Alternative Structural Systems for Simmons Hall

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Background and Problem Statement


Figure 1: Simmons Hall

Simmons Hall opened to great fanfare and controversy in the fall of 2002. As students took up residence in the new dormitory, discussions around campus centered on two primary questions: was the new dormitory "a nice place to live," and was it "worth the cost." These two questions were very related to the goals of MIT when Simmons was under consideration. The quality of life questions are very relevant to the Institute's primary goal of fostering a sense of community throughout the new living group. The cost question stemmed from MIT's desire for the new building to be architecturally significant.

Two years after the opening of the residence hall, the success of the dormitory in accomplishing the goals set forth for it is mixed. Specifically, the structure itself seems to have limited success in fostering a sense of community, and the overall quality of life does not differ significantly from that of other dormitories. In the architectural field, Simmons has become fairly well-known because of its unique design and the methods used in construction. However, the worth of the building and the benefit/cost analysis seems to be a subjective judgment which differs significantly between MIT administration and students.

Several techniques were used in design and construction which were intended to create a sense of community among the residents of Simmons Hall. For example, the walls of the building are packed with windows, attempting to create a sense of openness and light. Each floor has abundant common space, and the commons are linked to other floors by in-room staircases. Finally, the ground floor of the dormitory is devoted to shared space, including lounges and dining.

Unfortunately, many of these efforts seem to be only marginally effective at accomplishing the goals set forth. The lounges often sit empty, although some floors do use their space more than others. The dining facility is well used, but anecdotal reports indicate that most people eat with the same groups each night, thereby reducing the amount of socialization that could occur. The biggest challenge seems to be a failure with the "openness" desired by the architect and administration. On a walk through Simmons, it quickly becomes apparent that most people stay in their rooms with their doors shut. This nullifies the effect of the many windows, and has a negative impact on the sense of community which could otherwise develop.

The cost-effectiveness of Simmons Hall is a less objective issue. The dormitory was enormously expensive to construct, with the cost estimates ranging from $125 million to $300 million (the exact figure is "confidential"). Administrators almost unanimously support the costs, and President Vest often cites Simmons as being a bold statement and a worthwhile investment in community[1]. Students, however, have mixed opinions.

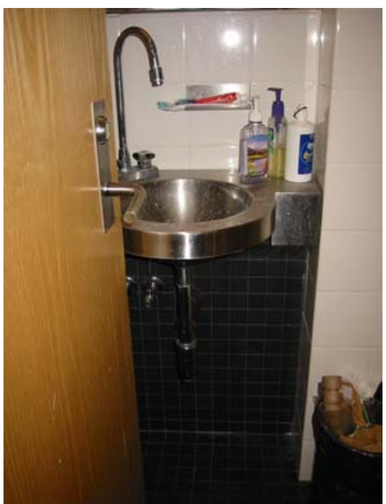


Figure 2: A bathroom in Simmons Hall

As we began to interview students about their opinions of Simmons Hall, we found that few students agreed on the value of the building. Several consensus opinions did emerge, however. The residents of Simmons were generally enthusiastic about the dormitory and liked the facilities. All the residents with whom we talked – both freshmen and upperclassmen – were especially satisfied with the size of the rooms. Also, residents were happy with the number of bathrooms. Many rooms have personal bathrooms or share one between 3 or 4 students. However, several freshmen remarked that the bathrooms were uncomfortably small.

Residents of the Hall are also appreciative of the efforts made to foster community. Rebecca Idell '07 stated that she likes that the "lounges are all connected to a different floor so you can meet people you don't normally see." Also, students are generally satisfied with the amount of shared-study area, and in contrast to the lounges, these study spaces seem to be well used. The kitchen facilities are also popular, and

[1] Cameron, Jay. "Vest Starts New Year at Convocation." *The Tech*, 26 Aug. 2003, A1.

many students reported that they had cooked for themselves at least occasionally. The first floor dining hall is also well used, and Idell stated that the communal tables are sometimes a good way to meet other residents.

Students are ambivalent about the design of the building, though. While there exists among MIT undergraduates a consensus that the building is unique, a majority of students seem to dislike the appearance. Many residents of other MIT undergraduate dormitories dislike the "sponge" theme, and SeongMin Kim '05 went so far as to call the building "an eyesore." Also, many engineering students questioned the necessity of the cutouts. "The missing sections seem like they would add a lot to the cost of the building, and have a questionable return on the investment," said John Huss '04, referring to the complex engineering necessary to support floors via hanging girders and cantilevered beams.

Figure 3: Simmons Hall dining room

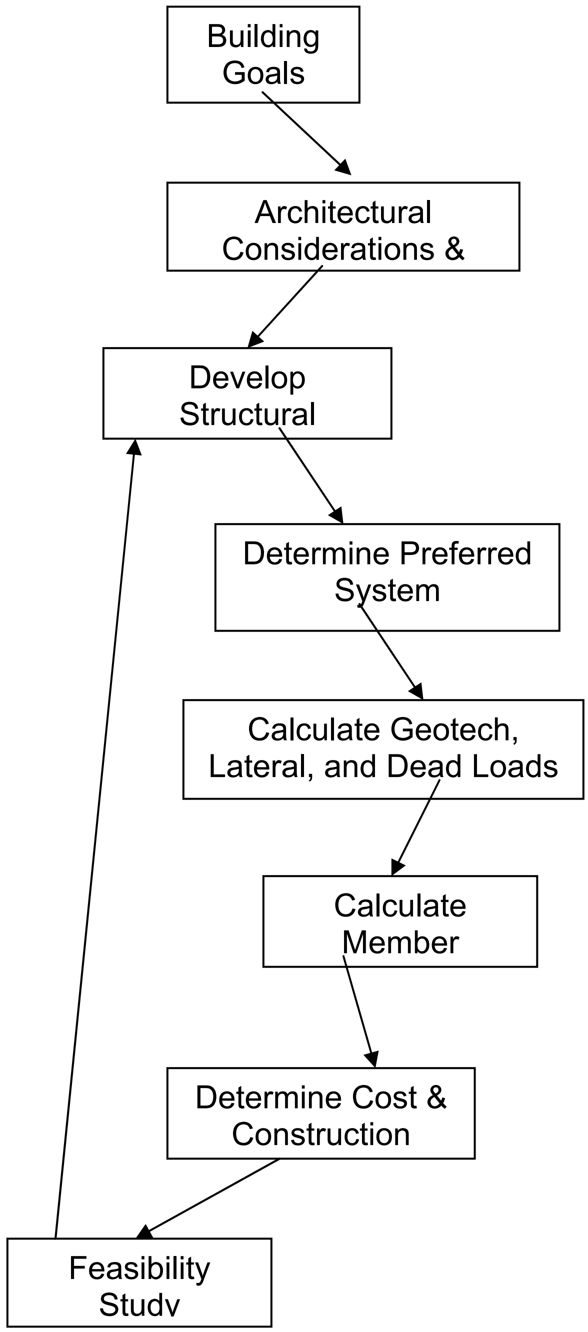
A general statement to summarize undergraduate opinion about Simmons would be that students like the facilities, but think that similar goals could have been accomplished with a more traditional design. "Simmons Hall is like Burton-Conner, but with a different exterior and a dining hall," said Alison Baker '05. Burton-Conner is one of MIT's oldest undergraduate dormitories. Simmons Hall may be architecturally significant, but student opinion seems to suggest that a more traditional design would have been preferred.

Architectural Considerations and Design

As the main function of Simmons Hall is act as a dormitory, this had to be the first consideration during design. It accomplishes this aspect, boasting 350 beds for students as well as additional beds for visiting professors. In addition to this main task though, it was very important that Simmons not only blend in to the surrounding community, but promote community as well. The dormitory has a large emphasis on open space for students to congregate so that they can bond with each other. In addition, Simmons has its own dining facilities so that students may eat with each other and feel more at home inside their dorm. Simmons was supposed to blend in with the buildings around it, but Simmons has become very well known for sticking out and catching the eye of passersby.

The design is revolutionary in several aspects. Not only is the sponge exterior a completely unique façade, but the structural integrity of the exterior is also one of a kind. MIT was willing to spend money to hire an architect who would deliver a design that would make a lasting impression on all who saw it. The final design can be described at porous. The outer membrane is perfcon, with squares cut out to create the sponge effect. Although this surface may seem to be purely aesthetic, it serves as a load bearing wall and it the key support for much of the building. Once all this had been decided upon, the building had to be built in time to accommodate the incoming freshmen class. The construction period was extremely time critical. The design had to incorporate this into the final concept. The perfcon had to be constructed quickly, and the final design for the foundation had just been completed right before construction began.

Design Flow Chart



Geotechnical Review

I: Site Conditions (all data Haley & Aldrich 1999):

The site has several distinct layers, as described below in order of increasing depth:

1. A layer of fill: sand with varying amounts of gravel, brick, and other material, ranging in depth from 5.5 to 8.5 feet
2. A layer of organic materials: very loose silt and peat with some sand and gravel, ranging in thickness from 0 to 8.5 feet
3. Medium to very dense, coarse to fine sand and gravelly coarse sand, with some clay and gravel, 13 to 33 feet thick
4. Very soft to hard silty clay, ranging from 167.5 to 176.5 feet in thickness
5. Medium to very dense silt, with mixed gravel, sand and clay, 6.5 to 11.5 feet thick.
6. Very dense silty coarse to fine sand, gravelly sand, with some gravel, clay, and boulders.

The final refusal depth was 224.5 feet.

Other factors that had to be considered included the water table at 12 feet below grade, which would require waterproofing of any foundation below that level, and the existence of neighboring buildings, roads, and railroad tracks, which would require special measures to prevent their settling while construction was occurring.

II: Original Engineer's Recommendations

The Haley & Aldrich report, 1999, made recommendations based on the size of the final building and the depth of basement to be used. A summary:

1. A tower of 9 to ten stories with a 1.5 story basement could use a concrete mat "floating" foundation. Resulting differential settlements would be within generally acceptable limits, an approximate distortion ratio less than 1:450.
2. Taller towers, greater than ten stories, would require successively deeper excavations to allow create a raft situation.
3. Towers greater than 20 stories in height would require end bearing piles. Site conditions indicate that end bearing piles would have to extend approximately 200 feet below grade, but significantly more site exploration would need to be performed to confirm the exact design of piles.

III: Final Design

The final design uses a 10 story tower with roughly 1.5 basement levels. It is based on a 4 foot thick slab foundation. The weight of the soil excavated is approximately equal to the final weight of the building, so a "mat" situation is created with virtually no settling (after allowing for "heave" during excavation and the small settling that follows).

IV: Construction Procedures

Because the foundation extended below the water table, much of it required special waterproofing. During construction de-watering wells were used to keep the site clear. Extra deep pads below the elevator shafts specifically required constant de-watering.

The excavation site was surrounded by sheet piles driven below the excavation level. These piles were braced both diagonally in the corners and horizontally across the lot's shorter dimension using large steel beams. These measures prevented any significant settling around the neighboring railroad tracks and road during construction.

V: Possible Alternate Geotechnical Designs

There are three major types of foundations available: spread foundations using slabs or footings, pile foundations, and pier foundations. In general spread foundations will be the most economical when they meet design requirements, because they don't require very deep excavations or driving of piles (Huntington, 42).

In this case, for the required building size it was possible to use a spread foundation, specifically a mat or "raft" foundation. Because site conditions would require end bearing piles to be approximately 200 feet in depth (extremely deep), costs associated with construction and driving of piles would be extremely large. The excavation and large slab required by the mat foundation are comparatively cheap and easy.

In the event that building design were to call for a building of greater than 20 stories (or if different loading conditions, say a heavy manufacturing plant on the same location that created the same loading conditions) end bearing piles would be necessary. With the current building design using 20 bays horizontally and 3 vertically, there are 84 points where columns contact the foundation. Using combined footings to connect some of these columns, the number of points at which piles would be driven could probably be reduced to approximately twenty. Because pile foundations require at least three piles per foundation to ensure stability (Huntington 43), approximately sixty piles would have to be driven to a depth of near 200 feet. Piles of this length are very near the limits of what can be done with available technology.

