Final Report

Hakuna Resort

Swift Water, Pennsylvania

Image Courtesy of LMN Development LLC

Young Jeon
Structural Option
Advisor: Heather Sustersic
April 8th, 2015

Abstract

Hakuna Resort

Swiftwater, Pennsylvania

Project Team

Owner: LMN Development, LLC
Architect: Architectural Design Consultant
General Contractor: Kraemer Brothers, LLC
MEP/Structural: Harwood Engineering Consulants, LTD
Civil: Pennoni Associates, INC

General Building Data

Construction Dates: March 2014 - Summer of 2015
Building Cost: (Information Requested)
Delivery Method: Design Bid Build
Size: 395.938 SF

Images Courtesy of LMN Development, LLC

Architecture

At the corners of building, architectural finish will be done to resemble ancient stone. Also little more distinctive color finishes will be used at the top of hotel façade to give tribal character to the building. The interior designs are also jungle theme. Most of the furniture in hotel have bark surface finishes.

Structural

The main structural system used in this building is masonry shear walls and precast planks. There are also concrete piers, spread and strip footings, walls and masonry walls in the foundation and steel framing system in areas that require more flexible open spaces. The roof system is also precast hollow core planks.

Lighting and Electrical

The public area lighting with occupancy sensors is a florescent lighting system Compact florescent down lights and line voltage halogen continuous run wall graze luminaries to provide uniform grazing on the vertical surface in the corridor. The primary feed is 208Y/120V system for general use and 480Y/277 for lighting.

Mechanical

The AHU operate with a Variable Air Volume System that has hook ups in the primary spaces, allowing the end user to monitor each space. A building automation system is provided to monitor various mechanical points throughout the building.

Young Jeon - Structural Option www.engr.psu.edu/ae/thesis/portfolios/2015/ybj5001/index.html

Executive summary

Hakuna Resort is a Savanna Desert themed hotel that includes a 217,703 square feet indoor water park as well as outdoor pool. The other side of the resort is convention centers which provides multiple meeting spaces. Divided into three distinctive spaces, the hotel is in between the indoor water park and convention space. These spaces are connected with expansion joints, therefore, can be looked at as three separate buildings.

The hotel building has total of eight stories above ground with total height of 101'-5" to the top of roof excluding the basement. With each floor having approximately 45,000 SF, the hotel portion of the resort has 395,938 SF by itself. Due to the shape of the building, which is very long and narrow, the hotel structure is further divided by another expansion joint. The scope of this thesis project is limited to the smaller hotel portion of the site which is rectangular geometry with dimensions of 66' – 8" by 236' – 6".

Taking the advantage of the repetitive and typical hotel room floor layout, the original design had chosen load bearing masonry shear wall with hollow core plank flooring system as its primary gravity and lateral system. This system is redesigned with new system called staggered truss framing system. This report contains the redesign calculation and process.

With the incorporation of the new system as structural system, architectural breadth study is also included in this report. In architectural breadth study, the rearrangement of first and second floor layout will be discussed. Also new fa çade design is included to help the building to be more exciting to the targeted occupants when first encountered. The material for the new fa çade design was kept the same as the original, exterior insulation finish system, but with different color.

With the change in structural system, the construction management data was evaluated in this report. In construction breadth study, cost and schedule differences was compared to the original design of load bearing masonry shear wall. While staggered truss system is adequate alternative structural system, it showed a significant increase in cost. However, the construction schedule is decreased slightly.

In conclusion, the staggered truss framing system is a valid alternative structural system for Hakuna Resort's hotel structure. However, while it reduces the construction schedule slightly, the cost increase is significant. Therefore, the redesign is not recommended but was a meaningful research experience.

Credits and Acknowledgements

I would like to thank the following people for helping me to complete this report.

- The engineers at Nitterhouse Concrete Products, especially Jonathan Kirk who supported and assisted me with the thesis project decision process and also provided meaningful advices throughout last summer and this academic year.
- Mark Taylor, also from Nitterhouse Concrete Products, for helping me to get the approval from the owner representative for this thesis project
- Dale Hensen, the construction manager from Kalahari Resort for approving me to use this project as my thesis topic
- Professor Heather Sustersic for providing very helpful advices throughout the academic year to improve this thesis project
- AE classmates for being great peers
- My friends who encouraged me
- My family for supporting, encouraging me and praying for me every time I face hardships

Documents Used to Create This Report

Masonry Standards Joint Committee

- Building Code Requirements and Specification for Masonry Structures
    - Building Code Requirements for Masonry Structures
        - TMS 402-11 / ACI 530-11 / ASCE 5-11
    - Specification for Masonry Structures
        - TMS 602-11 / ACI 530.1-11 / ASCE 6-11

Concrete Masonry Association of California and Nevada

- 2009 Design of Reinforced Masonry Structures

American Concrete Institute

- ACI 318-08 – Building Code Requirements for Structural Concrete and Commentary

American Institute of Steel Construction

- Steel Construction Manual 14th Edition
- Steel Design Guide Series 14 – Staggered Truss Framing System

Hakuna Resort Construction Documents

- Architectural and Structural Sets

Table of Contents

Final Report 1
Abstract 2
Executive summary 3
Credits and Acknowledgements 4
Documents Used to Create This Report 5
Table of Contents 6
Building General Information 8
Existing Structural System Overview 10
  Foundation 10
  Floor Systems 12
  Lateral Load Resisting Elements 13
  Framing System 15
    Typical Bay 15
  Columns 17
  Roofing System 18
  Joint Details 18
Proposal Statement 20
  Proposed Solution 20
  Breadth Studies 20
    Architecture 20
    Construction Management 20
Structural Depth 21
  Staggered Truss Background 21
  Truss Design 22
    Truss Layout 22
    Floor System 23
    Truss Members 23
    Computer Modeling 23
    Diagonal Members 24

Truss Chords ... 24
Columns ... 25
Deflections ... 25
Architectural Breadth Study ... 27
Floorplan Redesign ... 27
Façade Redesign ... 28
Construction Management Breadth Study ... 31
Conclusion ... 32
Appendices ... 33
Appendix A ... 34
A.1 Hand Calculations ... 34
A.2 Typical Floor Axial Load Calculation ... 38
A.3 Wind Load Calculation ... 39
A.4 Seismic Load Calculation ... 40
A.5 Truss Chord Design Values ... 41
A.6 Diagonal Member Design Values ... 41
A.7 Column Member Design Values ... 42
Appendix B ... 43
B.1 Original Floorplans ... 43
B.2 Redesigned Floorplans ... 44
Appendix C ... 45
C.1 Existing Load Bearing Masonry Shear Wall Cost & Schedule Estimate ... 45
C.2 Redesigned Staggered Truss System Cost & Schedule Estimate ... 46
C.3 Façade Redesign Cost & Schedule Estimate ... 47

Building General Information

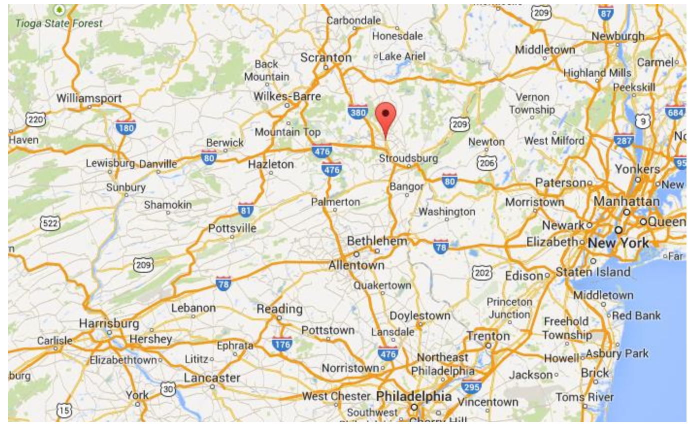
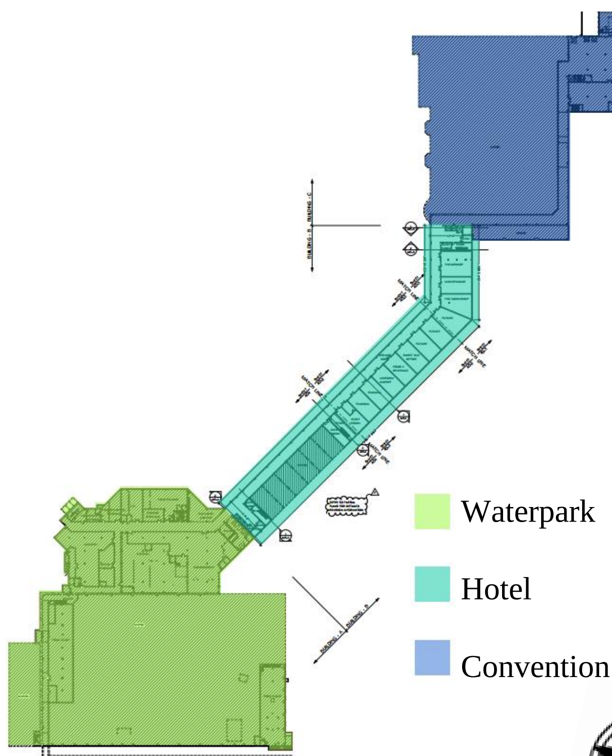


Figure 1 Project Location: Swiftwater, PA

Located in Shiftwater, Pennsylvania, Hakuna Resort is a jungle theme resort which includes both indoor waterpark and outdoor pool as well as convention centers while providing luxury hotel space. The indoor waterpark, located north-west to the hotel, has square footage of 143,798 SF in first floor and 73,905 SF in second. As can be seen in figure 1, the convention center is located the opposite, south-east side of the hotel. With basement space of 18,802 SF, the convention center has first floor space of 92,668 SF. The biggest space, however, is the hotel with total of 394,938 SF distributed throughout eight stories and a basement. For this project, only highlighted portion of the hotel with total area of 143,107 S.F. is to be analyzed in the figure below as it is also connected with another expansion joint.



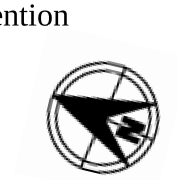


Figure 2 Project Floor Layout

Started constructing in March 2014, Hakuna Resort is to be completed and be open to public in summer of 2015. The project is also looking ahead for potential of three additions in the future (figure 2). The hotel, tallest part of the project, is 101’-5” tall and has the most visual impact when confronted to the site.

The fa çade of hotel building has color tone of brown, red, and grey to give earth-like feeling. Custom ancient stone architectural finishes, applied at the corners of the building, will keep the consistency of tribal jungle theme fa çade finishes. Also little more distinctive color finishes will be used at the top of hotel fa çade to give tribal character to the building. The interior designs are also jungle theme. Most of the furniture in hotel have bark surface finishes.

The floor plan layout is very simple in hotel building. Most of the hotel rooms are identical in plan, repeated in a regular array at each floor level. The rooms facing southern side of building has balconies and northern side does not. Also, the rooms at the angled middle corner section and all rooms in the top floor have bigger suite.

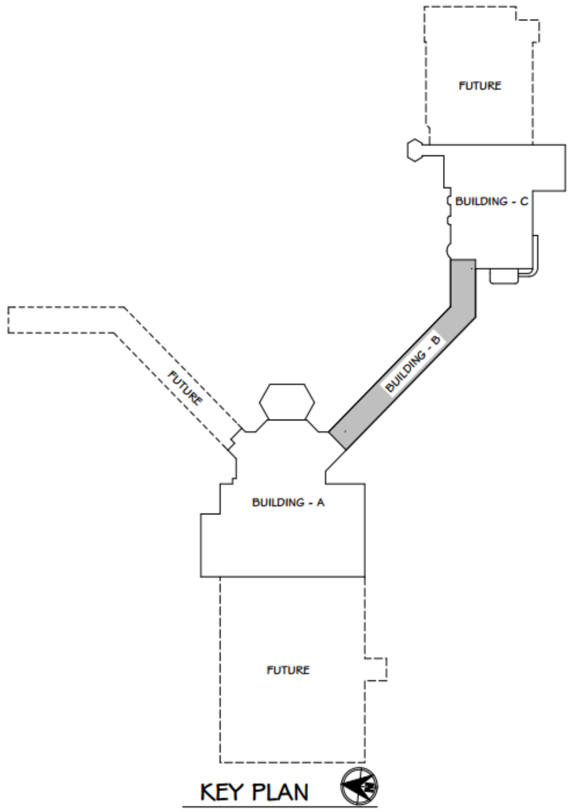


Figure 3 Project Future Additions

Figure 4 Hotel Building Rendering (looking from south)

Existing Structural System Overview

Hakuna Resort is composed with three major components: indoor waterpark, hotel, and convention center. These components are connected by expansion joints, which allows each section to be considered as separate independent buildings. As stated before, only the hotel building will be described in this report due to its size. The main structural system used in this building is masonry shear walls and precast planks. There are also concrete piers, spread and strip footings, walls and masonry walls in the foundation and steel framing system in areas that require more flexible open spaces. The roof system is also precast hollow core planks.

Foundation

The foundation of Hakuna Resort has spread and strip footings or varying sizes to support concrete columns, exterior walls, steel columns and concrete shear walls. According to the geotechnical report done by Pennoni Associates Inc., “spread footing foundations is feasible in dense natural soils, weathered rock or compacted load-bearing fill.” Both spread and strip footings have allowable bearing pressure of 4,000 and 6,000 psi with varying steel reinforcements.

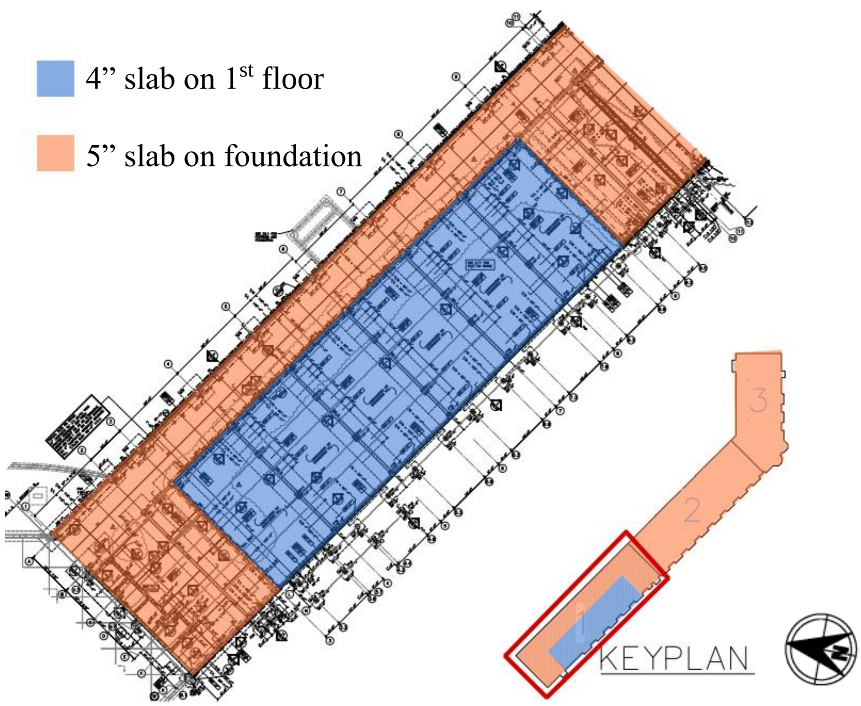


Figure 5 Partial Foundation Plan (S0.1)

For floor slabs, the geotechnical report approved using slab on grade with the usage of 4 inches thick layer of granular, free draining aggregate base course directly below the bottom of the slabs to provide a uniform bearing surface and improve overall slab performance. Figure 5 illustrates areas where 4" or 5" slab on grade is used.

