Final Report

Christina Landing Apartment Tower

Wilmington, DE

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Option: Structural Option
Advisor: Boothby
Semester: Spring 2006
Date: 4-03-2006

CHRISTINA LANDING APARTMENT TOWER

Wilmington, DE

Gregory R. Eckel

Structural Option

Project Team
Owner: Buccini Pollin Group
Website: http://www.bpgroup.net/
Architect and Engineering Disciplines: Kling
Website: http://www.kling.us
General Contractor: Gilbane Building Co.
Website: http://www.gilbanco.com

Project Overview
22 Story High Rise Apartment Building
Size: 248,884 sqft
Construction: April 2004 — October 2005
Cost: 60 million
Delivery Method: Design-Bid-Build

Structural
Cast-in-place concrete structure
Reinforced 8” concrete slab with perimeter beam
Reinforced concrete columns (square and round)
Main Wind Force Resisting System: Concrete shear walls
Foundation: Pile caps and H-piles

Architectural
Building Materials: Brick, Glass, Metal Cladding
173 one and two bedroom apartments
Part of a residential construction project including 63 townhouses and a park
Façade: Non-structural precast concrete panel with architectural brick veneer and aluminum framed glass curtain walls

Mechanical
Air Handling: Air to air heat pumps in apartments
Air to water heat pump for common areas
System also uses electric resistance heaters
Fire Protection: Entirely sprinkled wet system
Automated pressurization for smoke control

Lighting/Electrical
Two feeds (208/120V and 480/277V)
208/120V feeds 3 phase, 4 wire plug-in busway for apartments
Apartments metered individually
480/277V line serves mechanical equipment
500kW/625kVA generator serves emergency systems

CPEP: www.arche.psu.edu/thesis/eportfolio/current/portfolios/gre111/

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Executive Summary

The Christina Landing Apartment Tower is a 22 story apartment building located just outside center city Wilmington, DE. The tower provides 250,000 square feet of floor space. The structure is a predominately cast-in-place concrete building. Its floors are supported by a two way flat slab system. The typical floor system also incorporates small areas of reinforced concrete and post-tensioned beams to aid the lateral force resisting system. The floors are supported by square and round concrete columns. Lateral forces induced on the building are resisted by a box of four shear walls. All columns and shear walls rest on a foundation system of H-piles and pile caps. Typical floor loads are 130psf dead load and 40psf live load.

This thesis investigates two structural redesigns as well as an acoustic, and construction management study. The first of the structural alternates analyzes the feasibility of reducing the existing 8” reinforced concrete slab to a 7” post-tensioned concrete plate. This study proved to be quite effective decreasing both reinforcing and concrete volumes while also decreasing the maximum deflections. The second structural change involved negating the effect of the existing equivalent moment frames in the building and using an additional shear wall to replace their function. This analysis also proved successful decreasing the total building deflection over 3” in locations. The first of the breath topics covered was an acoustic study of transmission losses between floors and walls at various locations in the tower. It was found that the existing structure preformed well acoustically however the proposed redesign could be benefited acoustically by addition of sound absorbing elements around the post-tensioned slab. Finally, a construction management study was preformed. Its goal was to investigate the difference between the existing and proposed floor systems. While this analysis showed the post-tensioned system would save significant material cot it would also cause an increased project duration.

Building Introduction

The Christina Landing Apartment Tower is a 22 story apartment building located just outside center city Wilmington, DE. It will be one of the tallest buildings in Wilmington, and will have a significant impact on the city. The tower is part of a residential construction project on the south side of the Christina River which includes 63 townhouses, a condominium high rise, a river walk, and a two acre park. It is the first sizable development on the south side of the Christina River and the first riverfront residential project in recent history. The building owner and developer is The Buccini/Pollin Group, who were extremely confident in the project. After receiving favorable demand for the townhouses they decided to build a high rise condominium at the site. Buccini/Pollin contracted the architectural engineering firm Kling to design the tower. The project takes inspiration from the nearby river-walk trail and is centered on the creation of a park-like space bordering the river. The construction project was managed by Gilbane Co.. There were several notable construction issues for the project. Because the site is located directly on the Christina River the tower site was raised 5 feet above the flood plane before construction began. It was also necessary to drive H-piles up to 70ft deep for the building's foundation system. The floors were cast using a flying form system which allowed for quicker turnover time between floors due to forming time savings. Construction started August 2004, the building topped out in May 2005, and substantial completion was during November 2005.

Architectural Features

The tower itself consists of 173 one and two bedroom apartments with balconies. General areas include; a media room, fitness center, great room, bar, convenience store, dry cleaners, and on site parking. Floors 3-20 are typical of the building, each containing 6 single apartments and 3 double apartments. The first and second floors house the retail and common spaces, and the 21^{st} and 22^{nd} floors consist of two story penthouses. The tower provides 250,000 square feet of floor space and its footprint covers approximately 12,000 square feet. The typical floor to floor height (floors 3-20) is 10 feet, while the common spaces and the penthouses have 12 foot floor heights. The total building height is 230'.

The building envelope consists of two main wall systems and a roof system. The primary wall system which covers most of each of the east/west faces of the tower is a non-structural precast concrete panel with a thin architectural brick veneer. The panels are backed by a semi-rigid insulation and are hung from the building structure. The other sides of the building are comprised of an aluminum framed glass curtain wall system. The roofing system is a structural concrete slab topped with rigid insulation, coverboard, and a 2-ply roofing membrane. The building envelope also uses aluminum framed windows and sliding glass doors, metal panel wall assemblies, and louver assemblies.

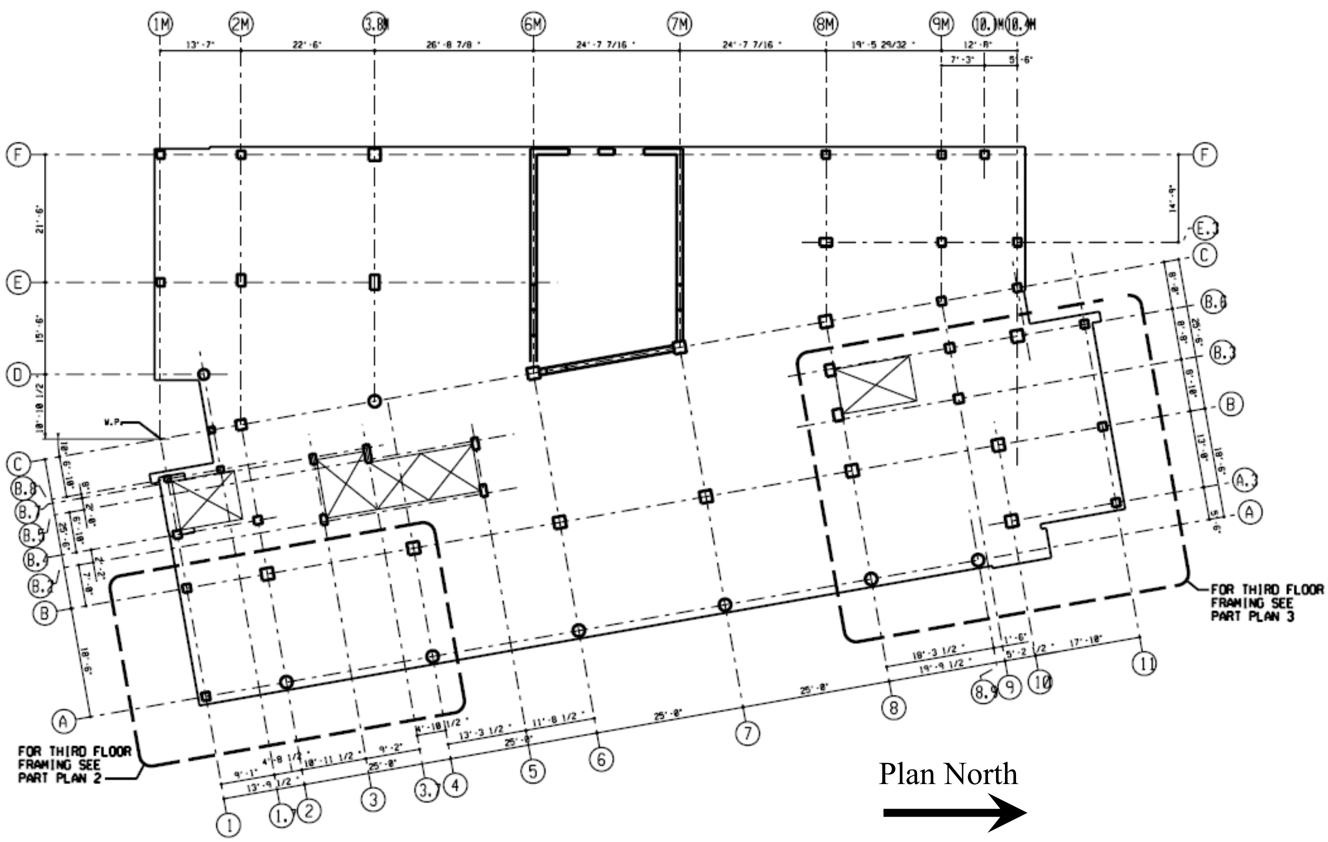
The building uses several different partition walls. The typical wall consists of gypsum wall board on various sizes of metal studs with sound attenuation blankets in critical areas.

Structural Introduction

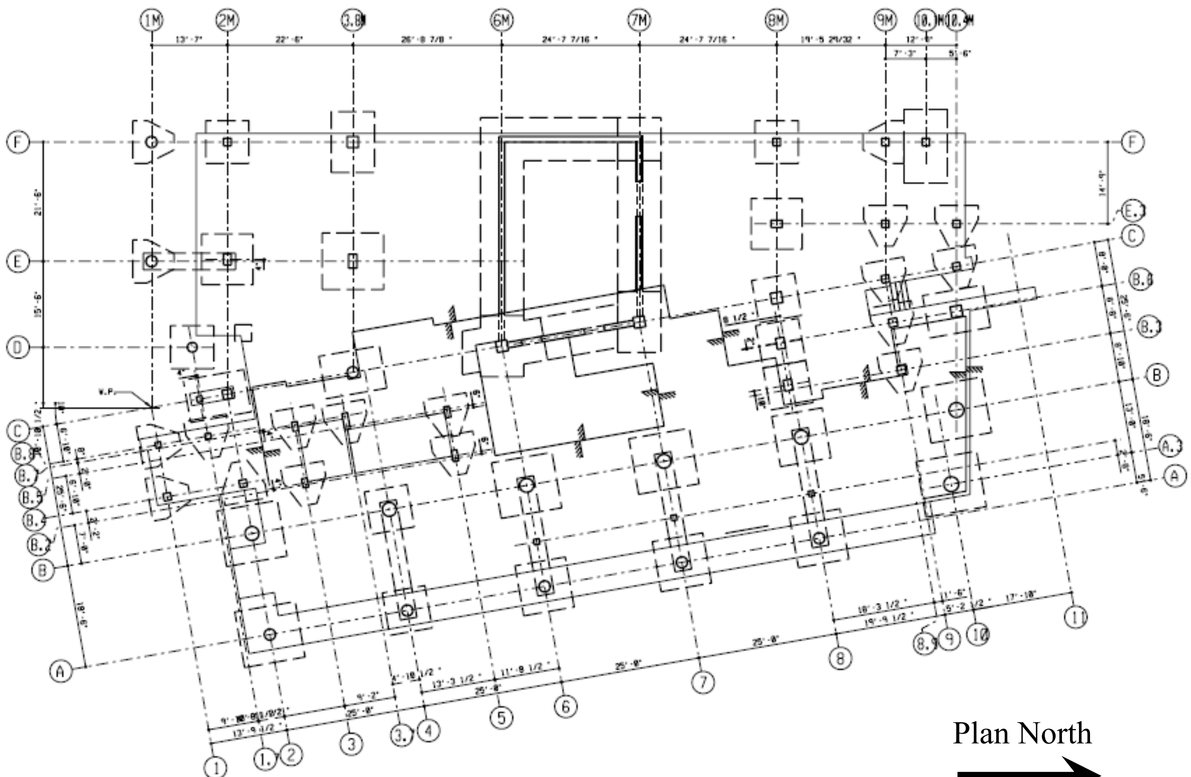
The structure is a predominately cast-in-place concrete building. Its floors are supported by a two way flat slab system. Spans between columns are on average approximately 20 to 25 feet. Other than the bays that contain slab openings, the typical panel ratios range from 1:1 to 1:1.5 (see page 8-9 for framing plans). The floors are supported by square and round concrete columns. Column sizes for typical bays are 2' square or 2' round columns. For columns that surround slab openings and support smaller spans, sizes range down to 12"*12". Column sizes seldom vary from floor to floor although reinforcement frequently changes (see page __ for column schedule). Lateral forces induced on the building are resisted by a box of four shear walls located in the center of the west wall. All columns and shear walls rest on a foundation system of H-piles and pile caps. Concrete strengths differ throughout the structure, ranging from 4000 psi to 8000 psi (see below for concrete strength schedule.)

Concrete Strength Schedule	
Element	28 Day Cylinder Strength (psi)
Pile Caps	4,000
Slabs 5th Floor and Above	4,500
Slabs Below 5th Floor	5,600
Columns 5th Floor and Above	5,000
Columns Below 5th Floor	8,000
Exterior Slabs and Paving	5,000
Shear Walls	5,000
Topping Fills	4,000

Typical Framing Plan

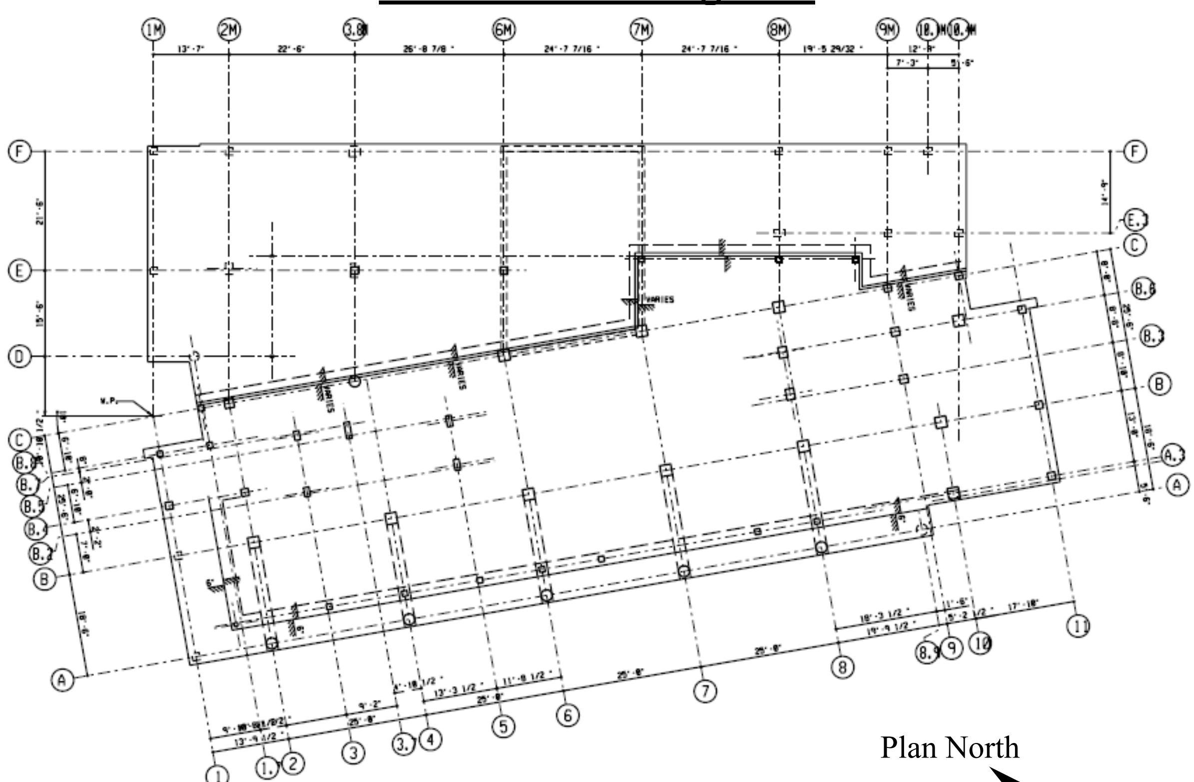


1st Floor Framing Plan



Plan North

21st Floor Framing Plan



Plan North

Existing Slab and Framing System

All the floors, including the roof and the ground floor, in the building have the same two way flat slab system. Spans between columns are on average approximately 20 to 25 feet. Other than the bays that contain slab openings the typical panel ratios range from 1:1 to 1:1.5 (see page 4 for framing plan). The slab is an 8" slab with #6 bars at 10" on center, each way in the top and #4 bars at 10" on center, each way in the bottom. The strength of the concrete in the floor system is 5,600psi from the ground floor to the fifth floor and 4,500psi above the fifth floor.

