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Architectural Engineering

Pennsylvania State University 2012



Southwest Housing, Arizona State University

Final Summary Report

Southwest Student Housing
Tempe, Arizona
Final Summary Report

Southwest Student Housing Building

1000 Apache Boulevard East, Tempe, Arizona

Project Team:

- **Owner** - Arizona State University
- **Architect** - BOKA Powell
- **Structural Engineer** - Thornton-Tomasetti
- **MEP Engineer** - WSP Flack+Kurtz
- **Agency CM** - Romani Group, Inc.

Construction:

- **Projected schedule** - 302 days total
125 days Preconstruction
177 days Construction
- 4 weeks for concrete cores
- 2 days for each half-floor
(Each floor is constructed on the ground in (2) halves, which are separately lifted into place with (6) 75-ton strand jacks, placed at the corners of the finished concrete cores. Interior work starts immediately on each floor once it is constructed and lifted into place.)

Basics:

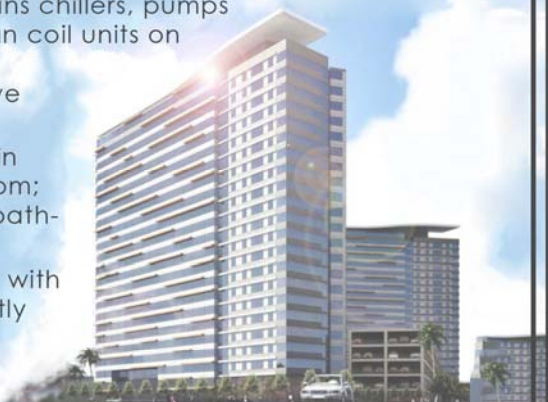
- **Purpose** - Dormitories
- **Size** - 268 units with 528 beds;
Area per story - 13,000 ft²
Total area - 260,000 ft²
- **Stories** - 20 stories
- **Estimated Total Cost** - \$37,358,413
- **Distinguishing features** - modular building (3 or 4 modules per floor), prefabricated bathrooms, constructed as a modified lift slab

Structural:

- **Foundation** - concrete piers (12 to 60" in diameter) penetrating 12.5 to 47.5' into the ground; piers and mats are located under each core to directly support gravity load
- **Superstructure** - composite floor (3-1/4" lightweight concrete on 3" metal deck) attached to structural steel frame made of wide flange beams (sizes from W14x22 to W24x176) using Hilti fasteners;
Typical bay size: 12'-6"x13'
- **Lateral System** - gravity and lateral system are the same: (3) 8" thick concrete cores, 25'x25' on center with a single layer of rebar running through center

MEP Systems:

- **Mechanical** - 1st floor mechanical room contains chillers, pumps and water tanks 4-pipe system with single fan coil units on each pipe supply hot and chilled water;
Bathroom exhaust fans located on roof, above stacked bathrooms
- **Fire Safety** - wet system in building, dry system in parking lot; jockey pumps in mechanical room;
1 sprinkler in each sleeping room, foyer and bathroom; all corridors are sprinkled
- **Lighting/Electrical** - 1 main panel at each floor with subs in each unit; Electrical design is currently incomplete and pending.

**Ksenia Tretiakova**
Structural Option**CPEP Website:**
<http://www.engr.psu.edu/ae/thesis/portfolios/2012/KKT5018/>

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Executive Summary

The Southwest Student Housing building is a new building to be constructed in Tempe, AZ. It will be built using slip-forms for the concrete core structure that will function both as the gravity and lateral system. The floors will be assembled using lift-slab construction for ease of assembly and speed of erection. The building will cost an estimated \$37 million, and 177 days of construction to complete.

In order to test the versatility of this type of building construction, this report focuses on redesigning the structural system to withstand increased seismic design loads. The new design loads for the building are about 325% greater than the original seismic design loads, which resulted in a doubling of the overall concrete volume (an increase in wall thickness from 8" to 16"), and quintupling of reinforcement volume.

To truly investigate the impact of transitioning the building design to SDC D, the building must be evaluated from a cost and scheduling standpoint as well. This report investigates the bare material cost difference between the original building design and the redesign geared toward SDC D. The overall cost difference totals to an increase of about 8% of the total building cost. The resulting impact on the schedule is minimal, with approximately 10 to 20 additional days needed to slip-form the larger cores. The construction method is the primary reason that the impact on the schedule is so small--the fluid pre-assembly of the majority of the components needed for construction allows for minimal time delays throughout the actual construction process.

In addition to cost and scheduling, this report includes investigations on the impact of the structural redesign on the typical building floor plan. To have a stiffer building, the cores were made to be as continuous as possible, which removed the majority of the openings that were previously present. These openings gave the residents access to the interior of the cores for usable apartment unit space. Making the walls continuous closes off that usable space, unless the floor plan is rearranged. The *Effect on Architecture* section provides several options for how to rearrange the apartment units to take advantage of some corridor space to provide access to the cores without having openings. The maximum decrease in unit area for the new floor plan options was found to be about 5% of the original apartment unit area, while some apartments received up to a 33% increase in area.

This report also discusses the potential for the Southwest Student Housing building to become LEED Certified. Ultimately, it is concluded that the building could become LEED Certified with relative ease and minimal cost investment if the owner and contractor take the time to plan ahead for certain things during the preconstruction phase of this project.

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Introduction

The Southwest Student Housing building is a 20-story project commissioned by Arizona State University (ASU). An artistic rendering of how the building will potentially appear can be found in Figure 1. This building will contain apartments that can be rented by students who attend ASU. There are 268 apartment units in the building plan, consisting of studio apartments and one to four bedroom units. In total, there will be 528 beds in the Southwest Student Housing building.

This project began in November of 2010 and is still currently in the works, due to several modifications to the overall building design. The Thornton-Termohlen Group is responsible for the structural design and the ideas for the construction methods that will be employed, which is a unique feature of this particular building design. The construction method was chosen for its high speed, ease, and low cost.

This building will be erected using lift-slab construction-- a construction method that has long been frowned upon in the United States as a result of the catastrophic collapse of L'Ambiance Plaza in 1987. According to individuals involved with the company, this method of construction has been used elsewhere around the world for the last 40 years; the intent of this project, and others by the Thornton-Termohlen Group, is to reintroduce the lift-slab construction method to the United States because of its speed, utility and low cost. With this construction method, the estimated time in construction is 177 days from pouring the foundations to completing the final interior finishes.

The Southwest Student Housing building has three concrete cores as its primary structural system. These cores will be the first item constructed. Once they are fully erected, each floor will be assembled one half at a time as indicated in Figure 2, and then lifted into place with 75-ton strand jacks located at the corners of each core. The floors will be erected from the top-down. They will contain all prefabricated partitions, bathrooms, and the building envelope for that floor.



Figure 1 - Rendering of potential building appearance



Figure 2 - Half a floor lifted into place

Site Information

The Southwest Student Housing building is located at 1000 Apache Boulevard East, in Tempe, Arizona. This location is on the Arizona State University's campus. From the satellite map pictured in Figure 3, it can be observed that the site is located in an urban area with an already-developed site. The dimensions of the site are about 150' x 300', with an additional piece of land sprouting out of the northwestern portion of the site (the irregular shape is highlighted in red in Figure 3). The site is located within 400 feet of 5 different bus stops.

The geotechnical report for this site indicates a 10 to 35 foot layer of Silty and Poorly Graded Sand followed by a layer of Sand Gravel Cobble down to a total of 100 feet into the ground.



Figure 3 - Site map, 1000 Apache Blvd East, Tempe, AZ
(modified from <http://maps.google.com>)

The building design has a long, thin footprint that is 52 feet wide and 250 feet long, which means the building will need to be oriented either with the long axis in the North-South direction, or diagonally on the site. This sort of orientation will provide considerable sun exposure to the windows of the building, allowing for natural daylighting to be an option in the design.



Figure 4 - Typical floor plan with bays marked

The floor plan is split into almost square bays, 13 ft (short direction) by 12.5 feet (long direction), as can be seen in Figure 4. The plan is very regular and modular to allow for speed and ease of construction. Each floor will capitalize on the use of prefabricated bathrooms, pre-plumbed and pre-wired partition walls, and mechanical modules that can be slid into the building through the corridor that runs across each floor in the long direction.

Building Envelope

The theme of "fast" and "modular" extends to the building envelope as well. The envelope consists of exterior insulation finishing system (EIFS) cladding and conventional glazing. To assemble the building, the EIFS and glazing are assembled on each floor before it is lifted into place. The envelope is erected at the edge of the floor, and then slid into place like a puzzle piece after the floor is lifted to its designated height/location.