A typical section of strip footings supporting the 1' wide concrete shear walls is shown in figure 6. Because these footings are supporting the lateral resisting system, their thickness range from 2' to 3'-6" whereas the strip footings of exterior walls are below 2'. The width of footings for

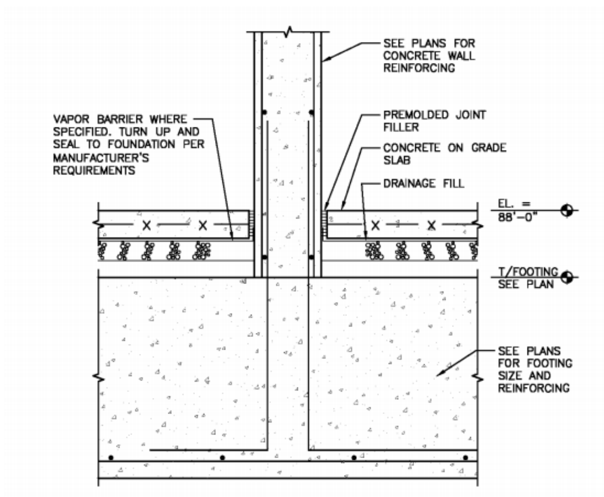


Figure 6 Concrete Wall Footing Section (S12.01, Drawing 14)

shear walls are also 12'-6" wide compared to exterior wall strip footing width, 2'-6". Similarly, the spread footings supporting concrete columns and steel columns are shown below in figure 7 and 8.

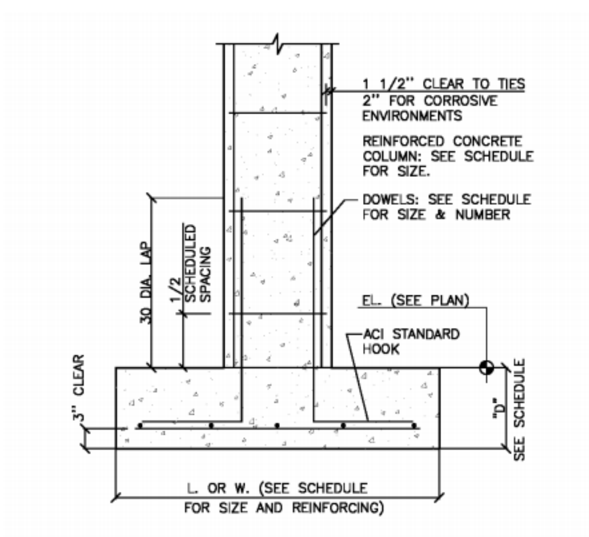


Figure 7 Typical Concrete Column Footing (S12.00 Drawing 10)

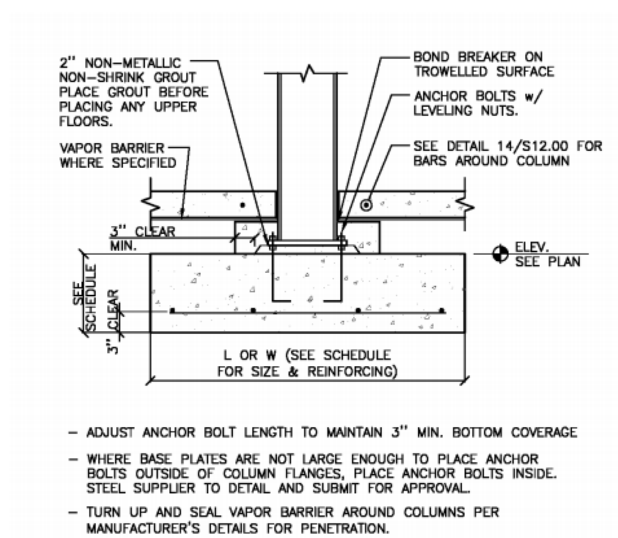


Figure 8 Steel Column on Footing (S12.00 Drawing 16)

Floor Systems

Hakuna Resort's main floor system is prestressed precast hollow core planks. The hotel is a very narrow rectangular building with slight turn at the south-east end. The north-west side is about 501'-6" by 69' and south-east is 151'-6" by 69'. Having precast planks spanning long direction allowed usage of load bearing walls in the other direction. This is a very effective choice of system while utilizing the architectural layout of hotel. Because the floor layout is repetitive with identical hotel rooms next to one another, putting loadbearing walls in between the rooms to support the precast planks is efficient approach.

There are two different thickness of precast planks. As shown in figure 9, there are 10" and 12" thick precast planks. 10" thick planks have six prestressed strands and are used throughout the building typically spanning 28'. The 12" thick planks, which also uses six strands, are only placed at the 45 °corner highlighted in orange in figure 9 below. At this location, bigger suites that have maximum span of 40' were designed. The balcony is also precast but solid plank that is 1'-½" thick which is supported by 1' x 1' precast columns at each exterior corner.

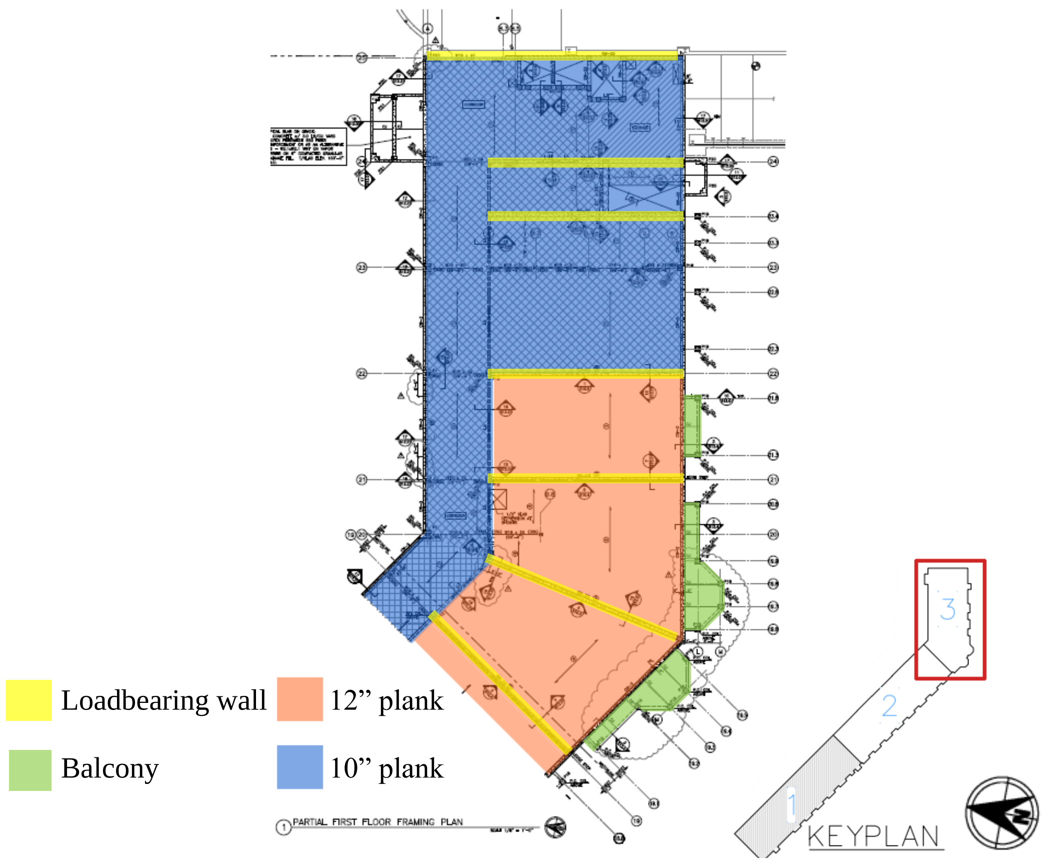


Figure 9 Partial First Floor Plan (S1.3)

Lateral Load Resisting Elements

The main lateral force resisting system for Hakuna Resort consists of solid grouted 12” thick masonry walls. These concrete masonry units are structured to have masonry piers at each ends and sometimes in the middle as well instead of steel columns. The masonry pier schedule can be found in figure 11. The blocks have F’m of 2000 psi which requires a net area compressive strength of 2800 psi and grouted with 3000 psi grout. The typical layout of masonry shear walls can be found in figure 10.

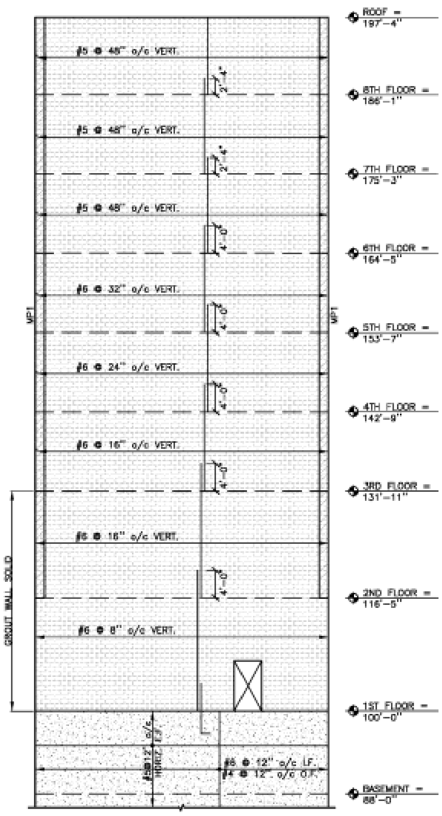
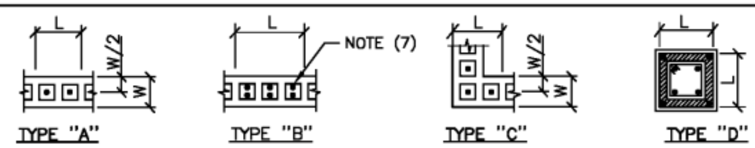


Figure 10 Masonry Shear Wall (S10.3 Drawing 2)

MASONRY PIER SCHEDULE



MARK	TYPE	SIZE (L) IN INCHES	VERTICAL REINFORCMENT (NOTE 2)	TIES	REMARKS
MP1	"A"	16	(2) – #5 BARS		
MP2	"A"	24	(3) – #5 BARS		
MP3	"A"	32	(4) – #5 BARS		
MP4	"A"	16	(2) – #5 BARS	NOTE (3), (8) #2 @ 8" o/c	
MP5	"A"	24	(3) – #5 BARS	NOTE (3), (8) #2 @ 8" o/c	
MP6	"A"	48	(6) – #5 BARS	NOTE (3), (8) #2 @ 8" o/c	
MP7	"C"	16 x 16	(3) – #5 BARS		
MP8	"C"	24 x 24	(5) – #5 BARS		
MP9	"A"	40	(5) – #5 BARS	NOTE (3), (8) #2 @ 8" o/c	
MP10	"D"	16 x 16	(4) – #5 BARS	NOTE (6) #3 TIES @ 16" o/c	

Figure 11 Masonry Pier Schedule (S13.3)

The size of vertical reinforcement for the masonry shear walls vary from #5 to #8. The spacing of the reinforcements also vary from 8” to 48” o.c. as the placement of reinforcing become higher in elevation. #5 bars, which is used the most throughout the shear walls, have 2’-4” of splice and #6 bars have 4’-0” splice.

Another lateral force resisting system is reinforced concrete shear walls that erect from the foundation and up to first and second level of the hotel structure. Varying from 12” to 14” thick, the concrete shear walls are vertically reinforced in two curtains with #5 or #6 for walls from basement to first floor and #7 for walls from basement to second floor with varying spacing from 12” to 16” o.c. The horizontal reinforcement uses #5 or #6 bars both at 10” o.c. spacing.

The last lateral force resisting system is steel moment frame. Due to the demand and purpose of certain spaces that require spacious area, reinforced concrete and masonry shear walls were not adequate. Therefore, to remove the abruptness of blocking space from solid shear walls, steel moment frames were chosen. Due to this transition, the load from the masonry shear wall will transfer to the moment frame, which will have an impact on the lateral system analysis. The spaces which required these moment frames are the theme shop located in the basement level, service area such as reception, massage, relaxation rooms on second floor, and deluxe suite located on eighth floor.

The most influential space out of these three is the service area. While the other two spaces only require moment frame that replaces half of shear walls in one grid line, the service area has entire gridline to have moment frame as illustrated in figure 12. The frame uses smallest beam of W27x102 to biggest size of W36x330. The columns of the moment frame vary from W12x65 to W14x120.

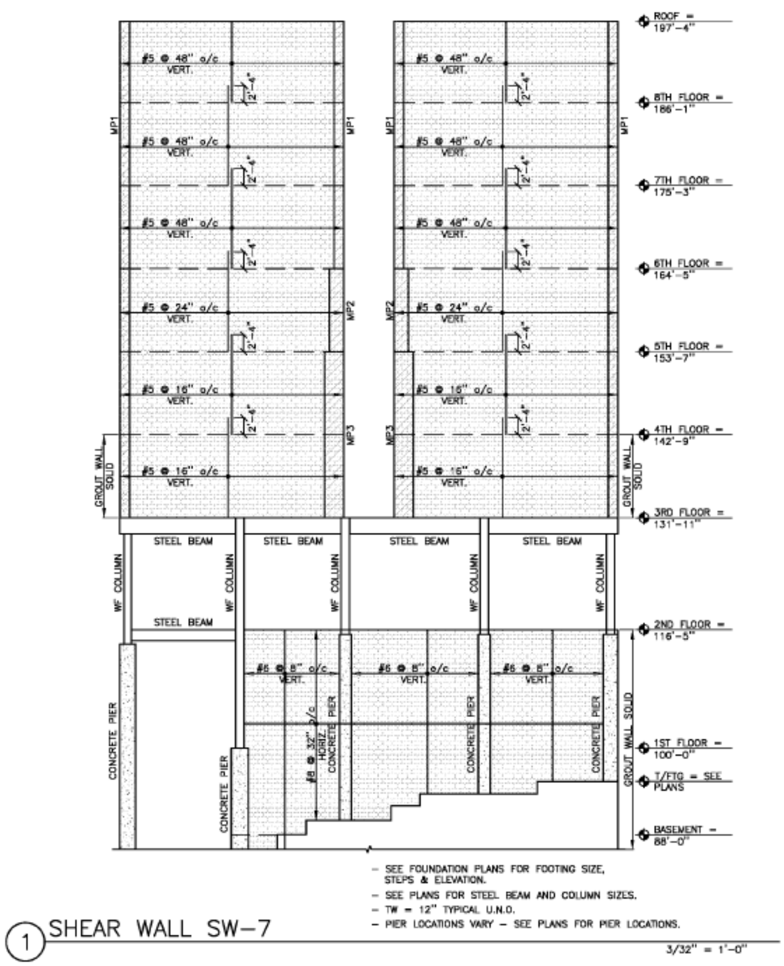


Figure 12 Shear Wall with Steel Moment Frames (S10.2 Drawing 1)

Framing System

As described above, the structure is mostly comprised of 10” or 12” precast plank supported by masonry loadbearing shear walls oriented in one direction. The shear walls use 12x8x16 blocks fully grouted. While this framing system is dominantly present in this project, there are steel moment frame systems in some portion of the structure as described above section of this report.

