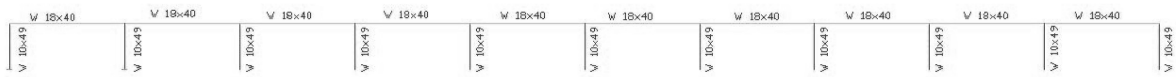
Steel Deck manual:

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:

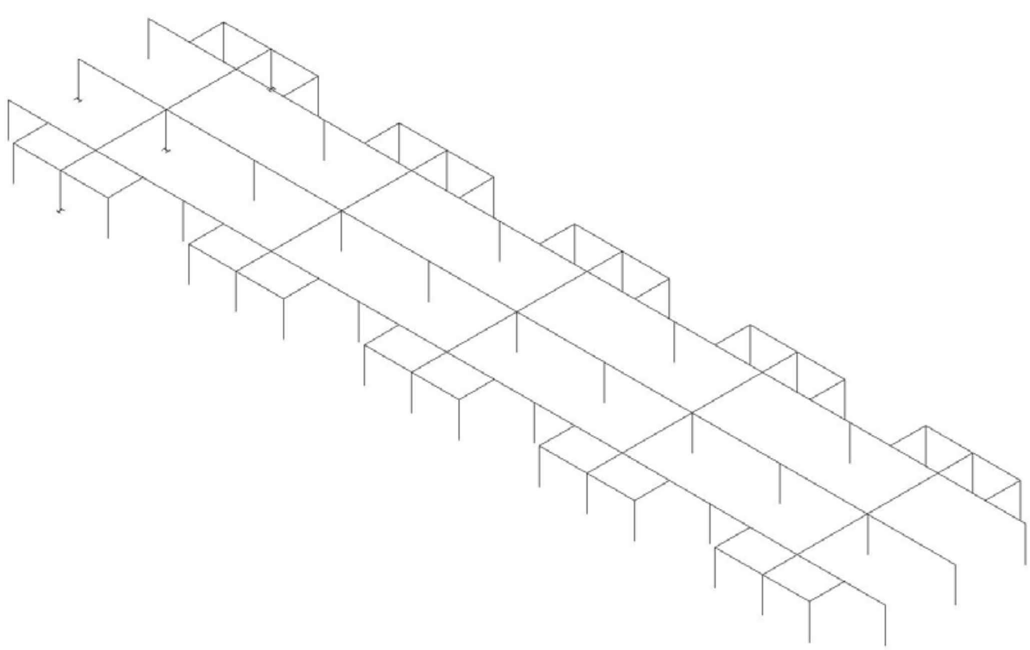
The frame in the long span direction of the building is to be considered to have full moment connections at the beam to column interfaces. The columns are oriented accordingly, so as to resist the moments transferred to them. The bases of the columns are modeled as pinned, so as to transfer as little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

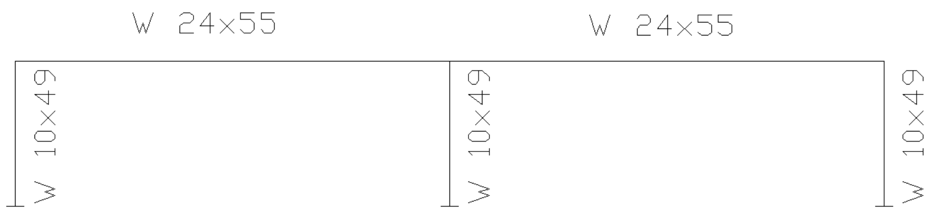
<u>Fourth Floor Design:</u>

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.

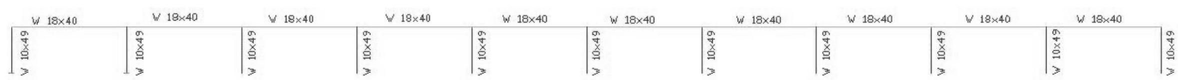
Table shows maximum design values

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.