Introduction of Lateral System

The lateral system of this building consists of two parts. The first part is comprised of a box of four shear walls located at the center of the west wall. The walls are connected at the corners and act in unison to allow for shear flow. For ease of analysis I assumed that all four walls are perpendicular to each other by conservatively adjusting their lengths. All of the walls are 12" thick with #4 bars at 12" on center each way in each face. Two of the walls are 32' and the other two are 24' long. The other lateral force resisting system is the equivalent frame created by the slab and columns. This system has far less stiffness than the shear walls, however it helps to resist the large torsional force generated by the eccentricity of the center of rigidity from the center of mass. The relationship between the rigidities of the lateral resisting elements was studied extensively in technical report 3 and the findings influenced the thesis proposal, to be discussed in more detail later.

Proposal Summary

Problem Statement

It has been shown during the first semester of thesis work that the Christina Landing Apartment Tower's existing framing and lateral systems are highly successful systems for the building type and location. In technical assignment 2 the existing 8" flat slab was found to be the thinnest possible slab for that type of system. In technical assignment 3 it was shown that the lateral system had a deflection of L/350. The goal of this thesis will be to attempt to make both the framing and the lateral systems more efficient by redesigning them. The goal of any structural engineer is to find the most economical design while keeping serviceability requirements in mind. Any change made to the existing structure will impact labor cost, material cost, and job schedule. It is important for engineers to remember these are the issues that should influence their design. The focus of this redesign will be to attempt to find alternate floor and lateral systems that improve the balance between cost, schedule, and serviceability.

Structural Depth Study

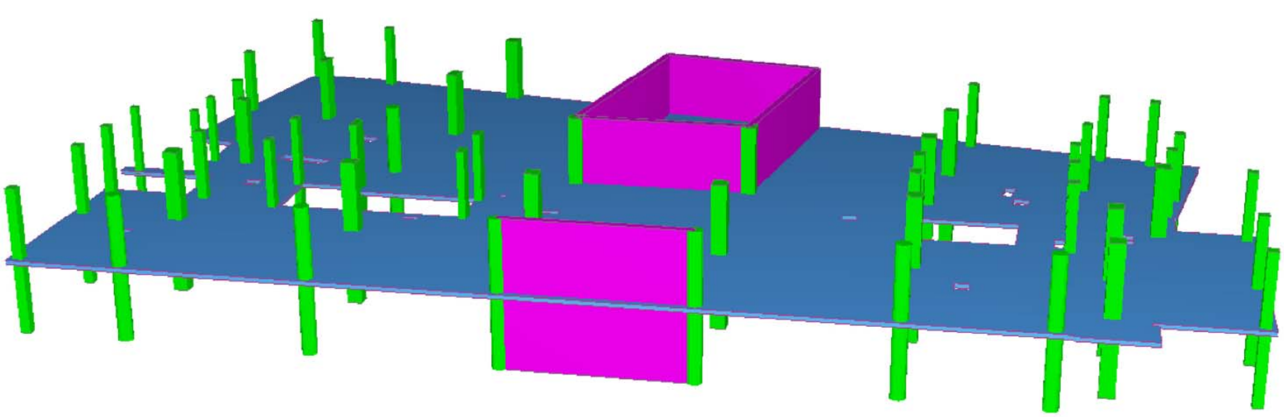
Floor System Redesign

Introduction

The Christina Landing Apartment Tower has a very unique slab shape and column layout to accommodate the apartment plans. In order to analyze the floor system as a whole two way system, it was determined it would be necessary to use a finite element modeling software. The program chosen was RAM Concept which has the ability to design two way post-tensioned structures.

Modeling the Floor

The first step of the design process was to model the slab and columns as they appear in the original design. This was achieved by using an AutoCAD drawing of the floor system and simply adding slab and column elements at the appropriate angle and location in Concept. Each of these elements was then given initial design characteristics. Choices included concrete strength and column fixity. It was determined in technical report 2, by a simple calculation, that for spans of 25’ a 7” post-tensioned slab would be a good starting point. This also covers a minimum depth for fire safety of 6” and is a reasonable depth to check for punching shear which typically controls post-tensioned flat plats.

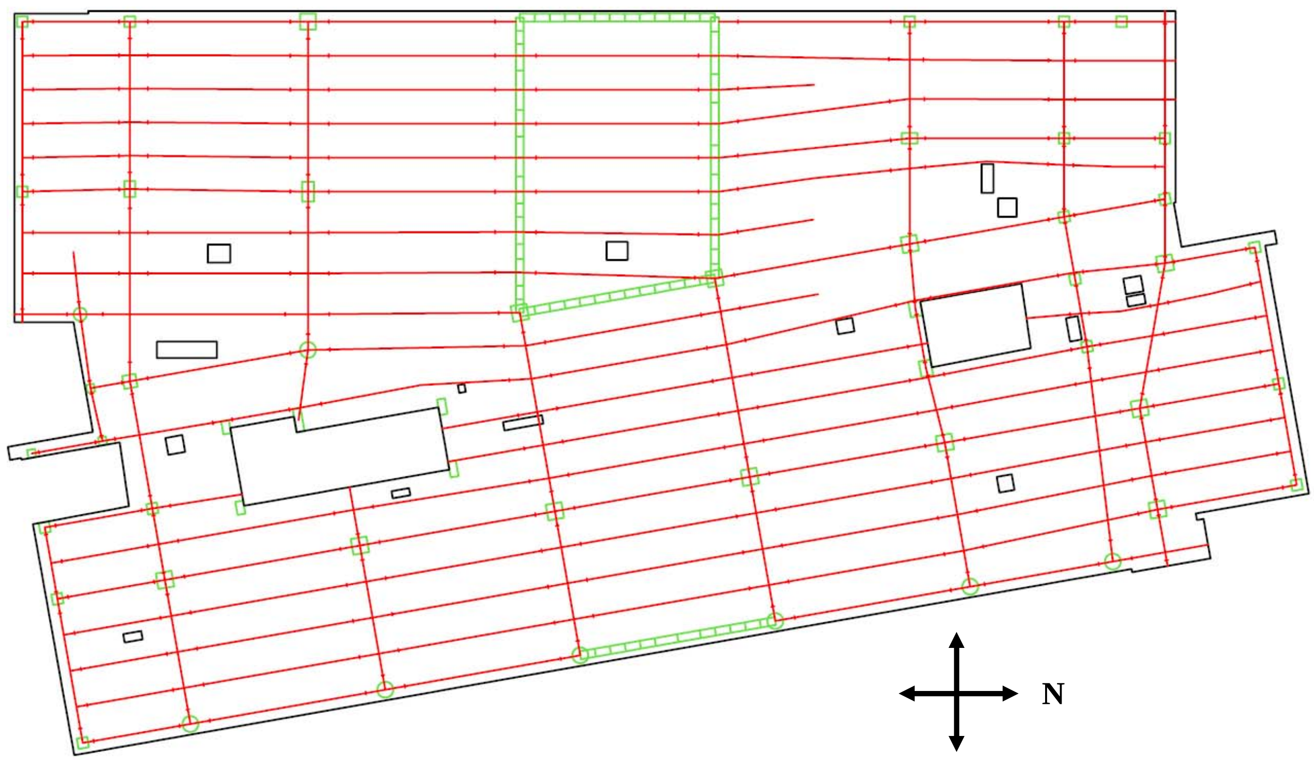


Post-Tensioning Tendon Layout

At this point in the modeling process it was necessary to determine tendon locations and profiles. The decision was made to use the technique of banded tendons in one direction and uniformly distributed tendons in the other direction. This is the typical method of post-tensioning two-way slabs in the United States. The banded tendons act virtually as the support beams. This simplified reinforcement system also speed the construction process.

Two key design decisions for the tendons were the use of ½” unbonded strands, and 1” of concrete cover (.75” minimum for fire safety). The tendon layout is shown below. The lines in the east-west direction represent the banded tendons along the column lines. Each line represents 15 strands. While the contractor will usually have some say in how these tendons are laid out it is typical to place 3 strands in each sleeve. The lines in the North-South direction represent the distributed tendons. Each line represents 4 strands. These strands will most likely be split along the entire tributary width of the tendon line. The maximum spacing for the strands in the distributed direction is 6 times the slab thickness, or in this case 42”.

Special care was taken to plan the placement of all the tendons. Considerations were made for strength requirements, slab openings, and constructability. Strength requirements came into play in several places. One such area was slab edges where combinations of torsional moments and unbalanced loading forced unique strand forces. Areas where the slab cantilevered over supports it was necessary to make sure that the strands remained in the top of the section profile for most of the span. Finally the North-West corner of the building contains cut off strands, where if they had been continued through the slab uplift forces would be too great and crack the slab at midspan. In order to accommodate slab openings tendons were either anchored at the edge, if there is no way to bypass the penetration, or be bent around the opening. Finally particular care was taken to keep profile points of the tendons consistent throughout the slab for ease of construction. The goal of the overall layout of strands was to make the placement very uniform throughout.



Reinforcement Design Strips

The next step in the design of the system was to lay out the design strips. This step tells the program where to place bonded reinforcement and how much to use. I changed several user variables in order to properly model my reinforcing. They included telling the program what size bars to use, what reinforcement ratio to use, whether or not to use a middle strip, and various others. After researching this topic more thoroughly I decided to use a reinforcement ratio of both .0009 in the top and bottom of the slab. These ratios yield and overall reinforcing minimum of .0018. When entered in this manner the program will reinforce both the top and bottom of the slab continuously throughout the slab. This is more reinforcing than what is needed, because technically the bonded reinforcement is only need in the tension areas of the slab. However, due to constructability it may be easier to lay the bars continuously. In this fashion lap spliced need not be added so long as the splices occur in the compression regions of the slab.

Building Loads

Building loads were added consistently with those of the original design. RAM Concept factors in the self-weight of the slab automatically, so the decrease due to the redesigned thinner slab was not needed to be accounted for. Loads added into the model included; superimposed dead, and live loads. A review of the loads on the building is listed below.

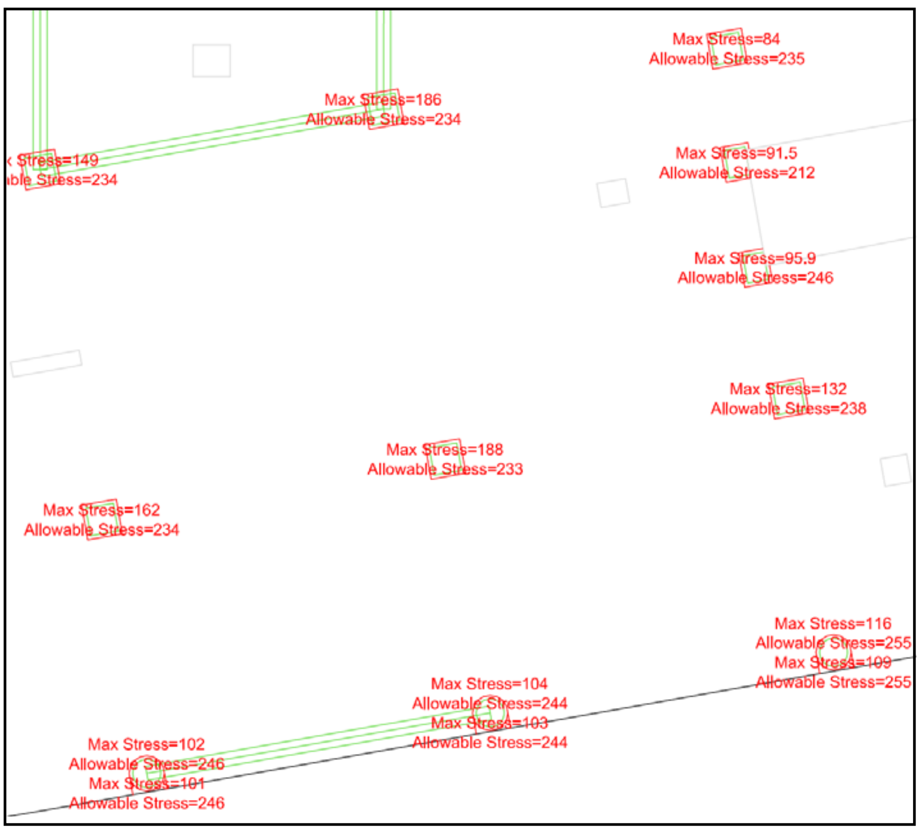
The loads used for this design are as follows:

Partitions =	20psf
Miscellaneous Dead Load =	10psf
Live Load =	40psf

For gravity loads the load case used was 1.2D+1.6L

Punching Shear

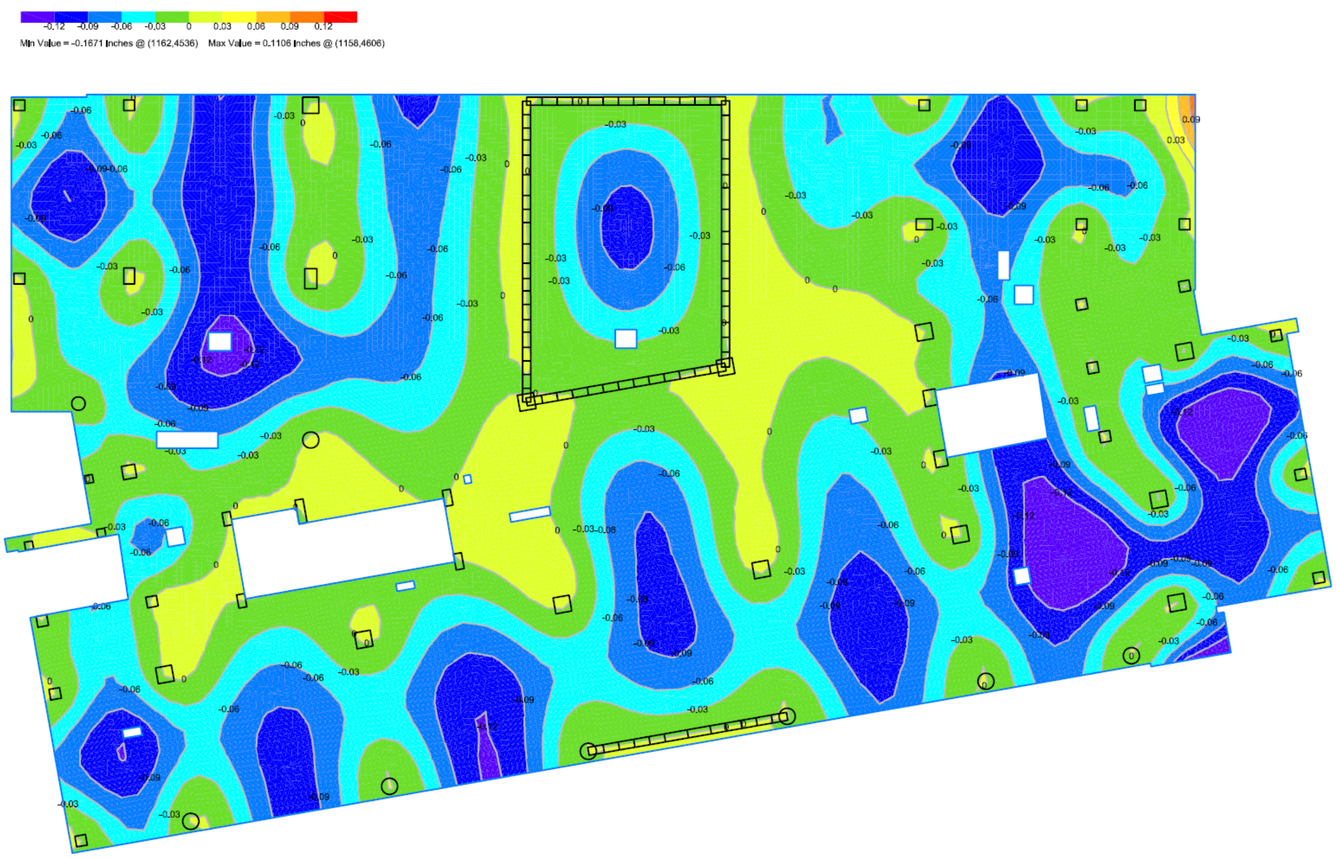
The most common element controlling slab thickness is punching shear. Concept allows the user to design for punching shear. I addition to using this design aid, worst case punching checks were done by hand calculations to verify the software output. The results obtained by the hand method were very close to the design shear forces and maximum allowable shear forces. By verifying the software in several locations it was assumed that the less critical sections would also pass shear tests. For more detailed assumptions and calculations see the appendix (page 60). The design and max allowable shears are shown on the plan below.