Typical Bay

The most replicated typical bay can be found in fourth floor layout, figure 13. This 67’ by 28’ bay is used from fourth floor to eighth floor. Due to precast planks forming stable frame system with masonry shear walls only in one direction, any need of beam spanning in the direction that is perpendicular of shear walls was eliminated; therefore, resulting such large typical bay.

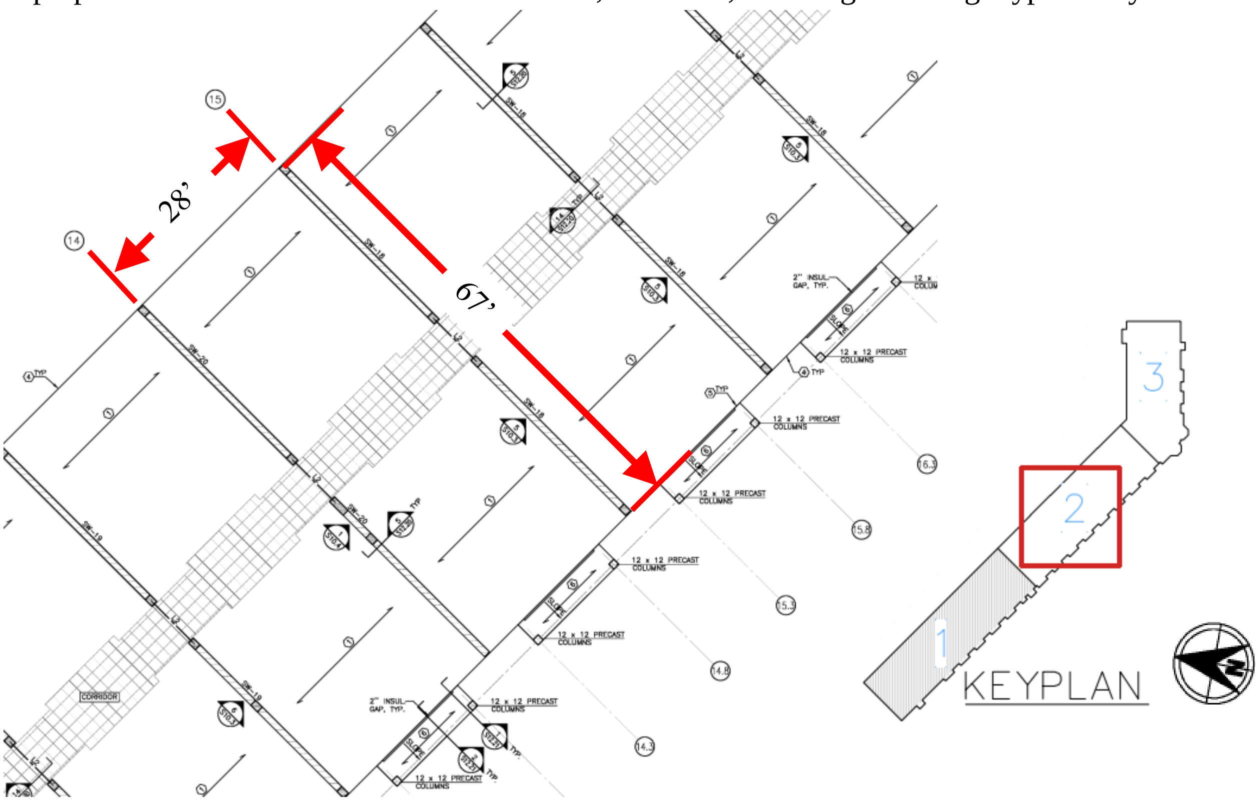


Figure 13 Typical Bay of Fourth Floor Plan (S4.2)

The 12” fully grouted masonry loadbearing shear walls with vertical reinforcement size of #5 with varying spacing per level are supporting 10” prestressed precast hollow core planks with 3” topping and bearing of 5.5”. These planks have 1 hour fire rating.

To leave the opening for the corridor but to not disrupt supporting planks, lintel system which consists of HHS 10x4x3/8 and steel plate of 1/2” deep and 12” wide is placed in between the two shear walls adjacent to the corridor, bearing 4” into the shear walls. As shown in figure 14, this lintel allows the precast planks to be supported, leaving an opening beneath.

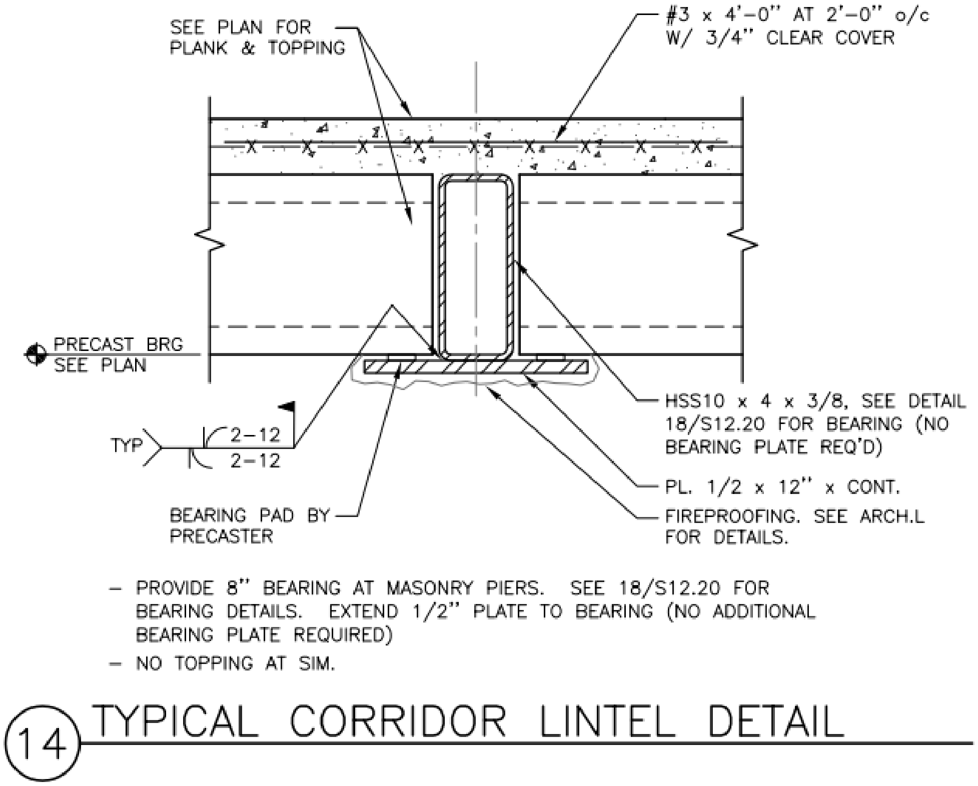


Figure 14 Typical Corridor Lintel Detail (S12.20)

Columns

Concrete piers were majorly used in basement and first level only where steel columns are located in order to support them. These concrete piers are in great number of various sizes. It ranges from a maximum size of 2' by 3'-4" to a minimum size of 16" by 16", shown below in figure 15. The steel columns that sits on top of concrete pier or right above foundation slab on grade have great number of varieties as well. To a minimum size of W10x49 to maximum of W14x120.

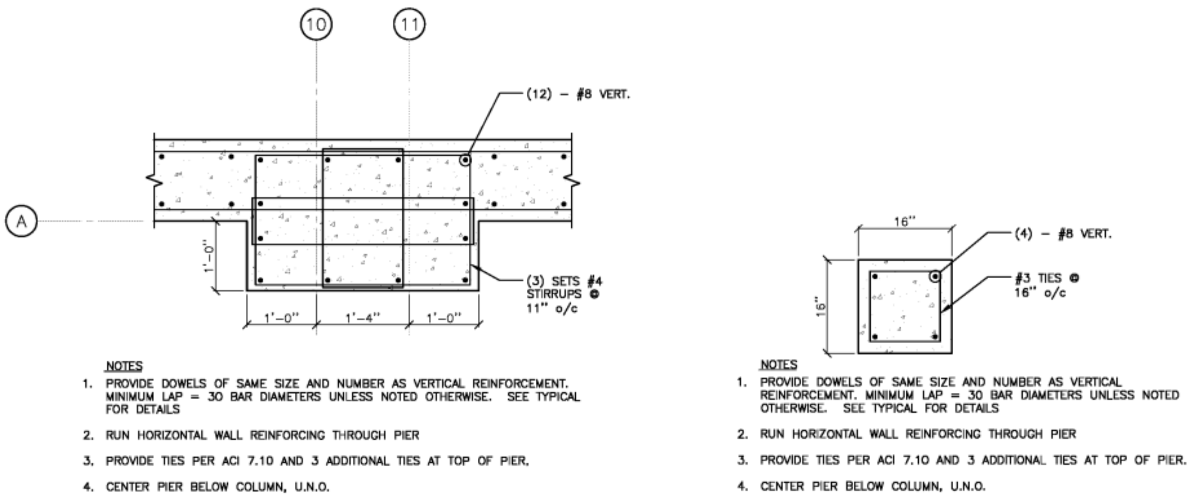


Figure 15 Concrete Piers (S12.02 Drawing 2 and 19)

There are also 12"x12" precast concrete columns that are supporting the balconies. Another interesting feature in columns from this structure is the canopy to support small roof that sheds an emergency exit, shown below in figure 16.

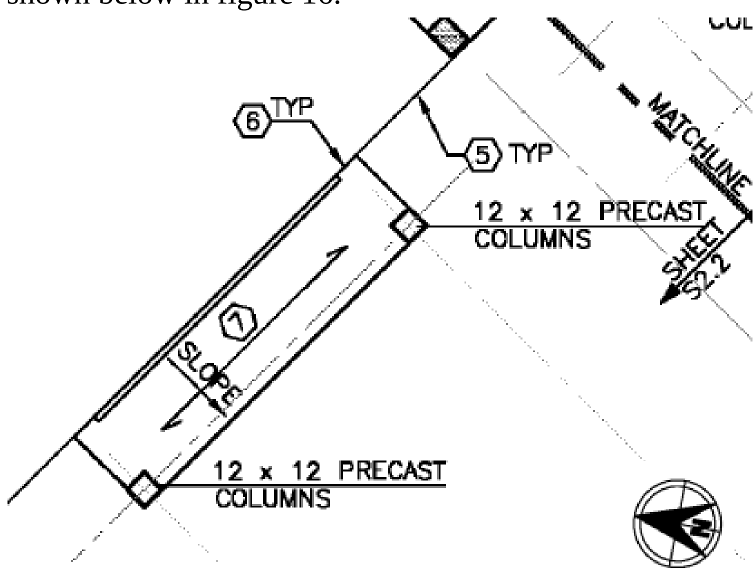


Figure 16 Typical Balcony Layout (S4.2)

Roofing System

Roofing uses exactly the same 10" and 12" thick precast planks at the same locations as floors below but except without toppings. As can be seen in figure 17, 6" galvanized lightgage metal stud parapet is connected by galvanized steel angle beam L4x4x3/8. There are also roofing above balconies (only on eighth floor) and entrances/exits. These hip roofs are supported by light steel trusses at 24" o.c.

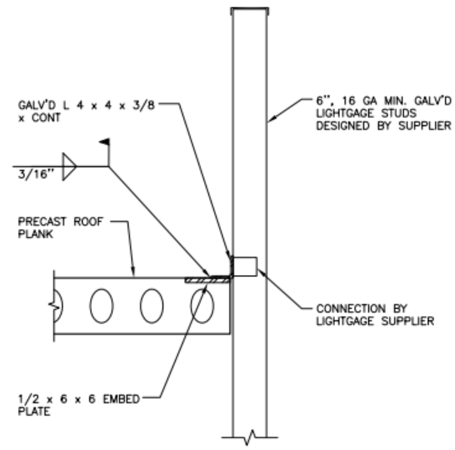


Figure 17 Typical Parapet Section (S12.30 Drawing 11)

Joint Details

As previously described, the precast planks bears on top of shear walls that are topped with masonry bond beams and sits on bearing strips (figure 20). The planks that are connected to the wide flange beams are set on top of weld anchor finished with grouted butt joint, shown in figure 19 below. Precast planks supported by steel column will be connected by steel angle with stiffener plate in its center, shown in figure 18.

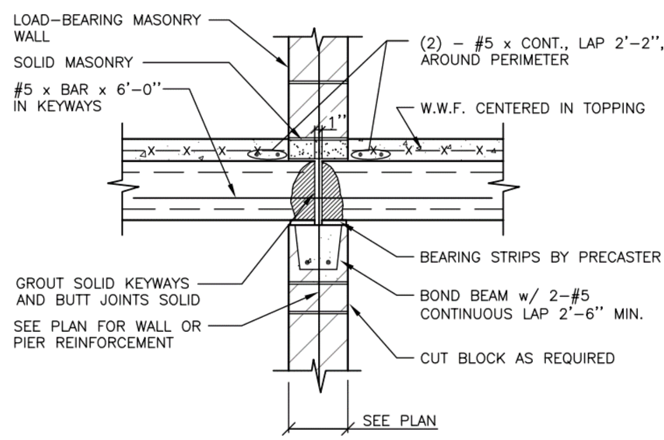


Figure 20 Precast Plank Bearing on Masonry Shear Wall (S12.20 Drawing 10)

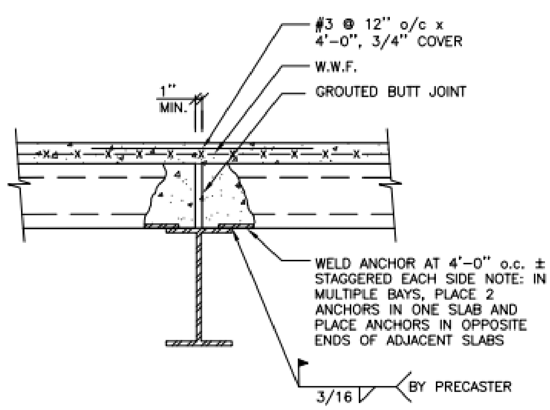


Figure 18 Precast Plank Bearing on Steel Beam (S12.20 Drawing 11)

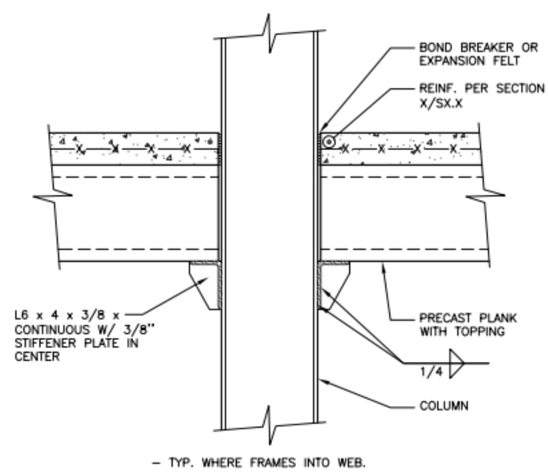
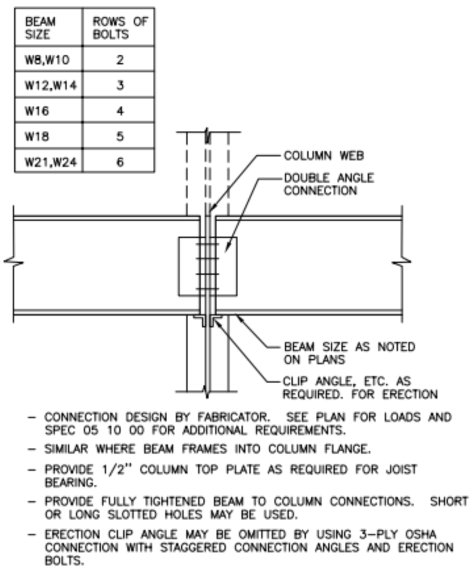


Figure 19 Precast Plank Support at Steel Column (S12.20 Drawing 8)

The typical steel framing section is as shown in figure 22. The column web holds double angle connection as well as clip angle to support wide flange beams. A typical steel moment connection shown in figure 21 has welded double angle connection with erection bolts.