Though construction of a pile foundation on the site for a building greater than 20 stories in height would not be unprecedented, it would almost certainly not be economically viable. The resulting costs and increased construction time would be high enough that we can essentially assume the building should be limited to 20 stories maximum, and that an appropriately deep basement to create a "raft" situation is a design requirement.

The final building as constructed required relatively large excavation, and an extremely large foundation slab, but this method appears to be much more efficient than any alternatives available, largely due to the soil conditions on site.

Perforated Concrete (Perfcon):

Background

The exterior skin of Simmons Hall has a unique appearance, but the "sponge look" is special for reasons other than just how it looks. Simmons Hall was designed with an exterior of perforated concrete. This material, commonly referred to as "perfcon," is fabricated of reinforced concrete and plays a crucial role in the support of gravitational loads in a structure. In the design of Simmons Hall, the vertical members of perfcon act as columns, each one supporting a small faction of the load.

Architectural considerations played a primary role in the use of perfcon on the outside of Simmons Hall. The many slender columns and the equally spaced horizontal members allowed the exterior of Simmons to be packed with windows: approximately 6,000 in all. Also, the inclusion of reinforced concrete members in the exterior shell meant that massive columns were not necessary along the exterior edges of the building. Accordingly, perfcon functioned as a load bearing shell and exterior of the dormitory.

However, because Simmons was one of the first buildings to use perfcon as load bearing members, it was difficult to find a manufacturing firm willing to build the perfcon segments. In the end, only one precasting contractor was willing to complete the job. The manufacturing process was difficult not only because of the complexity of each section, but also because of the precision necessary to ensure the sections were of appropriate strength. In addition, large amounts of reinforcing steel were necessary to provide tension strength because of the irregular design.

Figure 4: Construction of perfcon sections

Construction and erection of the perfcon sections also proved to be difficult and expensive. Because construction crews had little experience with the techniques necessary to build with perfcon, a mock up was constructed and tested to ensure that the technique would produce adequate results. This mock-up was necessary, but it added time and expense to the overall design of the dormitory.

Figure 5: Erection of perfcon mock-up

The joints between the sections of perfcon were especially difficult to construct. Joints between the sections were designed to an extremely high strength specification so that the members themselves would yield before the joints did. This strength was accomplished in vertical members by using spliced sleeve joints with 1-inch dry-packed grout. For horizontal couplings, wet joints and bar-lock couplers were used.

Although the joints were very strong, this strength came at a high price. The joining method itself was very expensive, but also required a large number of engineer-hours to oversee the joining process. Construction crews were unused to the joint methods, so an engineer had to personally supervise and direct the creation of all joints between each of the 290 panels. Other high costs as a result of perfcon included the transportation of each section from the manufacturing plants to the construction site.

Finally, quality control was closely monitored throughout the entire process. The manufacturer was held to a high standard of precision, which was ensured through spot-checks and mock-ups. The joints were each supervised by engineers, which helped to maintain the expected level of quality.

Joint Performance

Joints between perfcon sections were designed to be stronger than the perfcon members themselves. This design choice was made because the joints were considered to be more vulnerable than the members. Also, if the members were designed to support the necessary loads, and the strength of the joints exceeded that of the members, the shell of

Simmons would be very structurally sound. This choice also makes sense because it is easier to control the strength of pre-fabricated concrete than to control the strength of cast-in-place joints.

The strong joints proved to be very expensive because they used a great deal of steel. Over-designing the joints significantly increased the cost of construction, and the engineering hours required to make connections between the sections was very expensive. However, the joints did support the appropriate loads in compression tests.

Figure 6: Inspection of Perfcon section joints

Alternative Approach

An alternative to the load-bearing perfcon could have the majority of the gravitational loads supported by columns in the core of the building. The exterior of the building would be strong enough to resist wind forces, and would provide shear support, but would not bear as much of the gravitational load as the current system does. The same architectural effect and abundance of windows could be achieved through a steel and aluminum façade.

This approach would reduce the amount of reinforcement needed in the perfcon walls, which would result in more economical construction budgets. Also, the joints between the exterior sections would be less crucial, resulting in fewer engineering hours and a more efficient construction schedule. The beams and columns that would support that majority of the gravitational loads could be cast-in-place concrete, which would reduce transportation costs. Also, using a more traditional load-bearing support structure would make procurement of the necessary members easier because the construction teams would not be reliant on only one supplier. Finally, this alternative would allow more open space for utilities in the exterior of the building.

Load Calculation

Wind and Earthquake Loads –

Height (ft)	Wind Load (kips)	Earthquake Load (kips)
100	4.42	2.14
90	4.3	2.75
80	4.22	2.36
70	4.06	1.98
60	3.92	1.62
50	3.78	1.28
40	3.6	.958
30	3.38	.659
20	3.04	.389
10	2.92	.158

Live Loads –

The calculation of the live loads was carried out in the following manner. First the different uses of the floor space in Simmons were determined and the following categories were determined:

- corridors & stairways
- bedrooms
- lounges
- kitchens
- open

Then loading conditions were associated with each of these categories:

80 lb/ft^2 - corridors & stairways

50 lb/ft^2 - bedrooms

60 lb/ft^2 - lounges

60 lb/ft^2 - kitchens

0 lb/ft^2 – open

Then the total area associated with each category was determined:

4081 ft^2 - corridors & stairways

12243 ft^2 - bedrooms

3678 ft^2 - lounges

204 ft^2 - kitchen

Structural Component Selection and Placement

The gravity and lateral load resisting components of Simmons' structural system consist of slabs, beams, columns, and perfcon. All of these load resisting components are placed within the structural system in such a way so as to allow for the desired architectural considerations. These considerations are for the exterior shell to be pierced with many small openings as well as a few larger openings, open atrium spaces within the building, and as much uninterrupted room and lounge space as possible.

Load path of Simmons is somewhat unique due to the existence of the perfcon that surrounds the building. The perfcon is able to resist lateral loads much better than a weak façade, but the loads are still distributed through the building to the inner rows of vertical supports. These supports work with the perfcon to dissipate the effects of the forces. Gravity loads run from the concrete slabs that make up the floor to the beams that support the edges of these slabs. From the beams, the loads are transferred either out to the exterior walls that are made of perfcon, or to one of the central rows of columns that runs through the building. The perfcon and columns both transfer the loads down and into the ground.

Perfcon was specially designed and formed in precast sections able to perform structurally, even though the sections are pierced with multiple openings for windows. The perfcon itself serves to resist the majority of the wind loads because its stiffness and continuity allows it to resist the lateral loads as they are applied to it. The perfcon also supports a large percentage of the gravity loads as well. A load applied to the floor is transferred along the floor slab to the perfcon on the outer edge and then down to the ground. The perfcon allows for large atrium openings throughout the building because the majority of the weight is in a sense cantilevered out from the exterior walls and not supported by columns placed at frequent intervals.

The columns that are incorporated into Simmons are placed in two rows along the length of the building. This works very well because some sort of internal vertical support is needed, however when placed in this way, the columns run along the walls of the central corridor allowing them to avoid interrupting any open spaces.

Approximate Analysis

Approximate Analysis was carried out on the top two floors of Simmons. Since the wind loads dominate the earthquake loads, and there is no load combination for the two, the approximate analysis was done using wind, live, and dead loads only. The analysis was only done on the narrower of the two cross sections because this direction is less stable and we must design for the weakest scenario. It was approximated to simply be a system of beams and columns, neglecting the slabs that exist in the structure. Again, this was done to simplify the calculations. The only effect it will have on our calculations is to make them more conservative since the weight of the slabs is taken into account as dead load in the analysis while the stiffening properties are left out. The shear and moment diagrams that were found from this analysis are shown below. All units for the shear diagram are kips, and for the moment diagram are kips-ft.

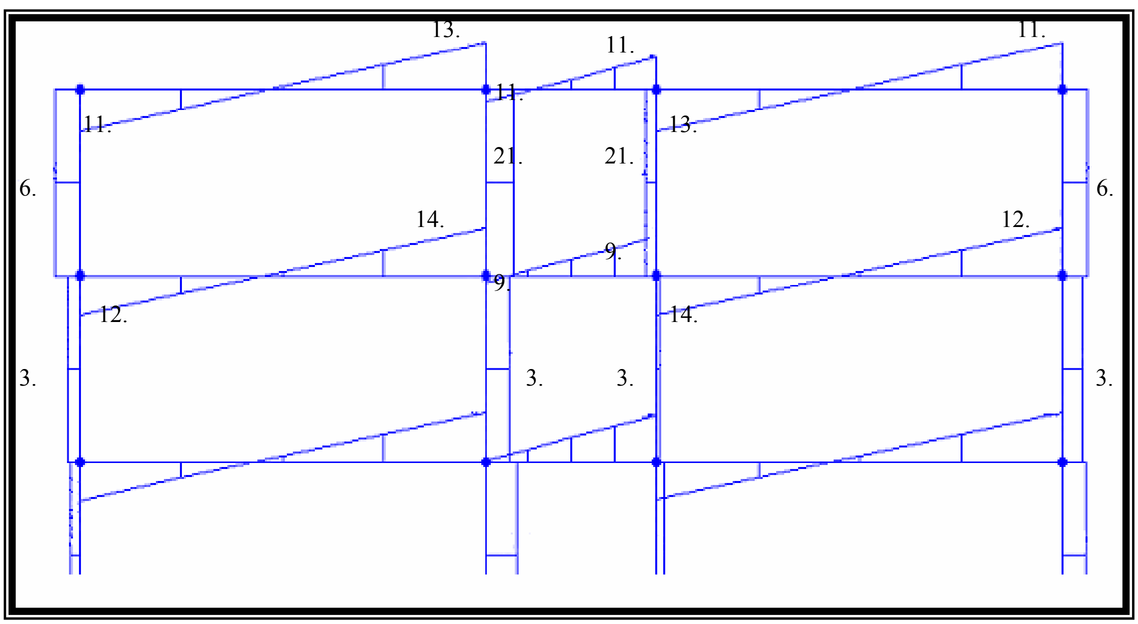


Figure 7: Shear Force Diagram for Floors 9, 10, and Roof

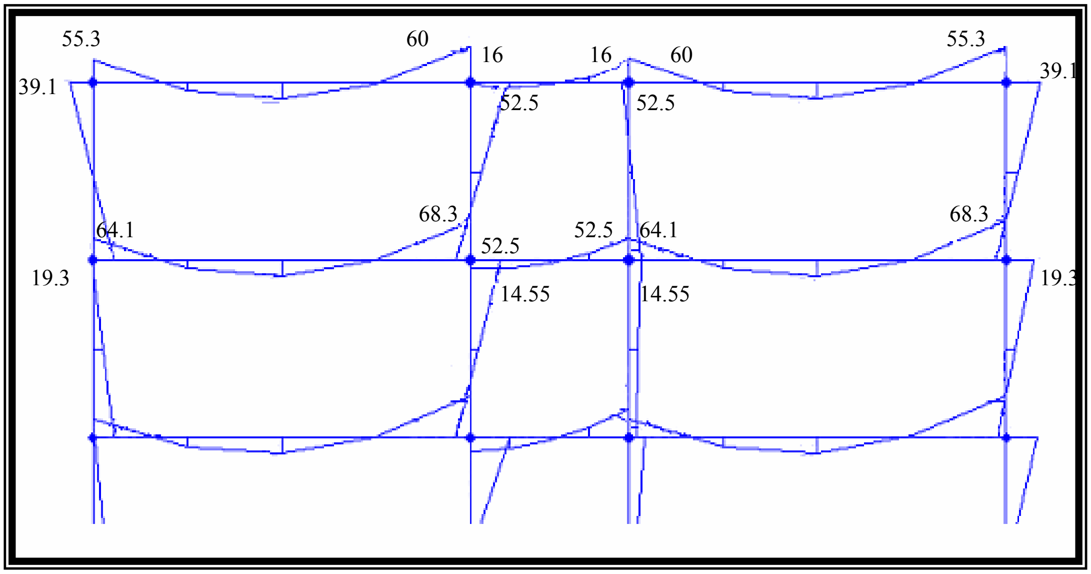


Figure 8: Critical Bending Moment Diagram for Floors 9, 10, and Roof

SAP 2000 Analysis

SAP Analysis was carried out on the building using the more narrow of the two cross sections because deformations in that direction will dominate. SAP Analysis includes the Dead, Live, Earthquake, and Wind loads. These loads were applied to the structure using the following load combination:

$$L_{total} = 1.4L_{dead} + 1.7L_{live} + 1.6L_{wind} + L_{earthquake}$$

The following are the results that are gained when these loads are applied to the structure. Again, all numerical values for shear are in kips, and for moment are in kip-ft.