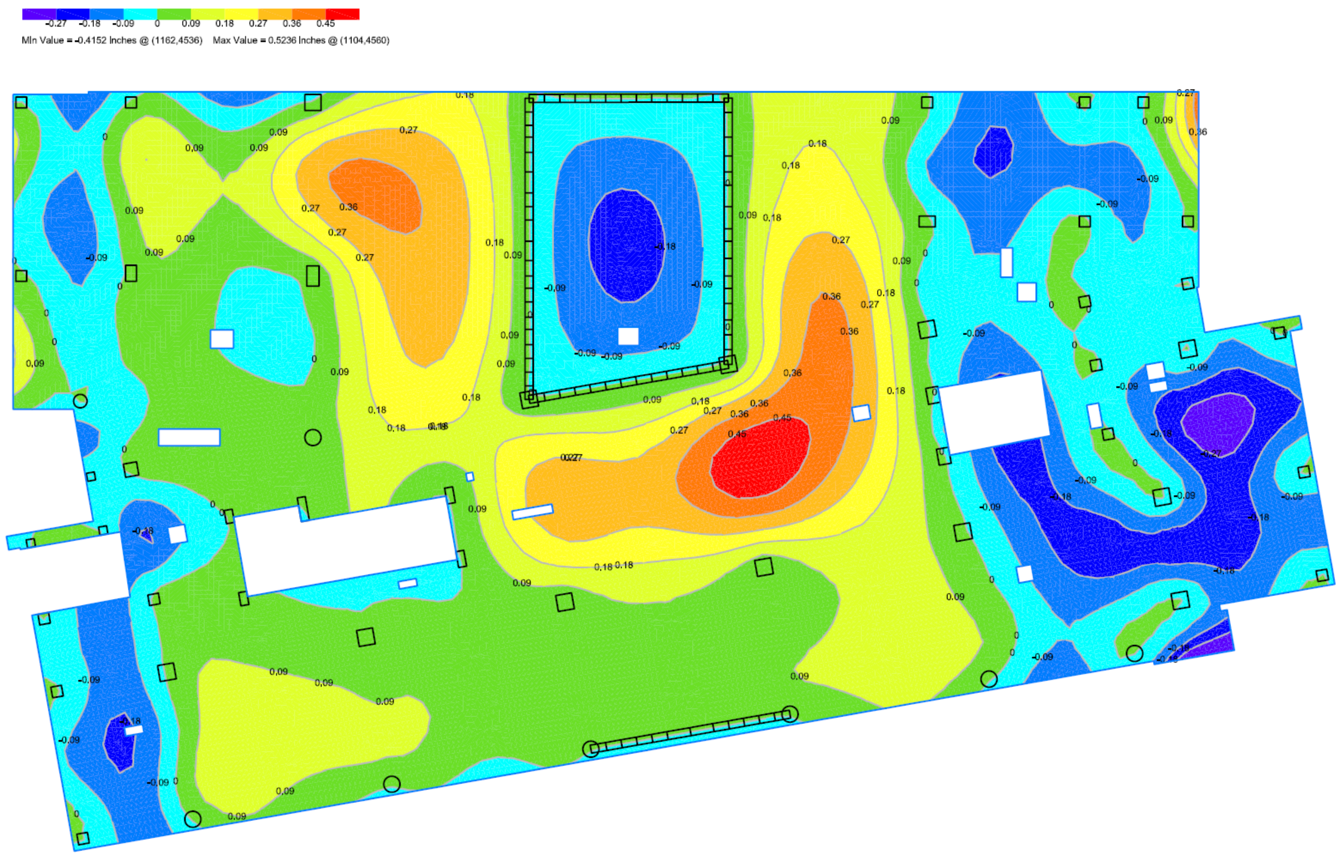
Design Results

The design summary for the slab passed for all spans. This shows that the slab is capable of meeting all code and strength requirements. This is not enough in itself to define the slab as satisfactory for construction. Once the slab was found to be sufficient both stress and deflection diagrams were checked to eliminate areas of excessive deflections and stresses. The final results of these diagrams are shown below. The first figure shows the bonded reinforcing layout. For the most part the design calls for #4 bars in the top and bottom of the slab at 31" on center. In addition to the computer output hand calculations were done to check maximum stresses in the slab verses maximum allowable stresses. The worst cases were checked and were within the allowable limit. See appendix for additional calculations and assumptions.

The next figure shows the initial service load case. This is a key diagram to study because in the tensioning process, before the load is applied, it is possible to put to much tension in areas of the slab causing failures. This becomes especially important in buildings with large loads because of the huge prestressing force needed. The maximum uplift in this load case was found to be -0.17" and the maximum deflection was 0.11".

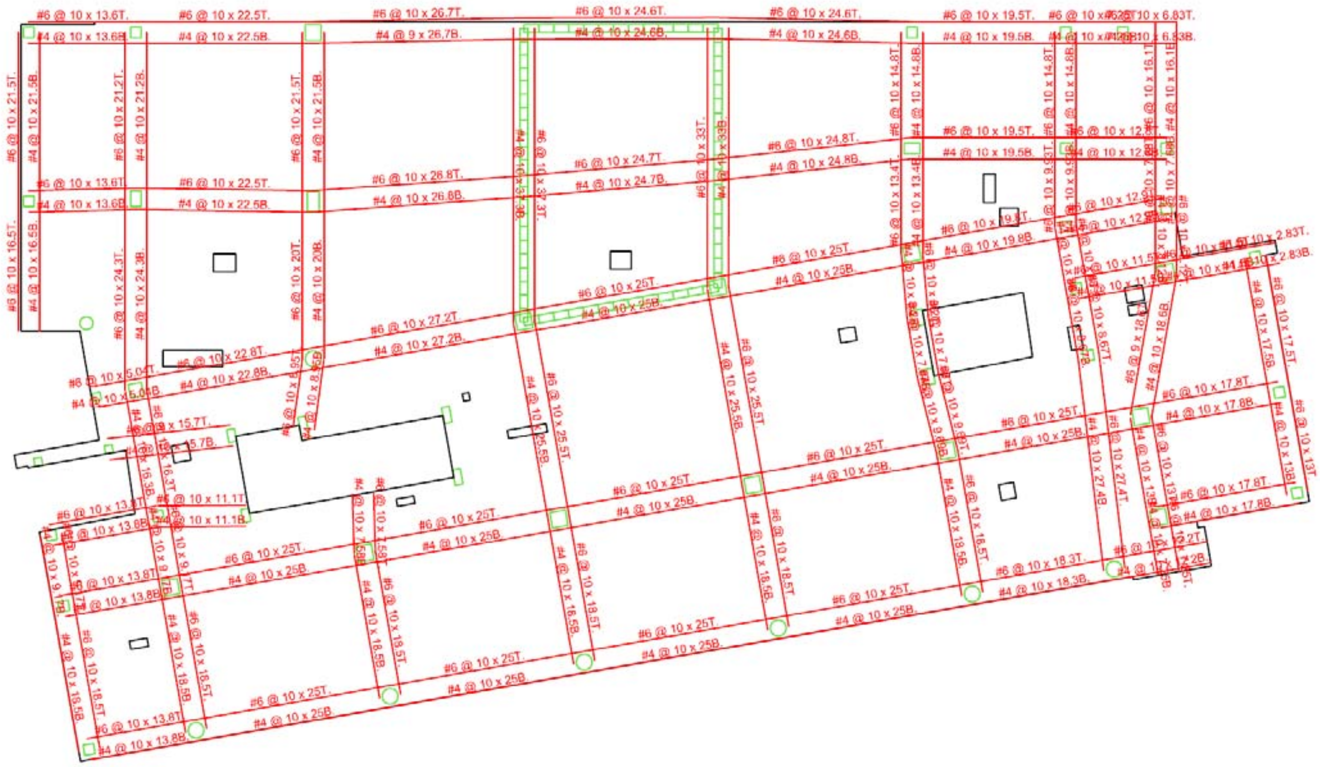


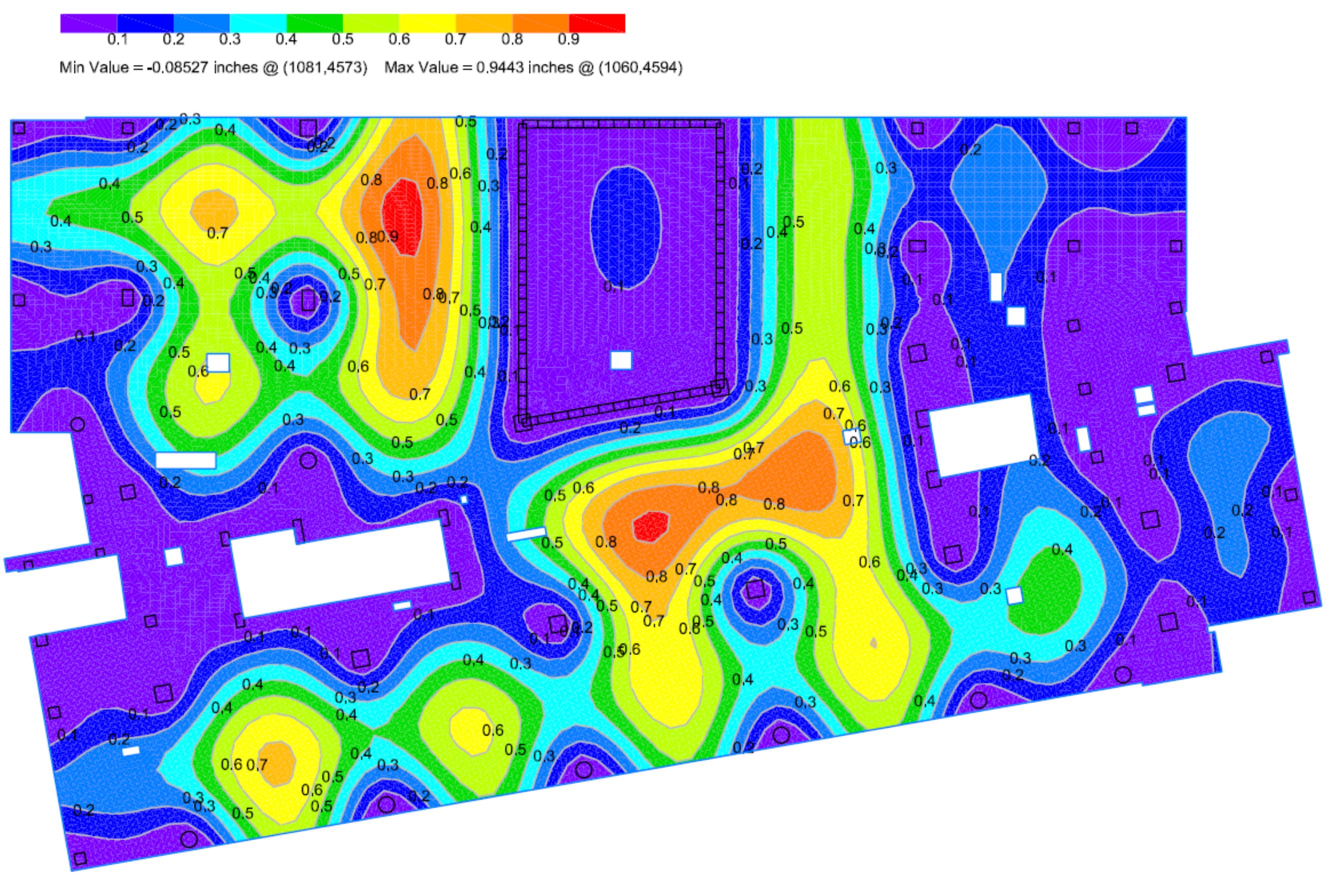
The next figure shows the long term deflection of the slab. This is an important diagram because it shows how well the slab maintains strength through its life. As time passes losses are inherent in both the concrete, due to creep and shrinkage, as well as in the tendons, due to relaxation. It is important to make sure the serviceability of the slab remains acceptable. The maximum uplift in this load case was found to be -0.41" and the maximum deflection was 0.52". Using a maximum deflection of L/480 which is quite conservative would yield a maximum of .63” of deflection, therefore the slab is acceptable.



Original Design

After completing the post-tensioned model it became apparent that the results would be more valuable if the original deflections and stresses were known. The first figure below shows the reinforcement in the slab. The top is reinforced with #6 bars at 10” on center and the bottom is reinforced with #4 bars at 10” on center. The second diagram shows the long term deflection of the slab. The maximum of which is .94”.




0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9
Min Value = -0.08527 inches @ (1081,4573) Max Value = 0.9443 inches @ (1060,4594)

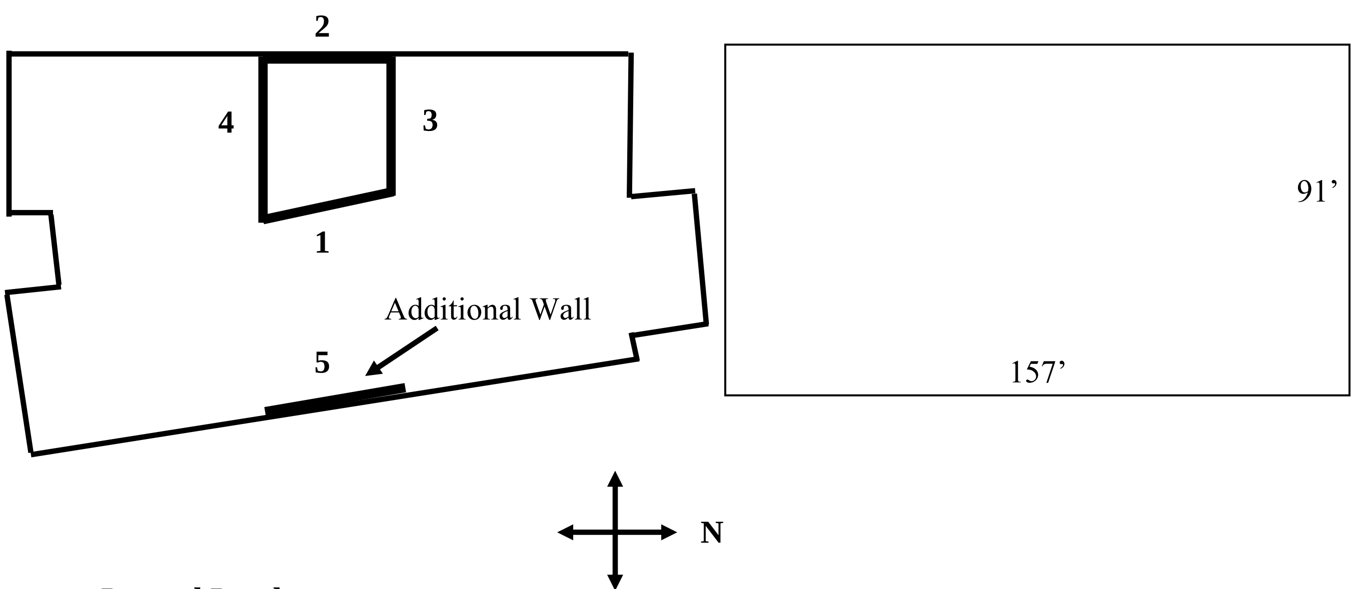
Conclusion

It was shown that both the existing and proposed redesign are viable floor systems for the Christina Landing Apartment Tower. The proposed post-tensioned redesign using the method of banded tendons results in small deflections throughout the floor. The existing condition was shown to have a maximum deflection of 0.94" while the post-tensioned system's deflection was .52". The new system also uses considerably less reinforcing. Where in the original design reinforcing was spaced at 10" on center the post-tensioned systems bonded reinforcement was spaced at 31" on center. One particular area of concern in thinning the slab was whether or not punching shear criteria would be met. It was verified by both hand calculation and Concept that all the columns were acceptable for punching shear.