BEAM SIZE	ROWS OF BOLTS
W8,W10	2
W12,W14	3
W16	4
W18	5
W21,W24	6

Figure 22 Typical Steel Framing Section

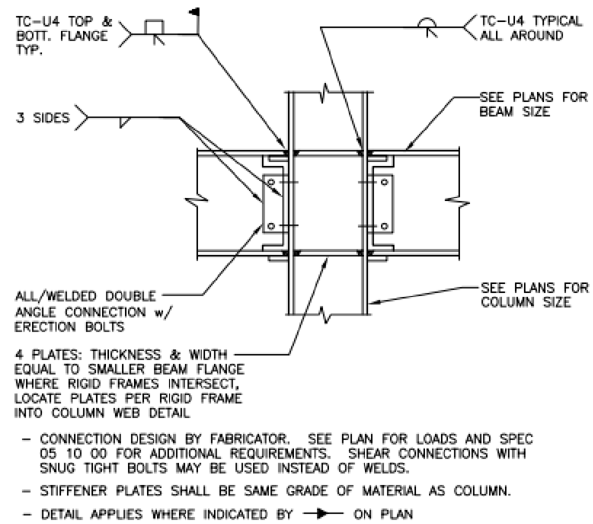


Figure 21 Typical Moment Connection

The steel column is connected to the baseplate shown in figure 23 with non-shrink grout that is injected between the baseplate and concrete pier. The anchor bolts with leveling nuts are installed under the base plate to level the baseplate prior to grouting.

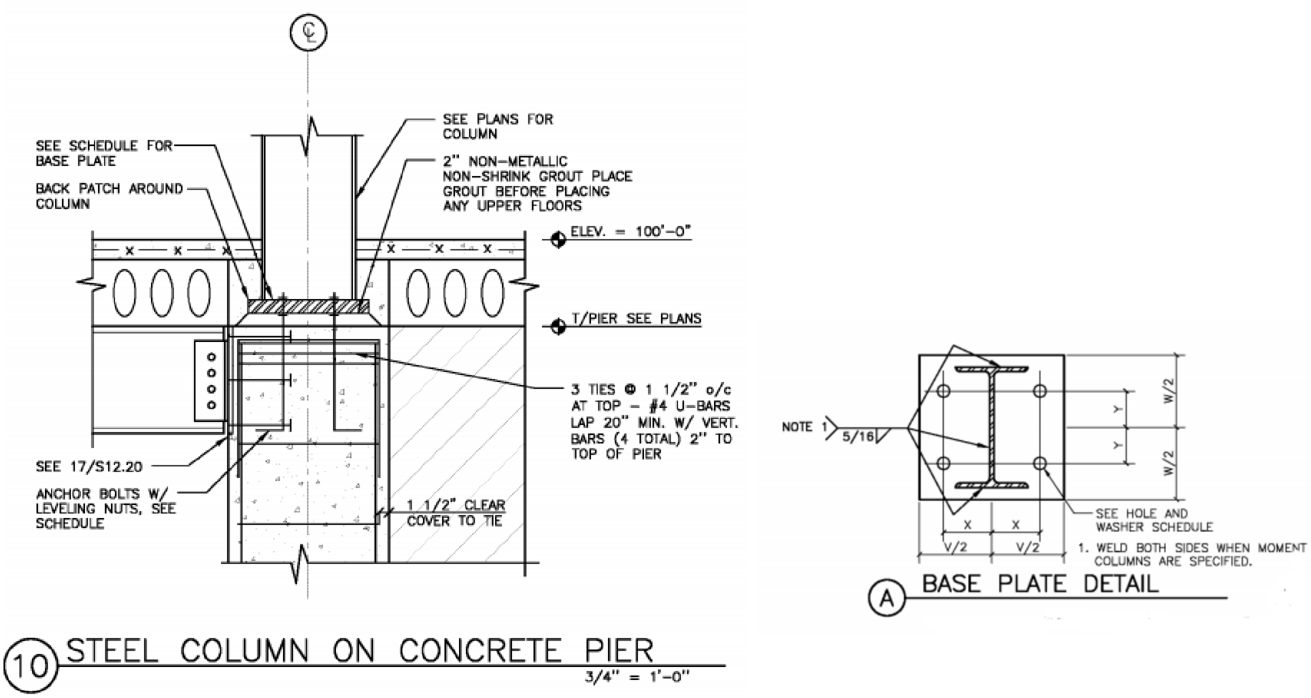


Figure 23 Steel Column on Concrete Pier and Base Plate Detail (S12.22 Drawing 10, S13.3 Drawing A)

Proposal Statement

The existing hotel structure of Hakuna Resort contains total of 39 load bearing masonry shear walls. Considering the location of the project is not a highly active seismic area, this is too many shear walls. Based on this information, a scenario was created to have an alternate structural system to see how it will behave compared to the original system in terms of strength, serviceability and cost.

Proposed Solution

To compare the efficiency of existing lateral system in terms of conservative design and cost, an alternate lateral system design with staggered steel truss system will be investigated and designed. By the nature of staggered truss system, the number of walls created by truss will be greatly reduced compared to the number of existing load bearing masonry shear walls. This solution will keep the original prestressed precast hollow core planks as floor system and replace gravity and lateral system to staggered truss system.

Breadth Studies

Architecture

The implementation of staggered steel truss system may have a big impact on floor plan layout in lower levels which includes public service areas that require open spaces. The existing structure handled this problem by using steel moment frame. The second floor contains vestibule, sauna, reception, relaxation rooms and massage treatment rooms, which does not follow the typical bay grid layout of hotel rooms above 3rd floor. Hence the floor plans of first and second levels need to be redesigned. The floor plans of 3rd level and above will remain the same to the original design to avoid any major conflict.

The exterior façade will also be redesigned to be more attractive and exciting. The existing façade follows brown color scheme to emulate earth, wood and nature, which resulted rather blend façade. By adding more variety of colors added with a pattern that resembles a tribal symbols as architectural finishes on the façade, the building will draw more excitement when families encounter the resort.

Construction Management

The change in material of lateral system will result change in cost analysis including material cost and labor cost. Also, because it is a totally different system, it will have different assembly sequence which affects the schedule of project. In addition, any changes made in floor and façade redesign will be considered in terms of cost and schedule. After examination, these cost and schedule data of staggered truss system will be compared to the existing lateral system to determine efficiency of each design.

Structural Depth

Staggered Truss Background

Staggered truss system is consisted of story tall steel trusses placed alternatingly in every other column lines on each floor. The floor system, typically precast concrete hollow core plank, is utilized by having planks spanning from the bottom chord of one truss to the top chord of the adjacent truss. Numerous hotel structures use staggered truss system due to the simple framing layout.

By having trusses arranged in staggered pattern as shown in figure 24, and letting the truss to support load from floor above and below eliminate the need for interior columns or load bearing walls to be continuous from bottom floor to the roof. Hence this allows more open floor area and be more flexible with architectural floor layout.

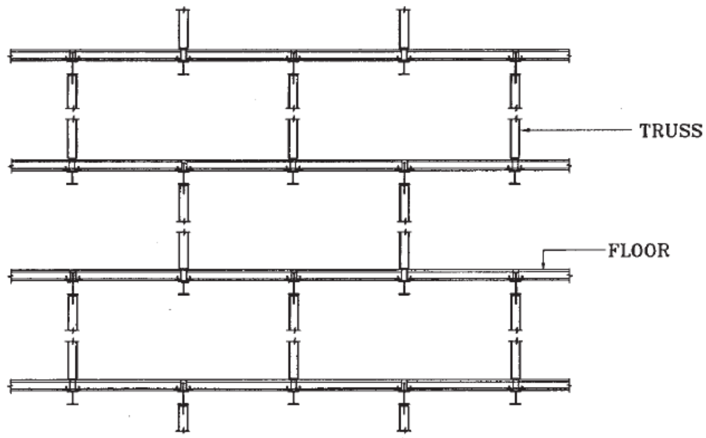


Figure 24 Staggered Truss System Vertical Staking Arrangement from AISC Design Guide 14

Figure 25 is the representation of typical truss that can be staggered. The AISC Design Guide 14 – Staggered Truss Framing System suggests top and bottom chords to be wide flange steel beams and rectangular HSS shape for the vertical and diagonal members.

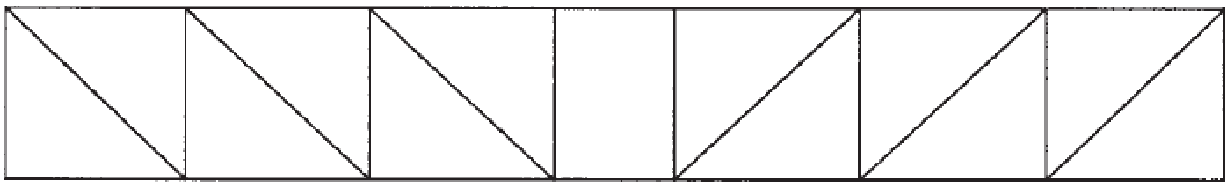


Figure 25 Typical Truss from AISC Design Guide 14

Truss Design

Truss Layout

The Hakuna Resort hotel's original design already had repetitive floor layout with uniform column grids that are spaced at 28 ft. Therefore, no change in structural gridline was made and truss was staggered as shown in figure 26 below. First and second floor had floor layout conflict when placing trusses. Therefore, the floor was redesigned and will be covered more in depth in Architecture Breadth Study later in this report. 3rd floor and up are hotel room floors, which had no conflict with floor layout, hence eased the process of truss layout.

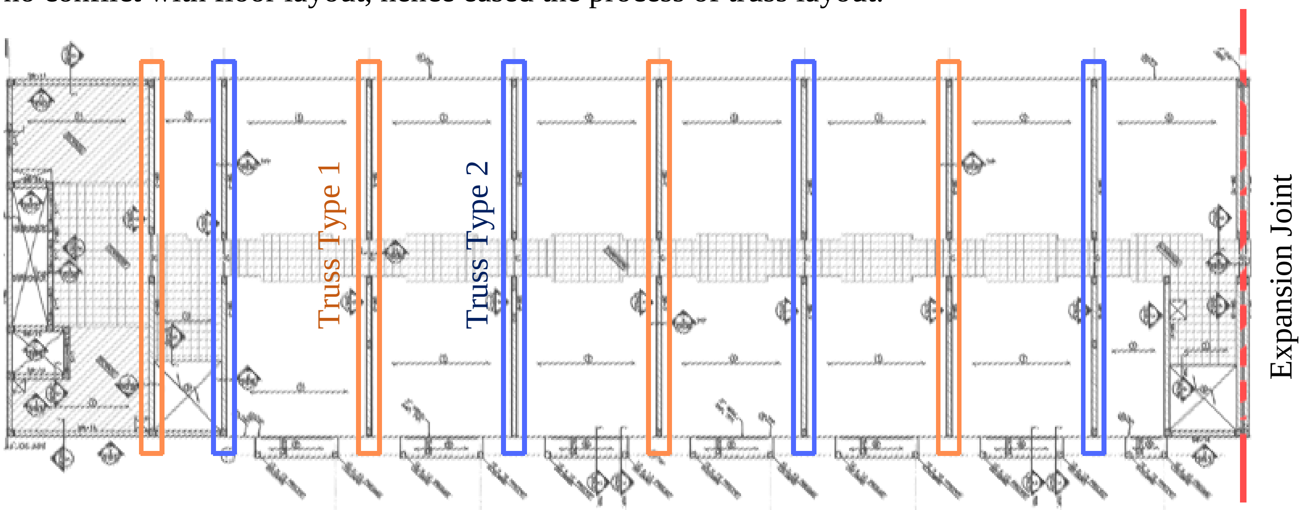


Figure 26 Staggered Truss Layout of Typical Floor Plan (4th Floor)

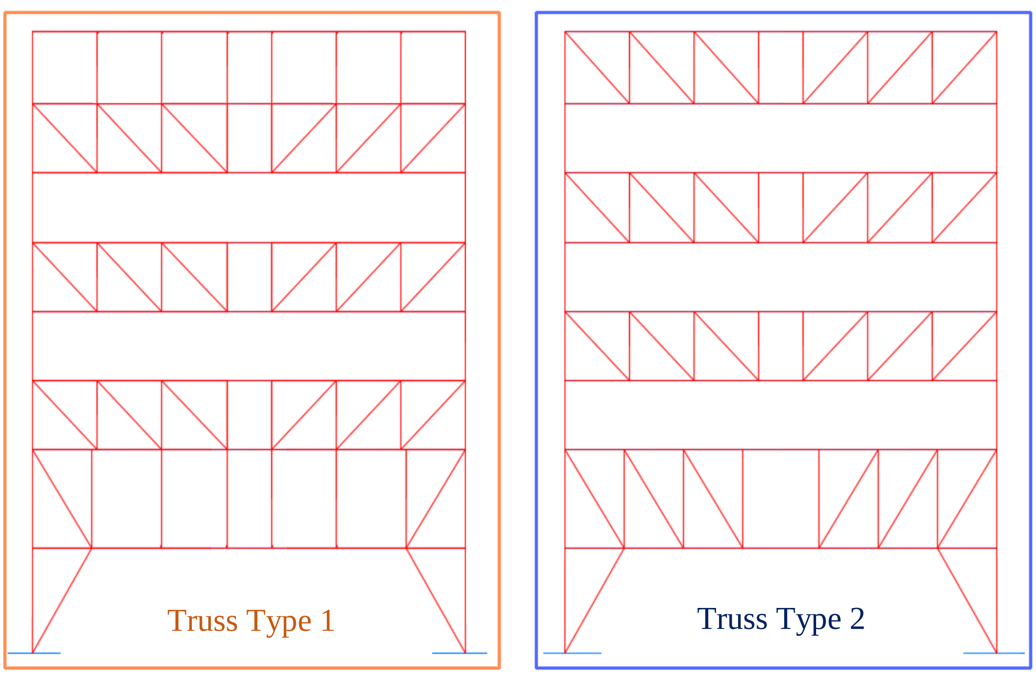


Figure 27 Truss Frame Elevation Views

Floor System

The existing floor system is kept the same as 10” & 12” precast prestressed concrete hollow core planks with 3” topping which work very well with staggered truss system. Therefore, when designing the truss system, same load types were taken from previous Technical Reports.