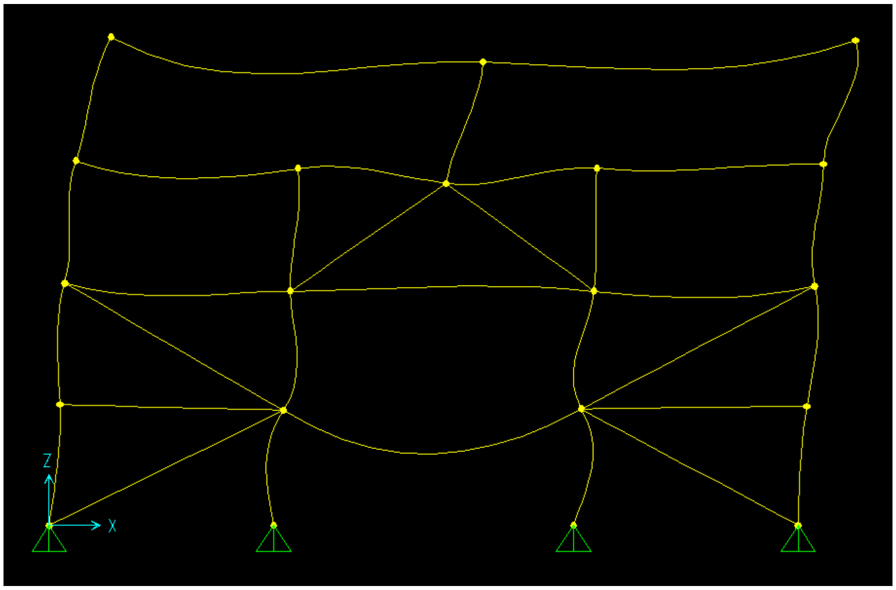
Long frame:

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:

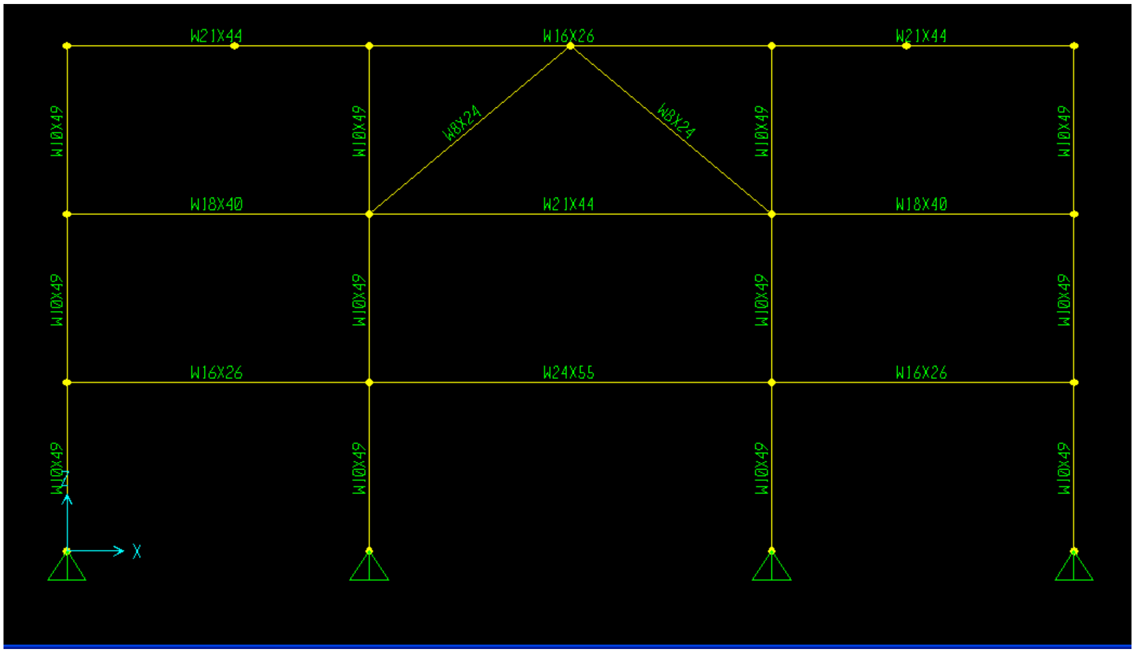


Story	**Total Lateral Displacement** (in)
1	.041
2	.079
3	.121
4	.276

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.