Lateral System Redesign

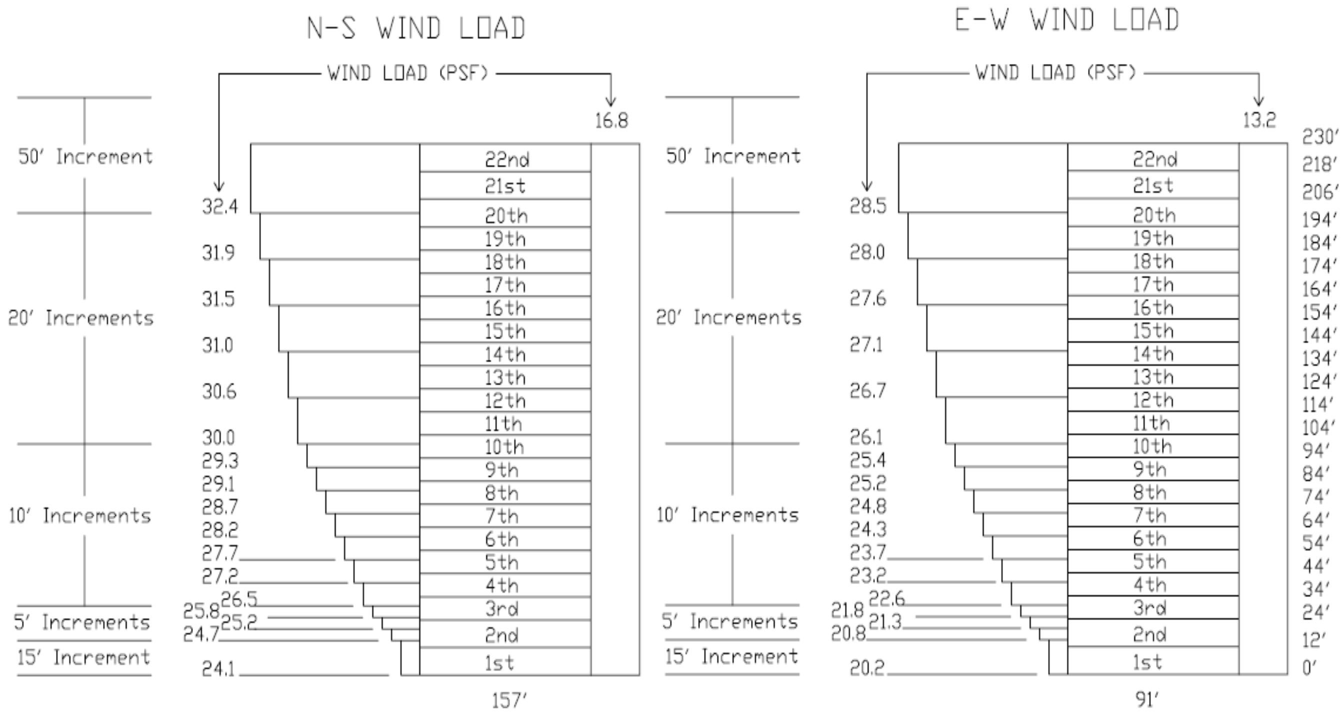
Introduction

Technical assignment 3 found the deflection of the building to be approximately 8" which is a large deflection for the height of 230'. It may be possible that this large deflection can be reduced by eliminating the large torsional shear force due to a north-south wind load. In order to eliminate these forces one option would be to ignore the effect of the equivalent concrete moment frames and instead to add another shear wall located on the east wall. The walls size and position will be determined by making their resultant center of rigidity as close as possible to the center of mass.



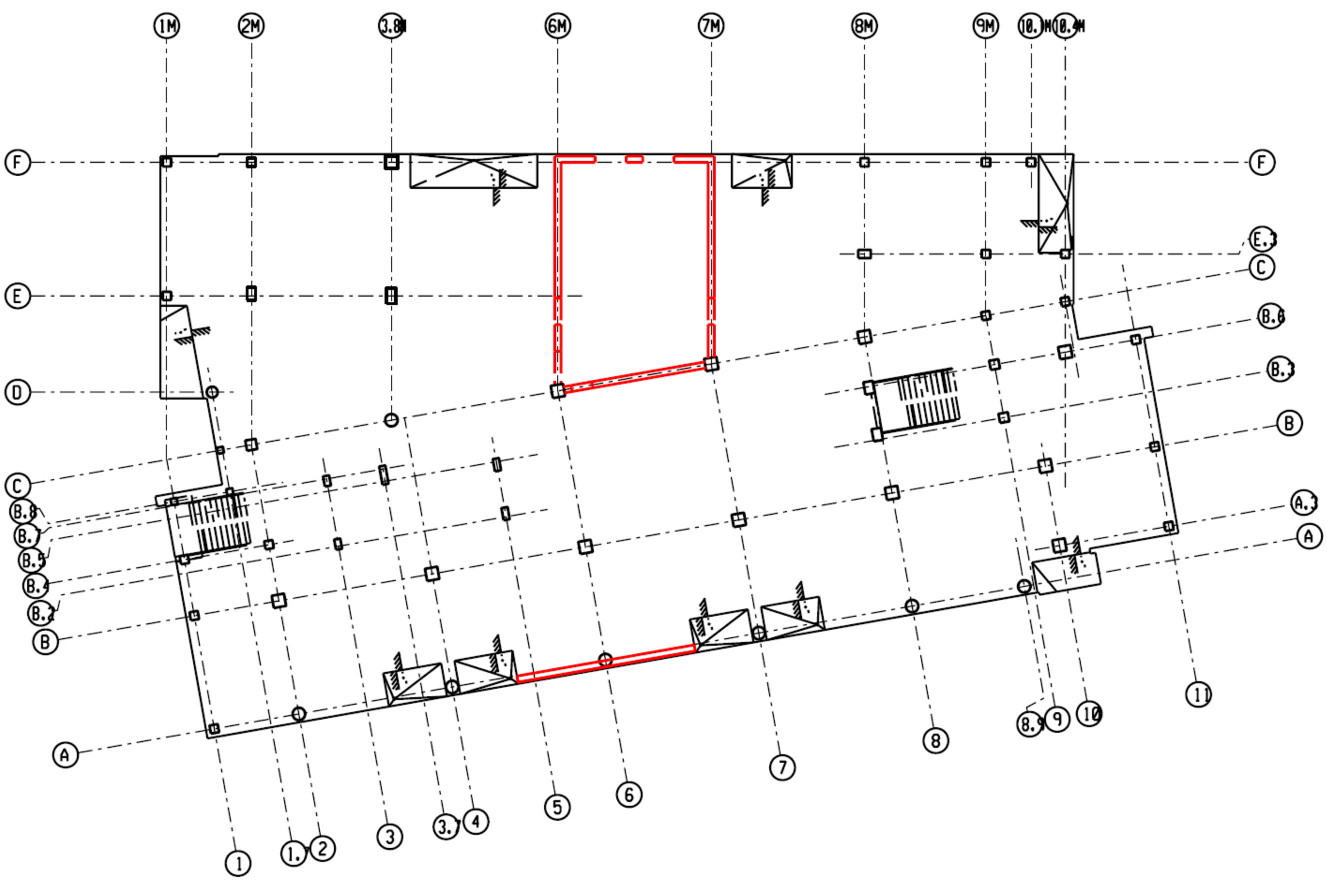
Lateral Loads

From Technical Report 1 wind was found to be the controlling lateral load. The images on the following page are wind loading diagrams for the apartment tower. For the calculations the building was estimated to be a 91'x157' rectangle. These dimensions are conservative and provide the loading for the worst case scenario pressures on the structure. In order to calculate the building pressures method 2 for high rise buildings was used from ASCE7. It was also determined that the tower was not able to be classified as a rigid structure and therefore a gust factor needed to be found. Other relevant information used in the wind loading calculations includes an importance factor of 1 and a wind exposure of class “C”. The total base shear on the building due to the North-South wind load is 968 k and the total resisting moment at the base of the structure is 114,795 ft-k. The total base shear on the building due to the East-West wind load is 1400 k and the total resisting moment at the base of the structure is 166,980 ft-k. All of the information presented here is generated from calculations and spreadsheets found on pages 44-50 in the appendix.



Design of New Shear Wall

In order to determine the size and the location of the new shear wall several hand calculations were preformed. In order to eliminate the forces due to the torsional moment it is necessary to make sure that the center of mass and the center of rigidity coincide. The center of mass is fixed and therefore the center of rigidity must change. It was decided that the shear wall should be located along the east wall in order to not interfere with apartment layouts and maximize the eccentricity from the existing center of rigidity. By this method it was determined the shear wall will be 28' long. The plan below shows the location of the new shear wall on the east face of the building between adjacent apartment balconies. See page 59 in the appendix for more detailed calculations.



Distribution of Loads

In order to determine the total building deflection the first step is to calculate how the force resisting system will distribute the loads. To divide the forces between resisting elements the proportion of rigidity carried by each wall at each level was found. In order to find the rigidities of the shear walls several different options were investigated. The three methods tried to find the rigidities were: first, to analyze the walls separately using the equation R=Et/(4(h/L)^3+3(h/L)); second, to analyze the walls separately using a unit load at a distance to find the relative stiffnesses of the walls compared to each other; third, to analyze the walls as if they acted as a box. During technical assignment 3 it would have been ideal to make the walls work as a single box, however by this method, it was only possible to relate their stiffnesses to each other and not to the equivalent moment frames as well. As it turned out analyzing the walls by the first method mentioned gave similar proportions to that of the preferred third method. This was quite convenient for technical report 3 because the first method was easily related to the moment frames in the structure (see page 36 in appendix for comparison). The same method was used for the lateral system redesign. Microsoft Excel was used for all the lateral redesign calculations. Starting with the equation R=Et/(4(h/L)^3+3(h/L)) and adjusting the height of the wall, the rigidity at each story was found. Comparing each rigidity to the total of all the walls rigidities acting in its direction, the proportion of stiffness for each wall, in each direction, at each floor was found. The next step is to apply the torsional moment on the structure to each wall, at each floor, to find the torsional shear in each brace. The torsional shears were then added to the direct shears in the locations where the forces would be additive due to the eccentricity. Where the forces act in opposite directions only the direct shear was used.

Distribution of story shears for all the lateral resisting elements in both the existing and redesigned systems are given on pages 51-53 of the appendix. To save space I left off floor numbers. The first number is the story shear at the 2nd floor which is the first slab above grade. The last two numbers in each list are the 22nd floor and the roof. All shears are in kips. All results are calculated from pages 24-31 in the appendix.

Lateral Element Deflections

In order to determine deflection of the lateral elements the story shears at each floor were compared to the element's stiffness at that level. When comparing this value to that of the floor below, the drift of the floor in question can be calculated. The total building deflection is determined by adding all the story drifts for the entire structure. The tables below show the stiffness, story deflection, and total deflection at each floor for both the proposed redesign as well as the original design. All deflections are given in inches.

Wall 1	**Proposed Redesign**	
Stiffness	Story Deflection	Total Deflection
26659.73	0.01	0.01
7735.04	0.03	0.04
3492.15	0.04	0.08
1817.53	0.06	0.14
1050.19	0.09	0.24
656.26	0.13	0.36
435.48	0.16	0.52
302.91	0.20	0.72
218.82	0.23	0.95
163.03	0.27	1.22
124.62	0.30	1.51
97.35	0.33	1.84
77.46	0.35	2.19
62.62	0.36	2.55
51.33	0.37	2.92
42.60	0.37	3.30
35.74	0.36	3.66
30.27	0.34	4.00
25.86	0.31	4.30
21.63	0.30	4.61
18.27	0.20	4.81
15.57	0.08	**4.89**

Wall 1	**Original Design**	
Stiffness	Story Deflection	Total Deflection
26659.73	0.03	0.03
7735.04	0.06	0.09
3492.15	0.10	0.20
1817.53	0.16	0.35
1050.19	0.22	0.58
656.26	0.29	0.87
435.48	0.36	1.23
302.91	0.42	1.65
218.82	0.48	2.13
163.03	0.53	2.66
124.62	0.57	3.23
97.35	0.60	3.83
77.46	0.62	4.45
62.62	0.63	5.08
51.33	0.62	5.70
42.60	0.60	6.29
35.74	0.56	6.85
30.27	0.51	7.36
25.86	0.44	7.80
21.63	0.42	8.23
18.27	0.27	8.50
15.57	0.10	**8.60**

Wall 2	**Proposed Redesign**	
Stiffness	Story Deflection	Total Deflection
26659.73	0.01	0.01
7735.04	0.03	0.04
3492.15	0.04	0.08
1817.53	0.07	0.15
1050.19	0.10	0.25
656.26	0.13	0.38
435.48	0.17	0.55
302.91	0.21	0.76
218.82	0.25	1.01
163.03	0.29	1.30
124.62	0.32	1.62
97.35	0.35	1.97
77.46	0.37	2.34
62.62	0.39	2.73
51.33	0.40	3.13
42.60	0.40	3.54
35.74	0.39	3.93
30.27	0.37	4.29
25.86	0.33	4.62
21.63	0.33	4.95
18.27	0.22	5.17
15.57	0.08	**5.25**

Wall 2	**Original Design**	
Stiffness	Story Deflection	Total Deflection
26659.73	0.02	0.02
7735.04	0.04	0.06
3492.15	0.07	0.12
1817.53	0.10	0.23
1050.19	0.15	0.38
656.26	0.20	0.58
435.48	0.25	0.83
302.91	0.30	1.13
218.82	0.35	1.48
163.03	0.40	1.88
124.62	0.44	2.32
97.35	0.47	2.78
77.46	0.49	3.27
62.62	0.50	3.78
51.33	0.51	4.28
42.60	0.50	4.78
35.74	0.47	5.25
30.27	0.43	5.69
25.86	0.38	6.07
21.63	0.37	6.44
18.27	0.24	6.68
15.57	0.09	**6.77**

Wall 5	**Proposed Redesign**	
Stiffness	Story Deflection	Total Deflection
32140.79	0.01	0.01
10106.12	0.02	0.04
4761.28	0.04	0.07
2542.18	0.06	0.14
1492.08	0.09	0.23
941.79	0.12	0.35
629.16	0.15	0.50
439.69	0.19	0.69
318.70	0.22	0.91
238.04	0.26	1.17
182.31	0.29	1.46
142.62	0.32	1.78
113.62	0.34	2.11
91.94	0.35	2.47
75.43	0.36	2.83
62.64	0.36	3.19
52.57	0.35	3.54
44.55	0.33	3.87
38.07	0.30	4.17
31.85	0.29	4.47
26.92	0.20	4.66
22.95	0.07	**4.74**

All	**Original Design**	
Stiffness	Story Deflection	Total Deflection
1401.40	0.05	0.05
343.66	0.13	0.18
186.46	0.15	0.33
132.78	0.17	0.50
99.03	0.23	0.73
79.53	0.26	0.99
65.70	0.31	1.30
55.80	0.34	1.63
48.41	0.36	2.00
42.41	0.40	2.40
37.58	0.42	2.82
33.77	0.41	3.23
30.54	0.42	3.66
27.68	0.44	4.09
25.32	0.41	4.50
23.26	0.39	4.89
21.33	0.39	5.28
19.75	0.32	5.60
18.22	0.30	5.91
16.63	0.29	6.19
15.13	0.20	6.39
13.73	0.08	**6.47**

Wall 3	Proposed Redesign	
Stiffness	Story Deflection	Total Deflection
39167.14	0.02	0.02
13381.74	0.03	0.05
6627.89	0.05	0.10
3655.10	0.08	0.18
2190.28	0.11	0.29
1401.56	0.15	0.43
945.12	0.19	0.62
664.90	0.23	0.84
484.27	0.27	1.11
363.02	0.31	1.42
278.80	0.35	1.77
218.58	0.38	2.15
174.43	0.41	2.56
141.35	0.44	3.00
116.10	0.45	3.45
96.49	0.46	3.91
81.05	0.46	4.37
68.73	0.44	4.81
58.77	0.41	5.22
49.20	0.44	5.66
41.60	0.37	6.03
35.48	0.25	**6.28**

Wall 3	Original Design	
Stiffness	Story Deflection	Total Deflection
39167.14	0.02	0.02
13381.74	0.03	0.05
6627.89	0.05	0.10
3655.10	0.08	0.18
2190.28	0.11	0.29
1401.56	0.15	0.43
945.12	0.19	0.62
664.90	0.23	0.84
484.27	0.27	1.11
363.02	0.31	1.42
278.80	0.35	1.77
218.58	0.38	2.15
174.43	0.41	2.56
141.35	0.44	3.00
116.10	0.45	3.45
96.49	0.46	3.91
81.05	0.46	4.37
68.73	0.44	4.81
58.77	0.41	5.22
49.20	0.44	5.66
41.60	0.37	6.03
35.48	0.25	**6.28**

Wall 4	Proposed Redesign	
Stiffness	Story Deflection	Total Deflection
39167.14	0.02	0.02
13381.74	0.03	0.05
6627.89	0.05	0.10
3655.10	0.08	0.18
2190.28	0.11	0.29
1401.56	0.15	0.44
945.12	0.19	0.63
664.90	0.23	0.86
484.27	0.27	1.13
363.02	0.31	1.44
278.80	0.35	1.79
218.58	0.39	2.18
174.43	0.42	2.60
141.35	0.44	3.04
116.10	0.46	3.50
96.49	0.47	3.97
81.05	0.46	4.43
68.73	0.45	4.87
58.77	0.42	5.29
49.20	0.45	5.74
41.60	0.37	6.11
35.48	0.25	**6.36**

Wall 4	Original Design	
Stiffness	Story Deflection	Total Deflection
39167.14	0.02	0.02
13381.74	0.03	0.05
6627.89	0.05	0.10
3655.10	0.08	0.18
2190.28	0.11	0.30
1401.56	0.15	0.45
945.12	0.19	0.64
664.90	0.23	0.87
484.27	0.28	1.14
363.02	0.32	1.46
278.80	0.36	1.82
218.58	0.39	2.21
174.43	0.42	2.63
141.35	0.44	3.07
116.10	0.46	3.53
96.49	0.47	4.00
81.05	0.46	4.46
68.73	0.45	4.91
58.77	0.42	5.32
49.20	0.45	5.77
41.60	0.37	6.14
35.48	0.25	**6.39**

Conclusion

Summary			
	Direction	Original	Redesign
Wall 1	N-S	8.60	4.89
Wall 2	N-S	6.77	5.25
Frames/Wall 5	N-S	6.47	4.74
Wall 3	E-W	6.28	6.28
Wall 4	E-W	6.39	6.36

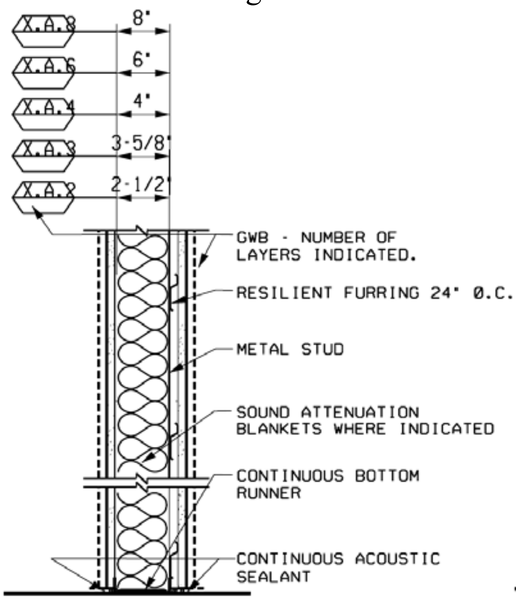
By negating the effect of the equivalent moment frames and replacing them with an additional shear wall torsional forces due to wind can be greatly reduced. This reduction in unison with the extra stiffness due to the new shear wall decreases the deflection up to 3.71". Before the redesign the total building drift was at its greatest L/320 in the north-south direction. After the addition of the wall the maximum building deflection became L/433 in the east-west direction.