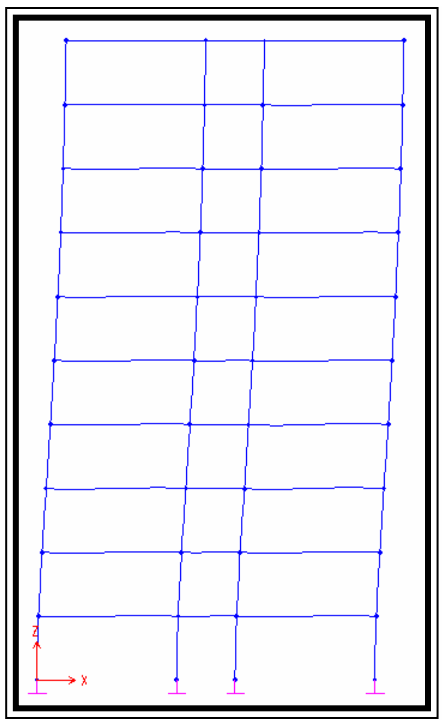


Figure 9: Frame Deformations under Positive Gravity, Wind, and Earthquake Loads

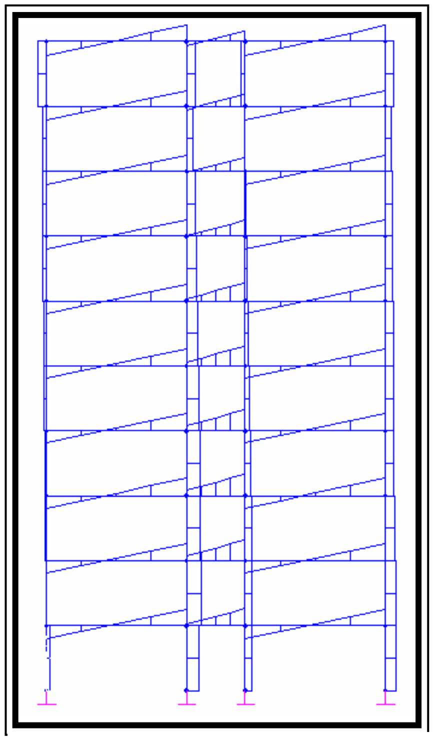


Figure 10: Shear Force Diagram under Positive Gravity, Wind,

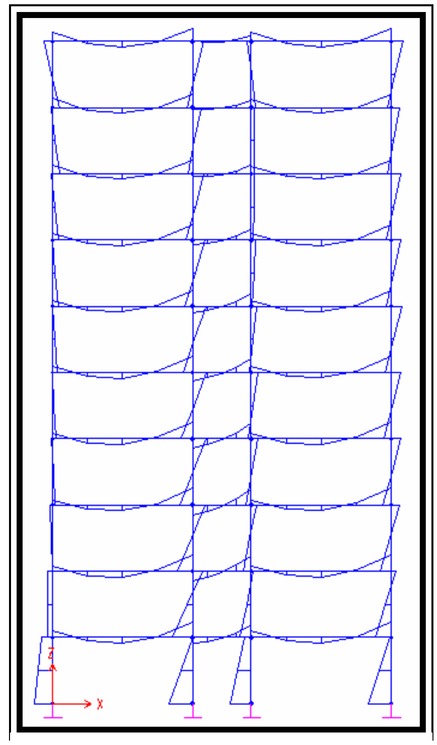


Figure 11: Bending Moment Diagram under Positive Gravity,

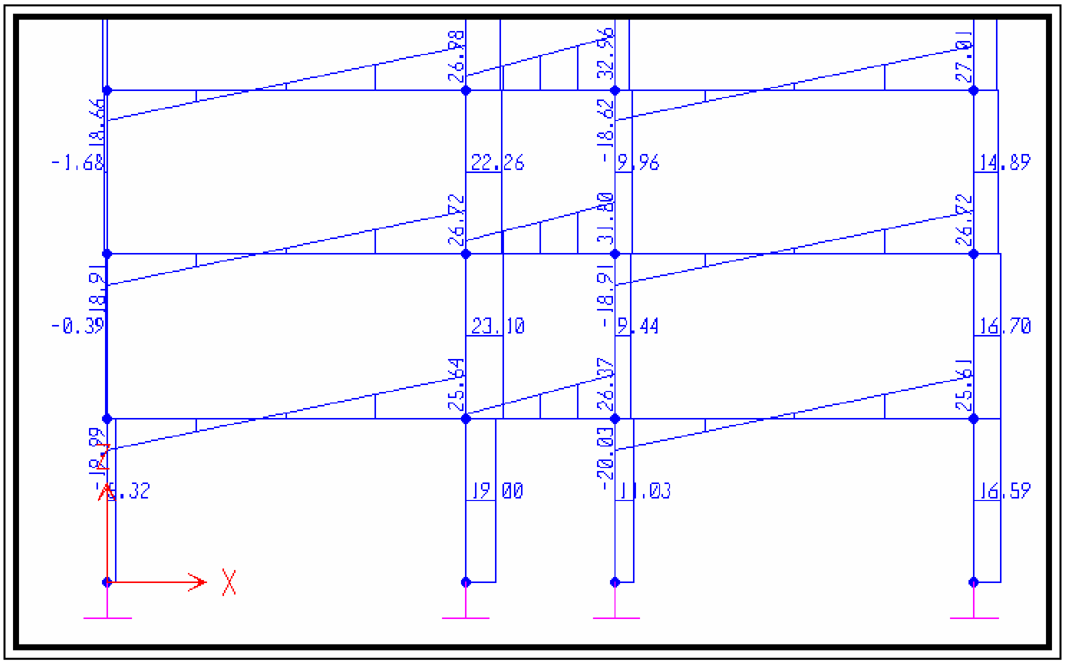


Figure 12: Shear Forces on Floors 1, 2, 3

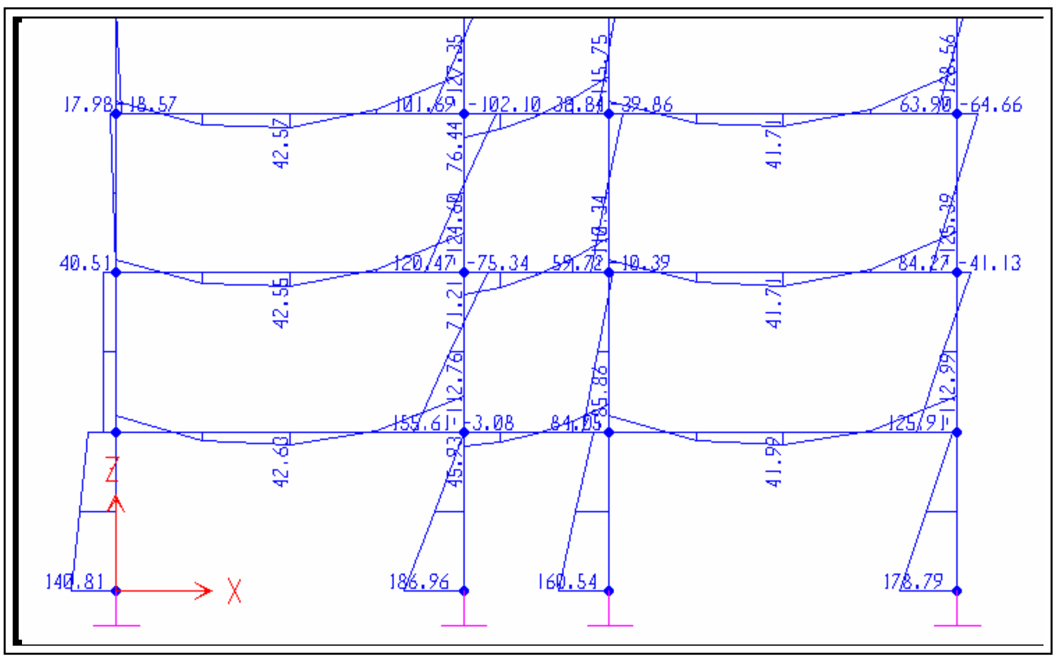


Figure 13: Critical Bending Moments for Floors 1, 2, 3

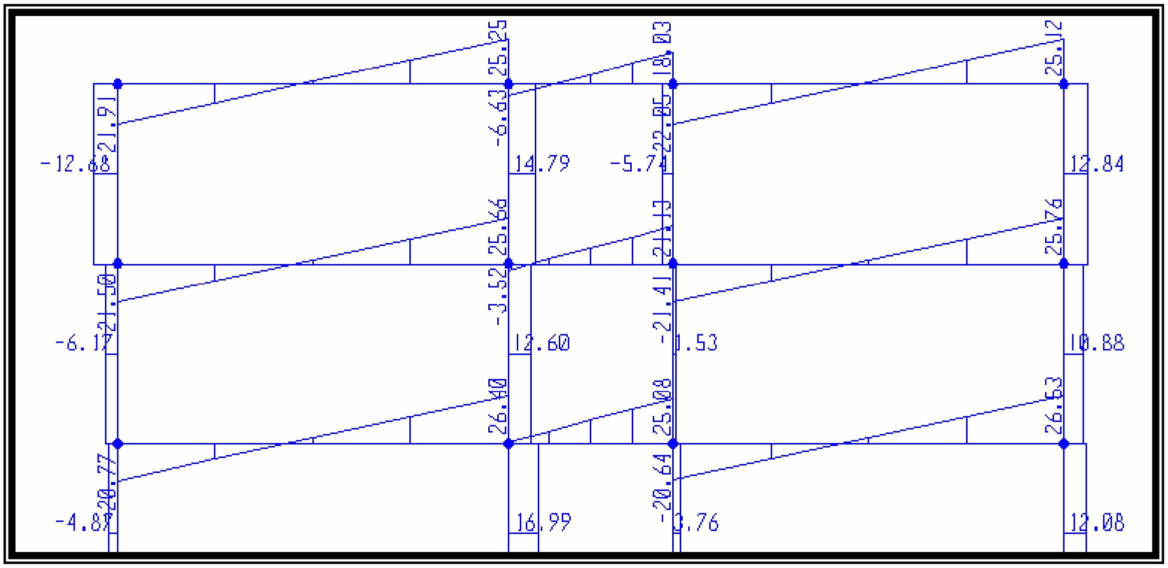


Figure 14: Shear Forces on Floors 9, 10, and Roof

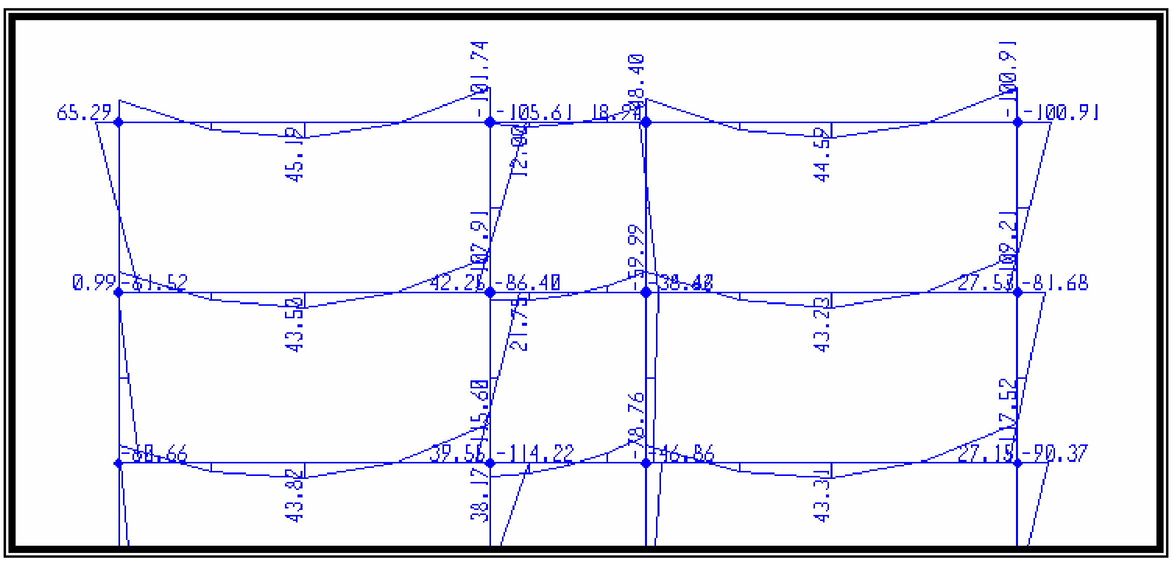


Figure 15: Critical Bending Moments for Floors 9, 10, and Roof

When these values determined by use of the SAP2000 model are compared to those determined from Approximate Analysis, they are of the same order of magnitude, however they are usually off by a factor of two, with the values from SAP being double those found from approximate analysis. This can be partially explained due to the fact that the SAP analysis included the live, dead, wind, and earthquake loads while the approximate analysis only included the live, dead, and wind loads. The other reason that the values are off may be connected with the points that are assumed to be zero moment points in the approximate analysis may in fact be incorrect.