Figure 5 shows the location of the cladding while the floor is being lifted into place. Once each floor is lifted and affixed to its end location, long aluminum tabs will be slid into the gaps created by the panels of EIFS, and all interior cracks and openings will be caulked closed.

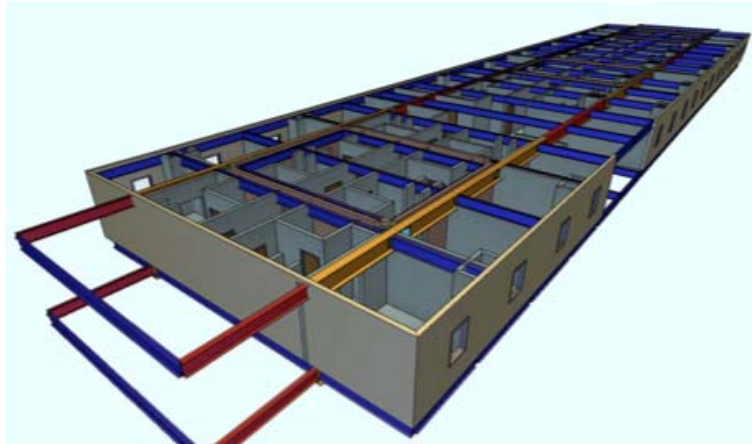


Figure 5 - How the cladding is lifted on the floor

Structural Systems

Foundation

According to the area geotechnical report, the Southwest Student Housing building will exert significant loads on the foundation elements. As a result, the report recommends a deep foundation system that penetrates through the second layer of soil on the site in order to limit settlement. Specifically, the report calls for drilled piers with no pier shaft sized with a diameter less than one foot. Each pier is expected to penetrate at least twice the shaft diameter into the second layer of soil. A potential foundation layout can be found in Appendix A, along with relevant calculations.

Officials working on this project have stated that, though the soils report requires deep foundations, the building designers are planning on having a mat foundation. The designers intend on proving that their design works for the soil conditions, and submitting the building design for peer review.

Floor System

The floor system is identical on every floor. It consists of 3-1/4" lightweight concrete topping on 3" metal deck, with minimum 20 gage. According to officials working on this building design, the concrete is not expected to work in composite with the steel. The deck runs in the long direction of the building footprint, and is supported by a structural steel frame, with clear spans of 12.5 feet. The structural steel wide flange sizes run from W14x22 infill beams to W24x176 interior girders, as indicated on the close-up of the framing plan in Figure 6. The girders span the length of the building while the beams span the entire short direction. An important thing to note is that each beam that runs through the girders is continuous at all points. The infill beams span either 13 feet (one bay in the short direction) or 24 feet (the inside distance of the concrete core walls). Each structural steel element in the floor system is cambered, with the interior girders cambered up to 4 inches at the ends of the building. Camber diagrams from the structural engineers can be found in Appendix A.

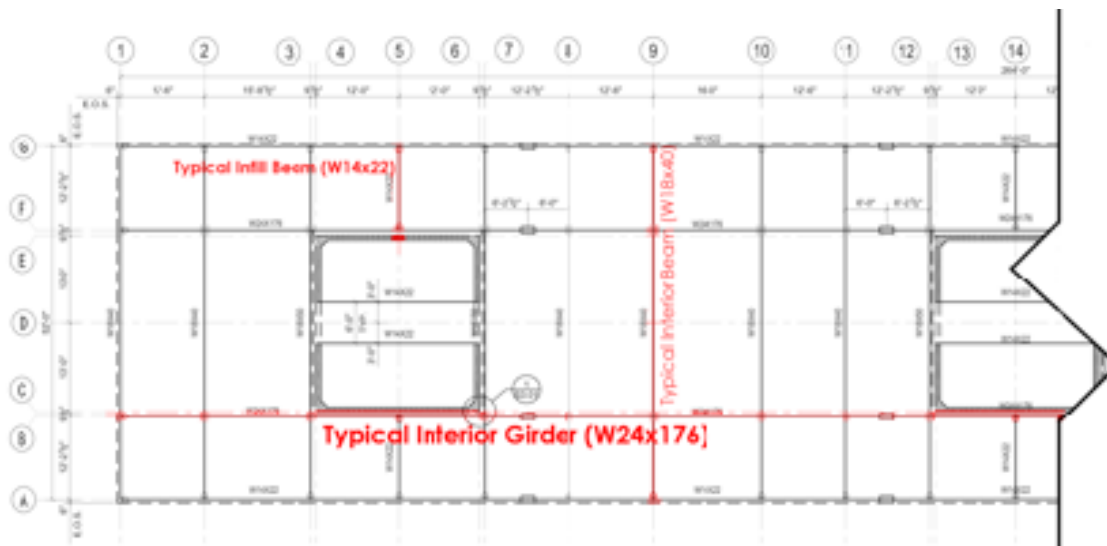


Figure 6 - Floor framing plan with key elements highlighted in red

Gravity and Lateral System

Unlike the majority of conventional construction, there are no columns in this building. The three concrete cores carry the entire gravity load of each floor. Each core is a hollow 25'x25' square tube punctuated by irregular openings to accommodate the doorways in the floor plan, as seen in Figure 7. The core walls are 8 inches thick and minimally reinforced according to ACI 318-05.

At each level where a floor is to be located, each corner of every core has an indentation. Once a floor is lifted to its target height, a wide-flange is inserted through a cutout in the girder to rest on the corner indentation, as demonstrated in Figure 8. This wide-flange detail takes all of the floor loads and transfers them into the cores.

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Figure 7 - Observe the multitude of openings in the concrete cores

The concrete cores are also the building's sole lateral force resisting system. Each core functions as a set of shear walls that resist the lateral forces transferred by the floor diaphragm. On each floor, the individual shear walls are connected by 2 foot coupling beams above the openings for the corridor and doors. Seismic forces do not govern this building's design because of the location in Tempe, AZ and the soil content of the site.

The design gravity loads for this building are summarized in Table 1 below. The design lateral loads can be found in Technical Report 1 on the CPEP website listed on the Thesis Abstract.

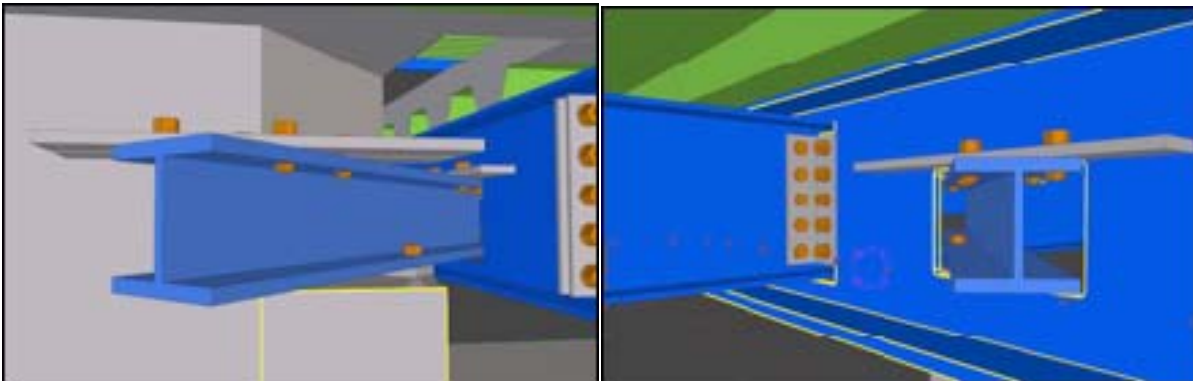


Figure 8 - Core corner detail (floor connection to concrete core)

Table 1 - Design gravity loads for original building

Load Type	Load Value (psf)
Construction Dead Load	59
Superimposed Dead Load	15
Live Load	80
Façade Load	15
Snow Load	0

Roof System

The roof system is a simple, long-lasting construction of the typical floor framing described in the Floor System section of this report, and an Ethylene Propylene Diene Terpolymer (EPDM) membrane on top. There is no mechanical equipment on the roof- the major elements of the mechanical system will be located on the ground floor, and will serve each unit in the building via a 2-pipe system.

Proposal Information and Thesis Scope of Work

Problem Statement

The design of the Southwest Student Housing building is very simple, but elegant. The balance of cost vs. speed of construction has been finely tuned, and the structural design is above adequate for the design loads in its location in Tempe, Arizona. Modularity is a key concept within the design. It is used to accelerate the construction process and have each floor equipped with façade walls and MEP systems within the 4 days it takes to pour and finish each floor. The floor system is a reasonable solution for the construction method when compared to other alternatives. Little can be altered in this design that could allow for faster construction or lower construction costs.