Acoustic Breadth Study

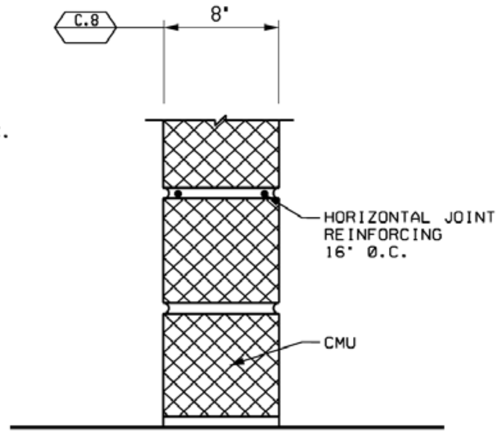
Introduction

This breadth study investigates the acoustic properties of both walls and floor systems in the Christina Landing Apartment Tower. The acoustic properties of walls and floors are very important in residential high rises. In order for the design to be successful and for the tenants to be happy, engineers have to take into consideration that two people of very different lifestyles might be sharing a wall. This study concentrates on two different areas where the effects of sound damping would be most significant. The first area investigated is a wall shared between two apartment units where loud music could transmit into a neighboring unit. The second area analyzed are the floor slabs between the gym and the apartment above, and lobby below, where the noise of music, banging weights, and people might disturb tenants.

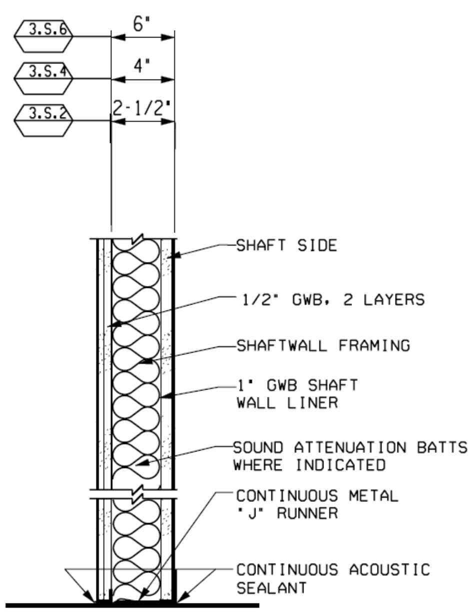
There are four factors that need to be considered when determining transmission of sound between two rooms or floors. The first factor is the level of noise generated by the source room. In this study the two source sounds considered were loud music of 80 decibels and the impact of dropping weights at 85 decibels. The second key factor in acoustic transfer is the transmission loss through the wall assembly or floor system. Transmission loss is the measure of how much sound energy is reduced in traveling through materials. Many different types of partition walls were used in the Christina Landing Apartment tower (see diagram below). For the study wall A was used because it had the smallest transmission loss of all the walls that separated dwelling units. The third factor deals with the physical properties of the source and receiving rooms. Noise reduction between rooms is increased by having a “dead” or very absorbent receiving room. It is also increased if the partition between the rooms has a small area in comparison to the size of the receiving room. The last factor needed to be considered is the level of background sound in the receiving room. If the level of noise generated by the occupant is greater than that of the transmitted sound it will drown out the neighboring noises. It can be assumed that on average an apartment unit will generate approximately 20-30 dB of sound. For the lobby the assumed level of sound is close to those of office activities or 50 dB. These values were used as design maximums for the amount of sound allowed to transmit into the receiving room.



A PARTITION TYPE A
ACOUSTIC PARTITION
SCALE:N.T.S.

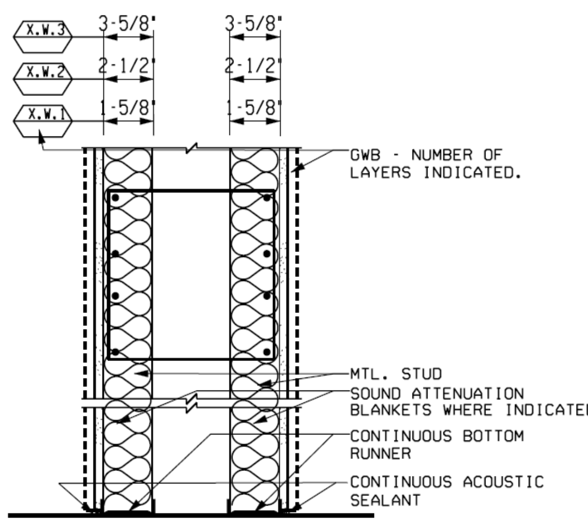


C PARTITION TYPE C - CMU
SCALE:N.T.S.



3 LAYERS GWB

S PARTITION TYPE S
SHAFTWALL
SCALE:N.T.S.

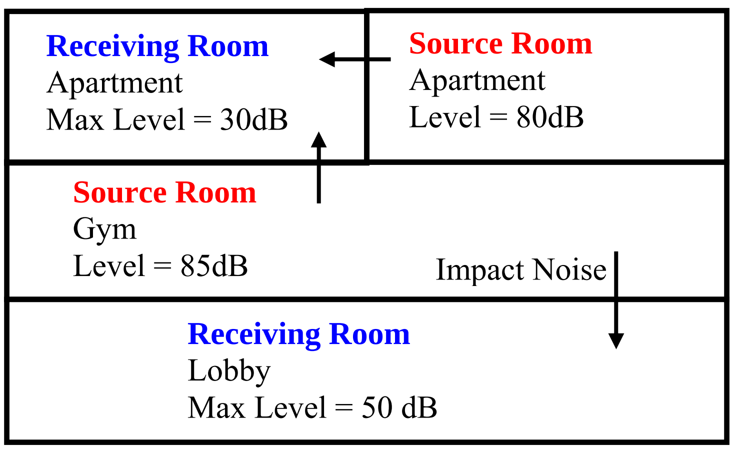


W PARTITION TYPE W
SCALE:N.T.S.

Results

Listed below are the five cases analyzed for acoustic transfer.

1. Original floor design between the gym and lobby
2. Original floor design between the gym and an apartment bedroom
3. New floor design between the gym and lobby
4. New floor design between the gym and an apartment bedroom
5. Wall between two adjacent bedrooms



Original floor – 8" Reinforced concrete flat slab.
New floor – 7" Post-tensioned concrete flat slab.
Wall – 3 5/8" metal studs with 2 layers of 5/8" gypsum board on both sides

Transmission Loss

-Original Floor	= 57 dB
-New Floor	= 55 dB
-Wall	= 57 dB

Impact Isolation Class

-Original Floor	= 36 dB
-New Floor	= 34 dB

RL = SL – NR (Receiving Level = Source Level – Noise Reduction)
NR = TL + 10*log(Σ(Sα)/S)
(See pages 62-64 in the appendix for more detailed calculations.)

Case 1

TL = 36.0 dB
NR = 36.0 dB
LS = 85.0 dB
LR = 49.0 dB < 50 dB **OK**

Case 2

TL = 57.0 dB
NR = 54.5 dB
LS = 85.0 dB
LR = 30.5 dB ≈ 30 dB **OK**

Case 3

TL = 34.0 dB
NR = 33.5 dB
LS = 85.0 dB
LR = 51.5 dB > 50 dB **NG**

Try to use acoustic board on ceiling in lobby
TL = 34.0 dB
NR = 35.0 dB
LS = 85.0 dB
LR = 50.0 dB = 50 dB **OK**

Case 4

TL = 55.0 dB
NR = 51.9 dB
LS = 85.0 dB
LR = 33.1 dB > 30 dB **NG**

Try to use carpet on foam rubber in apartment above
TL = 55.0 dB
NR = 55.5 dB
LS = 85.0 dB
LR = 29.5 dB < 30 dB **OK**

Case 5

TL = 57.0 dB
NR = 56.2 dB
LS = 80.0 dB
LR = 23.8 dB < 30 dB **OK**

Conclusions

The transmission loss through the slab, because of the decreased thickness, drops by approximately 2 dB in the redesigned post-tensioned slab, due to the transmission mass law. This would not normally be a great deal of concern, however, the receiving rooms above and below the gym were already near the design limits in the original design. By adding an acoustical drop ceiling in the lobby below the gym and foam rubber below the carpet in the bedroom above the decreased transmission loss can be offset. The cost of this design would be minimal relative to the total building cost and could provide the occupants a more comfortable living environment. The original floor slabs and partition walls were found to be acoustically satisfactory. The original design seems to take particular care in providing especially well performing partition walls between apartments. Continuous acoustic sealant was used at the base of the walls to prevent sound leaks.

Construction Management Breadth Study

Introduction

This breadth study investigates the differences in both cost and schedule between the existing and proposed floor systems. When using a post-tensioned system it is typical to have material savings in both concrete and reinforcement. However, this savings is usually offset by both the cost of post-tensioning strands, jacking equipment, and the increase in schedule.

Cost Analysis

The total volumes of concrete, tonnages of reinforcing, and areas of formwork for both the existing and redesigned systems were calculated using RAM Concept. For the existing condition the 8" slab was designed and the proper amount of reinforcement was achieved by setting a minimum reinforcement ratio. The three parts of the total cost affected by the redesign were concrete, post-tensioning, and reinforcing steel cost. In the proposed system 36.4 cubic yards of concrete were saved resulting in a cost savings of $13,510. Money was also saved on reinforcing steel. For the original design the additional reinforcing tonnage correlates to an increased cost of $30,487. All the additional costs for the redesign are in the post-tensioning material and labor. The total cost for the post-tension system's installment is $11,150. The total cost of formwork is the same for both systems. The total cost of the redesign comes out to be $32,900 cheaper per floor. This results in a total building savings of approximately $700,000 for the floor redesign

			Original Design		Proposed Redesign	
	Unit Cost Material	Unit Cost Labor	Quantity	Total Cost	Quantity	Total Cost
Concrete	232 /cy	140 /cy	291.2 cy	$108300	254.8 cy	$94790
PT Strands	.46 /lb	.72 /lb	0 lbs	$0	9449 lbs	$11150
Formwork	1.6 /sqft	2.94 /sqft	11790 sqft	$53540	11790 sqft	$53540
Reinforcing Steel	850 /ton	420 /ton	31.38 tons	$39850	7.373 tons	$9363
Totals	9.59 /sqft	7.514 /sqft	11790 sqft	**$201700**	11790 sqft	**$168800**

Schedule Analysis

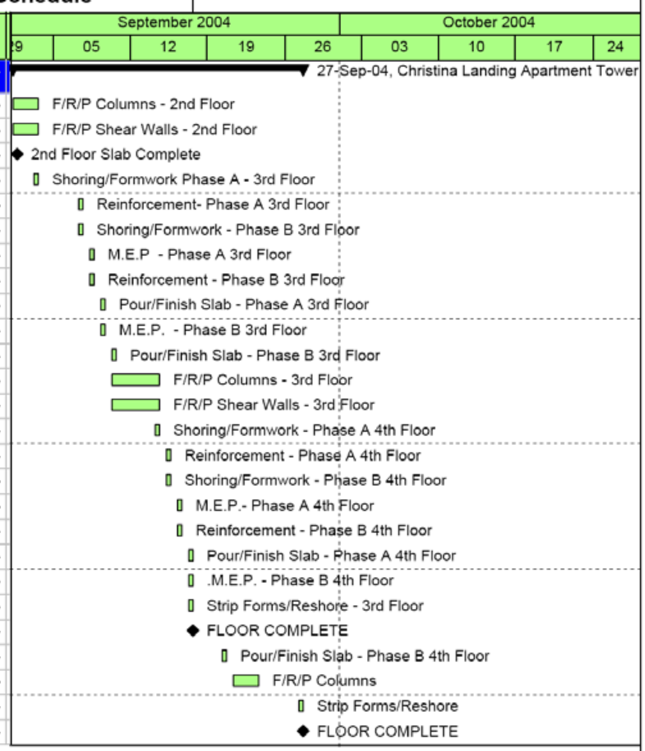
In order to make a recommendation for using the proposed floor system it is important to consider the impact it would have on the project's schedule. For this analysis a partial schedule was created for both floor systems. For both this shows the entire duration to complete one floor as well as the floor turnover rate. All construction processes can be seen in the schedules below. The major difference between the systems is the additional time needed during the phase in which the post-tensioning strands are placed. Other notable difference in the construction process which could pose delays are the tensioning of the tendons as well as the drilling of slab penetrations after curing. While the tensioning process can take place as work moves on it can require a significant amount of time. Drilling penetrations in the slab can also cause major delays and incur large costs if x-ray equiptment is needed to locate the tendons. If care is taken in laying the tendons out and marking their locations this costly procedure can be avoided.

The original design has a floor completion time of 11 days. However, work can move on to the floor above on the 7th day. Therefore the floor turnover time is 7 days. In the redesign additional time is needed to place the tensioning members. For this schedule it takes 12 days to complete one floor and it has a floor turnover time of 8 days. This shows that the proposed redesign is approximately one day slower than the original resulting in a 22 day longer total schedule. The pace will probably improve as the workers move up the building and familiarize themselves with placing post-tensioning strands. It helps that for this design each floor remains the same.

In order to relate the addition time on site to a cost general conditions fees were investigated. It was found that general conditions can be roughly assumed to be one percent of the total building cost through the duration of the job. This translates to approximately $30,000 per month. Therefore the additional 22 days on site would amount to a cost increase of $30,000. One way to potentially offset this cost would be to increase crew sizes in various phases of construction in order to shorten the overall schedule.