Member Dimensions

The following member dimensions are the results of both the Approximate Analysis and SAP Analysis. The 9th floor beams and columns are based solely on the numbers derived from the Approximate Analysis, while the 2nd floor beams and columns were designed using the results from the SAP analysis.

Interior & Exterior Beams-2nd Floor

Exterior Beams

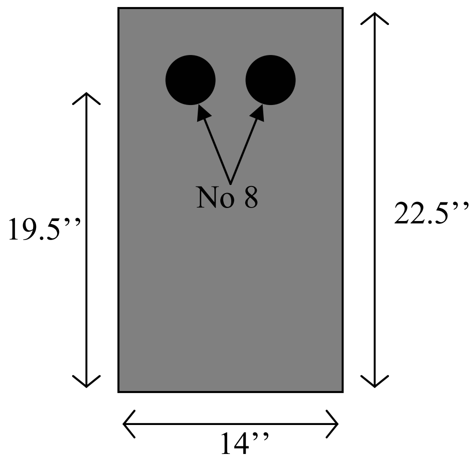


Figure 16: Second Floor, Exterior Beam, End Section

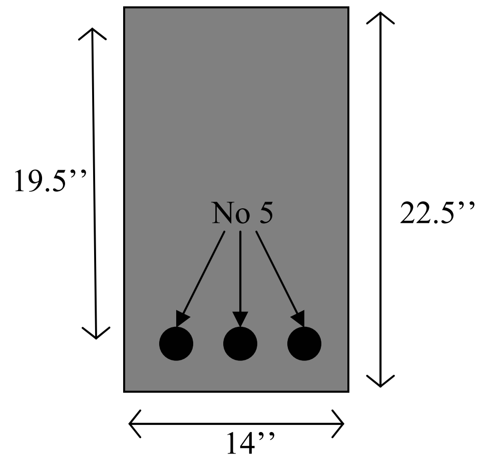


Figure 17: Second Floor, Exterior Beam, Mid-span Section

Interior Beams

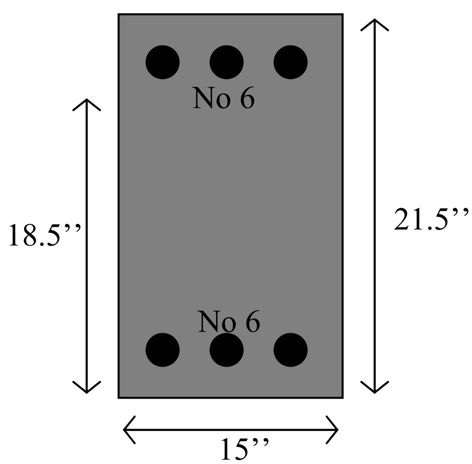


Figure 18: Second Floor, Interior Beam, End Section

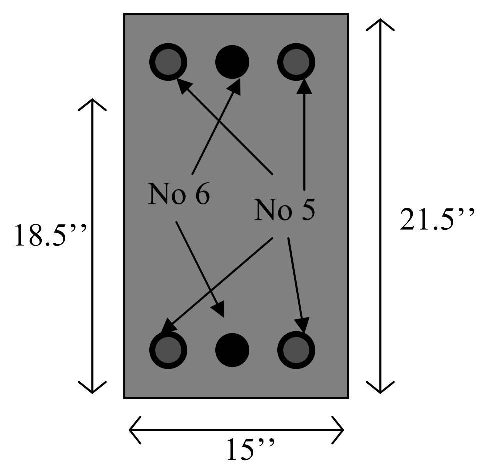


Figure 19: Second Floor, Interior Beam, Mid-span Section

Interior & Exterior Beams-9th Floor

Exterior Beams

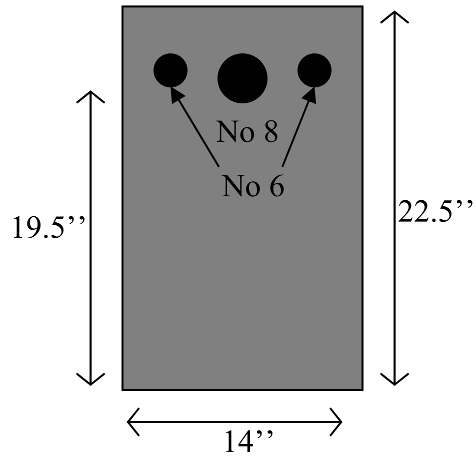


Figure 20: Ninth Floor, Exterior Beam, End Section

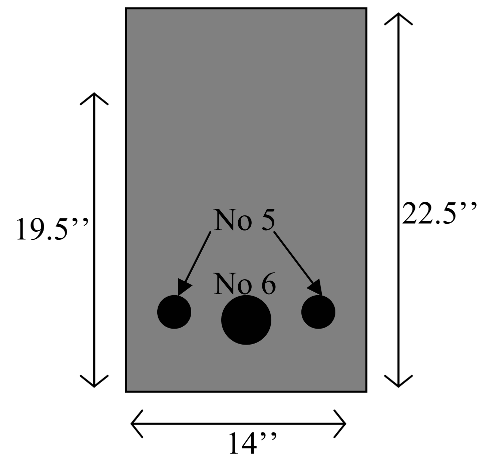


Figure 21: Ninth Floor, Exterior Beam, Mid-span Section

Interior Beams

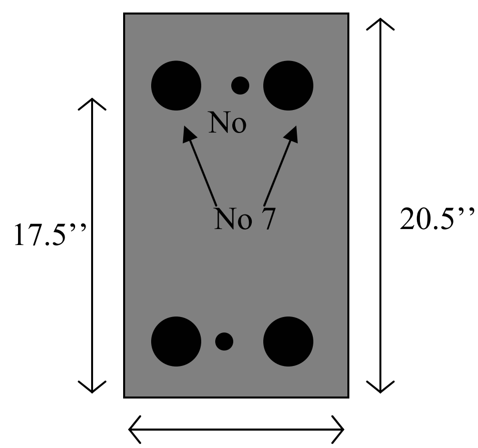


Figure 22: Ninth Floor, Interior Beam, End and Mid-span Sections

The loads on the exterior beams are:

LL = 725 lb/ft
DL = 281 lb/ft

The loads on the interior beams are:

LL = 695 lb/ft
DL = 616 lb/ft

Interior & Exterior Columns - 2nd Floor

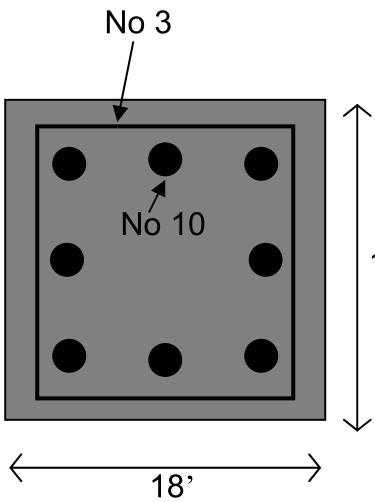


Figure 23: Second Floor, Exterior Column

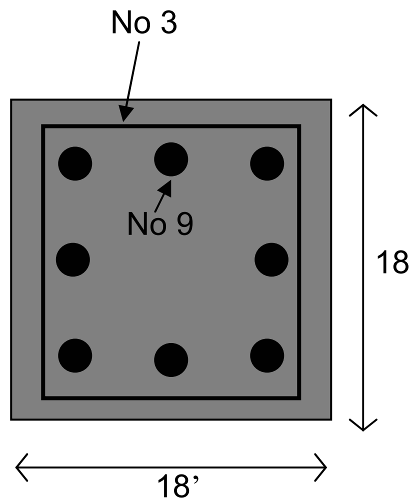


Figure 24: Second Floor, Interior Column

* Stirrups spaced at 12 inches.

Interior & Exterior Columns - 9th Floor

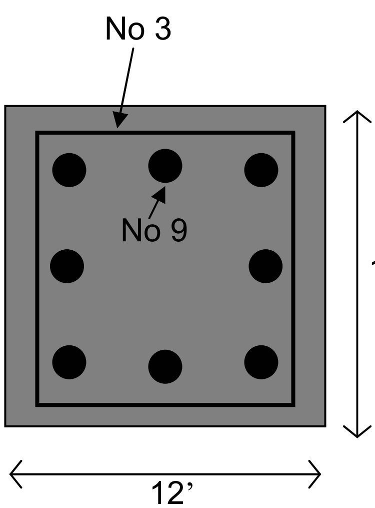


Figure 25: Ninth Floor, Exterior Column

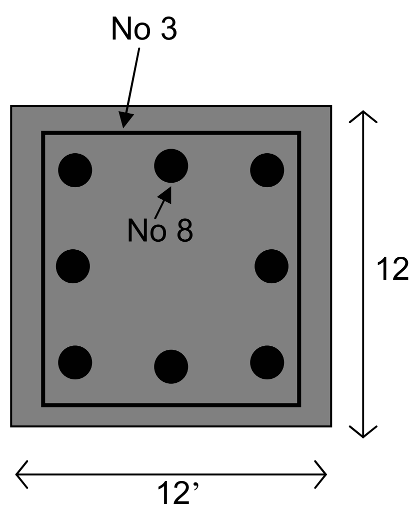


Figure 26: Ninth Floor, Interior Column

* Stirrups spaced at 18 in

The loads on the interior columns are: LL = 11.3 kips
DL = 28.1 kips

The loads on the exterior columns are: LL = 18.7 kips
DL = 35.6 kips

Interior & Exterior Slabs

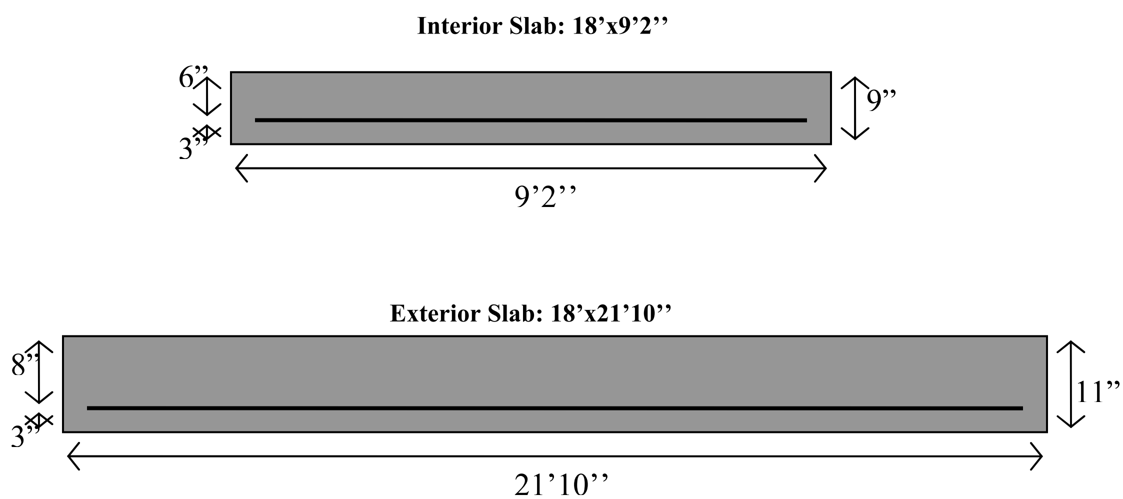


Figure 27: Slab Design

Alternate Structural System

Description

The structural system that was used in building Simmons revolved around the Perfcon elements that form the skin of the building. Not only do the Perfcon elements make up the exterior façade of the building, but they also serve as the main support system for the structure. Building Simmons in this fashion greatly added to the overall cost of construction because these elements had to be made of precast concrete so that they would be able to withstand the necessary loading conditions. Our suggestion for an alternative structural system reduces these construction costs, by eliminating the need for the Perfcon to serve a significant structural purpose. This structural system involves cantilevering the floors off of five central cores that will also serve as the elevator shafts, one running up the center of each tower, and two rising between the towers. If it is determined necessary, trusses can also be cantilevered off of existing stairways. This structural system has been used previously to construct the IBM Building as well as New York's World Trade Center. Shown below is a diagram of the World Trade Center that depicts the layout of the trusses that are cantilevered from the central core to support each floor (left). Also below is a close up that cuts through a floor to illustrate the trusses used and the layers involved in connecting the trusses to form a cohesive floor (right).