Truss Members

To begin designing members, hand calculation was done prior to the computer modeling for better understanding of system. During this hand calculation process, example calculation procedure from AISC Design Guide 14 was followed as the guide. The AISC Design Guide example records the top and bottom chords to carry axial loads and moments while the vertical and diagonal members to only carry axial load.

For the hand calculation, typical truss located on 4th floor, the hotel room that is replicated up until top floor, was chosen to be calculated. The gravity loads from Technical Report 2 was taken when calculating member load. The uniform gravity loads were converted to concentrated loads which are applied at each joint of the truss. According to the design guide, the gravity loads produces shear in the top and bottom chords at the Vierendeel panel, but this could be ignored due to symmetry. This allows the truss to be statically determinate. With these assumptions, the hand calculation was done in method of joints and can be found in Appendix A.

The hand calculation was done in unfactored gravity loads. After the member loads were determined, load combinations were applied to find the actual applied loads and also to find the worst load case for member sizing. Microsoft Excel spreadsheet was used to tabulate the load values for each load combination and can be found in Appendix A. As the result, 1.2D + 1.6L was controlling in majority of members.

Computer Modeling

After finding out the controlling load combination and axial loads of each member, ETABS model was created for the typical floor truss only for one floor (4th floor) first to verify the values from hand calculation. When creating computer model, AISC design guide stated that steel truss members’ behavior may vary due to the flexible nature of modeled truss and concrete floor and cause the tensile stress to be not efficiently transmitted. As a solution, the design guide suggested creating two different models – one for gravity loads only and one for lateral loads only, and then the results are combined using load factors.

The lateral loads were recalculated with the parameters from revised lateral load calculated done in Technical Report 4 and new parameters that staggered truss system brought. These values can be found in Appendix A. These values which were calculated and verified with model output were combined when sizing the members as AISC design guide suggested.

Diagonal Members

As stated before, the values from gravity model and lateral model were combined. The data tabulated below is truss on gridline 6. Only three load combinations were used because the others were eliminated for obvious non-governing coefficients. The phi is the percentage of lateral base shear each floor take for each lateral load type. As the design guide recommended, HSS shape members were selected for the diagonal members and the member size for each level is listed below. The same size member is to be used for vertical members.

Diagonal Member

	wind		seismic		Load Combinations			
Floor	phi	Applied Load (kips)	phi	Applied Load (kips)	1.2D+.8W	1.2D+1.6 W+L	1.2D+E+L	Section
Roof	9%	13.80	25%	44.70	99.48	149.42	**172.04**	**HSS8x6x1/2**
8	24%	36.85	43%	76.27	117.92	186.30	**203.61**	**HSS8x6x1/2**
7	36%	55.61	59%	103.62	132.93	216.31	**230.96**	**HSS8x6x1/2**
6	48%	74.00	72%	126.77	147.64	**245.74**	254.11	**HSS8x6x1/2**
5	60%	91.98	82%	145.78	162.03	**274.51**	273.12	**HSS10x8x1/2**
4	72%	109.47	91%	160.72	176.02	**302.49**	288.06	**HSS10x8x1/2**
3	85%	130.00	97%	171.64	192.44	**335.34**	298.98	**HSS10x8x1/2**
2	100%	153.07	100%	177.00	210.90	**372.25**	304.34	**HSS10x8x1/2**
Gound								

Table 1 Diagonal Member Size Selections and Design Values

Truss Chords

When finding the size of members, member moments from the gravity and lateral model were taken as well as the axial load from gravity load. As shown in the table above, wind load case is governing, which is why moment values from wind load was used for this calculation. The Mu is the sum of moment from gravity (Mug) and wind load (Muw). In order to avoid the floor to floor height to be large, W10 sections were chosen and detailed member size for each level is tabulated below.

Truss Chord

Floor	phi	M_{ug}	M_{uw}	Mu	Pu	Section
Roof	9%	44.4	27.25	71.65	476.4	**W10x60**
8	24%	44.4	72.78	117.18	476.4	**W10x60**
7	36%	44.4	109.83	154.23	476.4	**W10x77**
6	48%	44.4	146.17	190.57	476.4	**W10x77**
5	60%	44.4	181.68	226.08	476.4	**W10x88**
4	72%	44.4	216.23	260.63	476.4	**W10x88**
3	85%	44.4	256.78	301.18	476.4	**W10x112**
2	100%	44.4	302.34	346.74	476.4	**W10x112**

Table 2 Truss Chord Member Size Selections and Design Values

Columns

Like truss chords and diagonal members, columns design calculation was done by following the example procedure in AISC design guide 14. While the staggered truss system eliminates the need for interior columns, exterior columns are faced with great increase in the tributary area and subsequent load that each edge column will carry. Tabulated below are the individual floor loads on each column, and the sizes of columns selected. More detailed table with prerequisite values is included in Appendix A.

Load Combinations				Section
1.4D		1.2D+1.6L		
Pu	Mu	Pu	Mu	
289.8	77	**351.032**	66	**W12x65**
289.8	0	**370.232**	0	**W12x65**
579.6	91	**721.2641**	78	**W12x87**
579.6	0	**740.4641**	0	**W12x87**
869.4	107.8	**1091.496**	92.4	**W12x120**
869.4	0	**1110.696**	0	**W12x120**
1159.2	114.8	**1461.728**	98.4	**W12x152**
1159.2	0	**1480.928**	0	**W12x152**
1449	135.8	**1831.96**	116.4	

Table 3 Column Member Size Selection and Design Values

Deflections

After sizing the members, the chord deflections from gravity loads were checked. The figure XX shows the deflection shape and table 4 shows the maximum values of deflection for each chord size. With the chord span of 66'-8", the deflection limit was determined to be L/240 = 3.35". The live load deflection was L/360 = 2.23".

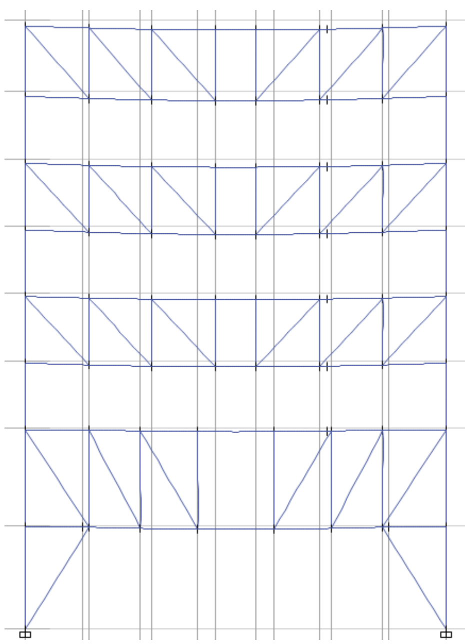


Figure 28 Gravity Load Model Deflection Shape

Gravity Deflections (in)		
Chord Size	1.2D+1.6L	1.6L
W10x60	0.919	0.29
W10x77	0.883	0.243
W10x88	0.854	0.18
W10x112	0.691	0.183

Table 4 Maximum Chord Member Deflections

The drift data were also taken separately and then combined together with load combinations. The story drift of wind and seismic with load combination applied is compared in table 5. This data indicates the wind load is still governing for the lateral drifts as well. The roof displacement of wind load case is then checked with the deflection limit L/400 = 2.01". With the roof displacement of 0.526", the structure is well under the limit and therefore the design is valid.

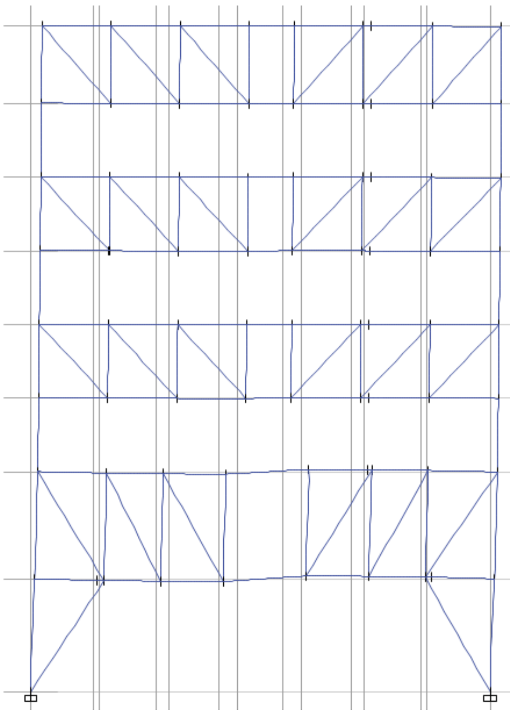


Figure 29 Lateral Load Model Deflection Shape

Lateral Story Drifts (in)		
Level	1.2D+L+1.6W	1.2D+L+E
Roof	0.009	0.017
8	0.014	0.024
7	0.025	0.032
6	0.027	0.035
5	0.031	0.034
4	0.043	0.063
3	0.147	0.115
2	0.23	0.182
1	0	0
Total	0.526	0.502

Table 5 Lateral Story Drifts

Architectural Breadth Study

Floorplan Redesign

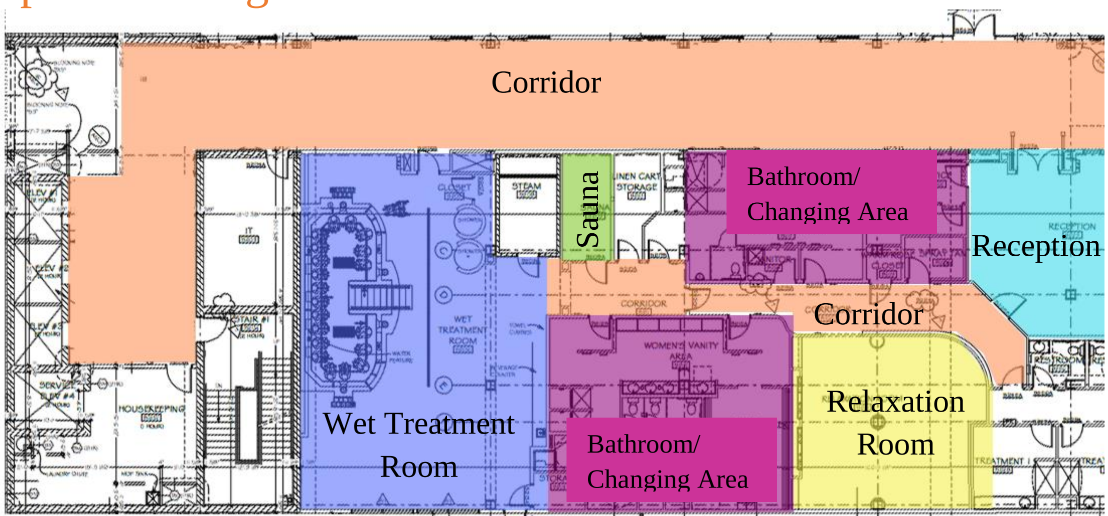


Figure 30 Original 2nd Floor Plan

In the lower levels of the hotel structure exist service areas such as massage room, hair salon, sauna and rest areas. With these types of activities, the rooms need to be bigger and opened. The very first conflict with staggered truss framing system is that it requires every 56 feet to be closed with full story tall trusses as walls. Figure 30 shows west portion of second floor layout. As indicated by color, the hall way is against the north side wall and all other service rooms concentrated on the other side. Figure 31 is the redesigned floor plan with staggered truss layout and Vierendeel panel in the middle for the hallway. Also the wall placements were carefully arranged so that where staggered truss will be located will have wall separating between rooms. The square footage of each space was kept relatively equal to that of the original design. More detailed redesign and original floor layout comparison can be found in Appendix B.

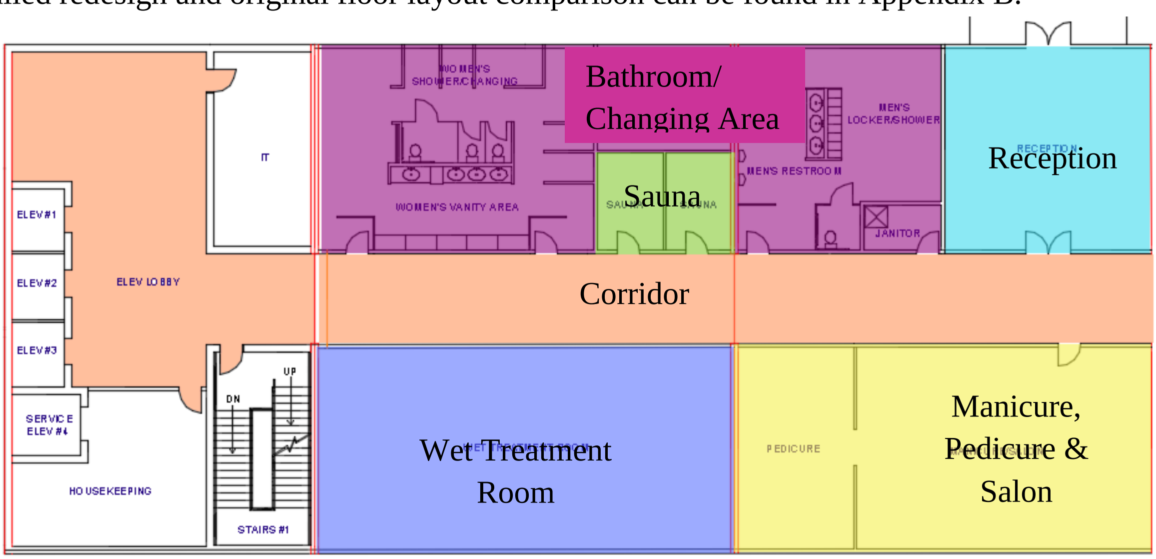


Figure 30 Redesigned 2nd Floor Plan

Fa çade Redesign

Figure 31 North Driveway Entrance of Hakuna Resort

Hakuna resort has overall brown color scheme. As mentioned before, Hakuna Resort carries Savannah Desert theme and is quite evident that the original fa çade is trying to replicate the desert scene by using earth-like tone throughout the entire hotel building. Figure XX below is a rendering taken from the northern driveway entrance, which has a very good view of the north fa çade of the hotel building. The north fa çade is rather flat and has very basic pattern that with a few different colors: red, brown and gray. With such a huge building, the tallest of the entire resort project, the blend look of fa çade gives somewhat underwhelming feel to the whole project cite.

The existing structure uses exterior insulation finish system (EIFS) with different colored finish. This means that pretty much all area of fa çade is same material except the color of the finish surface. Since the original building already utilized different colors, goal for new design was set to keep the EIFS but use different color scheme to minimize change in cost and construction schedule. So how the new design needs to keep the flat profile while revamping the pattern of the existing fa çade for more excitement to Hakuna Resort.