Christina Landing Apartment Tower | **Original Design Schedule** | Greg Eckel

Activity ID	Activity Name	Original Duration	Remaining Duration	Start	Finish
Christina Landing Apartment ...		18	18	01-Sep-04	27-Sep-04
A1010	F/R/P Columns - 2nd Floor	3	3	01-Sep-04	03-Sep-04
A1020	F/R/P Shear Walls - 2nd Floor	3	3	01-Sep-04	03-Sep-04
A1000	2nd Floor Slab Complete	0	0		01-Sep-04
A1030	Shoring/Formwork Phase A - 3rd Fl...	1	1	03-Sep-04	03-Sep-04
A1050	Reinforcement- Phase A 3rd Floor	1	1	07-Sep-04	07-Sep-04
A1080	Shoring/Formwork - Phase B 3rd Fl...	1	1	07-Sep-04	07-Sep-04
A1060	M.E.P - Phase A 3rd Floor	1	1	08-Sep-04	08-Sep-04
A1100	Reinforcement - Phase B 3rd Floor	1	1	08-Sep-04	08-Sep-04
A1070	Pour/Finish Slab - Phase A 3rd Floor	1	1	09-Sep-04	09-Sep-04
A1110	M.E.P. - Phase B 3rd Floor	1	1	09-Sep-04	09-Sep-04
A1120	Pour/Finish Slab - Phase B 3rd Floor	1	1	10-Sep-04	10-Sep-04
A1130	F/R/P Columns - 3rd Floor	3	3	10-Sep-04	14-Sep-04
A1140	F/R/P Shear Walls - 3rd Floor	3	3	10-Sep-04	14-Sep-04
A1150	Shoring/Formwork - Phase A 4th Fl...	1	1	14-Sep-04	14-Sep-04
A1170	Reinforcement - Phase A 4th Floor	1	1	15-Sep-04	15-Sep-04
A1200	Shoring/Formwork - Phase B 4th Fl...	1	1	15-Sep-04	15-Sep-04
A1180	M.E.P.- Phase A 4th Floor	1	1	16-Sep-04	16-Sep-04
A1220	Reinforcement - Phase B 4th Floor	1	1	16-Sep-04	16-Sep-04
A1190	Pour/Finish Slab - Phase A 4th Floor	1	1	17-Sep-04	17-Sep-04
A1230	.M.E.P. - Phase B 4th Floor	1	1	17-Sep-04	17-Sep-04
A1137	Strip Forms/Reshore - 3rd Floor	1	1	17-Sep-04	17-Sep-04
A1138	FLOOR COMPLETE	0	0		17-Sep-04
A1240	Pour/Finish Slab - Phase B 4th Floor	1	1	20-Sep-04	20-Sep-04
A1250	F/R/P Columns	3	3	21-Sep-04	23-Sep-04
A1270	Strip Forms/Reshore	1	1	27-Sep-04	27-Sep-04
A1280	FLOOR COMPLETE	0	0		27-Sep-04



Actual Work | Critical Remaining Work | Summary | Remaining Work | Milestone

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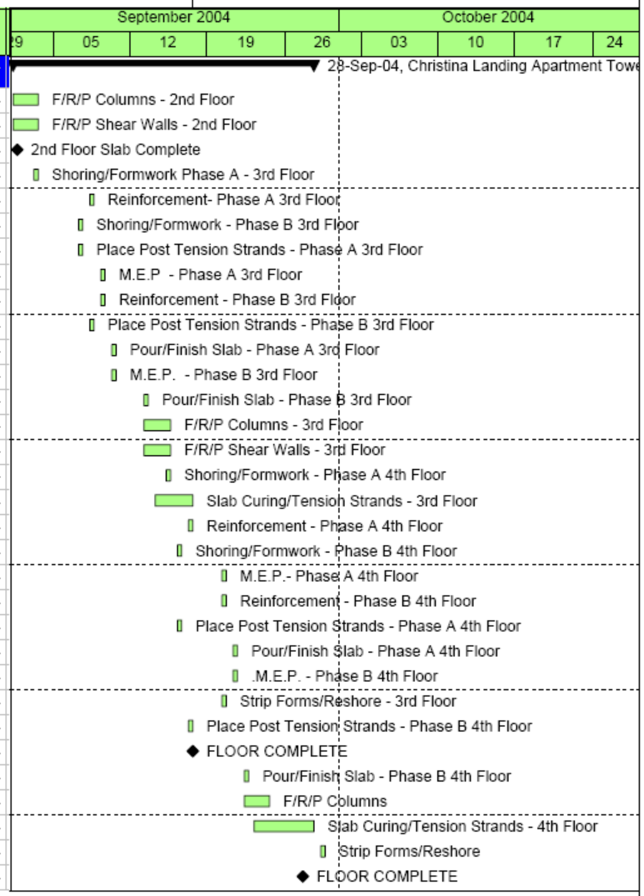
Christina Landing Apartment Tower — **Proposed Redesign Schedule** — Greg Eckel

Activity ID	Activity Name	Original Duration	Remaining Duration	Start	Finish
Christina Landing Apartment ...		20	20	01-Sep-04	28-Sep-04
A1010	F/R/P Columns - 2nd Floor	3	3	01-Sep-04	03-Sep-04
A1020	F/R/P Shear Walls - 2nd Floor	3	3	01-Sep-04	03-Sep-04
A1000	2nd Floor Slab Complete	0	0		01-Sep-04
A1030	Shoring/Formwork Phase A - 3rd Fl...	1	1	03-Sep-04	03-Sep-04
A1050	Reinforcement- Phase A 3rd Floor	1	1	07-Sep-04	07-Sep-04
A1080	Shoring/Formwork - Phase B 3rd Fl...	1	1	07-Sep-04	07-Sep-04
A1040	Place Post Tension Strands - Phas...	1	1	07-Sep-04	07-Sep-04
A1060	M.E.P - Phase A 3rd Floor	1	1	08-Sep-04	08-Sep-04
A1100	Reinforcement - Phase B 3rd Floor	1	1	08-Sep-04	08-Sep-04
A1090	Place Post Tension Strands - Phas...	1	1	08-Sep-04	08-Sep-04
A1070	Pour/Finish Slab - Phase A 3rd Floor	1	1	09-Sep-04	09-Sep-04
A1110	M.E.P. - Phase B 3rd Floor	1	1	09-Sep-04	09-Sep-04
A1120	Pour/Finish Slab - Phase B 3rd Floor	1	1	10-Sep-04	10-Sep-04
A1130	F/R/P Columns - 3rd Floor	3	3	10-Sep-04	14-Sep-04
A1140	F/R/P Shear Walls - 3rd Floor	3	3	10-Sep-04	14-Sep-04
A1150	Shoring/Formwork - Phase A 4th Fl...	1	1	14-Sep-04	14-Sep-04
A1135	Slab Curing/Tension Strands - 3rd ...	4	4	14-Sep-04	17-Sep-04
A1170	Reinforcement - Phase A 4th Floor	1	1	15-Sep-04	15-Sep-04
A1200	Shoring/Formwork - Phase B 4th Fl...	1	1	15-Sep-04	15-Sep-04
A1180	M.E.P.- Phase A 4th Floor	1	1	16-Sep-04	16-Sep-04
A1220	Reinforcement - Phase B 4th Floor	1	1	16-Sep-04	16-Sep-04
A1160	Place Post Tension Strands - Phas...	1	1	16-Sep-04	16-Sep-04
A1190	Pour/Finish Slab - Phase A 4th Floor	1	1	17-Sep-04	17-Sep-04
A1230	.M.E.P. - Phase B 4th Floor	1	1	17-Sep-04	17-Sep-04
A1137	Strip Forms/Reshore - 3rd Floor	1	1	17-Sep-04	17-Sep-04
A1210	Place Post Tension Strands - Phas...	1	1	17-Sep-04	17-Sep-04
A1138	FLOOR COMPLETE	0	0		17-Sep-04
A1240	Pour/Finish Slab - Phase B 4th Floor	1	1	20-Sep-04	20-Sep-04
A1250	F/R/P Columns	3	3	21-Sep-04	23-Sep-04
A1260	Slab Curing/Tension Strands - 4th ...	4	4	23-Sep-04	28-Sep-04
A1270	Strip Forms/Reshore	1	1	27-Sep-04	27-Sep-04
A1280	FLOOR COMPLETE	0	0		27-Sep-04



Actual Work — Critical Remaining Work — Summary — Remaining Work — Milestone

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Conclusions

This construction management breadth study shows the relationship between material cost and job schedule. The total cost of the material for the redesign comes out to be $32,900 cheaper per floor. However, the addition cost related to general conditions due to the prolonged schedule of the post-tensioned floor system carry a $30,000 cost increase. The savings in material cost is all but offset by the extended schedule time. With a more detailed takeoff and cost analysis it would be possible to show more evidence of savings in one or the other floor system. However, this analysis shows that redesigning the building with a post-tensioned system will have little impact on the total building cost and could, if planned thoroughly, save money on the job.

Conclusion

This conclusion section will summarize each of the previous individual conclusion sections.

Two structural redesigns were undergone in the depth study. First a 7" post-tensioned slab was analyzed as an alternate floor system. The slab was shown to be acceptable in flexure, deflection, and punching shear, by both hand calculations and a RAM Concept model. By these calculations and computer outputs the post-tensioned system was determined to be a viable alternate floor system.

The second structural redesign involved negating the affect of the building's equivalent moment frames and replacing their function with a shear wall on the building's east wall. The wall was determined to be 28' long in order to eliminate the effect of torsion on the building due to a north-south wind. By removing the torsional shear in unison with the extra stiffness due to the new shear wall the deflection one of the walls is decreased 3.71". The overall building deflection changes from L/320 to L/433.

For the acoustic breadth study, partition walls between apartments as well as floor elements in both the existing and proposed redesign were investigated. The acoustic transfer between apartments it was analyzed for a source of 80dB transferred into an apartment with a maximum allowable receiving level of 30dB. The gypsum board on metal studs with acoustic blankets was found to be acceptable for this condition. The transfer between the building's weight room and both the apartment above and the lobby below were also investigated. In the original design both slabs were found to be satisfactory. However, with the thinner slab in the redesign, it was suggested that an acoustic drop ceiling be added in the lobby and in the apartment above that rubber flooring be added under the carpet.

The construction management breadth explored the difference in cost between the existing and post-tensioned floor systems. It was found that the post-tensioned system would save approximately $30,000 in material cost due to the significant reinforcing and concrete savings. However, after schedules were calculated for each system it was found that the proposed redesign would increase the overall duration by 22 days. This can be quantified as an additional general conditions cost of approximately $30,000. The overall construction management breadth shows that there is little cost reason to suggest one floor system over the other.

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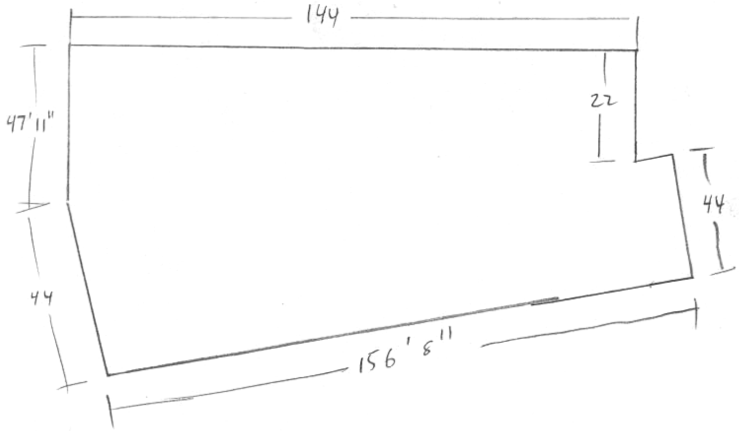
Acknowledgements

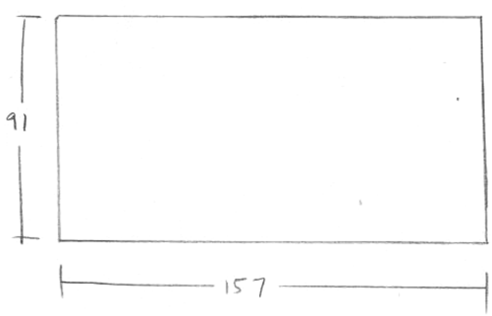
I would like to thank The Buccini/Pollin Group for allowing me to use the Christina Landing Apartment Tower for my senior thesis. Thank you to Kling who supplied me with all my drawings and helped me find and get my building approved. I would also like to thank my advisor professor Boothby, as well as professor Hanagan, professor Parfitt, and the rest of the AE faculty. Thank you to all my friends in the department who helped me through the past 5 years. Special thanks goes to all those who helped me with my thesis. Finally thank you to my family who has supported me throughout my collegiate experience.

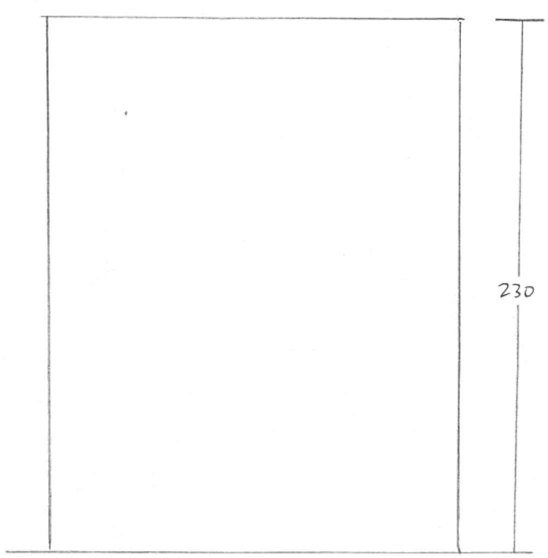
Appendix

Wind Load Analysis



assume





floors	1+2	12'0"
floors	3 – 19	10'6"
floors	20 – 21	12'0"
floor	22	12'0"
total		230'0"

BASIC WIND SPEED: 75 mph (1609.3) use 90 mph

IMPORTANCE FACTOR I = 1.05 (1609.5)

WIND EXPOSURE "C" OPEN TERRAIN

1. find V 6.5.4
   find Kd
2. find I 6.5.5
3. find K_z, K_h 6.5.6
4. find K_{zt} 6.5.7
5. find G or Gf 6.5.8
6. enclosure classification 6.5.9
7. internal pressure coeff. GC_{pi} 6.5.11.1
8. external pressure coeff 6.5.11.2,3
   C_p or GC_{pf} C_t
9. velocity pressure q_z or q_h 6.5.10
10. design wind load p or F 6.5.12

1. V = 90 mph
   K_d = .85
2. Building category II
   I = 1.0
3. K_z, K_h = 1.50
4. K_{zt} = 1.0 flat ground
5. C_t = .016 x = .9

   $T = .016(230)^{.9} = 2.14$

   $n_1 = \frac{1}{2.14} = .467$ FLEXIBLE

Gust factor calcs.

$$G = .925\left(\frac{1+1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_{\bar{z}}}\right)$$

$g_v = g_Q = 3.4$

$$g_R = \sqrt{2\ln[(3600)(.467)]} + \frac{.577}{\sqrt{2\ln[3600(.467)]}} = 4.00$$

n_1 n_1

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53 + .47 R_L)}$$

$z_{min} = 15$ ft

$c = .20$

$\bar{z} = .6(230) = 138$ ft

$\ell = 500$ ft

$\bar{\epsilon} = 1/5.0$

$$L_{\bar{z}} = 500\left(\frac{138}{33}\right)^{1/5} = 665.6$$

$\bar{b} = .65$

$\bar{\alpha} = 1/6.5$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = .65\left(\frac{138}{33}\right)^{1/6.5} 90\left(\frac{88}{60}\right) = 106.9$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{V_{\bar{z}}} = \frac{.467(665.6)}{106.9} = 2.91$$

$$R_n = \frac{7.47 N_1}{(1+10.3 N_1)^{5/3}} = \frac{7.47(2.91)}{[1+10.3(2.91)]^{5/3}} = .0712$$

$$n_h = 4.6 n_1\left(h/\bar{V}_{\bar{z}}\right) = 4.6(.467)\left(\frac{230}{106.9}\right) = 4.62$$

$$n_B = 4.6\left(\frac{n_1}{\bar{V}_{\bar{z}}}\right) = 4.6\left(\frac{.467}{106.9}\right) = .020$$

$$n_L = 15.4 n_1\left(\frac{L}{\bar{V}_{\bar{z}}}\right) = 15.4(.467)\left(\frac{157}{106.9}\right) = 10.56$$

$$R_\ell = \frac{1}{n} - \frac{1}{2n^2}\left(1-e^{-2n}\right)$$

$$R_h = \frac{1}{4.62} - \frac{1}{2(4.62)^2}\left(1-e^{-2(4.62)}\right) = .193$$

$$R_B = \frac{1}{.02} - \frac{1}{2(.02)^2}\left(1-e^{-2(.02)}\right) = .987$$

$$R_L = \frac{1}{10.56} - \frac{1}{2(10.56)^2}\left(1-e^{-2(10.56)}\right) = .090$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B(.53 + .47 R_L)} = \sqrt{\left(\frac{1}{.05}\right) .0712\, (.193)(.987)\left[.53+.47(.09)\right]}$$

$R = .394$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .2\left(\frac{33}{138}\right)^{1/6} = .158$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}} = \sqrt{\frac{1}{1+.63\left(\frac{91+230}{665.6}\right)^{.63}}} = .846$$

$$G_f = .925\left[\frac{1+1.7(.158)\sqrt{3.4^2(.846)^2+(4)^2(.394)^2}}{1+1.7(3.4)(.158)}\right] = .909$$

Velocity Pressure

$$q_z = .00256\, K_z K_{zt} K_d V^2 I$$

see spreadsheet

$$p = \overset{q_z}{q}\, G_f C_p - \overset{q_h}{q_i}(G C_{pi})$$

$G C_{pi} = .18$

$C_p = .8$ for windward

$-.35$ for leeward E-W

$-.5$ for leeward N-S

Gust Factor N-S

$$n_L = 15.4(.467)\left(\frac{91}{106.9}\right) = 6.12$$

$$R_L = \frac{1}{6.12} - \frac{1}{2(6.12)^2}\left[1 - e^{-2(6.12)}\right] = .15$$

$$R = \sqrt{\frac{1}{.05}(.0712)(.193)(.987)\left[.53 + .47(.15)\right]} = .404$$

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{157+230}{665.6}\right)^{.63}}} = .831$$

$$G_f = .925\left[\frac{1 + 1.7(.158)\sqrt{3.4^2(.831)^2 + (4)^2(.404)^2}}{1 + 1.7(3.4)(.158)}\right] = .906$$

WIND CALCULATIONS (see calcs. for additional info.)