An important item to consider is the applicability of this design to other areas of the United States, such as areas with high seismic activity. This design is intended for construction in a wide variety of locations and would benefit from refinement to make it suited to high seismicity areas. Therefore, it is highly pertinent to investigate the functionality and alterations of this building design and construction method for an area such as St. Louis, Missouri. St. Louis was chosen to minimize the number of design parameters that influence the cost of construction. Appendix C shows a comparison of the costs of construction and the seismic design coefficients for several U.S. cities with high seismic activity. The cost of construction in St. Louis, MO is the closest to that of the existing location. In order to bring the building into SDC D, the site class will have to be altered from C to D. To accommodate these changes, the structural design would need to be reexamined (especially the floor-to-core connections), as would the cost for the new system design and any changes to the construction schedule. There is also a great potential need to alter the floor plans and modules to accommodate the structural design changes. The building envelope system might also need to be examined for ability to accommodate seismic drift, if time permits.

Additionally, the current building design is not LEED certified. Sustainability has been an important design aspect of many buildings in the twenty-first century, and should be considered with each new building design. As a result, it is crucial to consider what it would take for the building to achieve, at minimum, LEED certified status. More importantly, any changes or additions to the building design would need to be practical and appropriate for the occupancy, budget and location.

Proposed Solutions

If this building were relocated to and redesigned for St. Louis, MO with site class D, it would need to be designed as a building in Seismic Design Category D. To alter the design of this building for SDC D, the new seismic design loads would need to be calculated and compared to new wind design loads.

The concrete cores would have to increase in cross-sectional area and reinforcing. Potentially, a higher strength concrete might prove useful when changing building locations to SDC D. A careful review of the floor plans would be necessary, in order to change the floor plan to minimize openings in the core walls.

Additionally, the floor-to-core connections would need to be investigated and altered to satisfy special seismic provisions. If time permits, the cladding would need to be investigated and redesigned to accommodate seismic drifts and seismic design forces on nonstructural components.

The solutions to the problem statement would also require analysis and design in several breadths for thoroughness, and to ensure that these redesigns can truly be compared to the original design of the Southwest Student Housing building.

Breadth Studies

To truly be able to compare the original design to the design in an area of high seismicity, it is necessary to take an in-depth look at the construction costs and schedules. The impact of altering the design for high seismic lateral loads would be great: the cost for construction would increase significantly, due in part to increased material cost. It would be necessary to analyze the area prices for different strengths of concrete and compare the costs to the current building design. It would also be important to consider the schedule, which could potentially be prolonged, resulting in the owner (Arizona State University) losing potential profits from opening the building earlier. An analysis of the profit change due to schedule change would also be necessary.

Additionally, the floor plans and modules in the building would need to be redesigned to accommodate the alterations to the structure. Potential streamlining of the module design might accelerate the construction schedule and provide a greater profit that would need to be considered in cost and schedule evaluations. The module design might also have separate changes relating back to initial manufacturing costs, which should be examined if time permits. Additionally, it would also be beneficial to examine the potential for a module design that is applicable to this type of building design in both SDC B and SDC D.

As a result of the need for in-depth cost and schedule evaluation, one of the breadth studies can be classified as a Construction Management breadth. The other breadth study would be with regards to Architecture, and how the floor plan and module design would need to be modified as a result of the changes to the structural system. The Architecture breadth would involve module design and floor plan design, including an evaluation of the locations of openings, stairs and elevators.

A third breadth study to carry out will center on Sustainability. This breadth will require evaluation of LEED points throughout the building, as well as analysis of potential changes that can be made to bring the building to LEED certified status. Ultimately, if LEED certified status can be achieved, a cost and schedule evaluation will follow to gauge the impact of expanding the sustainability of the building design.

Structural Investigations (Depth Studies)

Design Codes, References and Standards

Model Code:

International Building Code, 2006 Edition

Design Codes:

American Institute of Steel Construction "Specifications for Structural Steel Buildings", AISC 360-05 (13th ed.) and AISC 360-10 (14th ed.)

American Concrete Institute "Building Code Requirements for Structural Concrete", ACI 318-05

Structural Standards:

American Society of Civil Engineers "Minimum Design Loads for Buildings and other Structures", ASCE7-05

Deflection Criteria:

Limit Unfactored Live Load deflections to $L/360$ or less

Limit Total (Service) Load deflections to $L/240$ or less

Limit building drift to $h/400$ or less

Materials

Structural Steel:

- All Rolled Shapes – ASTM A992 Grade 50
- All Plates and Connection Material – ASTM A36
- All Tubular Sections – ASTM A500 Grade B
- All Pipe Sections – ASTM A53 Grade B
- Anchor Rods – ASTM F1554

Cast-in-Place Concrete:

- Foundations – 4000 psi normal weight
- Slab on Grade – 4000 psi normal weight
- Structural Slab on Grade – 5000 psi normal weight
- Lightweight Concrete – 4000 psi
- Walls (core) – 4000 – 5000 psi

Reinforcement:

- Deformed Bars – ASTM A615 Grade 60 typ.
- Welded Wire Fabric – ASTM A195

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Welding Electrodes:

- E70xx Low Hydrogen

Bolting Materials:

ASTM 325 or A490

St Louis Site Information

The choice of St Louis, MO as the new site for this building was very important. The goal of this location choice was to create a loading situation where the building would experience strong seismic loads (Seismic Design Category D) without having the building undergo loads that are considered extreme, such as those in Alaska or California. Putting the building site in St Louis brought the overall seismic design category of the design loads to Category C; changing the soil type to Site Class D brings the seismic design category to Category D.

Table 2 - Seismic design forces for different locations and wall thicknesses

Floor #	Seismic Design Forces					
	Tempe, AZ; 8" walls		St Louis, MO; 8" walls		St Louis, MO; 16" walls	
	fi (k)	Mz (k-ft)	fi (k)	Mz (k-ft)	fi (k)	Mz (k-ft)
20	29.5	368.4	68.5	856.8	81.0	1011.9
19	27.0	705.9	90.2	1984.5	111.9	2410.3
18	24.6	1013.6	83.6	3029.0	103.2	3699.8
17	22.3	1292.9	77.1	3992.4	94.7	4883.7
16	20.2	1544.9	70.8	4876.9	86.5	5965.2
15	18.1	1770.8	64.6	5684.6	78.6	6947.7
14	16.1	1972.0	58.7	6417.9	71.0	7834.7
13	14.2	2149.6	52.9	7079.0	63.6	8629.8
12	12.4	2304.9	47.3	7670.6	56.5	9336.6
11	10.8	2439.4	42.0	8195.1	49.8	9959.0
10	9.2	2554.2	36.8	8655.2	43.4	10500.9
9	7.7	2650.8	31.9	9053.8	37.2	10966.4
8	6.4	2730.6	27.2	9393.7	31.5	11359.8
7	5.2	2795.0	22.7	9678.0	26.1	11685.5
6	4.0	2845.5	18.6	9910.1	21.0	11948.4
5	3.0	2883.5	14.7	10093.6	16.4	12153.3
4	2.2	2910.7	11.1	10232.3	12.2	12305.8
3	1.4	2928.6	7.9	10330.4	8.5	12411.6
2	0.8	2939.0	5.0	10392.9	5.2	12477.2
1	0.4	2943.8	2.9	10428.8	3.2	12517.5

Design Loads:

The transition to a site in St. Louis, Missouri creates alternate design loads for the building. The floor gravity loads and wind loads do not change from the loads in Tempe, AZ, but the Snow and Earthquake loads change significantly. The new seismic lateral loads increase by about 400% overall. In Table 2, the seismic loads for the original

site can be compared to the new seismic loads for walls the same thickness, as well as walls twice the thickness. Related calculations, design coefficients and design factors can be found in Appendix B. Table 3 shows the loads used to design and model the building under the site conditions in St Louis, MO.

Table 3 - Loads used to design and model the building in St Louis, MO

Gravity Loads			
Construction Dead Load		59	psf
Superimposed Dead Load		15	psf
Live Load		80	psf
Façade Load		15	psf
Snow Load		20	psf
Base Shear		1001.4	psf
Model Masses			
Roof	1.725 E-06	k-s2/in2	
Typical Floor	2.574E-06	k-s2/in2	
First Floor	3.158E-06	k-s2/in2	

Core Design

The redesign of the concrete cores began with choosing the wall thickness for the new seismic base shear. The first portion of the core design calculations can be found on page 4 of the Core Design section of the Structural Investigations Appendix (Appendix C). The initial sizing for the walls was based off of the assumption that the openings from the original design are no longer present, and that the cores will only have openings for the 6-foot corridor running through the middle of the floor plan.