Figure 32 View of Hotel Building from South

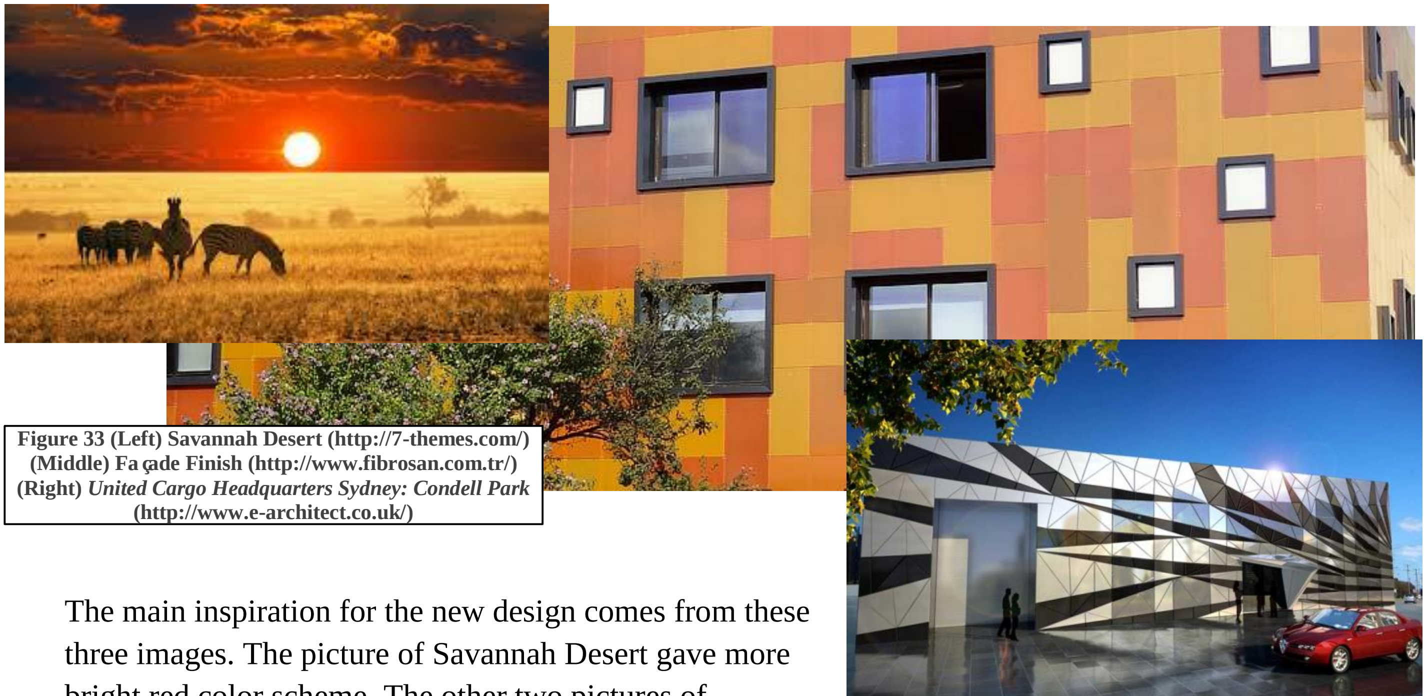


Figure 33 (Left) Savannah Desert (http://7-themes.com/) (Middle) Fa çade Finish (http://www.fibrosan.com.tr/) (Right) *United Cargo Headquarters Sydney: Condell Park* (http://www.e-architect.co.uk/)

The main inspiration for the new design comes from these three images. The picture of Savannah Desert gave more bright red color scheme. The other two pictures of buildings share the flat surface of fa çade and yet keep the buildings intriguing to the eyes. Combining these key ideas, the redesigned fa çade is as shown in Figure 34. Figure 35 shows the south fa çade where, unlike north fa çade, balconies and columns add more character to the fa çade. When the same finish pattern were to be used on the south fa çade, there is too much features that are meshed up together that it would be rather exhausting than exciting. Therefore, simpler pattern yet complementing consistency of balcony pattern, square block pattern was chosen.

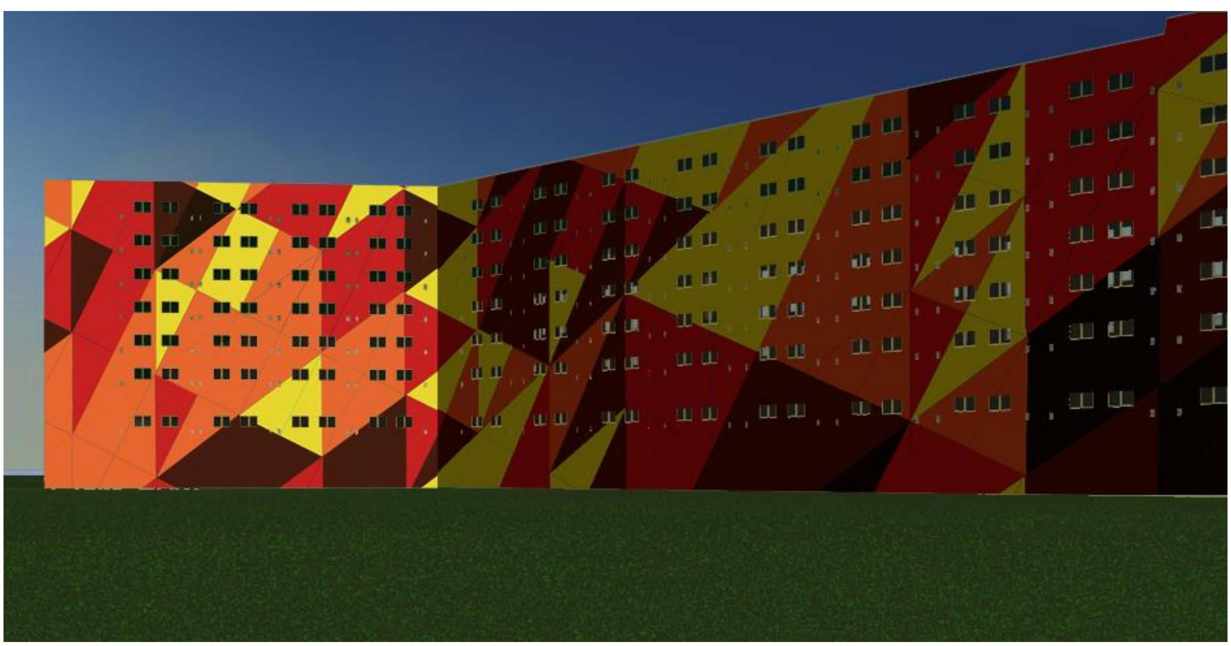


Figure 34 Redesigned North Fa çade

Figure 35 Redesigned South Façade

Construction Management Breadth Study

In order to compare the validity of the new staggered truss system, other aspect of design must be observed. Changing the existing load bearing masonry shear wall to steel structure completely raised a question if it is adequate to do so economically.

Because the cost and schedule data of the original project was not available, the cost of one typical masonry shear wall was estimated using Building Construction Data and Assemblies Cost Data by RSMeans. Then it was multiplied by the number of shear walls in the focused portion of the hotel structure to get the total value. The new system's cost was estimated the same way. All steel members' lengths were measured then multiplied by the cost per linear feet for each steel member size. For member sizes that were not listed on RSMeans were linearly interpolated by two nearest member sizes' cost.

The total cost of original design came out to be approximately $1 million and the new staggered truss system's total cost was about $1.2 million. What brings an interesting idea is on their construction schedule.

When cost data was recorded, each material's daily output and labor hour were recorded as well. The labor hour was divided by daily output to obtain total hour it takes for workers to finish that material. The prefabrication of staggered truss system allowed the schedule to be decreased significantly. The connection schedule was estimated by increasing the total hour by 20%. The original design was estimated to take 9 days, whereas the new staggered truss system were estimated to take only a day. More detailed calculation of these estimates can be found in Appendix C.

For the cost and schedule estimate for the architectural breadth, no definite numbers were estimated. Due to lack of information on the finish material "other than E.I.F.S., it was assumed that the material was E.I.F.S. cement board sheathing, 3-5/8" metal studs, 16" o.c. with painting finish. Because the material was kept the same in the redesign, material cost is assumed to be the same as well. In terms of schedule, due to the complexity of paining of new façade design, 10% to the original exterior wall schedule was added.

It is questionable if one system is better than the other simply by looking at the construction aspect. Is $200,000 worth to pay in order to decrease the duration of construction by a week? If the project were to be in tight schedule, this is definitely a better option. However, if the project is not under the pressure of time, the original design is best option for the owner.

Conclusion

This report consisted of an analysis and redesign of Hakuna Resort at Shiftwater, Pennsylvania. During the fall semester, analyses of the existing load bearing masonry shear walls were analyzed as gravity and lateral system. It was determined that the original designs were adequate for strength and therefore, valid design. Due to this, a scenario was made in which the existing structural system were to be redesigned to staggered truss framing system.

The staggered truss framing system redesign was completed using AISC Design Guide 14 and its example design procedure. Hand calculation was done prior to making ETBAS model. After basic hand calculation of typical truss was done, a gravity load model and a lateral load model was created as recommended by the design guide. The outputs from these two models were then combined using spreadsheet to incorporate load combinations. After finding the controlling load cases for each member types, the member sizes were determined then checked with displacement limit. With the data obtained throughout the process, it was determined that the new design was adequate and valid.

Although staggered truss system worked really well with Hakuna Resort's hotel building, few problem arose due to the redesign of structural system. There were service areas that requires more open space and hence must be more flexible with the room layout than the limited area constrained by staggered truss pattern. In order to overcome this, the architectural breadth study was done to redesign the floorplan of first, second and basement level to accommodate the staggered truss constraints. In addition, fa çade was redesigned as well to revamp the traditional fa çade to more modern and exciting while keeping the same finish materials.

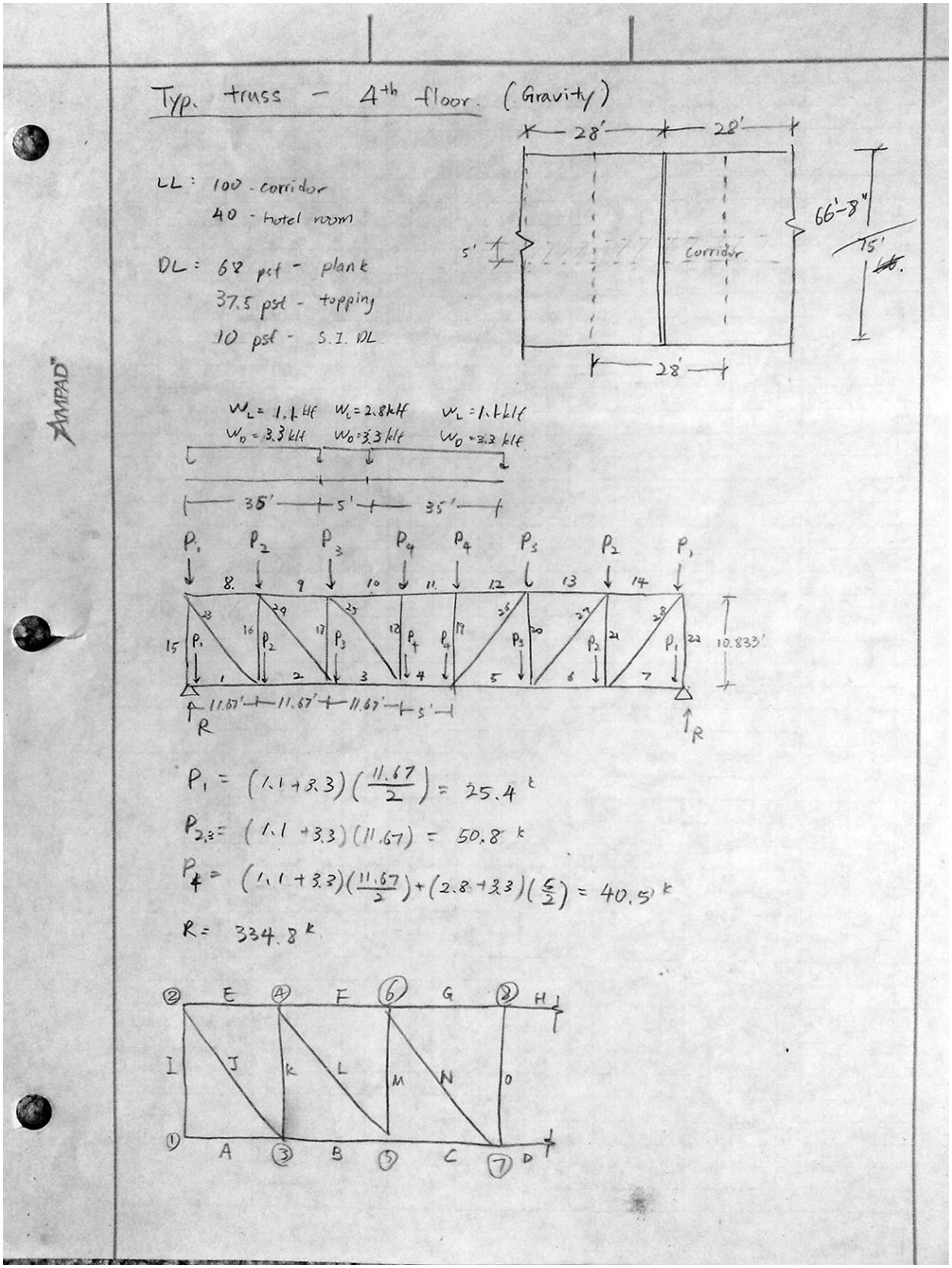
For the second breadth study, a cost and schedule analysis was completed to help determine the feasibility of the staggered truss system and architectural changes. Through this study, it was determined that the staggered truss system would offer a decrease in the construction schedule while increasing the cost by $200,000.