Kzt=	1
Kd=	0.85
V=	90
I=	1
Gf (E-W)=	0.909
Gcpi=	0.18
Cp windward=	0.8
Cp leeward=	-0.35
Cp leeward=	-0.5
Gf (N-S)=	0.906

E-W

Height	Kz	qz		p(windward)	p(leeward)	pressure (psf)
0-15	0.85	14.982		20.161	-13.188	33.348
20	0.9	15.863		20.801	-13.188	33.989
25	0.94	16.568		21.314	-13.188	34.502
30	0.98	17.273		21.827	-13.188	35.015
40	1.04	18.331		22.596	-13.188	35.784
50	1.09	19.212		23.237	-13.188	36.425
60	1.13	19.917		23.749	-13.188	36.937
70	1.17	20.622		24.262	-13.188	37.450
80	1.21	21.327		24.775	-13.188	37.963
90	1.24	21.856		25.159	-13.188	38.347
100	1.26	22.208		25.416	-13.188	38.603
120	1.31	23.090		26.056	-13.188	39.244
140	1.36	23.971		26.697	-13.188	39.885
160	1.39	24.500		27.082	-13.188	40.270
180	1.43	25.205		27.595	-13.188	40.782
200	1.46	25.733		27.979	-13.188	41.167
250	1.53	26.967		28.876	-13.188	42.064
230	1.502	26.474		28.517	-13.188	41.705

story	elev.	trib. H below	trib. H above	trib. range	V(lb)	V(k)	M(ft*k)
ground	0		6	0-6	31414.158	31.414	0.000
1	12	6	6	6-18	63130.164	63.130	757.562
2	24	6	5	18-29	59745.807	59.746	1433.899
3	34	5	5	29-39	56059.602	56.060	1906.026
4	44	5	5	39-49	57085.886	57.086	2511.779
5	54	5	5	49-59	57910.938	57.911	3127.191
6	64	5	5	59-69	58715.867	58.716	3757.815
7	74	5	5	69-79	59520.796	59.521	4404.539
8	84	5	5	79-89	60144.616	60.145	5052.148
9	94	5	5	89-99	60567.203	60.567	5693.317
10	104	5	5	99-109	61512.994	61.513	6397.351
11	114	5	5	109-119	61613.610	61.614	7023.952
12	124	5	5	119-129	62519.155	62.519	7752.375
13	134	5	5	129-139	62619.771	62.620	8391.049
14	144	5	5	139-149	63163.098	63.163	9095.486
15	154	5	5	149-159	63223.468	63.223	9736.414
16	164	5	5	159-169	63947.904	63.948	10487.456
17	174	5	5	169-179	64028.397	64.028	11140.941
18	184	5	5	179-189	64571.723	64.572	11881.197
19	194	5	6	189-200	71095.302	71.095	13792.489
20	206	6	6	200-212	79248.862	79.249	16325.266
21	218	6	6	212-224	79248.862	79.249	17276.252
22	230	6	0	224-230	39286.361	39.286	9035.863
						1400.375	166980.368

Base Shear=	1400.375
Base Resisting Moment=	166980.4

N-S

Height	Kz	qz		p(windward)	p(leeward)	pressure (psf)
0-15	0.85	14.982		24.096	-16.758	40.853
20	0.9	15.863		24.734	-16.758	41.492
25	0.94	16.568		25.245	-16.758	42.003
30	0.98	17.273		25.756	-16.758	42.514
40	1.04	18.331		26.523	-16.758	43.281
50	1.09	19.212		27.162	-16.758	43.919
60	1.13	19.917		27.673	-16.758	44.430
70	1.17	20.622		28.184	-16.758	44.941
80	1.21	21.327		28.695	-16.758	45.452
90	1.24	21.856		29.078	-16.758	45.836
100	1.26	22.208		29.333	-16.758	46.091
120	1.31	23.090		29.972	-16.758	46.730
140	1.36	23.971		30.611	-16.758	47.369
160	1.39	24.500		30.994	-16.758	47.752
180	1.43	25.205		31.505	-16.758	48.263
200	1.46	25.733		31.888	-16.758	48.646
250	1.53	26.967		32.783	-16.758	49.540
230	1.502	26.474		32.425	-16.758	49.183

story	elev.	trib. H below	trib. H above	trib. range	V(lb)	V(k)	M(ft*k)
ground	0		6	0-6	22305.971	22.306	0.000
1	12	6	6	6-18	44786.321	44.786	537.436
2	24	6	5	18-29	42138.185	42.138	1011.316
3	34	5	5	29-39	39315.670	39.316	1336.733
4	44	5	5	39-49	39908.559	39.909	1755.977
5	54	5	5	49-59	40385.196	40.385	2180.801
6	64	5	5	59-69	40850.207	40.850	2614.413
7	74	5	5	69-79	41315.218	41.315	3057.326
8	84	5	5	79-89	41675.602	41.676	3500.751
9	94	5	5	89-99	41919.733	41.920	3940.455
10	104	5	5	99-109	42466.121	42.466	4416.477
11	114	5	5	109-119	42524.248	42.524	4847.764
12	124	5	5	119-129	43047.385	43.047	5337.876
13	134	5	5	129-139	43105.512	43.106	5776.139
14	144	5	5	139-149	43419.394	43.419	6252.393
15	154	5	5	149-159	43454.270	43.454	6691.958
16	164	5	5	159-169	43872.780	43.873	7195.136
17	174	5	5	169-179	43919.281	43.919	7641.955
18	184	5	5	179-189	44233.164	44.233	8138.902
19	194	5	6	189-200	48694.844	48.695	9446.800
20	206	6	6	200-212	54098.172	54.098	11144.223
21	218	6	6	212-224	54098.172	54.098	11793.401
22	230	6	0	224-230	26853.781	26.854	6176.370
						968.388	114794.600

Base Shear= 968.3878
Base Resisting Moment= 114794.6

Distribution of Forces

Shear Walls	Direction	E (ksi)	floor	t (in)	h (ft)	L (ft)	Rigidity	Proportion of Rigidity	Percent Rigidity
Wall1/ Wall 2	N-S	4287.00	ground	12.00	0	24.58	#DIV/0!	#DIV/0!	
	N-S	4287.00	2	12.00	12	24.58	26659.73	0.31	31.20
	N-S	4287.00	3	12.00	24	24.58	7735.04	0.30	30.24
	N-S	4287.00	4	12.00	34	24.58	3492.15	0.30	29.73
	N-S	4287.00	5	12.00	44	24.58	1817.53	0.29	29.42
	N-S	4287.00	6	12.00	54	24.58	1050.19	0.29	29.23
	N-S	4287.00	7	12.00	64	24.58	656.26	0.29	29.11
	N-S	4287.00	8	12.00	74	24.58	435.48	0.29	29.03
	N-S	4287.00	9	12.00	84	24.58	302.91	0.29	28.97
	N-S	4287.00	10	12.00	94	24.58	218.82	0.29	28.93
	N-S	4287.00	11	12.00	104	24.58	163.03	0.29	28.90
	N-S	4287.00	12	12.00	114	24.58	124.62	0.29	28.88
	N-S	4287.00	13	12.00	124	24.58	97.35	0.29	28.86
	N-S	4287.00	14	12.00	134	24.58	77.46	0.29	28.84
	N-S	4287.00	15	12.00	144	24.58	62.62	0.29	28.83
	N-S	4287.00	16	12.00	154	24.58	51.33	0.29	28.82
	N-S	4287.00	17	12.00	164	24.58	42.60	0.29	28.82
	N-S	4287.00	18	12.00	174	24.58	35.74	0.29	28.81
	N-S	4287.00	19	12.00	184	24.58	30.27	0.29	28.80
	N-S	4287.00	20	12.00	194	24.58	25.86	0.29	28.80
	N-S	4287.00	21	12.00	206	24.58	21.63	0.29	28.79
	N-S	4287.00	22	12.00	218	24.58	18.27	0.29	28.79
	N-S	4287.00	roof	12.00	230	24.58	15.57	0.29	28.79

Wall3/ Wall 4	E-W	4287.00	ground	12.00	0	32.42	#DIV/0!	#DIV/0!	
	E-W	4287.00	2	12.00	12	32.42	39167.14	0.5	50.00
	E-W	4287.00	3	12.00	24	32.42	13381.74	0.5	50.00
	E-W	4287.00	4	12.00	34	32.42	6627.89	0.5	50.00
	E-W	4287.00	5	12.00	44	32.42	3655.10	0.5	50.00
	E-W	4287.00	6	12.00	54	32.42	2190.28	0.5	50.00
	E-W	4287.00	7	12.00	64	32.42	1401.56	0.5	50.00
	E-W	4287.00	8	12.00	74	32.42	945.12	0.5	50.00
	E-W	4287.00	9	12.00	84	32.42	664.90	0.5	50.00
	E-W	4287.00	10	12.00	94	32.42	484.27	0.5	50.00
	E-W	4287.00	11	12.00	104	32.42	363.02	0.5	50.00
	E-W	4287.00	12	12.00	114	32.42	278.80	0.5	50.00
	E-W	4287.00	13	12.00	124	32.42	218.58	0.5	50.00
	E-W	4287.00	14	12.00	134	32.42	174.43	0.5	50.00
	E-W	4287.00	15	12.00	144	32.42	141.35	0.5	50.00
	E-W	4287.00	16	12.00	154	32.42	116.10	0.5	50.00
	E-W	4287.00	17	12.00	164	32.42	96.49	0.5	50.00
	E-W	4287.00	18	12.00	174	32.42	81.05	0.5	50.00
	E-W	4287.00	19	12.00	184	32.42	68.73	0.5	50.00
	E-W	4287.00	20	12.00	194	32.42	58.77	0.5	50.00
	E-W	4287.00	21	12.00	206	32.42	49.20	0.5	50.00
	E-W	4287.00	22	12.00	218	32.42	41.60	0.5	50.00
	E-W	4287.00	roof	12.00	230	32.42	35.48	0.5	50.00

Wall 5	E-W	4287.00	ground	12.00	0	28.00	#DIV/0!	#DIV/0!	
	E-W	4287.00	2	12.00	12	28.00	32140.79	0.38	37.61
	E-W	4287.00	3	12.00	24	28.00	10106.12	0.40	39.51
	E-W	4287.00	4	12.00	34	28.00	4761.28	0.41	40.54
	E-W	4287.00	5	12.00	44	28.00	2542.18	0.41	41.15
	E-W	4287.00	6	12.00	54	28.00	1492.08	0.42	41.53
	E-W	4287.00	7	12.00	64	28.00	941.79	0.42	41.78
	E-W	4287.00	8	12.00	74	28.00	629.16	0.42	41.94
	E-W	4287.00	9	12.00	84	28.00	439.69	0.42	42.05
	E-W	4287.00	10	12.00	94	28.00	318.70	0.42	42.14
	E-W	4287.00	11	12.00	104	28.00	238.04	0.42	42.20
	E-W	4287.00	12	12.00	114	28.00	182.31	0.42	42.25
	E-W	4287.00	13	12.00	124	28.00	142.62	0.42	42.28
	E-W	4287.00	14	12.00	134	28.00	113.62	0.42	42.31
	E-W	4287.00	15	12.00	144	28.00	91.94	0.42	42.33
	E-W	4287.00	16	12.00	154	28.00	75.43	0.42	42.35
	E-W	4287.00	17	12.00	164	28.00	62.64	0.42	42.37
	E-W	4287.00	18	12.00	174	28.00	52.57	0.42	42.38
	E-W	4287.00	19	12.00	184	28.00	44.55	0.42	42.39
	E-W	4287.00	20	12.00	194	28.00	38.07	0.42	42.40
	E-W	4287.00	21	12.00	206	28.00	31.85	0.42	42.41
	E-W	4287.00	22	12.00	218	28.00	26.92	0.42	42.42
	E-W	4287.00	roof	12.00	230	28.00	22.95	0.42	42.43

Center of Rigidity and Mass

Center of Rigidity			Center of Mass		Difference	
Floor	Distance from West Face	Distance from South Face	Distance From West Face	Distance from South Face	E-W	N-S
ground						
2	40.1945731	75.15	40	73.9	-0.194573	1.25
3	41.4097872	75.15			-1.409787	
4	42.0624497	75.15			-2.06245	
5	42.456253	75.15			-2.456253	
6	42.6983846	75.15			-2.698385	
7	42.8538224	75.15			-2.853822	
8	42.9581318	75.15			-2.958132	
9	43.0309638	75.15			-3.030964	
10	43.0835819	75.15			-3.083582	
11	43.1227169	75.15			-3.122717	
12	43.1525536	75.15			-3.152554	
13	43.1757897	75.15			-3.17579	
14	43.1942202	75.15			-3.19422	
15	43.2090742	75.15			-3.209074	
16	43.2212142	75.15			-3.221214	
17	43.2312595	75.15			-3.231259	
18	43.2396629	75.15			-3.239663	
19	43.246762	75.15			-3.246762	
20	43.2528123	75.15			-3.252812	
21	43.258961	75.15			-3.258961	
22	43.2641375	75.15			-3.264138	
roof	43.2685359	75.15			-3.268536	

Story Shears and Torsional Forces

	Story Force N-S Wind	Story Shear N-S Wind	Story Force E-W Wind	Story Shear E-W Wind	Torsional Force N-S wind	Torsional Force E-W wind
ground	22.31	968.39	31.414158	1400.37455	0	
2.00	44.79	946.08	63.130164	1368.96039	-184.0820342	1711.20049
3.00	42.14	901.30	59.745807	1305.83022	-1270.634855	1632.28778
4.00	39.32	859.16	56.059602	1246.08442	-1771.96874	1557.60552
5.00	39.91	819.84	57.085886	1190.02481	-2013.738483	1487.53102
6.00	40.39	779.93	57.910938	1132.93893	-2104.559445	1416.17366
7.00	40.85	739.55	58.715867	1075.02799	-2110.538302	1343.78499
8.00	41.32	698.70	59.520796	1016.31212	-2066.839843	1270.39015
9.00	41.68	657.38	60.144616	956.791327	-1992.502423	1195.98916
10.00	41.92	615.71	60.567203	896.646712	-1898.582525	1120.80839
11.00	42.47	573.79	61.512994	836.079509	-1791.774729	1045.09939
12.00	42.52	531.32	61.61361	774.566514	-1675.017951	968.208143
13.00	43.05	488.80	62.519155	712.952904	-1552.31568	891.19113
14.00	43.11	445.75	62.619771	650.433749	-1423.821634	813.042186
15.00	43.42	402.64	63.163098	587.813977	-1292.114006	734.767472
16.00	43.45	359.22	63.223468	524.650879	-1157.138963	655.813599
17.00	43.87	315.77	63.947904	461.427411	-1020.335428	576.784264
18.00	43.92	271.90	64.028397	397.479508	-880.8559619	496.849384
19.00	44.23	227.98	64.571723	333.451111	-740.190745	416.813889
20.00	48.69	183.74	71.095302	268.879388	-597.6878956	336.099234
21.00	54.10	135.05	79.248862	197.784085	-440.1230906	247.230106
22.00	54.10	80.95	79.248862	118.535223	-264.2383079	148.169029
roof	26.85	26.85	39.286361	39.286361	-87.77254779	49.1079512
base shear	968.39		1400.3745			

Direct Shears on Walls

N-S						
		Story Shear		Wall 1	Wall 2	Wall 5
ground	22.31	968.39		#DIV/0!	#DIV/0!	#DIV/0!
2	44.79	946.08		295.13	295.13	355.812371
3	42.14	901.30		272.58	272.58	356.135857
4	39.32	859.16		255.44	255.44	348.274494
5	39.91	819.84		241.22	241.22	337.397641
6	40.39	779.93		228.00	228.00	323.933649
7	40.85	739.55		215.29	215.29	308.962037
8	41.32	698.70		202.83	202.83	293.038316
9	41.68	657.38		190.46	190.46	276.460893
10	41.92	615.71		178.13	178.13	259.442096
11	42.47	573.79		165.83	165.83	242.130224
12	42.52	531.32		153.43	153.43	224.458587
13	43.05	488.80		141.06	141.06	206.672076
14	43.11	445.75		128.57	128.57	188.599634
15	43.42	402.64		116.09	116.09	170.455139
16	43.45	359.22		103.54	103.54	152.142339
17	43.87	315.77		90.99	90.99	133.787869
18	43.92	271.90		78.33	78.33	115.235336
19	44.23	227.98		65.67	65.67	96.6468698
20	48.69	183.74		52.92	52.92	77.91251
21	54.10	135.05		38.89	38.89	57.277681
22	54.10	80.95		23.31	23.31	34.3400436
roof	26.85	26.85		7.73	7.73	11.3933
base shear	968.39					