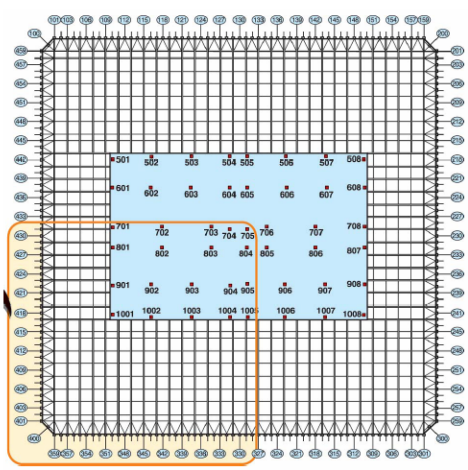


Figure 28: WTC Truss Layout

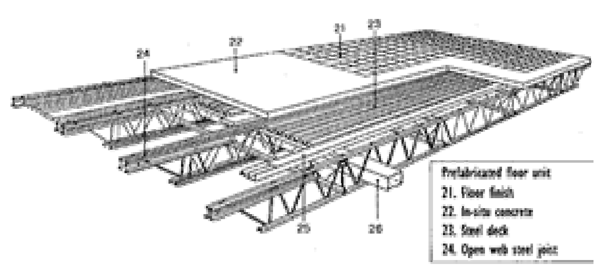


Figure 29: Truss Layers

By having the elevator cores, and possibly stairways, take 80% of the gravitational loads, there will be many beneficial consequences to the building other than simply reducing construction costs. The first of these benefits is that there is no need for any interior load bearing columns. This means that the architect will have much more freedom to develop the interior of the building with large open areas. Since it was important to the architect for the exterior of the building to be composed of many small windows, this can still be accomplished. However, this will now be done with a concrete façade which could be constructed much more readily than when major loading conditions had to be considered. Constructing this outer concrete shell with cast in place concrete, instead of precast concrete, greatly reduces both the construction cost and the construction time.

The main purpose of this façade will be to act as wind bracing for the building, although it will also be responsible for carrying up to 20% of gravity loads. The floors will be cantilevered off of the load bearing cores using prefabricated trusses. This method allows for the trusses to be constructed off site, and then transported to the site and immediately incorporated into the building. This will reduce construction time immensely. These trusses extending from the cores to the exterior façade will also serve as bracing for the façade to increase its ability to resist lateral loads. In order to assure the distribution of forces is more on the core columns, the trusses will be connected to the interior columns with a welded moment connection, and only simply supported on the exterior end.

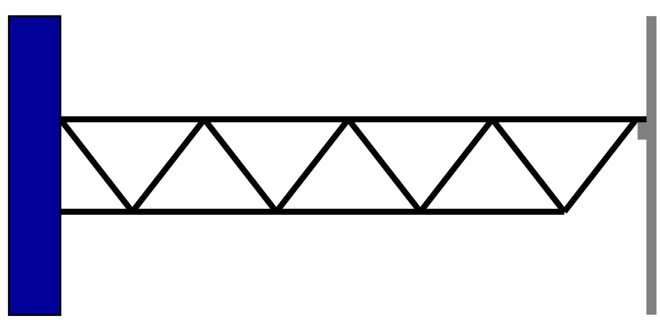


Figure 30: Sample Truss Connection

Load Path

The major difference between our alternate structure and the existing Simmons structure is that our alternate structure is based on the idea of the core columns taking the majority of the loads and the existing structure is built so that the exterior shell can support as much of the loads as possible. In the case of our alternate structure, lateral loads are mainly resisted by the exterior façade, just as in the existing structure. They are sufficiently anchored to the ground, and are stiffened by the lateral trusses to such a degree that they can deflect most of the wind and earthquake loads.

The gravity loads are transferred from the floors to the frequently spaced steel trusses. These trusses then transfer the loads either to the larger, less frequently spaced, trusses, which then transfer the load to the core columns, or the smaller trusses transfer the load directly to the core columns.

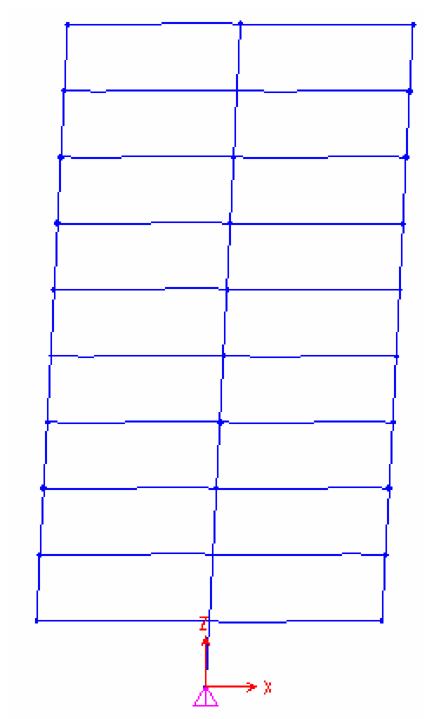


Figure 31: Frame Deformations under Gravity, Wind, and

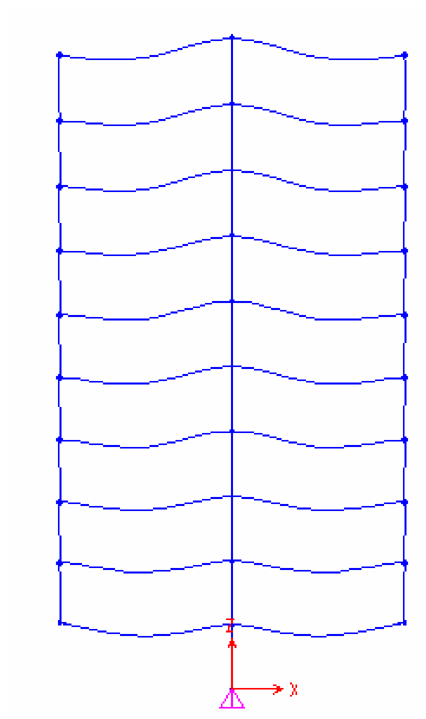


Figure 32: Frame Deformations under Gravity Loads

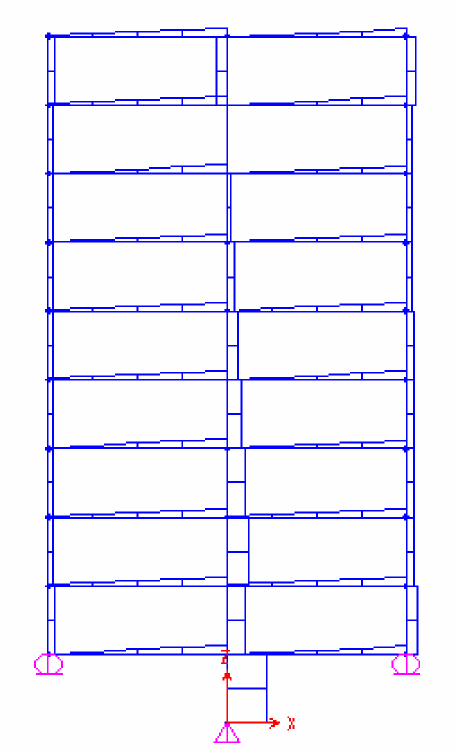


Figure 33: Shear Forces for Alternate Structure

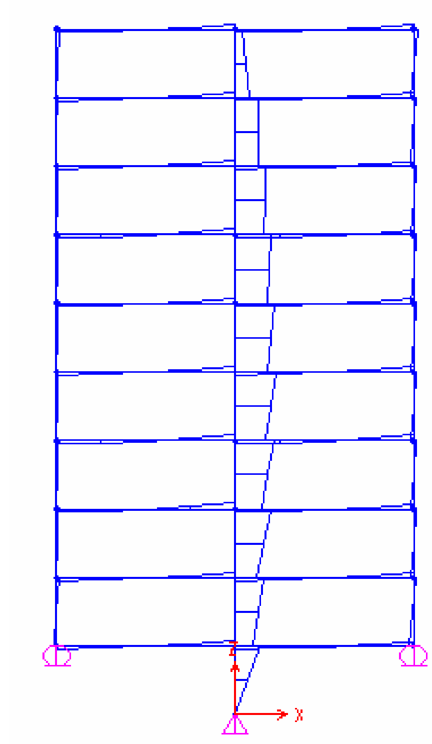


Figure 34: Critical Bending Moments for Alternate Structure

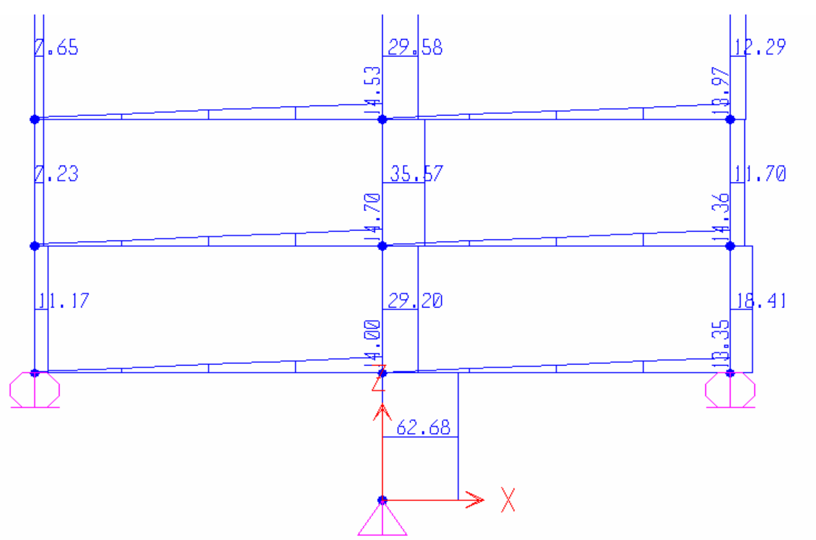


Figure 35: Shear Forces for Alternate Structure close up of lower three floors.