Initial hand calculations for a base shear of 1001 kips yielded a need for a wall thickness of at least 9.3 inches, if 4000 psi concrete is used in St Louis. For an initial trial size of 16-inch walls, the building period was found to be 2.17 seconds when the building was modeled in ETABS using rigid diaphragms and cracked sections (0.5 section multiplier for walls, 0.25 for coupling beams).

The cores are in compression even when the building is subjected to 120% (accounting for torsion) of the overturning moment created by the seismic design base shear. This compression is due to the overall building self-weight counteracting any tensile force the building experiences. As a result of the entire building cross-section experiencing only compressive stress, only the minimum moment reinforcement for SDC D is necessary in the walls, while adhering to Chapter 21 of ACI 318-05. The choice was made to also include boundary elements to maintain structural integrity under seismic lateral loads when also holding gravity loads. When running through hand calculations, the boundary elements were seen to function essentially as columns. Overall, the maximum compressive stress in the concrete under loading was found to be 0.253f_c, with 120% of the design overturning moment applied to

Table 4 - Modes and deflections of original building and new building under St. Louis seismic forces

Design	Original	New Design
Max Deflection	16.202	6.126
Min Deflection	-4.296	-0.888
Mode 1	3.943	2.167
Mode 2	3.521	2.025
Mode 3	3.319	1.797

account for torsion.

Ultimately, the cores were designed to have 16" walls (still 25 feet long on center), a concrete strength of 4000 psi, reinforced with #4 bars spaced at 6 inches on center. The boundary elements extend 118 feet up, and are discontinued when the axial stress of the concrete is under 0.15f_c. Figure 9 shows a drawing of the core layout while Figure 10 shows the boundary reinforcement details. Table 4 shows a comparison of periods and deflections of the original design and the new design.

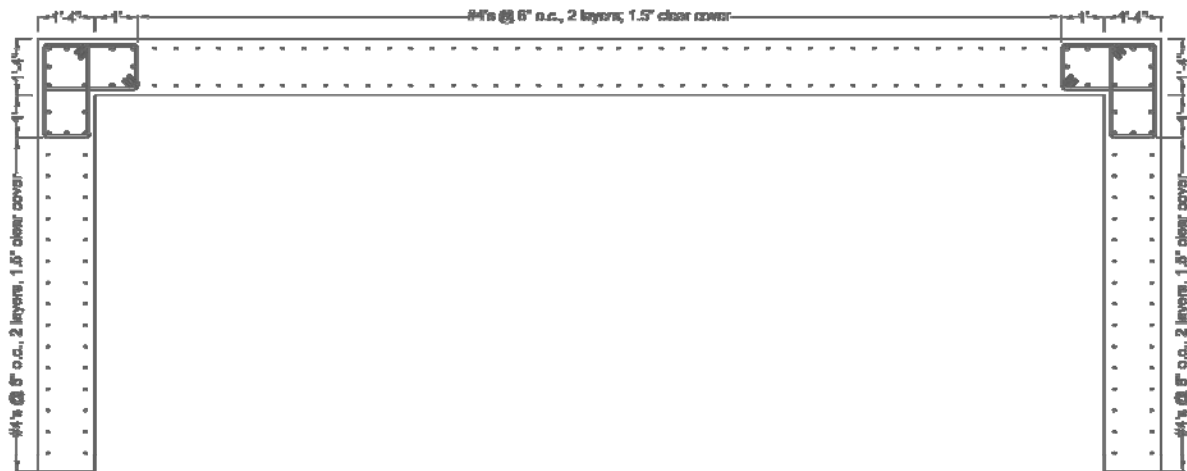


Figure 9 - Core wall reinforcement detailing

Once the core walls were designed and modeled, the coupling beam reinforcement was designed. The shear at the end of a coupling beam was found at each level for 120% of the seismic design load; the values can be found on page 9 of the core design calculations in Appendix C. The maximum shear was found to be about 131 kips. To design the reinforcement for the coupling beams, 80% of the maximum shear was taken to design the moment reinforcement, then divided by 0.9 to change the Φ factor to 1.0 (because of the equation used- see page 9 of the core design calculations), then multiplied by 1.25 to account for potential reinforcement yield over strength (see Figure R21.3.4, note 2 of ACI 318-05). This value was used to compute the necessary longitudinal reinforcement areas; shear reinforcement areas were computed using the assumption that the concrete will take no shear. Figure 11 shows the final coupling beam redesign. Diagonal reinforcement was deemed unnecessary because the aspect ratio of the coupling beam was under the ACI 318-05 limit of 4. Clear cover for all concrete was assumed to be 1.5 inches or more.

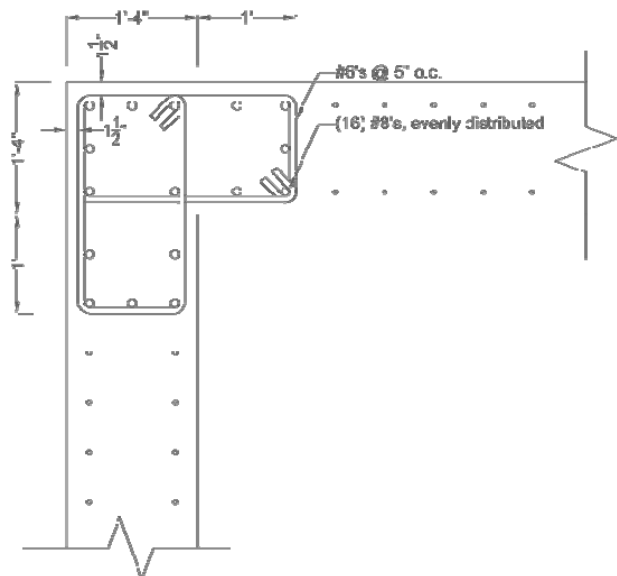


Figure 10 - Boundary element reinforcement corner detail

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In addition to this design, the primary design (or, "New Design 1"), the *Effect on Architecture* section describes another core design that was briefly investigated as a secondary option to better accommodate a potential new and more modular floor plan. As such, the *Effect on Architecture* section also includes an additional table of periods and deflections to compare the original and both new designs.

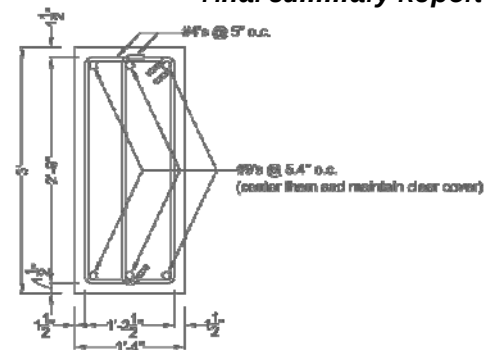


Figure 11 - Coupling beam reinforcement detail

Floor System Design

The floor system itself was not redesigned. The reason for engaging in a floor system design was the need to better transfer lateral forces through the floor diaphragm and into the concrete cores to satisfy special seismic design requirements. The simple corner supports designed by the Thornton-Termohlen Group would be insufficient for Seismic Design Category D. There were many iterations of design to create a reliable solution for the attachment of the floor system to the core- these can be found in Appendix D. Ultimately, two solutions were chosen as most pragmatic for maintaining the current construction method and minimizing any interference with the construction schedule. Figures 12 and 13 on the next two pages show these two designs.

Steel Collar Design

Figure 12 shows the *Steel Collar* design, which consists of a 3 foot deep steel plate embedded in the concrete core at the level of each coupling beam. The steel plate is attached to the core via a multitude of shear connector studs (designed to withstand the gravity loads from the floors, as well as the tension produced by the moment applied by the cantilevered portions of each floor).

Once a floor is lifted into place along the steel plate, corbels will be welded close to the corners of the core walls, under the main girders running lengthwise through the building. The floor would then be rested on these corbels (designed only for gravity loads) while the more permanent connection of the floor slab to the core would be made through the concrete topping. In order to make this connection, the concrete would need to be stopped about a foot from the core initially, before the floor is lifted into place. Once the floor is lifted, shear studs can be welded to the steel plate, in the middle of the concrete topping. Finally, additional concrete will be hand-poured to fill in the foot-wide gap that was initially left free of concrete.