E-W						
				Wall 3	Wall 4	
ground	31.41	1400.37		700.19	700.19	
2.00	63.13	1368.96		684.48	684.48	
3.00	59.75	1305.83		652.92	652.92	
4.00	56.06	1246.08		623.04	623.04	
5.00	57.09	1190.02		595.01	595.01	
6.00	57.91	1132.94		566.47	566.47	
7.00	58.72	1075.03		537.51	537.51	
8.00	59.52	1016.31		508.16	508.16	
9.00	60.14	956.79		478.40	478.40	
10.00	60.57	896.65		448.32	448.32	
11.00	61.51	836.08		418.04	418.04	
12.00	61.61	774.57		387.28	387.28	
13.00	62.52	712.95		356.48	356.48	
14.00	62.62	650.43		325.22	325.22	
15.00	63.16	587.81		293.91	293.91	
16.00	63.22	524.65		262.33	262.33	
17.00	63.95	461.43		230.71	230.71	
18.00	64.03	397.48		198.74	198.74	
19.00	64.57	333.45		166.73	166.73	
20.00	71.10	268.88		134.44	134.44	
21.00	79.25	197.78		98.89	98.89	
22.00	79.25	118.54		59.27	59.27	
roof	39.29	39.29		19.64	19.64	
base shear	1400.37					

Example of Torsional Shear on Floors 2 and 3

Torsion	Floor 2				
Wall	R	x	Rx^2	Rx/Rx^2	Torsional Shear
Wall 1 (N-S)	26659.73	-7.794573	1619721.8	-0.0019319	0.35562126
Wall 2 (N-S)	26659.73	-40.19457	43071561	-0.0099621	1.833845756
Wall 5 (N-S)	32140.79	39.80543	50926177	0.01189397	-2.189467017
Wall 3 (E-W)	39167.14	-12.35	5973870.1	-0.0044969	-7.695164737
Wall 4 (E-W)	39167.14	12.35	5973870.1	0.00449694	7.695164737
			107565200		

Torsion	Floor 3				
Wall	R	x	Rx^2	Rx/Rx^2	Torsional Shear
Wall 1 (N-S)	7735.04	-9.009787	627901.72	-0.0021103	2.681453967
Wall 2 (N-S)	7735.04	-41.40979	13263820	-0.0096992	12.32420208
Wall 5 (N-S)	10106.12	38.59021	15050085	0.01180957	-15.00565605
Wall 3 (E-W)	13381.74	-12.35	2041016.4	-0.0050044	-8.16862008
Wall 4 (E-W)	13381.74	12.35	2041016.4	0.0050044	8.16862008
			33023839		

Total Shears on Walls in Both Systems

Wall 1	Proposed Redesign	
Direct	Torsional	Total
295.13	0.36	295.49
272.58	2.68	275.26
255.44	3.89	259.33
241.22	4.51	245.73
228.00	4.77	232.77
215.29	4.82	220.11
202.83	4.74	207.57
190.46	4.59	195.05
178.13	4.38	182.51
165.83	4.14	169.97
153.43	3.88	157.31
141.06	3.60	144.66
128.57	3.30	131.88
116.09	3.00	119.09
103.54	2.69	106.23
90.99	2.37	93.36
78.33	2.05	80.38
65.67	1.72	67.39
52.92	1.39	54.31
38.89	1.02	39.91
23.31	0.61	23.92
7.73	0.20	7.93

Wall 1	Original Design	
Direct	Torsional	Total
460.93	292.23	753.15
440.85	266.16	707.02
418.41	235.98	654.39
395.48	207.32	602.79
372.41	182.40	554.81
348.65	159.03	507.68
324.84	137.87	462.71
300.97	118.54	419.51
277.19	101.00	378.20
253.88	85.50	339.38
230.85	71.80	302.65
208.27	59.53	267.80
186.17	48.91	235.08
164.88	39.88	204.75
144.08	31.96	176.04
124.03	25.17	149.20
104.70	19.54	124.24
85.94	14.67	100.61
67.93	10.67	78.61
48.77	6.91	55.68
28.62	3.70	32.33
9.32	1.11	10.42

Wall 2	Proposed Redesign	
Direct	Torsional	Total
295.13	1.83	296.97
272.58	12.32	284.90
255.44	16.92	272.37
241.22	19.05	260.27
228.00	19.78	247.78
215.29	19.76	235.05
202.83	19.30	222.13
190.46	18.57	209.03
178.13	17.67	195.80
165.83	16.65	182.48
153.43	15.56	168.99
141.06	14.41	155.47
128.57	13.21	141.78
116.09	11.98	128.08
103.54	10.73	114.27
90.99	9.46	100.45
78.33	8.16	86.49
65.67	6.86	72.52
52.92	5.54	58.45
38.89	4.08	42.96
23.31	2.45	25.75
7.73	0.81	8.54

Wall 2	Original Design	
Direct	Torsional	Total
460.93	neg value	460.93
440.85	neg value	440.85
418.41	neg value	418.41
395.48	neg value	395.48
372.41	neg value	372.41
348.65	neg value	348.65
324.84	neg value	324.84
300.97	neg value	300.97
277.19	neg value	277.19
253.88	neg value	253.88
230.85	neg value	230.85
208.27	neg value	208.27
186.17	neg value	186.17
164.88	neg value	164.88
144.08	neg value	144.08
124.03	neg value	124.03
104.70	neg value	104.70
85.94	neg value	85.94
67.93	neg value	67.93
48.77	neg value	48.77
28.62	neg value	28.62
9.32	neg value	9.32

Wall 5	Proposed Redesign	
Direct	Torsional	Total
355.81	neg value	355.81
356.14	neg value	356.14
348.27	neg value	348.27
337.40	neg value	337.40
323.93	neg value	323.93
308.96	neg value	308.96
293.04	neg value	293.04
276.46	neg value	276.46
259.44	neg value	259.44
242.13	neg value	242.13
224.46	neg value	224.46
206.67	neg value	206.67
188.60	neg value	188.60
170.46	neg value	170.46
152.14	neg value	152.14
133.79	neg value	133.79
115.24	neg value	115.24
96.65	neg value	96.65
77.91	neg value	77.91
57.28	neg value	57.28
34.34	neg value	34.34
11.39	neg value	11.39

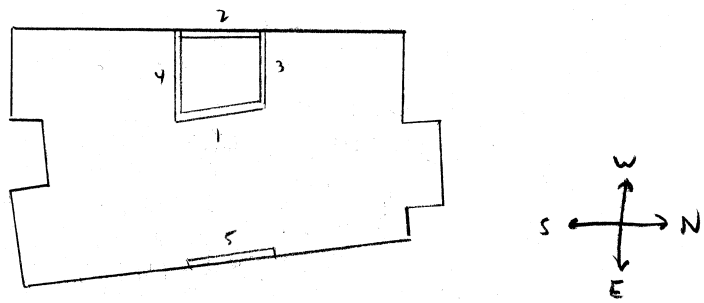
All Frames	Original Design	
Direct	Torsional	Total
24.23	48.97	73.20
19.59	37.83	57.41
22.34	40.66	63.00
28.89	48.43	77.32
35.12	55.13	90.25
42.25	61.69	103.94
49.01	66.60	115.61
55.45	69.88	125.33
61.32	71.44	132.75
66.03	71.10	137.14
69.61	69.15	138.76
72.25	65.92	138.18
73.41	61.48	134.89
72.89	56.13	129.02
71.06	50.13	121.19
67.71	43.70	111.41
62.50	37.07	99.57
56.09	30.39	86.48
47.88	23.85	71.73
37.51	16.85	54.36
23.70	9.70	33.40
8.22	3.08	11.30

Wall 3	**Proposed Redesign**	
Direct	Torsional	Total
700.19	neg value	700.19
684.48	neg value	684.48
652.92	neg value	652.92
623.04	neg value	623.04
595.01	neg value	595.01
566.47	neg value	566.47
537.51	neg value	537.51
508.16	neg value	508.16
478.40	neg value	478.40
448.32	neg value	448.32
418.04	neg value	418.04
387.28	neg value	387.28
356.48	neg value	356.48
325.22	neg value	325.22
293.91	neg value	293.91
262.33	neg value	262.33
230.71	neg value	230.71
198.74	neg value	198.74
166.73	neg value	166.73
134.44	neg value	134.44
98.89	neg value	98.89
59.27	neg value	59.27

Wall 3	**Original Design**	
Direct	Torsional	Total
700.19	neg value	700.19
684.48	neg value	684.48
652.92	neg value	652.92
623.04	neg value	623.04
595.01	neg value	595.01
566.47	neg value	566.47
537.51	neg value	537.51
508.16	neg value	508.16
478.40	neg value	478.40
448.32	neg value	448.32
418.04	neg value	418.04
387.28	neg value	387.28
356.48	neg value	356.48
325.22	neg value	325.22
293.91	neg value	293.91
262.33	neg value	262.33
230.71	neg value	230.71
198.74	neg value	198.74
166.73	neg value	166.73
134.44	neg value	134.44
98.89	neg value	98.89
59.27	neg value	59.27

Wall 4	**Proposed Redesign**	
Direct	Torsional	Total
700.19	7.70	707.88
684.48	8.17	692.65
652.92	8.29	661.21
623.04	8.23	631.27
595.01	8.03	603.04
566.47	7.74	574.21
537.51	7.40	544.91
508.16	7.02	515.18
478.40	6.62	485.01
448.32	6.19	454.52
418.04	5.76	423.80
387.28	5.31	392.60
356.48	4.86	361.33
325.22	4.40	329.61
293.91	3.93	297.84
262.33	3.46	265.78
230.71	2.98	233.70
198.74	2.50	201.24
166.73	2.02	168.75
134.44	1.49	135.93
98.89	0.89	99.78
59.27	0.30	59.56

Wall 4	**Original Design**	
Direct	Torsional	Total
700.19	19.41	719.59
684.48	20.65	705.13
652.92	20.28	673.19
623.04	19.25	642.29
595.01	17.95	612.96
566.47	16.47	582.94
537.51	14.96	552.48
508.16	13.46	521.62
478.40	12.01	490.41
448.32	10.65	458.98
418.04	9.38	427.42
387.28	8.19	395.47
356.48	7.09	363.57
325.22	6.09	331.31
293.91	5.17	299.07
262.33	4.32	266.64
230.71	3.55	234.26
198.74	2.84	201.58
166.73	2.19	168.92
134.44	1.53	135.97
98.89	0.88	99.77
59.27	0.28	59.55



DETERMINE LENGTH, THICKNESS, AND LOCATION OF 5 SO C.M. AND C.R. IN THE N-S DIRECTION ARE THE SAME.

C.M. = 40.3' from W WALL

MAKE C.R. = 40'

$$C.R. = \frac{R_1(32.4) + R_2(0) + R_5(80)}{R_1 + R_2 + R_3} = 40'$$

$$\frac{34536(32.4) + R_5(80)}{(69072 + R_5)} = 40'$$

$$1118966 + 80(R_5) = 2762880 + 40(R_5)$$

$$R_5 = 41098$$

$$R_5 = 41098 = \frac{Et}{4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)} = \frac{4287(12)}{4\left(\frac{120}{L}\right)^3 + 3\left(\frac{120}{L}\right)}$$

$$L = 336 \text{ in}$$

$$L = 28'$$

Punching Shear Check For Worst Case Col.

no shear reinf. needed unless $V_u > \phi V_c$

check punching shear @ $d/2$

$$v_c = \beta_p \sqrt{f'_c} + .3\,\sigma_g + V_p\, b_o d_e$$

$$\beta_p = \min \begin{cases} 3.5 & \leftarrow \text{controls} \\ 1.5 + \dfrac{\alpha_s d_e}{b_o} = 1.5 + \dfrac{40\,(6.25)}{96} = 4.1 \end{cases}$$

$$v_c = 3.5\sqrt{5000} + .3\,(175) + 15 = 315 \text{ psi}$$

$$\phi v_c = .75\,(315) = 236 \text{ psi}$$

$$v_u = \frac{V_u}{A_c} + \frac{\gamma_v M_u c_3}{J_c} = \frac{122000}{756.3} + \frac{.4\,(37.1)(15.1)(12000)}{92347} = 190.4$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{24 + 6.25}{24 + 6.25}}} = .4$$

$$A_c = 2\left(24 + 24 + 6.25(2)\right) 6.25 = 756.3$$

$$c_3 = \frac{(6.25 + 24)}{2} = 15.1$$

$$J_c = \frac{6.25\,(24 + 6.25)^3}{6} + \frac{(24 + 6.25)\,6.25^3}{6} + \frac{6.25\,(24 + 6.25)\,(6.25 + 24)^2}{2}$$

$$= 4614 + 1230.9 + 86502 = 92347$$

Max Stress = 190.4 psi

Max Allowable Stress = 315 psi OK

Check PT Slab

worst case moment is at the first interior support

$\sigma_t > -6\sqrt{f'c}$

$\sigma_t = \sigma_g + \frac{M}{Z_b} = 175\,psi - \frac{.75\,(28.6)(12000)}{2450} = -69.9 \quad > -6\sqrt{f'c} = -424.3$

$Z_b = \frac{bh^2}{6} = \frac{25\,(12)(7)^2}{6} = 2450\ in^3$

$\sigma_b = 106.5 + 175 = 281.5 < .45(f'c) = 2250$

1.8

check bonded reinforcement @ ultimate

$a = \frac{A_{ps} f_{ps} + A_s f_y}{.85(f'c)(b)} = \frac{15(.153)(175) + 3.33(60)}{.85\,(4)\,(25)(12)} = .6\ in$ ← #4 @ 18" o.c.

$\phi M_n = .9\left(2.3\,(175) + 3.3\,(60)\right)\left(6.25 - \frac{.6}{2}\right)/12 = 268\ ft{\cdot}k$

$M_u < \phi M_n \quad \therefore$ **OK**

Acoustic Calculations

ACOUSTIC BREADTH STUDY CALCS

CASES TO CHECK

1. ORIGINAL FLOOR DESIGN B/W GYM AND FLOORS ABOVE AND BELOW
2. STR. REDESIGN FLOOR B/W GYM AND FLOORS ABOVE AND BELOW
3. TYPICAL WALL ASSEMBLY B/W TWO APT UNITS

① ORIGINAL FLOOR DESIGN

8" RC SLAB

IIC RATING = 36

α for walls in source + recieving room

125	250	500	1000	2000	4000	
.28	.12	.10	.07	.13	.09	2 layers gypsum board

α for floors

.02	.03	.03	.03	.03	.02	rubber on concrete
.02	.06	.14	.37	.60	.65	carpet on concrete

α for ceiling

.15	.10	.05	.04	.07	.09	gyp. board

recieving room 1 $\boxed{\text{LOBBY}}$

floor A = 3750 ft²

ceiling A = 3750

wall A = 4030

recieving room 2 $\boxed{\text{APT.}}$

floor A = 187

ceiling A = 187

wall A = 560

Acceptable sound level in recieving room

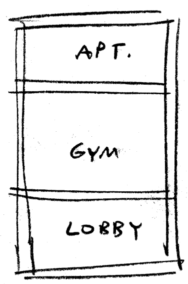
Apt 30 dB

Lobby 50 dB

$L_R = L_S - NR$

recieving level = source level - noise reduction

$NR = TL + 10 \log \frac{\Sigma S\alpha}{S_{eir}}$



Source level = 85 dB

Calc Gym to Apt case 1

$TL = 57\, dB$ for 8" RC floor

$\Sigma S\alpha = 187(.14) + 187(.05) + 560(.10) = 91.5$

$NR = 57 + 10 \log \left(\frac{91.5}{187}\right) = 53.9\, dB$

$L_R = L_S - NR = 85 - 53.9 = 31.1 \approx 30.0\, dB$ OK

Gym to Lobby case 1

IIC rating for impact = 36 dB

$\Sigma S\alpha = 3750(.14) + 3750(.15) + 4030(.1$
$= 1491$

$NR = 36 + 10 \log \left(\frac{1491}{1500}\right) = 36\, dB$

$L_R = 85 - 36 = 49\, dB < 50\, dB$ OK

Case 2

Gym → Apt

TL = 55 dB

$NR = 55 + 10 \log\left(\frac{91.5}{187}\right) = 51.9$ dB

$L_R = 85 - 51.9 = 33.1 > 30$ dB consider increasing ceiling absobtion.

w/ improved flooring in apt.

TL = 55

$NR = 55 + 10 \log\left(\frac{210}{187}\right) = 55.5$ dB

$L_R = 85 - 55.5 = 29.5 < 30$ dB OK w/ improved flooring in apt.

Gym → Lobby

IIC = 34 will not pass by observation

try w/ improved ceiling below

$NR = 34 + 10 \log\left(\frac{1729}{1500}\right) = 34.62$

$L_R = 85 - 34.6 = 50.4 \approx 50$ OK

Case 3

TL = 57

source = 80 dB

apt level wanted = 30 dB

$NR = 57 + 10 \log\left(\frac{91.5}{116}\right) = 56.2$

$L_R = 80 - 56.2 = 23.8$ dB < 30 dB OK