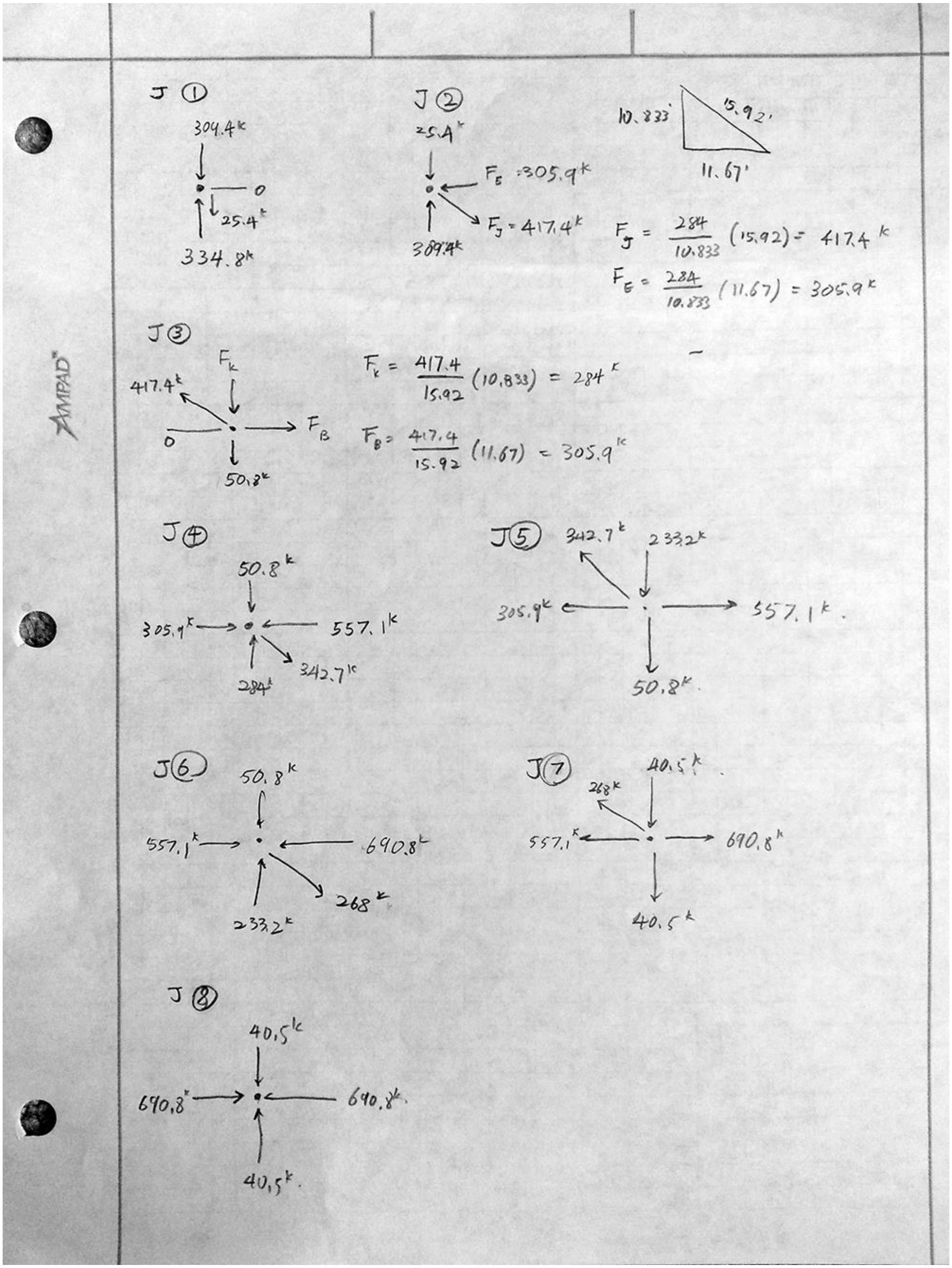
It was determined that the staggered truss system is ultimately a feasible alternative structural system. It did not show any definitive advantages compared to the original load bearing masonry shear walls. It was decided that it is up to owner if the project requires shorter construction schedule, staggered truss system is recommended with slight increase in project cost. If the schedule is not a critical matter, then the original design is better choice. Overall this project was very educational for learning a new structural system.

Appendices

Appendix A

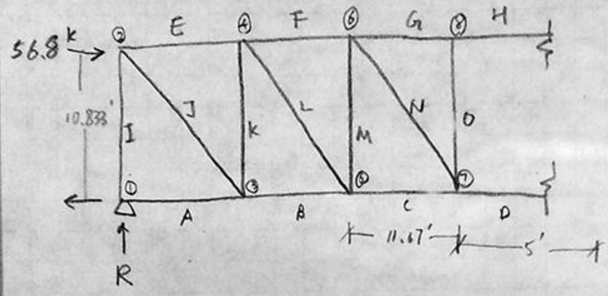
A.1 Hand Calculations

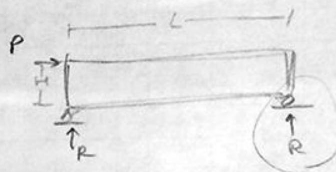



J ①
304.4k
0
25.4k
334.8k
J ②
25.4k
F_E = 305.9k
F_J = 417.4k
309.4k
10.833'
15.92'
11.67'
F_J = 284 / 10.833 (15.92) = 417.4 k
F_E = 284 / 10.833 (11.67) = 305.9 k
J ③
F_K
417.4k
F_B
0
50.8k
F_K = 417.4 / 15.92 (10.833) = 284 k
F_B = 417.4 / 15.92 (11.67) = 305.9 k
J ④
50.8 k
305.9k
557.1k
284k
342.7k
J ⑤
342.7k
233.2k
305.9k
557.1k
50.8k
J ⑥
50.8 k
557.1k
690.8k
268 k
233.2k
J ⑦
40.5 k
268k
557.1k
690.8k
40.5 k
J ⑧
40.5k
690.8k
690.8k
40.5k

Typ. truss – 4th floor (Lateral)

wind load = 56.8^k (from tech report 4).



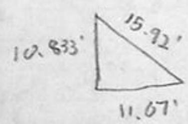


$$R = \frac{2(10.833)(56.8)}{75} = 16.4^k$$

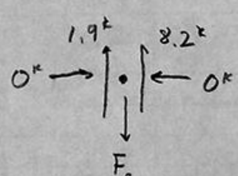
$$V_{midspan} = \frac{56.8(10.833)}{37.5}\left(\frac{1}{2}\right) = 8.2^k$$

$$M_{joint\,8} = 8.2\left(\frac{5'}{2}\right) = 20.5^{'k}$$

$$V_{left\ of\ jt\ 8} = \frac{20.5}{10.833'} = 1.9^k$$

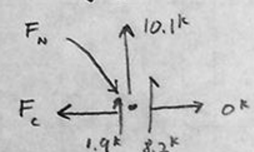


Joint (8)



$F_o = 10.1^k$

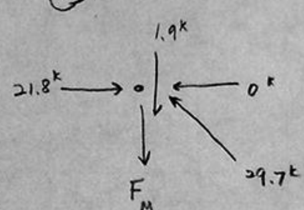
J (7)



$$F_N = \frac{(10.1+1.9+8.2)(15.92)}{10.833} = 29.7^k$$

$$F_c = \frac{29.7(11.67)}{15.92} = 21.8^k$$

J (6)



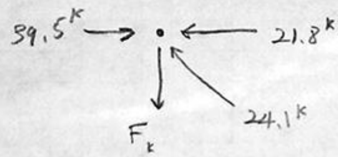
$F_M = 20.2 - 1.9^k = 18.3^k$

J 5

$$F_L = \frac{(18.3 - 1.9)(15.92)}{10.833} = 24.1^k$$

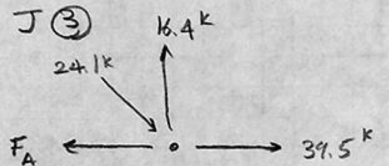
$$F_B = \frac{24.1\,(11.67)}{15.92} + 21.8^k = 39.5^k$$

J 4



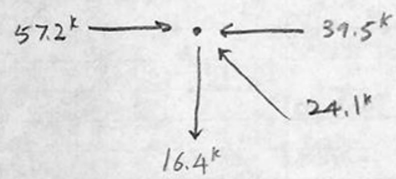
$$F_k = \frac{24.1\,(10.833)}{15.92} = 16.4^k$$

J 3

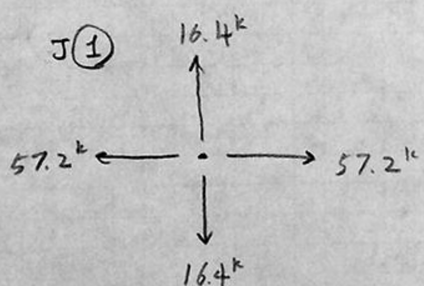


$$F_A = \frac{24.1(11.67)}{15.92} + 39.5^k = 57.2^k$$

J 2



J 1



A.2 Typical Floor Axial Load Calculation

Typical Floor Member Axial Loads (4th Floor)												
Unfactored Load (kips)						Factored Load (kips)						
Member	Dead	Live	Seismic	Wind		1.4D	1.2D+1.6L	1.2D+.8W	1.2D+1.6W+L	1.2D+E+L	0.9D+1.6W	0.9D+E
1	0.0	0.0	35.9	29.2		0.0	0.0	23.4	**46.7**	35.9	46.7	35.9
2	171.4	67.4	24.6	20.0		240.0	**313.5**	221.7	305.1	297.7	186.3	178.9
3	284.0	115.2	14.7	11.9		397.6	**525.1**	350.3	475.1	470.7	274.7	270.3
4	334.5	141.9	0.0	0.0		468.3	**628.4**	401.4	543.2	543.2	301.0	301.0
5	284.0	115.2	14.7	11.9		397.6	**525.1**	350.3	475.1	470.7	274.7	270.3
6	171.4	67.4	24.6	20.0		240.0	**313.5**	221.7	305.1	297.7	186.3	178.9
7	0.0	0.0	35.9	29.2		0.0	0.0	23.4	**46.7**	35.9	46.7	35.9
8	171.4	67.4	24.6	20.0		240.0	**313.5**	221.7	305.1	297.7	186.3	178.9
9	284.0	115.2	14.7	11.9		397.6	**525.1**	350.3	475.1	470.7	274.7	270.3
10	334.5	144.9	0.0	0.0		468.3	**633.2**	401.4	546.2	546.2	301.0	301.0
11	334.5	141.9	0.0	0.0		468.3	**628.4**	401.4	543.2	543.2	301.0	301.0
12	334.5	144.9	0.0	0.0		468.3	**633.2**	401.4	546.2	546.2	301.0	301.0
13	284.0	115.2	14.7	11.9		397.6	**525.1**	350.3	475.1	470.7	274.7	270.3
14	171.4	67.4	24.6	20.0		240.0	**313.5**	221.7	305.1	297.7	186.3	178.9
15	201.3	78.1	11.8	9.6		281.9	**366.5**	249.2	335.0	331.4	196.5	193.0
16	150.5	61.1	11.3	9.2		210.7	**278.3**	187.9	256.3	252.9	150.1	146.7
17	86.9	39.7	13.1	10.7		121.7	**167.8**	112.8	161.1	157.1	95.3	91.4
18	25.9	15.2	7.8	6.3		36.2	55.3	36.1	**56.3**	54.0	33.3	31.0
19	25.9	15.2	7.8	6.3		36.2	55.3	36.1	**56.3**	54.0	33.3	31.0
20	86.9	39.7	13.1	10.7		121.7	**167.8**	112.8	161.1	157.1	95.3	91.4
21	150.5	61.1	11.3	9.2		210.7	**278.3**	187.9	256.3	252.9	150.1	146.7
22	201.3	78.1	11.8	9.6		281.9	**366.5**	249.2	335.0	331.4	196.5	193.0
23	250.0	98.2	16.5	13.4		350.0	**457.1**	310.7	419.7	414.7	246.4	241.5
24	164.4	69.9	14.5	11.8		230.1	**309.0**	206.7	286.0	281.6	166.8	162.4
25	73.7	38.9	21.5	17.4		103.2	**150.7**	102.4	155.3	148.8	94.2	87.8
26	73.7	38.9	21.5	17.4		103.2	**150.7**	102.4	155.3	148.8	94.2	87.8
27	164.4	69.9	14.5	11.8		230.1	**309.0**	206.7	286.0	281.6	166.8	162.4
28	250.0	98.2	16.5	13.4		350.0	**457.1**	310.7	419.7	414.7	246.4	241.5

A.3 Wind Load Calculation

Wall Pressure North - South Direction

oor Numb	Height above ground	Story Height (ft)	q_z	q_h	Windward (psf)	Leeward (psf)	Tributary Height (ft)	Tributary Area (ft^2)	Force (k)	Story Shear (k)	Moment at Each Story (ft-k)
Ground	0	0	15.25	22.18	10.31	-9.37	8.21	1958	38.53	535.54	0.0
2	16.417	16.417	15.25	22.18	10.31	-9.37	15.96	3806	74.90	497.02	1229.7
3	31.917	15.5	17.54	22.18	11.86	-9.37	13.17	3140	66.66	422.11	2127.6
4	42.75	10.833	18.65	22.18	12.61	-9.37	10.83	2584	56.79	355.45	2427.8
5	53.583	10.833	19.56	22.18	13.22	-9.37	10.83	2584	58.38	298.66	3128.2
6	64.417	10.834	20.33	22.18	13.75	-9.37	10.83	2584	59.73	240.28	3847.6
7	75.25	10.833	21.01	22.18	14.20	-9.37	10.83	2584	60.91	180.55	4583.4
8	86.083	10.833	21.61	22.18	14.61	-9.37	13.09	3121	74.84	119.64	6442.8
Roof	101.42	15.337	22.37	22.18	15.12	-9.37	7.67	1829	44.80	44.80	4543.7
Base Shea	535.5435										
Total											
Overturn	28330.73										

A.4 Seismic Load Calculation

$T = 0.02\,(101.42)^{.75} = 0.639 < 6\,s = T_L$ $\therefore$ max = Eq-12.8-3

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.0669}{0.639(3/1)} = 0.035 = \text{max.}$$

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.1768}{3/1} = 0.059$$

N-S Direction

$$V = C_s W = (0.035)(19113) = 669^k$$

From Tech 4, $V = 965.2^k$.