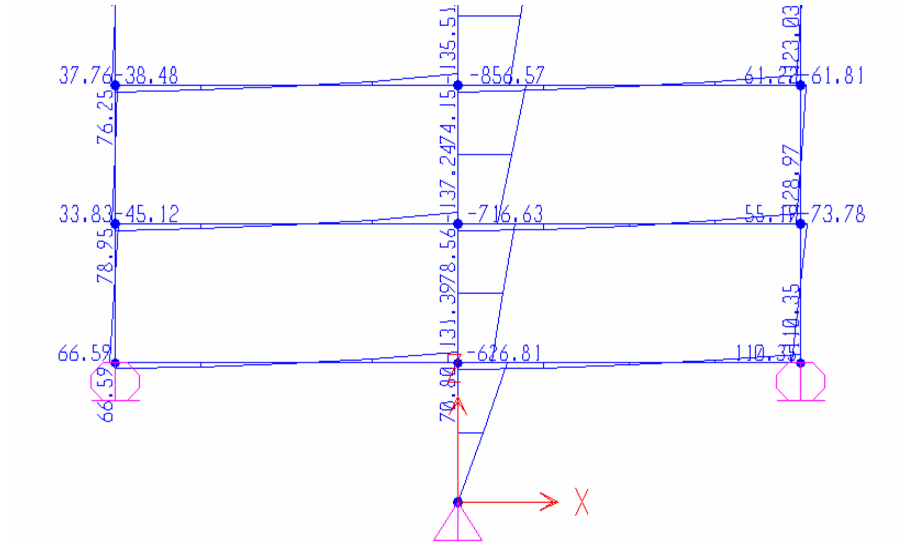


Figure 36: Moment Diagram for Alternate Structure (close up of lower three floors)

Alternative Supports for Group 3 Specific Sections

Several alternative structural designs for areas 3a and 3b were developed on a conceptual level. Each of these is briefly described with an approximate graphical representation below.

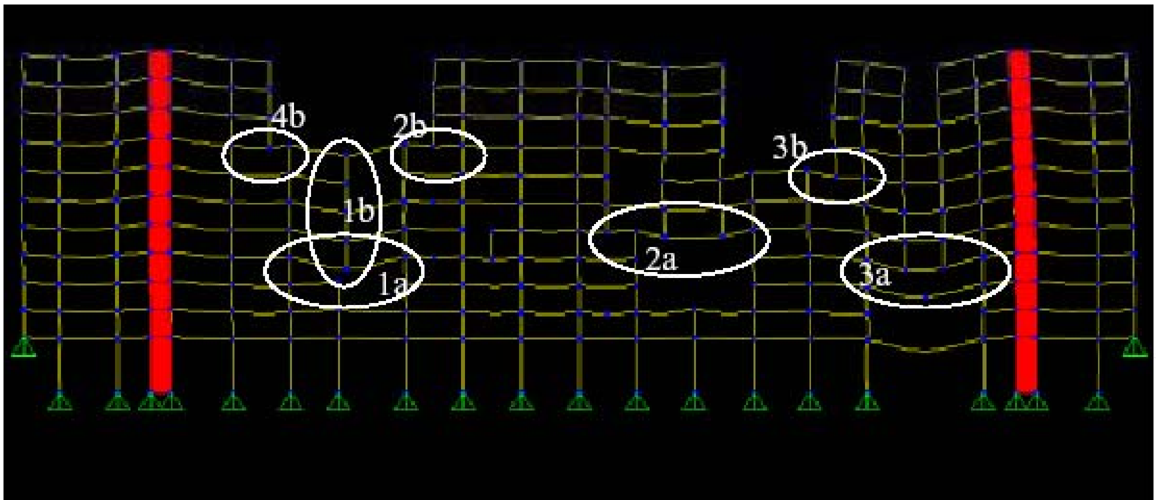


Figure 37: Specific group assignments for section design

The regions our group needs to analyze are sections 3a and 3b.

Section 3a

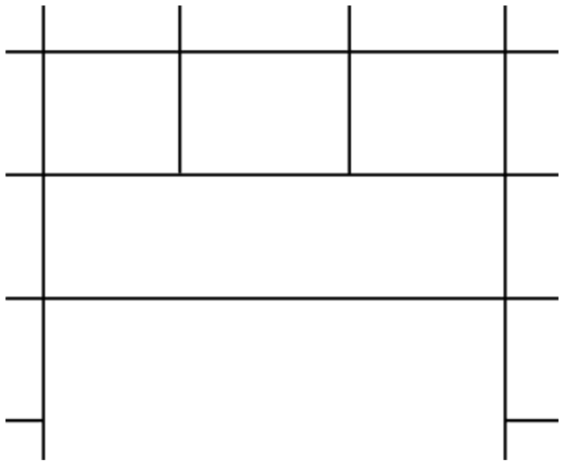


Figure 38: Existing Section 3a

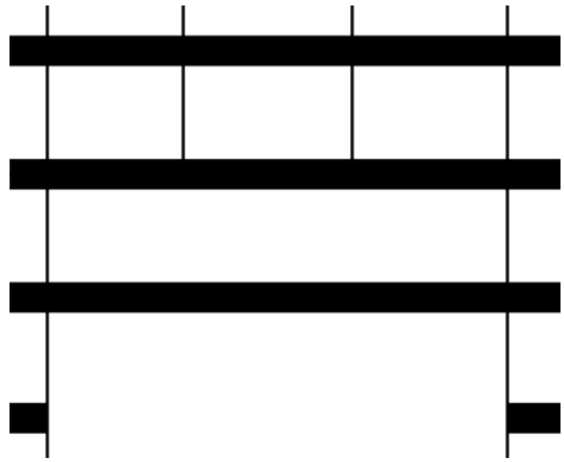


Figure 39: Alternative Design, Section 3a

Section 3a occurs in the eastern wing of Simmons Hall. It is a space over a cutout in the building, and prevents direct support from either perfcon or a column/slab design. Section 3a alternative includes large, reinforced columns on either side of the cutout. The floors above the cutout would be supported by the two large columns, and beams would run between the columns to support the floor slabs. This alternative would be bulky and unnecessary with our overall design of Simmons.

For Section 3a, design calculations were performed for alternative 1. This was the design of two steel beams used to bridge the large two-bay width gaps that are related to Section 3a. The lower of the two beams is required to support an evenly distributed load, while the upper beam is designed to also support two large point loads created by columns supporting upper floors. The re-design of this upper beam is an alternative to the system currently used, whereby this floor is actually hung from the floors above it. This floor now supports the floors above it.

Section 3b

Section 3b occurs between the 6th and 7th floors of Simmons Hall, and is the bottom corner of another cutout. The main design difficulty here is that a column rises to the 7th floor and above, but is centered between two separate columns rising from the ground.

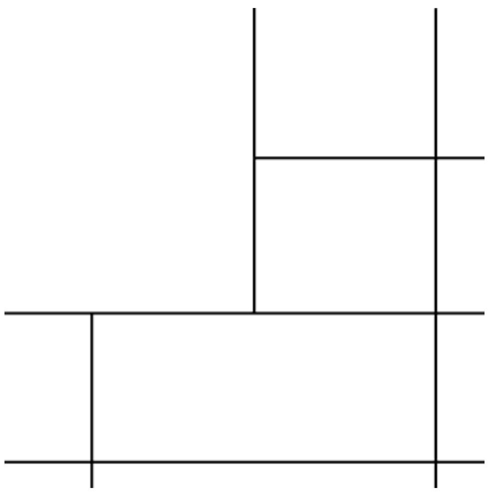


Figure 40: Existing Section 3b

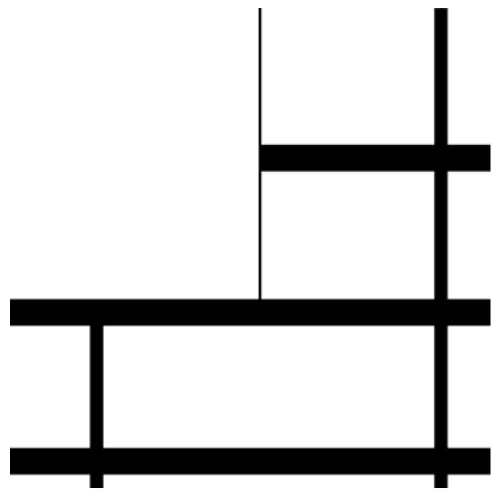


Figure 41: Alternative Design, Section 3b

For Section 3b, design calculations were performed for alternative 2. This was the design of a single steel beam over a standard bay width supporting both the usual distributed load and a large point load resulting from the non-standard bay spacing above the area.

Final beam design calculations and member selections are shown in appendix G. Major sections of the process included:

1. Estimation of distributed and point loads

2. Analysis to find maximum moment, shear, and deflection. These calculations are straightforward superpositions of case formulas for combined loading conditions.

3. Calculation of required section properties based on maximum moments and deflection. Shear calculations were not performed because it is known that shear will limit only in extremely short beams. Applying intuition to the required section properties indicates that deflection, in the form of I required, will almost certainly be the limiting factor in member selection.

4. Final member selection yielded two reasonably economical choices for each beam. In each case a slightly deeper beam would allow lower weight (and therefore cost), while a less deep beam would have higher weight (and therefore be more costly). In each case the thinner beam was selected because the small increase in cost due to weight was outweighed by the aesthetic and practical gains of a thinner section.

The alternative methods described here require only minimal modification to the columns in the area, so a detailed column re-design was not performed. The only major change necessary will be in connection details because the standard CIP concrete columns and beams are not designed to interface directly with large steel members.

Bibliography

Haley & Aldrich, Inc. Report On Preliminary Geotechnical Engineering and Environmental Investigation, Proposed Vassar Street Student Housing at Massachusetts Institute of Technology, Cambridge, Massachusetts. Boston, JAN 1999.

Huntington, WC. Huntington Building Construction. New York: John Wiley & Sons, 1956.

Appendix A: Additional Student Comments Regarding Simmons Hall

-- "The windows make everything rather claustrophobic, and take forever to open and close. The window curtains were a complete waste of time. They didn't block the light, didn't completely cover the window, and came off really easily."

-- "I wouldn't have liked to have lived in a room with a strange wall though, because I know it really messed with your floor space and decorating possibilities."

-- "It was rather cool to live in one of the newest (and most expensive) dorms, especially when Simmons ended up in the news, but that coolness didn't make up for the fact that many things did not work and the construction was a real drag for a long time."

-- "Personally, I like the suite arrangement in Burton Conner better than the arrangement at Simmons, but I preferred Simmons's common areas to Baker's."

-- "People did use the common spaces, at least the ones that were close to them. The 6th floor was one of the better ones because it had access to about 4 lounges and also had a kitchen (which wasn't really used much, but was nice to have around)."

Appendix B: Wind Load Calculations

Wind Loads

method 2 – Analytical Procedure

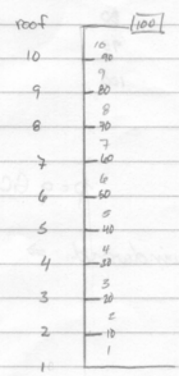
1. $V = 110$ K_d ← N/A $K_d = .85$

2. category II ⇒ $I = 1.0$

3. surface roughness ⇒ B
exposure ⇒ B

	K_h or K_z
h = 10	.57
20	.62
30	.7
40	.76
50	.81
60	.85
70	.89
80	.93
90	.96
100	.99

case #2



4. K_{zt} ← N/A

?? 5. $G = .85$ ~ for rigid building

6. enclosure classification – closed

7. $GC_{pi} = \pm .18$

8. GC_{pf}

$\frac{L}{B} = \frac{385}{58} = 7$ windward wall $C_p = .8$ leeward wall $= -.2$
$GC_{pf} = .7$ $GC_{pf} = -.8$

$q_z = (.00256)(.85)(110)^2\,(1)\,K_z = 26.3\,K_z$

	q_z	windward, p	leeward, p	p lb/in²
h= 10	15	10.2	-4.4	14.6
20	16.3	11.1	"	15.5
30	18.4	12.5	"	16.9
40	20	13.6	"	18
50	21.3	14.5	"	18.9
60	22.4	15.2	"	19.6
70	23.4	15.9	"	20.3
80	24.5	16.7	"	21.1
90	25.2	17.1	"	21.5
100	$26 = q_h$	17.7	"	22.1

$p = q\, GC_p - q_i\, (GC_{pi})$

windward ⇒ $G = .85$

$C_p = .8$

$GC_{pi} = .18$ $\qquad P_w = (q_z)(.85)(.8) - (26)(.18)$

$q = q_z$

$q_i = q_h$

leeward ⇒ $G = .85$

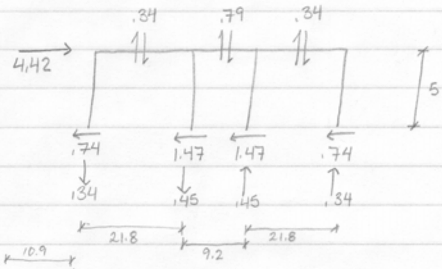
$Cp = -.2$

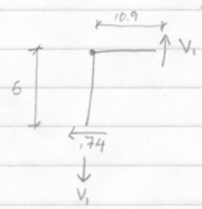
$GC_{pi} = .18$ ~ negative?