04.04.2012

Ksenia Tretiakova, Structural Option
AE Consultant: Dr. Andres Lepage

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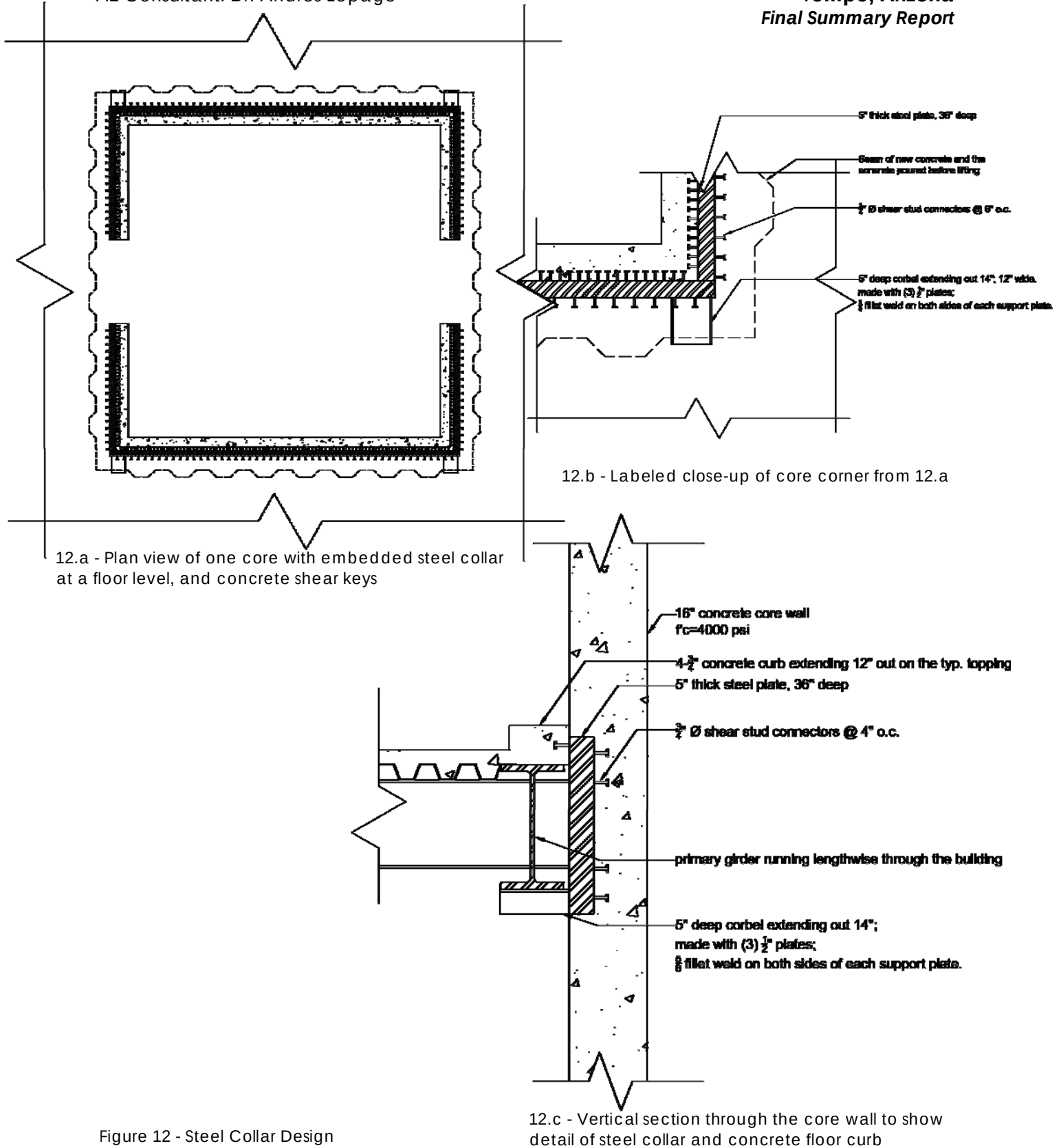


Figure 12 - Steel Collar Design

04.04.2012

Ksenia Tretiakova, Structural Option
AE Consultant: Dr. Andres Lepage

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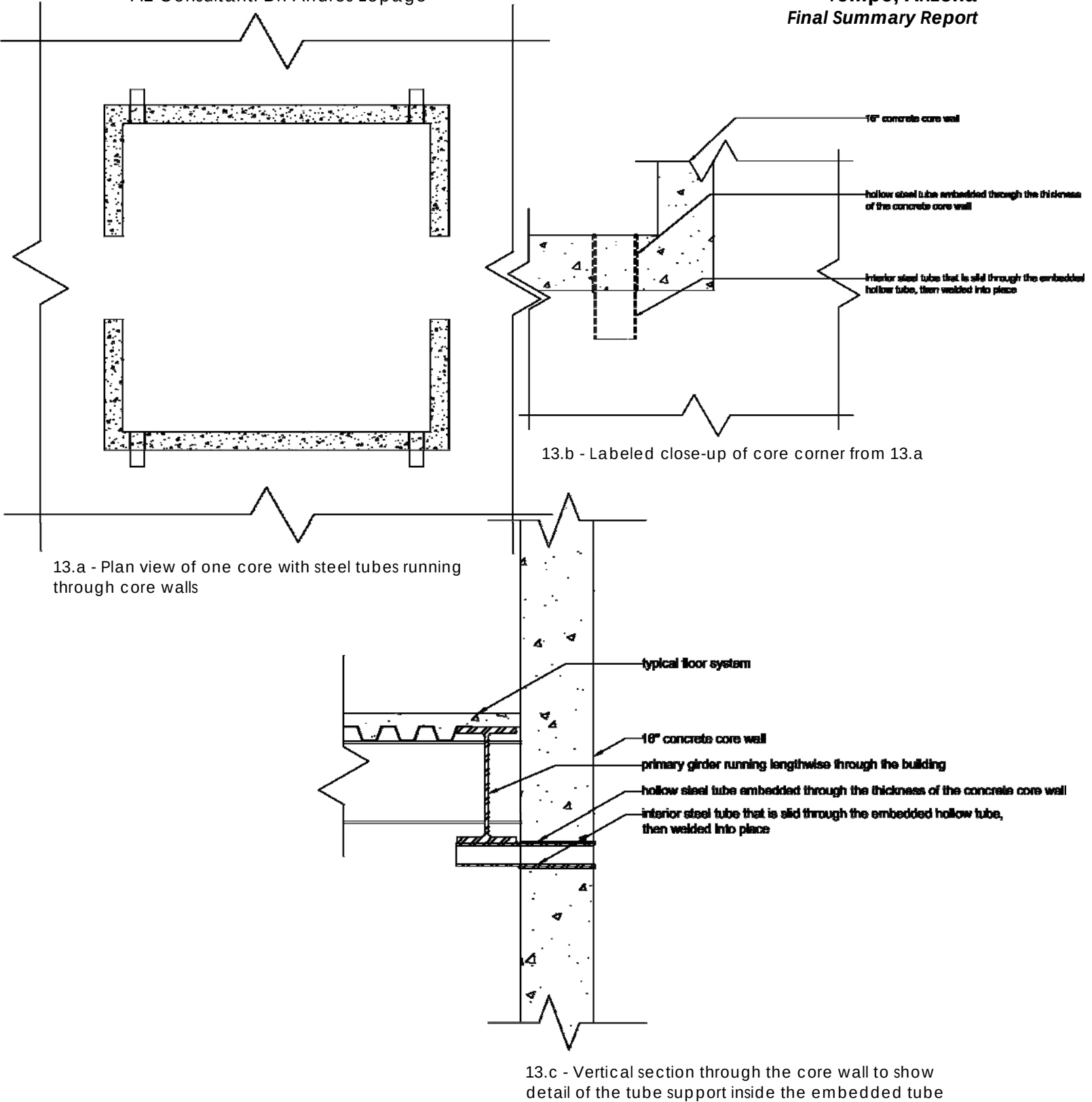


Figure 13 - Drag Strut Design

The shear studs and concrete topping were checked using IBC 2006 and PCI handbook, with spot checks from Appendix D of ACI 318-05 for concrete breakout strength in tension and shear, as well as concrete pryout strength. These calculations can be found on pages 14 to 17 of the floor system design section of the calculations in Appendix D. It was found that the original design's 3-1/4" concrete topping would be insufficient for withstanding the maximum floor diaphragm shear (found at the 17th floor to be 179 kips) multiplied by the overstrength factor for special reinforced concrete shear walls (2.5) found in Table 12.2-1 of ASCE 7-05. The Steel Collar design would require the presence of a curb around the core where the concrete topping was 8 inches thick for a foot past the core walls on all sides, as shown in Figure 12.

Drag Strut Design

Figure 13 shows an alternate design that was explored toward the end of the work and research for this report. This design would take advantage of the presence of continuous beams running through the main girders and running along the core walls by using them as collector elements for the diaphragm forces through the use of shear studs and composite action, as well as (potentially) additional reinforcement in the slab. This design could be constructed by installing hollow tubes running through the concrete cores. Once a floor is lifted into place, another tube could be inserted under the primary girders to act as corbels (similar to the Steel Collar design), and then welded into place at both faces of the core walls. To transfer the diaphragm forces, the deck would need about a shear stud per foot of beam length-- each collector element would have to take about 75 kips of shear.