Story Forces (North - South)						
Floor Number	Height above ground	Story Height (ft)	W (k)	Wh^k	C_{vx}	Forces (k)
2	16.417	16.417	2334	46615	0.030	20.27
3	31.917	15.5	2334	94943	0.062	41.28
4	42.75	10.833	2334	129796	0.084	56.44
5	53.583	10.833	2334	165280	0.107	71.86
6	64.417	10.834	2334	201275	0.131	87.52
7	75.25	10.833	2334	237696	0.154	103.35
8	86.083	10.833	2334	274487	0.178	119.35
Roof	101.42	15.337	2773	388538	0.253	168.94
					Base Shear:	669

A.5 Truss Chord Design Values

Truss Chord

Floor	phi	M_{ug}	M_{uw}	Mu	Pu	Section
Roof	9%	44.4	27.25	71.65	476.4	**W10x60**
8	24%	44.4	72.78	117.18	476.4	**W10x60**
7	36%	44.4	109.83	154.23	476.4	**W10x77**
6	48%	44.4	146.17	190.57	476.4	**W10x77**
5	60%	44.4	181.68	226.08	476.4	**W10x88**
4	72%	44.4	216.23	260.63	476.4	**W10x88**
3	85%	44.4	256.78	301.18	476.4	**W10x112**
2	100%	44.4	302.34	346.74	476.4	**W10x112**

A.6 Diagonal Member Design Values

Diagonal Member

	wind		seismic		Load Combinations			
Floor	phi	Applied Load (kips)	phi	Applied Load (kips)	1.2D+.8W	1.2D+1.6 W+L	1.2D+E+L	Section
Roof	9%	13.80	25%	44.70	99.48	149.42	**172.04**	**HSS8x6x1/2**
8	24%	36.85	43%	76.27	117.92	186.30	**203.61**	**HSS8x6x1/2**
7	36%	55.61	59%	103.62	132.93	216.31	**230.96**	**HSS8x6x1/2**
6	48%	74.00	72%	126.77	147.64	**245.74**	254.11	**HSS8x6x1/2**
5	60%	91.98	82%	145.78	162.03	**274.51**	273.12	**HSS10x8x1/2**
4	72%	109.47	91%	160.72	176.02	**302.49**	288.06	**HSS10x8x1/2**
3	85%	130.00	97%	171.64	192.44	**335.34**	298.98	**HSS10x8x1/2**
2	100%	153.07	100%	177.00	210.90	**372.25**	304.34	**HSS10x8x1/2**
Gound								

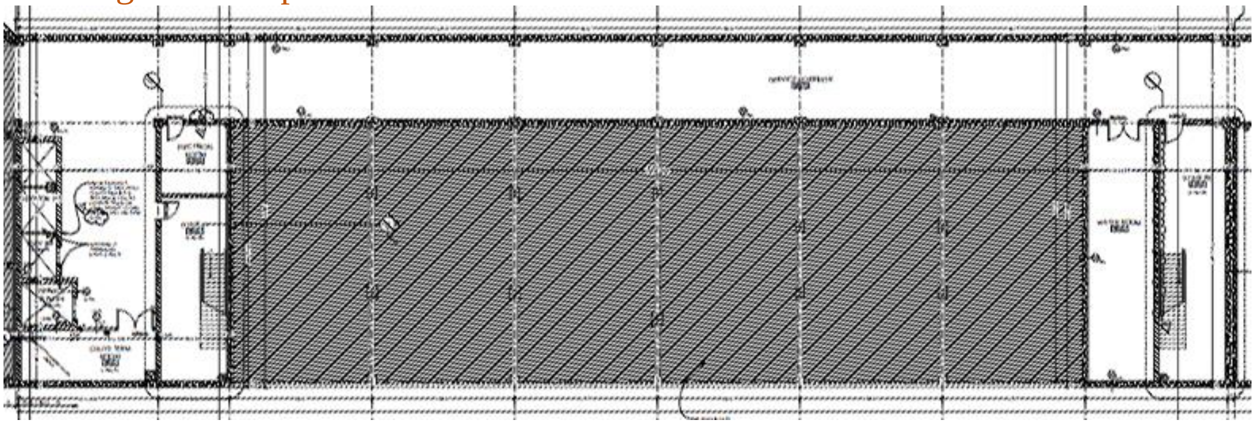
A.7 Column Member Design Values

Column 6A

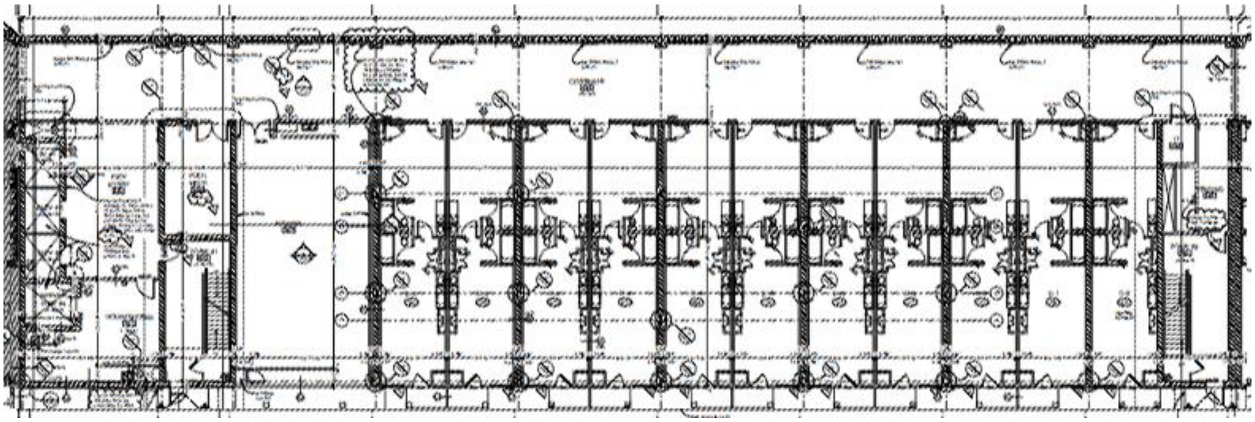
	Axial Forcs						Moment	Load Combinations				Section
	floor		Ext Wall	total		Ext Wall	DL	1.4D		1.2D+1.6L		
Floor	DL	DL+RLL		DL	DL+RLL			Pu	Mu	Pu	Mu	
Roof	207	264	16	207	264	16	55	289.8	77	**351.032**	66	**W12x65**
8			16	207	264	32		289.8	0	**370.232**	0	**W12x65**
7	207	264	16	414	528	48	65	579.6	91	**721.2641**	78	**W12x87**
6			16	414	528	64		579.6	0	**740.4641**	0	**W12x87**
5	207	264	16	621	792	80	77	869.4	107.8	**1091.496**	92.4	**W12x120**
4			16	621	792	96		869.4	0	**1110.696**	0	**W12x120**
3	207	264	16	828	1056	112	82	1159.2	114.8	**1461.728**	98.4	**W12x152**
2			16	828	1056	128		1159.2	0	**1480.928**	0	**W12x152**
Gound	207	264	16	1035	1320	144	97	1449	135.8	**1831.96**	116.4	

Appendix B

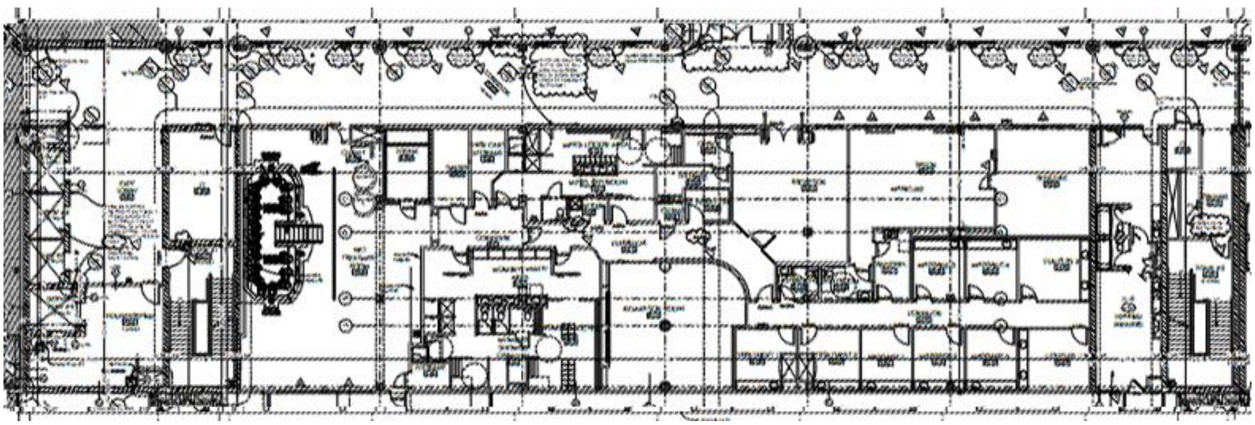
B.1 Original Floorplans



Basement

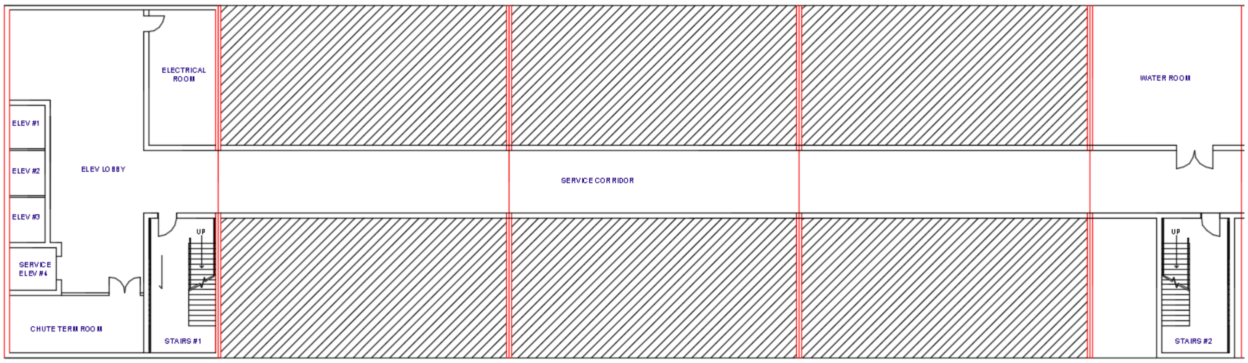


Level 1

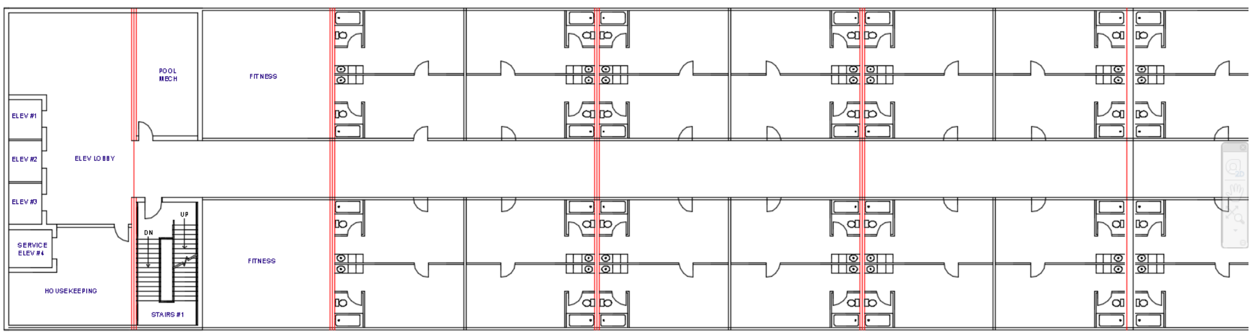


Level 2

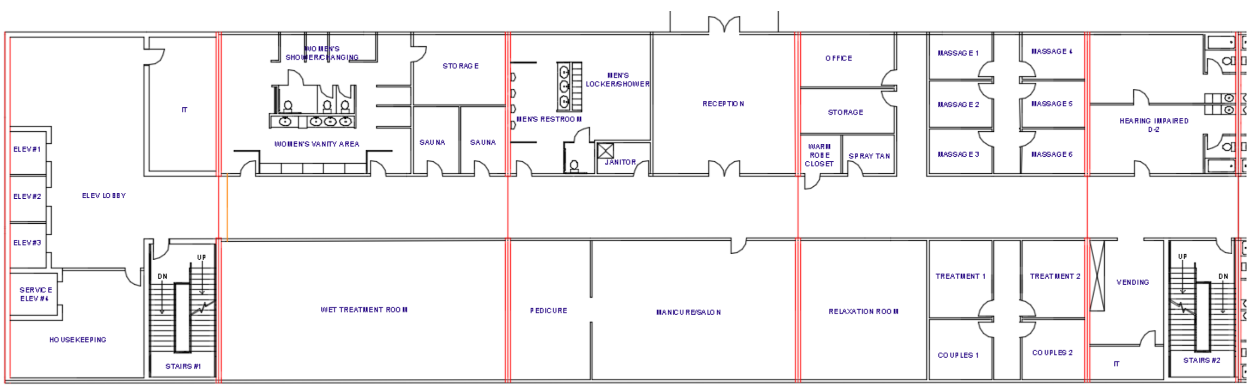
B.2 Redesigned Floorplans



Basement



Level 1



Level 2

Appendix C

C.1 Existing Load Bearing Masonry Shear Wall Cost & Schedule Estimate

Existing Load Bearing Masonry Shear Wall Cost Estimate

			Unit		Cost per S.F.	cost	daily output	labor	labor hour per S.F	Total Hour
Reinf. Conc Masonry Wall			Area (S.F.)							
Hollow	12x8x16	105 p.c.f.	2201.1	#5@48	13.9	$ 30,595.29	300	0.16	0.000533	1.17392
			933.8	#5@24	16.75	$ 15,641.15	300	0.16	0.000533	0.498027
			3200	#5@16	17.7	$ 56,640.00	300	0.16	0.000533	1.706667
Momenr Frame			Length (ft)							
	W14x109		32		350	$ 11,200.00	965	0.578	0.000599	0.019167
	W18x40		7		73.5	$ 514.50	960	0.83	0.000865	0.006052
			Cost per wall	$ 114,590.94		3.404				
			# of walls	8		8				
			Total Cost	**$ 1,031,318.46**	**Total Hour**	**27.23**				
						4 days				

C.2 Redesigned Staggered Truss System Cost & Schedule Estimate

Typical Frame S6

Truss Chord

Floor	Section	total length	cost/lf	total cost	daily output	labor	labor hour per L.F.	total hour	to comple floor
Roof	W10x60								
8	W10x60	133.32	101	13465.32	500	0.102	0.000204	0.02719728	0.217578
7	W10x77								
6	W10x77	133.32	130	17331.6	500	0.102	0.000204	0.02719728	0.217578
5	W10x88								
4	W10x88	133.32	149	19864.68	450	0.102	0.000227	0.0302192	0.241754
3	w10x112								
2	w10x112	133.32	160	21331.2	450	0.102	0.000227	0.0302192	0.241754
Gound									

Diagonal Member

Floor	Section	total #	cost/lf	total cost	daily output	labor	labor hour per L.F.	total hour
Roof	HSS8x6x1/2	6	566	3396	54	1.037	0.019204	0.115222222
8	HSS8x6x1/2							
7	HSS8x6x1/2	6	566	3396	54	1.037	0.019204	0.115222222
6	HSS8x6x1/2							
5	HSS10x8x1/2	6	863	5178	50	1.037	0.02074	0.12444
4	HSS10x8x1/2							
3	HSS10x8x1/2	6	863	5178	50	1.037	0.02074	0.12444
2	HSS10x8x1/2	2	863	1726	50	1.12	0.0224	0.0448
Gound								

* Diagonal member's schedule was excdluded due to prefrabrication of staggered truss system.

Column

Floor	Section	total length	cost/lf	total cost	daily output	labor	labor hour per L.F.	total hour	to comple floor
Roof	W12x65								
8	W12x65	44	99.25	4367	1000	0.056	0.000056	0.002464	0.019712
7	W12x87								
6	W12x87	44	131.48	5785.12	984	0.057	5.79E-05	0.00254878	0.02039
5	W12x120								
4	W12x120	44	179.59	7901.96	960	0.058	6.04E-05	0.002658333	0.021267
3	W12x152								
2	W12x152	68	226.24	15384.32	936.72	0.059	6.3E-05	0.00428303	0.034264
Gound									

per wall	$ 149,166.24
# of walls	8
total cost	**$ 1,193,329.92**

Total Hour	**1.217156**
	1 day

C.3 Façade Redesign Cost & Schedule Estimate

Exteior Walls		B2010152	I.I.F.S. Cement board sheathing, 3-5/8" metal studs, 16" O.C., 4" EPS			
Cost per S.F.	**Total Area (S.F.)**	**daily output**	**labor**	**labor hour per S.F**	**Total Hour**	**Total Cost**
$ 18.30	54913.7	250	0.16	0.00064	35.14	$ 1,004,920.71
					5 days	
With 10% increase in scheudle:					38.66	6 days