$q = q_h$

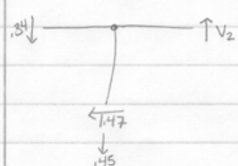
$q_i = q_z$ $\qquad P_L = (26)(.85)(-.2) - q_z(.18)$

Approximate Analysis – wind loads pg 1





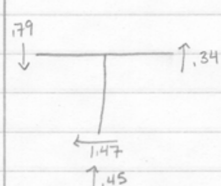
$(V_1)(10.9) = (.74)(5) \qquad V_1 = .34$



$(.34)(10.9) + (V_2)(4.6) = (1.47)(5)$

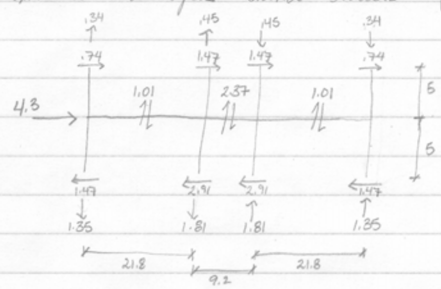
$V_2 = .79$

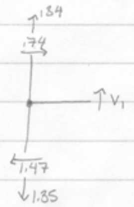
$.79 - .34 = .45$



$(4.6)(.79) + (V_3)(10.9) = (5)(10.9) \qquad V_3 = .34$

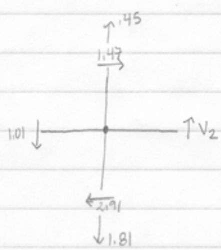
Approximate Analysis - wind loads pg 2





$$(.74 + 1.47)(5) = (V_1)(10.9) \qquad V_1 = 1.01$$

$$1.01 + .34 = 1.35$$



$$(1.47 + 2.91)(5) = (1.01)(10.9) + (V_2)(4.6)$$

$$V_2 = 2.37$$

$$2.37 + .45 - 1.01 = 1.81$$

Appendix C: Earthquake Load Calculations

EarthQuake Loads

determining max considered earthquake

1. S_S .2s spectral response 25 %g = .25

S_1 1s spectral response 8 %g = .08

2. site class – E

3. $F_a = 2.5$ $S_{ms} = (2.5)(.25) = .625$

$F_v = 3.5$ $S_{m1} = (3.5)(.08) = .28$

4. $S_{DS} = \frac{2}{3}(.625) = .417$

$S_{D1} = \frac{2}{3}(.28) = .187$

5. $T_o = .2\left(\frac{.187}{.417}\right) = .0897$

$T_1 = .448$

6. $T_a = (.016)(100)^{.9} = 1.01$ sec

$T \le T_o$ $S_a = (.417)\left(.4 + (.6)\frac{T}{(.0897)}\right)$

*can make graph S_a vs T (%g) (sec)

$T_o \le T \le T_s$ $S_a = .417$

$T > T_s$ $S_a = \frac{.187}{T}$

Base shear

category II
seismic use group I

$.33g \leq S_{DS} < .5g \Rightarrow C$ ~ seismic design category

equivalent lateral force procedure

W = dead load

$V = C_s W$ $\quad C_s = \frac{S_{DS}}{R/I} = \frac{.417}{R}$ $\quad R = 7$ $\quad I = 1.0$ $\quad C_s = \frac{.417}{7} = .06$

not ok

$max \Rightarrow C_s = \frac{.187}{(1.01)(7)} = .0264$ just use max

need building weight

base shear $V = (.0264)($

②

EarthQuake

$V = (.0264)(W) = (.0264)(\overset{\text{kips}}{542}) = 14.3$ kips ← base shear

$T_a = 1.01 \qquad .5 < 1.01 < 2.5$

$$k = 1 + \frac{(1.01 - .5)}{2.5 - .5} = 1.3$$

$F_x = C_{vx} V \qquad C_{vx} = \dfrac{w_x h_x^{1.3}}{\sum w_i h_i^{1.3}}$

	w_i	h_i	$w_i h_i^k$	F_i (kips)
1	56	10	1117.3	.158
2	56	20	2751.2	.389
3	56	30	4660.6	.659
4	56	40	6774.3	.958
5	56	50	9054.2	1.28
6	56	60	11476	1.62
7	56	70	14022	1.98
8	56	80	16680	2.36
9	56	90	19440	2.75
ceiling 10	38	100	15128	2.19
		total	101103.6	14.3

$w_i\,(1\text{–}9) = \underbrace{(724.7 + 281)(2)(21.8)}_{43849} + \underbrace{(695.2 + 615.7)(9.2)}_{12060} = 55910 = 56$ kips

$w_i\,(10) = (724.7)(2)(21.8) + (695.2)(9.2) = 37993 = 38$ kips

↳ roof, no live loads

Appendix D: Dead and Live Load Calculations

Approximate Analysis - dead & live loads pg 1

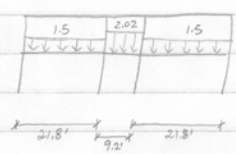
exterior beams: DL = 725 lb/ft
LL = 281 lb/ft

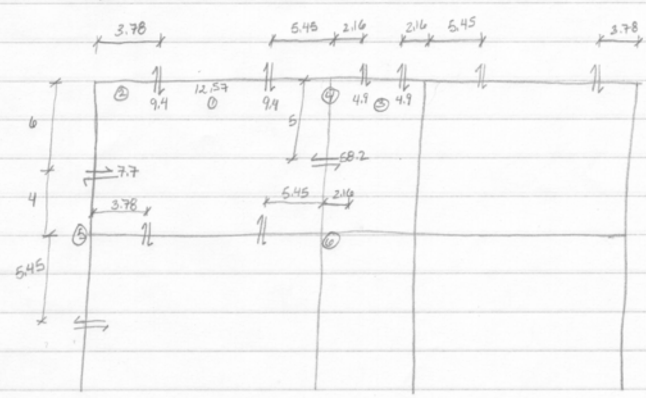
$(1.4)(725) + (1.7)(281) = 1493 \approx 1.5$ kips/ft

interior beams: DL = 695 lb/ft
LL = 616 lb/ft

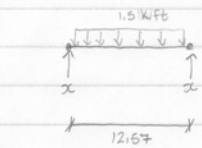
$(1.4)(695) + (1.7)(616) = 2020$ lb/f = 2.02 kips/ft

* assume coefficients from Oguz's solutions *

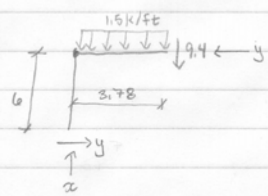




Approximate Analysis – dead & live loads pg 2

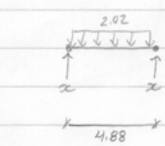
①


$$x = \frac{(1.5)(12.67)}{2} = 9.4$$

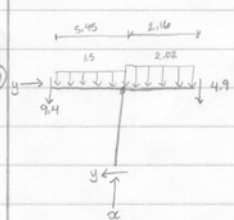
②


$$x = (1.5)(3.78) + 9.4 = 15.1$$

$$(9.4)(3.78) + \left(\tfrac{1}{2}\right)(3.78)^2(1.5) = (6)(y) \qquad y = 7.7$$

③


$$x = \frac{(2.02)(4.88)}{2} = 4.9$$

④


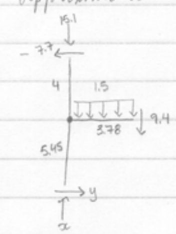
$$(4.9)(2.16) + (2.02)(2.16)^2\left(\tfrac{1}{2}\right) = 15.3$$

$$(9.4)(5.45) + (1.5)(5.45)^2\left(\tfrac{1}{2}\right) = 73.5$$

$$73.5 - 15.3 = 58.2 \qquad \frac{58.2}{5} = 11.64 = y$$

$$x = 9.4 + 4.9 + (1.5)(5.45) + (2.02)(2.16) = 26.8$$

Approximate Analysis – dead & live loads pg 3

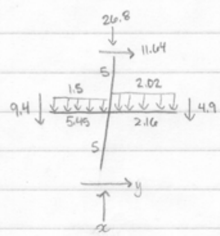
5 

$(4)(7.7) = 30.8$

$(9.4)(3.78) + (1.5)(3.78)(3.78/2) = 46.2$

$\frac{46.2 - 30.8}{5.45} = 2.8 = y$

$x = 15.1 + 9.4 + (1.5)(3.78) = 30.2$

6 

$x = 26.8 + 9.4 + 4.9 + (1.5)(5.45) + (2.02)(2.16) = 53.6$

$(11.64)(5) + (4.9)(2.16) + (2.02)(2.16)(2.16/2) = 73.5$

$(9.4)(5.45) + (1.5)(5.45)^2(1/2) = 73.5$

$y = 73.5 - 73.5 = 0$

Appendix E: Column Calculations

Column design calculation

9th Floor - Exterior Column

From approximate analysis, column loads are:

$$P_u = 28kips$$

$$M_u = 100kips - ft$$

Eccentricities, e = $\frac{100}{28}$ = 3.6 ft = 43 in
Choose trial size 12"x12", hence b = h = 12". Cover is 2".

$$\gamma = \frac{12-4}{12} = 0.67$$

So, we use interaction diagram for $\gamma = 0.7$

$$\begin{aligned} R_n &= \frac{P_u e}{\phi f'_c A_g h} \\ &= \frac{100}{0.65 \times 4 \times 12 \times 12 \times \frac{12}{12}} \\ &= 0.27 \\ R_n &= \frac{P_u}{\phi f'_c A_g} \\ &= \frac{28}{0.65 \times 4 \times 12 \times 12} \\ &= 0.07 \end{aligned}$$

From interaction diagram, $\rho_g = 0.05$. $A_s = \rho_g bh = 0.05 \times 12 \times 12 = 7.2in^2$. We use 8 #9 bars.

9th Floor - Interior Column

From approximate analysis, column loads are:

$$P_u = 32kips$$

$$M_u = 87kips - ft$$

Eccentricities, e = $\frac{87}{32}$ = 2.71 ft = 33 in
Choose trial size 12"x12", hence b = h = 12". Cover is 2".

$$\gamma = \frac{12-4}{12} = 0.67$$

So, we use interaction diagram for $\gamma = 0.7$

$$\begin{aligned} R_n &= \frac{P_u e}{\phi f'_c A_g h} \\ &= \frac{87}{0.65 \times 4 \times 12 \times 12 \times \frac{12}{12}} \\ &= 0.23 \\ R_n &= \frac{P_u}{\phi f'_c A_g} \\ &= \frac{32}{0.65 \times 4 \times 12 \times 12} \\ &= 0.08 \end{aligned}$$

1

From interaction diagram, $\rho_g = 0.04$. $A_s = \rho_g bh = 0.04 \times 12 \times 12 = 5.8in^2$. We use 8 #8 bars.

2nd Floor - Exterior Column

From approximate analysis, column loads are:

$$P_u = 142kips$$

$$M_u = 162kips - ft$$

Eccentricities, e = $\frac{162}{142}$ = 1.14 ft = 13.7 in
Choose trial size 18"x18", hence b = h = 18". Cover is 2".

$$\gamma = \frac{18-4}{18} = 0.77$$

So, we use interaction diagram for $\gamma = 0.8$

$$\begin{aligned} R_n &= \frac{P_u e}{\phi f'_c A_g h} \\ &= \frac{162}{0.65 \times 4 \times 18 \times 18 \times \frac{18}{12}} \\ &= 0.128 \\ R_n &= \frac{P_u}{\phi f'_c A_g} \\ &= \frac{142}{0.65 \times 4 \times 18 \times 18} \\ &= 0.169 \end{aligned}$$

From interaction diagram, $\rho_g = 0.03$. $A_s = \rho_g bh = 0.03 \times 18 \times 18 = 9.72in^2$. We use 8 #10 bars.