Additional investigation should be taken into this design. The collector elements would need to be designed as beam-column elements because of the axial load they would experience as a result of acting as a collector element. The welds for the tubes would also need to be carefully designed, especially since they would experience a unique and unusual loading condition due to the seismic forces acting in torsion, horizontally and also vertically when the building is in motion.

Structural Investigations Summary

In order to accommodate the loads for Seismic Design Category D, the concrete volume for this building would need to be doubled from its original design. The original design contains about 8 cubic yards of concrete per foot of height while the new design contains almost 17. The steel detailing is considerably greater as a result of the use of boundary elements and heavily reinforced coupling beams to withstand the design seismic shear forces. The material takeoffs and comparison of these construction materials can be found in the *Effect on Cost and Scheduling* section of this report. Additionally, the key detail of the attachment of the floor diaphragm to the core is very elaborate, instead of a simple connection like the original Thornton-Termohlen Group design. This building design and construction method can still potentially be used in areas that require special seismic designs, but would require additional insight through peer review. There was a key assumption made initially that ultimately was not met by the design: that there would be no extreme torsional irregularity (to meet height limit exception 12.2.5.4 of ASCE 7-05). Unfortunately, due to the long, thin shape of this

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building's footprint, it experiences considerable torsional irregularity. With the current design, the torsional amplification factor is about 2.1 when using the deflections computed by ETABS with 120% of the design base shear. Even if the wall thickness is increased to 24 inches, and a high strength concrete (8000 psi) is used, the torsional amplification factor is about 1.7. The geometry of the core walls would need to be greatly altered to meet the height limit exception in section 12.2.5.4 of ASCE 7-05 (which allows special reinforced concrete shear wall structures greater than 100 feet tall), thus sparking the need for additional peer review of the design.

Effect on Architecture (Breadth Study)

The structural investigations carried out on this building yielded architectural repercussions. The main impact of the alterations to the core wall design was the need to eliminate all openings in the cores, excluding the corridor. This led to the typical floor plan for the building to be rearranged. Options 1 and 2 offer different floor plans that accommodate the new design requirements. Appendix E contains the notes taken while working through the Architectural breadth study of this report.

Option 1

Option 1 was the first floor plan layout that manifested as a result of the new seismic design. The need for the cores to be closed off completely on two sides brought out the necessity to cut the central corridor short in order to still provide tenants access to the inside of the two outer cores. Figure 14 shows the Option 1 floor plan, including the layout of the bedrooms and prefabricated bathrooms. Table 5 in the *Summary* subsection of this section shows the area comparisons between the original floor plan and both new floor plans that accommodate the seismic design.

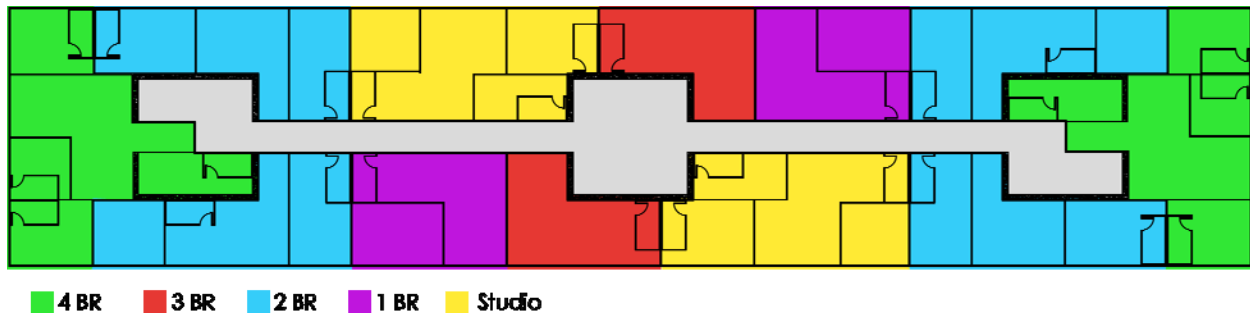


Figure 14 - Option 1 floor plan

Option 2

The second option was brought about after considering the theme of "modularity" that echoes throughout this entire project. Option 1 presents some difficulty with retaining modularity as a result of the shape of some of the apartments. The position of the bathrooms also deviates from the original design, which focused on keeping the bathrooms against the wall of the central corridor, or against the core walls as much as possible. Presumably, this placement pattern was to ensure ease of plumbing the building. Thus, the second option tries to emulate more of the patterns observed in the original floor plan design by maintaining more modular apartment boundaries and placing the bathrooms against the corridor as much as possible. This option also takes an idea from Option 1 of cutting off the ends of the central corridor in order to create better access to the cores.

Another very important factor of Option 2 (see Figure 15) is the clearly different core wall shape: instead of having continuous walls in the long direction and walls cut into two by the corridor in the short direction, each core is symmetric, so the building

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would be supported by cores that have mainly corner support. Making symmetric cores allows for better prediction of shear wall performance, as well as more stable stress distribution throughout each wall. Having the openings all in the central portion of the core walls also allows for the boundary elements in the design, which were not possible if the layout of the openings from the original design was used.



Figure 15 - Option 2 floor plan

Architectural Summary

Table 5 - Area comparisons for all options; % change is the % of the original unit size

Room Type	Original Design	Option 1	% Change	Option 2	% Change
Studio	431	541	26	431	0
1 Bedroom	541	719	33	719	33
2 Bedroom	A 913	877	-1	938	3
	B 863			858	-1
3 Bedroom	1150	1091	-5	1060	-8
4 Bedroom	1438	1396	-3	1488	3

Two alternate floor plan options were presented in this section of the Final Summary Report on the Southwest Student Housing building. These floor plans both accommodate the new seismic design

Table 6 - Modes and deflections for the original design and both redesign options

Design	Original	Option 1	Option 2
Max Deflection	16.20	6.13	9.74
Min Deflection	-4.30	-0.89	-2.55
Mode 1	3.94	2.17	2.78
Mode 2	3.52	2.03	2.49
Mode 3	3.32	1.80	2.33

described in the *Structural Investigations* section of this report. Option 1 allows for the use of continuous shear walls in the long direction of the building, ensuring the structural stability of the building in that direction against lateral forces. The wall continuity in Option 1 is also useful for limiting the already extreme torsional irregularity that this building experiences, while Option 2 propagates the aforementioned irregularity and increases the building's period further. Table 6 shows a comparison of the building modes and drifts for both options, as well as for the original design.

In order to attach the floor system to the walls in Option 2, the current Steel Collar design would need to be revisited to re-portion the steel plates to only the corners,

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which has the potential to cause some problems with the shear stud layout for the embedment of the plate in the core.

On the other hand, Option 2 allows for easier separation of each floor into modules that can be shipped in separately and slid into place before each floor is lifted, allowing for slightly greater ease of construction. Figure 16 shows the basic blocking of potential modules on the floor plans for both new designs as well as the original design. The areas of every unit in both options are never reduced by more than 8% of each original unit's area. In some cases, the apartment units even experience a great increase in area, up to 33% of the original area for that unit type! This increase in area is made possible by the removal of a portion of the corridor that is a little over 70 feet long, total (around 35 feet off per side).

Overall, the floor plan option should be chosen after a peer evaluation is conducted on the structural design. The peer evaluation can shed light on what direction to go in terms of the structural design. Additionally, Figure 17 shows a core layout that would be more versatile for floor plans, making it worthwhile to investigate further.