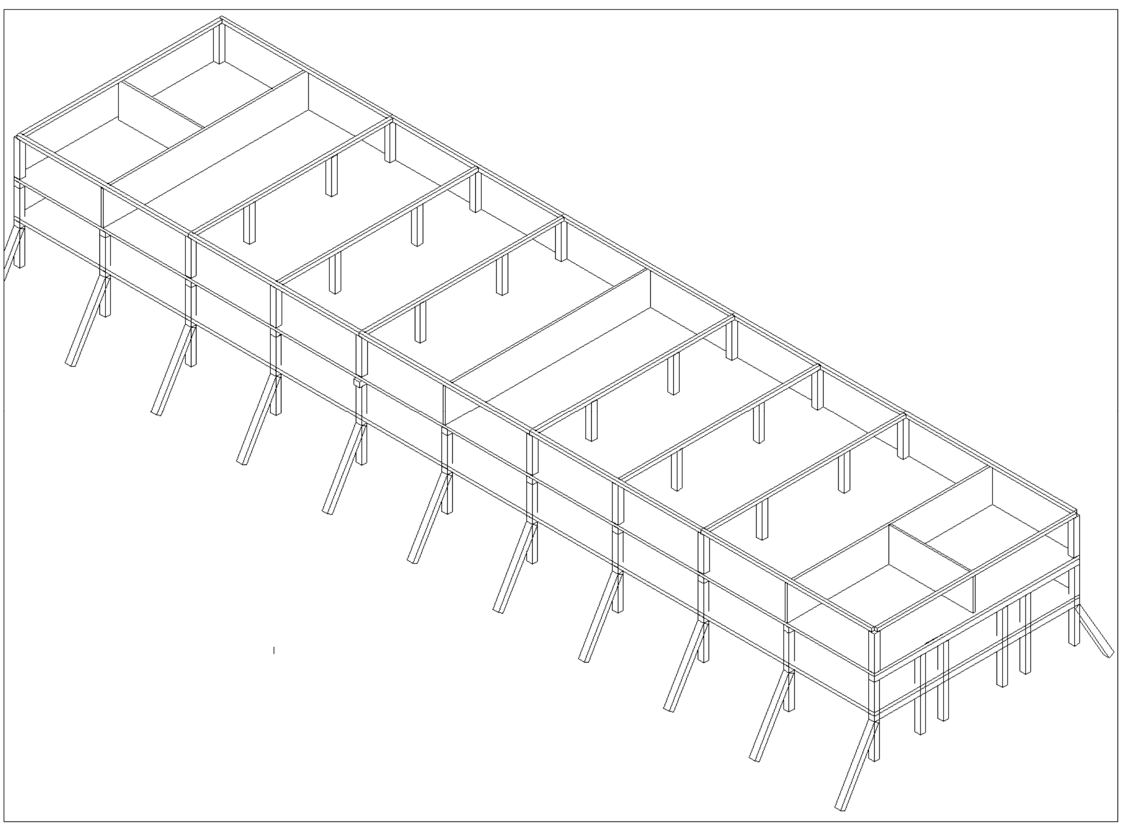
One issue that needs to be mentioned is that of moisture damage, since this structure is susceptible to high moisture content, it is only logical to try to prevent as much moisture damage as possible. One possible solution to this is to paint the exposed steel members; this would provide a fairly cost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