2nd Floor - Interior Column

From approximate analysis, column loads are:

$$P_u = 221kips$$

$$M_u = 207kips - ft$$

Eccentricities, e = $\frac{207}{221}$ = 0.94 ft = 11.24 in
Choose trial size 18"x18", hence b = h = 18". Cover is 2".

$$\gamma = \frac{18-4}{18} = 0.77$$

So, we use interaction diagram for $\gamma = 0.8$

$$\begin{aligned} R_n &= \frac{P_u e}{\phi f'_c A_g h} \\ &= \frac{207}{0.65 \times 4 \times 18 \times 18 \times \frac{18}{12}} \\ &= 0.16 \\ R_n &= \frac{P_u}{\phi f'_c A_g} \\ &= \frac{221}{0.65 \times 4 \times 18 \times 18} \\ &= 0.26 \end{aligned}$$

From interaction diagram, $\rho_g = 0.02$. $A_s = \rho_g bh = 0.02 \times 18 \times 18 = 6.48 in^2$. We use 8 #9 bars.

3

Appendix F: Slab Calculations

Calculating h_{min} for deflection requirements (ACI 9.5.3.3)

We are taking two of the most common slab dimension in Simmons Hall:

1. 18'x9'2" slab for corridor (interior slab)

$$\begin{aligned} \alpha &= \frac{18}{9.17} = 1.96 \geq 0.2 \\ h_{min} &= \frac{l_n(0.8 + \frac{f_y}{200000})}{36 + 9\beta} \\ &= \frac{(18 \times 12 - 7.5) \times (0.8 + \frac{50000}{200000})}{36 + 9 \times \frac{18\times12-7.5}{110-7.5}} \\ &= 4.03in \end{aligned}$$

2. 18'x21'10" slab for public and private spaces (exterior slab)

$$\begin{aligned} \alpha &= \frac{18}{21.83} = 0.8244 \geq 0.2 \\ h_{min} &= \frac{l_n \times (0.8 + \frac{f_y}{200000})}{36 + 9\beta} \\ &= \frac{(262 - 7.5) \times (0.8 + \frac{50000}{200000})}{36 + 9 \times \frac{262-7.5}{216-7.5}} \\ &= 5.69in \end{aligned}$$

Calculating Moments

For the short span:

$$\begin{aligned} M_0 &= \frac{1}{8} \times 218 \times 18 \times 8.5^2 \\ &= 35438 ft - lb \\ &= 35.4 kips - ft \end{aligned}$$

Negative design moment = 0.65 × 35.4 = 23.0 kips-ft
Positive design moment = 0.35 × 35.4 = 12.4 kips-ft

For the long span:

$$\begin{aligned} M_0 &= \frac{1}{8} \times 218 \times 9.17 \times 17.4^2 \\ &= 75.6 kips - ft \end{aligned}$$

Negative design moment = 0.65 × 75.6 = 49.1 kips-ft
Positive design moment = 0.35 × 75.6 = 26.5 kips-ft

Calculating required d

We assume $f_y = 50$ ksi and $f'_c = 4$ ksi, hence from table A.4 of textbook $\rho_{max} = 0.0248$

1

1. For short span

$$d^2 = \frac{23 \times 12}{0.9 \times 0.0248 \times 50 \times 12 \times (1 - 0.59 \times 0.0248 \times \frac{50}{4})}$$
$$d = 5.02in$$

Use d = 6 in

2. For long span

$$d^2 = \frac{49.1 \times 12}{0.9 \times 0.0248 \times 50 \times 12 \times (1 - 0.59 \times 0.0248 \times \frac{50}{4})}$$
$$d = 7.33in$$

Use d = 8 in

Calculating steel reinforcement

1. For short span

$$\rho_{min} = 0.0018 \times \frac{7 \times 12}{8 \times 12} = 0.002$$

With minimum reinforcement, design strength is twice than that required, so we use minimum reinforcement to save on steel cost.

$$\begin{aligned} A_s &= \rho bd \\ &= 0.002 \times 12 \times 8 \\ &= 0.192in^2 \end{aligned}$$

Use a #4 reinforcement bar

2. For long span

$$\rho_{min} = 0.0018 \times \frac{7 \times 12}{6 \times 12} = 0.002$$

Similarly, this will give design strength greater than that required. Hence, use a #4 reinforcement bar.

Check Shear Strength

$$\phi V_c = 0.75 \times 2 \times \sqrt{4000} \times 9.17 \times 12 \times 8 = 83.5kips$$
$$V_u = \frac{218 \times 9.17 \times 18}{2} = 17.9kips < \phi V_c$$

2

Appendix G: Group 3 Specific Sections Beam Calculations

Alternative Design of Specific sections

Region 3a

We choose to strengthen the floor beams (alternative #1 in presentation).

Load Calculation

Each of the two columns supports $P_1 = 9$ kips / floor
We're counting half a floor at boundary conditions

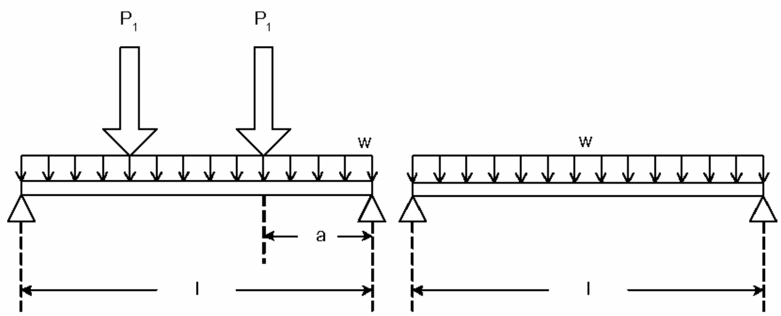


Figure 1: 4th and 3rd floor beam

$$\begin{aligned} P_1 &= 4 \text{ floors} + \tfrac{1}{2} \text{ floor} + \tfrac{1}{2} \text{ floor} &= 5 \text{ floors} \\ P_1 &= 5 \text{ floors x } 9 \text{ kips/floor} &= 45 \text{ kips} \end{aligned}$$

For 4^{th} floor beam:

$$M = \frac{wl^2}{8} + P_1 a$$

For 3^{rd} floor beam:

$$M = \frac{wl^2}{8}$$

$$\begin{aligned} w &= \text{distributed load on floor} &= (1.2)(616) + (1.6)(695) \\ & &= 1851 \text{ lbs / ft} \end{aligned}$$

$$\begin{aligned} l &= 36 \text{ ft} &= 432\text{''} \\ a &= 12 \text{ ft} &= 144\text{''} \end{aligned}$$

For 4^{th} floor beam:

$$\begin{aligned} M &= \tfrac{(1.8)(36^2)}{8} + (45)(12) &= 831.6 \text{ kips-ft} \\ V &= (wl + P_1 \cdot 2) \cdot \tfrac{1}{2} &= 77.4 \text{ kips} \end{aligned}$$

For 3^{rd} floor beam:

$$\begin{aligned} M &= \tfrac{(1.8)(36^2)}{8} &= 292 \text{ kips-ft} \\ V &= \tfrac{1.8 \times 36}{2} &= 32.4 \text{ kips} \end{aligned}$$

1

Deflections

For 4^{th} floor beam:

$$\Delta_{max} = \frac{5wl^4}{384EI} + \frac{Pa}{24EI}(3l^2 - 4a^2)$$

$$\begin{aligned} w &= 1.8 \text{ kips/ft} = 0.15 \text{ kips/in} = 150 \text{ lbs/in} \\ l &= 36 \text{ ft} = 432 \text{ in} \\ E &= 29000 \text{ ksi} = 2.9 \times 10^7 \text{ psi} \\ P &= 45 \text{ kips} = 45000 \text{ lb} \\ a &= 12 \text{ ft} = 144 \text{ in} \end{aligned}$$

$$\begin{aligned} \Delta_{max} &= \frac{(5)(150)(432^4)}{(384)(2.9 \times 10^7)I} + \frac{45000 \times 144}{24 \times (2.9 \times 10^7)I} \times (3 \times 432^2 - 4 \times 144^2) \\ &= \frac{2346}{I} + \frac{4440}{I} \\ &= \frac{6786}{I} \end{aligned}$$

$$\begin{aligned} \Delta_{max} &= \frac{l}{360} = \frac{432}{360} = 1.2 \text{ in} \\ \Delta_{max} &= \frac{6786}{I} \\ I &= \frac{6786}{\Delta_{max}} = \frac{6786}{1.2} = 5655\ in^4 \end{aligned}$$

For 3^{rd} floor beam:

$$\Delta_{max} = \frac{5wl^4}{384EI}$$

$$\begin{aligned} w &= 1.8 \text{ kips/ft} = 0.15 \text{ kips/in} = 150 \text{ lbs/in} \\ l &= 36 \text{ ft} = 432 \text{ in} \\ E &= 29000 \text{ ksi} = 2.9 \times 10^7 \text{ psi} \end{aligned}$$

$$\begin{aligned} \Delta_{max} &= \frac{(5)(150)(432^4)}{(384)(2.9 \times 10^7)I} \\ &= \frac{2346}{I} \end{aligned}$$

$$\begin{aligned} I &= \frac{2346}{\Delta_{max}} \\ \Delta_{max} &= \frac{l}{360} = \frac{432}{360} = 1.2 \text{ in} \\ I &= 1955\ in^4 \end{aligned}$$

Choosing section

For 4^{th} floor beam:

Bending:

$$s = \frac{10140000}{36000} = 282$$

Deflection:

$$I_{req'd} = 5655$$

2

Hence, we choose W24x176 for space efficiency or W27x161 for cost efficiency.

For 3^{rd} floor beam:

Bending:

$$s = \frac{3504000}{36000} = 98$$

Deflection:

$$I_{req'd} = 1955$$

Hence, we choose W18x119 for space efficiency or W24x76 for cost efficiency.

Region 3b

We choose to strengthen the floor beams (alternative #1 in presentation).

Load Calculation

Column supports $P_2 = 9$ kips / floor
We're counting half a floor at boundary conditions

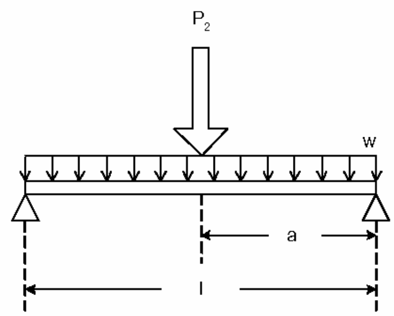


Figure 2: load on beam

$$\begin{array}{lcll} P_2 & = & \frac{1}{2}\text{ floor} + \frac{1}{2}\text{ floor} + \frac{1}{2}\text{ floor} + \frac{1}{2}\text{ floor} & = \text{ 2 floors} \\ P_1 & = & \text{2 floors x 9 kips/floor} & = \text{ 18 kips} \end{array}$$

$$M = \frac{P_1 l}{4} + \frac{wl^2}{8}$$

$$V_{max} = \frac{P+wl}{2} = \frac{18+1.8\times18}{2} = 25.2kips$$

3

Deflections

$$\Delta_{max} = \frac{5wl^4}{384EI} + \frac{Pl^3}{48EI}$$

$$\begin{array}{lllllll} w & = & 1.8 \text{ kips/ft} & = & 0.15 \text{ kips/in} & = & 150 \text{ lbs/in} \\ l & = & 18 \text{ ft} & = & 216 \text{ in} & & \\ E & = & 29000 \text{ ksi} & = & 2.9 \times 10^7 \text{ psi} & & \\ P & = & 18 \text{ kips} & = & 18000 \text{ lb} & & \end{array}$$

$$\begin{aligned} \Delta_{max} &= \frac{147}{I} + \frac{130}{I} \\ &= \frac{277}{I} \\ I &= \frac{177}{\Delta_{max}} \\ \Delta_{max} &= \frac{l}{360} \\ &= 0.6in \\ I &= 295in^4 \end{aligned}$$

Choosing section

Bending:

$$s = \frac{1846800}{36000} = 51.3$$

Deflection:

$$I_{req'd} = 295$$

Hence, we choose W14x38 for space efficiency or W16x36 for cost efficiency.
We have assumed a 60 ksi steel for our alternative beam design.

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