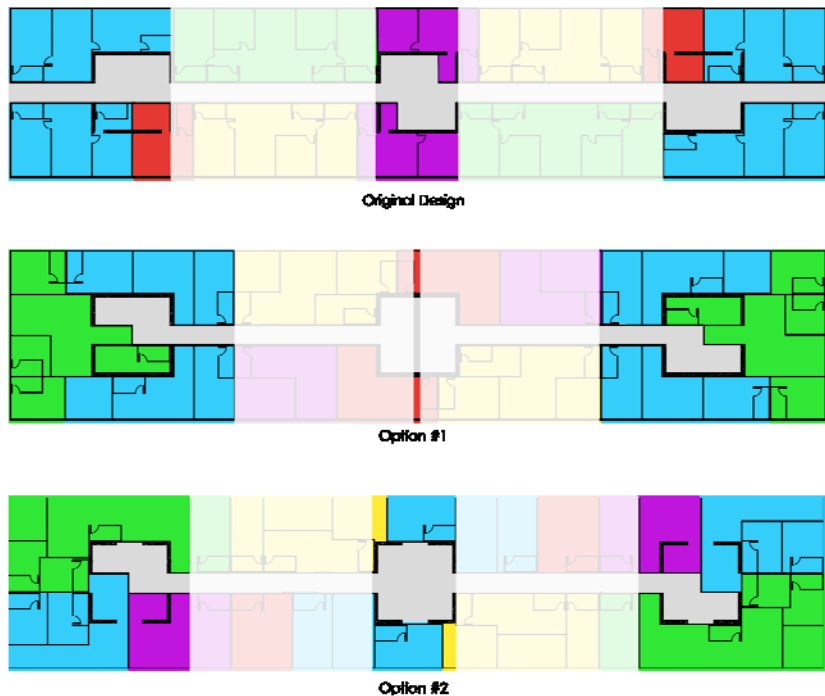


Figure 16 - Potential module breakup for each design: each change of gradient (from vivid to light to vivid) indicates a potential module

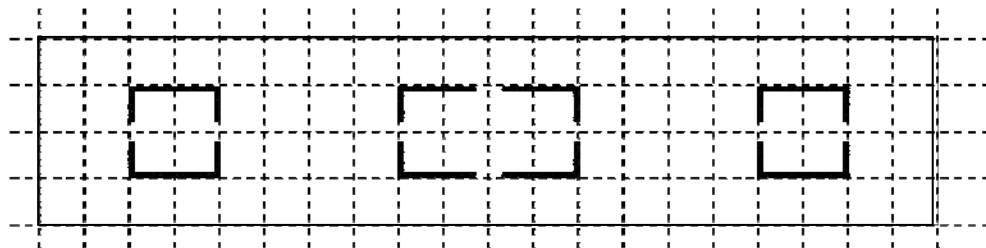


Figure 17 - Potential core layout worth investigating: central core is extended out

Effect on Cost and Scheduling (Breadth Study)

Construction Costs

The transition from Seismic Design Category B to D is generally an expensive one for any construction project. The Southwest Student Housing building is no different. As mentioned in the *Structural Investigations* section of this report, the concrete volume more than doubled as a result of this transition. The quantity of reinforcement greatly increased due to increased minimum reinforcement ratio requirements by ACI 318-05, as well as the addition of boundary elements and beefed up coupling beam reinforcement. In addition to these volume increases, the material cost for added features such as the steel collar plates, shear stud connectors, welds and corbels must be taken into account. The overall building cost was not re-evaluated for each design because the only changes were made to the structural system, which allowed for a narrow scope of investigation.

Table 7 shows the general bare material costs for the main changes to the structural system-- the concrete, the steel reinforcement, the steel plates, the shear stud connectors, and the welds. The two designs in Table 7 refer to Options 1 and 2 of the core layout that were mentioned in the *Effect on Architecture* section of this report.

More detailed tables of cost analysis, including base prices per unit and the material takeoff calculations, can be found in Appendix F, along with additional calculations and notes related to this section. All of these costs were based on Tempe, AZ prices in order to allow for direct comparisons. The costs displayed are bare material costs only, and do not take into account labor. Labor costs and additional assumptions are noted in Appendix F, but not considered in this analysis because of the great deviation in labor costs from area to area.

The steel plates used for the Steel Collar design were estimated using the weight of steel, with the material cost being taken from the Steel Projects section of RS Means. The line numbers for each item are available in the overall material costs table in Appendix F. All costs took into account 10% added waste for each material.

Ultimately, the adjustments to the building design to accommodate a change from Seismic Design Category B to D would incur a material cost increase of approximately \$3 million, which is about 8% of the total building cost, not taking into account any additional labor costs or schedule increases.

Table 7 - General bare material costs for primary changes to structural system

Bare Material Costs			
Item	Original Design	Option 1	Option 2
Concrete	113372.92	247340.18	216552.68
Reinforcement	74384.57	381027.09	432257.53
Welds	0.00	1080.00	1080.00
Shear Studs	0.00	70553.60	6364.80
Other Steel	0.00	2069809.20	2069809.20
Total	187757.49	2769810.07	2726064.21
Difference from Original	0.00	2582052.59	2538306.72

Construction Schedule

Several key assumptions were made before evaluating this project's schedule: the first was that everything would be assembled at the same rate for both projects. The second was that all of the added steel parts for the floor system connections would not take additional time (justification for this is provided below). The final assumption was that the welders could work easily at a similar productivity level as what was listed in RS Means while suspended in the air under a lifted floor slab.

The assumption that all the added steel parts for the floor system connections would take no additional time to assemble comes from the construction methodology. Slip-formed concrete walls have to have the reinforcement pre-assembled above the slip-form as it rises, so that the concrete pour rate does not depend on how much of the steel is assembled. The steel plates and corbels for the collar design would have to come prefabricated. As a result of these items coming prefabricated, the main labor for them would be the effort it takes to use a crane to lift each plate into place. While this job is not easy, it is certainly doable; especially when the concrete walls would be rising at half the speed of the original design.

The third assumption, that welders can work easily at a similar productivity level to what is listed in RS Means while suspended under the lifted floor slab, is necessary to be able to quantify the length of time it would take for a single team to complete any of the welds necessary for the floor system-to-core connections.

Overall, the schedule shouldn't be altered greatly as a result of the switch to SDC D. Since the concrete volume is about doubled, it would take double the time to slipform the cores: initially, it takes 10 days to slipform all of the cores up the full building height. Consequently, it would take 20 days to slipform the newer, thicker cores up to the full building height.

Beyond that, a single welding team would have two days to weld the corbels for half a floor. Each floor contains about 20 feet of 5/8" continuous fillet weld, so half a floor would contain 10 feet of weld. A single welding team can complete 12 feet of 3/4" welds per day, and it takes 2 days to prepare a half-floor to be lifted. Taking all of this into account results in the conclusion that a single welding team would have 2 days to complete a welding task that should take no longer than 1 day according to RS Means - a reasonable amount of time, even when taking into account the potential for any sort of delays due to the unusual circumstances of where the welding would take place.

Apart from the doubled concrete volume, the only other item that would have a true effect on the construction schedule would be the installation of shear studs and the final pour of the concrete curb around the core in order to create a strong connection from the floor diaphragm to the core. There is also a potential that the schedule would not be altered at all, because a team could start pouring the concrete as all the other floors are lifted into place. This would result in the completion of all of the connections at most a day or two after all of the floors are lifted into place, which is when the upper floors are being finished (this process takes 12 days per floor according to the Thornton-Termohlen Group). A conclusive answer to this would need to be investigated further with more information from the original design team about how the current construction schedule is split up.

Evaluation of Sustainability (Breadth Study)

The goal of the sustainability study of this building was to evaluate the current status of the building in terms of LEED ratings. Additionally, this study was meant to find a cost effective way to bring the building up to LEED Certified status, with minimal effort on the part of the owner and contractor. This study demonstrates that LEED Certified can be attainable by almost any project, as long as the owner and contractors dedicate more time to planning before starting the construction portion of the project.

This section discusses the LEED point breakdown by the groups defined in the New Construction rating system found on the United States Green Building Council (USGBC) website. The credits are listed and tallied by grouping under each subsection.

Ultimately, the current design has 20 LEED points and the potential to get up to 41 points total, yielding a LEED Certification for the building.

Sustainable Sites

Original Design

The original design possesses 12 of the Sustainable Sites credits, as seen in Table 8 below. These credits are primarily in relation to the urbanized area, proximity to bus stops, and previously developed site.

New Design

An additional 7 credits can be obtained with some planning:

- 1 credit for having covered storage facilities for 15% of the residents

A cost analysis for this credit can be found on page 2 of the Sustainability notes in Appendix G; estimated at approximately \$70/sf, with an assumption that it would require about 400 sf to complete this requirement, which totals to a cost of about \$28,000.

- 2 credits for providing no new parking

Currently, the presence of resident parking on the first level of the building is still up in the air. Planning on no parking costs nothing and achieves 2 credits.

- 1 credit for restoring or protecting a certain portion of the site

The site will require landscaping anyway. The cost was estimated by making an assumption about the cost/sf via estimating the shaded area provided by one full-grown tree; Calculations can be found in Appendix G. Overall estimated cost for this credit is \$170,000.

- 1 credit for shading the same portion of the site

See above credit for restoring or protecting the site, these two credits go hand in hand and would share the cost overall.

1 credit for changing the roof EPDM color to white-on-black instead of conventional black (no cost)

	Currently has	Can plan to attain
SS Prerequisite 1 (Points - 0): Erosion and Sedimentation Control	0	
SS Credit 1 (Points - 5): Site Selection	1	
SS Credit 2 (Points - 5): Development Density and Community Connectivity	5	
SS Credit 3 (Points - 1): Brownfield Redevelopment		
SS Credit 4 (Points - 12): Alternative Transportation	6	3
SS Credit 5 (Points - 2): Site Development-- Protect or Restore Habitat		1
SS Credit 6 (Points - 2): Stormwater Management		2
SS Credit 7 (Points - 2): Heat Island Effect		2
SS Credit 8 (Points - 1): Light Pollution Reduction		

Water Efficiency

The original design should be able to meet the prerequisites because of the vast availability of water-efficient fixtures. Other than that, the other credits in this section would cost a significant amount of money to obtain because the efficiency of the most readily available fixtures is never more than about 20% efficient, according to various major plumbing manufacturing sites.

	Currently has	Can plan to attain
WE Prerequisite (Points - 0): Water Use Reduction		0
WE Credit 1 (Points - 2 to 4): Water Efficient Landscaping		
WE Credit 2 (Points - 2): Innovative Wastewater Technologies		
WE Credit 3 (Points - 2 to 4): Water Use Reduction		

Energy and Atmosphere

This section was not evaluated because the designers provided no information on whether or not the building project met the prerequisites for the Energy and Atmosphere section. If the prerequisites are not met, the cost of the credits for this section rise rapidly, which is against the goal of the Sustainability study carried out in this report.

Materials and Resources

Original Design

The original design possesses only 2 of the Materials and Resources credits. These credits relate to the use of materials from within 500 miles of the site. The concrete for the project would be local, from very close to the Tempe, AZ city premises, if not from inside the city itself. As a result, with the large volume of concrete on this project, the Regional Materials credit is met very easily.

New Design

An additional 7 credits can be obtained with some planning:

1 credit for having a waste management plan before starting construction

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ASU already advocates for waste management and recycling on construction projects, and would provide bins to make it easier. Should not cost anything.

2 credits for using 10% by cost salvaged, refurbished or reused materials

Can plan to use salvaged finishing items, including doors, frames, casework, maybe even flooring; Should not incur additional cost.

2 credits for using 20% recycled materials, by cost

Use of recycled steel and concrete aggregate, as well as salvaged goods for furnishings, casework, finishes and flooring, even partition walls, would easily achieve these credits. Should not incur additional cost.

1 credit for use of rapidly renewable materials for 2.5% of total cost

Whatever flooring, finishes or casework present can be both salvaged and rapidly renewable resources (such as bamboo or linoleum); formwork can also be counted, if necessary. There might be costs incurred due to the specificity, but it is a cost that would be difficult to estimate.

1 credit for the use of certified wood

Can be achieved by simply following the previous credit- using certified and recycled/salvaged wood for flooring, casework, doors, frames and so on. Should cost nothing.

	Currently has	Can plan to attain
MR Prerequisite 1 (Points - 0): Storage and Collection of Recyclables	0	
MR Credit 1 (Points - 1 to 4): Building Reuse--Maintain Existing Walls, Floors and Roof		
MR Credit 2 (Points - 2): Construction Waste Management		1
MR Credit 3 (Points - 1 to 2): Materials Reuse		2
MR Credit 4 (Points - 1 to 2): Recycled Content		2
MR Credit 5 (Points - 2): Regional Materials	2	
MR Credit 6 (Points - 1): Rapidly Renewable Materials		1
MR Credit 7 (Points - 1): Certified Wood		1

Indoor Environment Quality*Original Design*

The original design can easily meet the prerequisites for this group of credits simply by following the ASHRAE design standards. The original design has 6 of the credits in this group. These credits come mostly from the fact that every apartment unit will have thermostats and a large quantity of daylighting due to the building's position on site. The IAQ management plan is a simple credit to obtain (mainly paperwork), and the thermal comfort credit involves simply meeting an ASHRAE standard for thermal comfort in the building.

New Design

An additional 5 credits can be obtained with some planning:

4 credits can be obtained by using low-emitting materials

These credits simply take some research: in the preconstruction phase, a member of the construction team can look up materials such as adhesives and paints that meet the requirements of both the owner and

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the LEED credits. Money will already be spent to obtain these materials, so taking the time to plan ahead and be careful with the choices will cost nothing but time.

1 credit can be obtained by sending out a survey to the residents to evaluate whether or not they are thermally comfortable after the first 6 to 18 months of occupancy. Will come with a printing cost if paper surveys are sent out, or zero cost if the residents are sent emails.

	Currently has	Can plan to attain
IEQ Prerequisite 1 (Points - 0): Minimum IAQ Performance	0	
IEQ Prerequisite 2 (Points - 0): Environmental Tobacco Smoke (ETS) Control	0	
IEQ Credit 1 (Points - 1): Outdoor Air Delivery Monitoring		
IEQ Credit 2 (Points - 1): Increased Ventilation		
IEQ Credit 3 (Points - 2): Construction IAQ Management Plan	2	
IEQ Credit 4 (Points - 4): Low-Emitting Materials		4
IEQ Credit 5 (Points - 1): Indoor Chemical and Pollutant Source Control		
IEQ Credit 6 (Points - 2): Controllability of Systems	2	
IEQ Credit 7 (Points - 2): Thermal Comfort	1	1
IEQ Credit 8 (Points - 2): Daylighting and Views	1	

Innovation in Design*Original Design*

The original design has no credits in this group

New Design

An additional 1 credit can be obtained with some planning:

1 credits can be obtained by having a LEED Accredited Professional on staff for this project; Chances are that there is already a LEED AP on this project, but there are cost-free options in the case that a LEED AP is not present: the many consulting companies for this project include Limbach, WSP Flack & Kurtz and Thornton-Tomasetti; Among the many staff in these companies, there are many LEED APs available that could be brought in as part of the consulting group.

	Currently has	Can plan to attain
ID Credit 1 (Points - 1 to 5): Innovations in Design		
IAPP Credit 2 (Points - 1): LEED Accredited Professional		1

Summary and Conclusions

The transition from Seismic Design Category B to D increased the overall seismic design loads from about 235 kips to about 1000 kips-- an increase of approximately 325% of the original base shear. In order to resist this force, the concrete volume in the building had to be increased by 100%, while the reinforcement redesign yielded an overall volume increase of about 420%. The overall building cost (bare material only) increased by about 8% of the total cost. The schedule would increase minimally-- perhaps by 10 to 20 days more than the original 177-day construction schedule. As a result of the minimal schedule changes, the building revenue would be minimally affected.

Ultimately, designing for higher seismic loads always requires bulking up an existing system, which means an increase in material and cost. There is no way to avoid additional expenses and time delays when a project simply ends up having a more complex design. The Southwest Student Housing building has a relatively simple original design, with a much more complicated seismic design as a result of the floor system-to-core connections; this feature makes the SW Student Housing building a great example of the changes and complications involved with altering a building design to withstand higher seismic forces. A key point mentioned in the *Structural Investigations* portion of this report was the reoccurring issue with extreme torsional irregularity in the building. This issue would need to be addressed before this building design could be considered even remotely viable for SDC D design. Thus, the need for additional peer review is great.

Apart from looking at this design from a structural and construction management perspective, it is also important to examine it from an architectural perspective. The *Effect on Architecture* section covers the necessity to change the floor plan to accommodate the newly reshaped concrete cores of the structure. With the current typical building floor plan, there are about 70ft of corridor space that could be used as apartment space. If these 70ft of corridor space (approximately 420 sf) were to be used for apartment space, the building would be able to easily accommodate the need to have continuous shear walls running in the long direction. Additionally, the extra corridor space allows for an apartment unit to have up to a 33% increase of area compared to the original unit dimensions. The *Effect on Architecture* section also shows several potential floor plan layouts that incorporate the extra corridor space, yielding at most a decrease of 5% of the original unit area in any given apartment under the new arrangement.

As to the subject of sustainability, the *Evaluation of Sustainability* section demonstrates that, with some simple planning ahead in the preconstruction phase, the building design could be turned into a LEED Certified design at close to no additional cost.

Credits and Acknowledgements

This project was a long, tough ride that was made many times easier by the following individuals, who never ceased to provide support and assistance throughout the entire process:

A very huge thank you to Charlie Thornton for being so receptive and providing me with a building for this thesis project. This gratitude extends to the Thornton-Termohlen Group for providing the project, as well as to Anthony Kelly for his help with the architectural side of things.

Brian Howells, who took the time to speak to me about the parking technology that TTG will be incorporating into their buildings.

Thornton-Tomasetti, for hosting TTG in the Philadelphia office, where I had the opportunity to meet many important and great people in my industry. The meeting was a phenomenal and eye-opening experience that cemented my decisions about my future.

Dr. Andres Lepage, who would, without fail, always see me or talk to me on the phone in order to sort out my countless issues and mistakes. I'm sorry for using up 2 hours for every 1-hour meeting slot you gave me!

Dr. Thomas Boothby, who went out of his way to talk to me about my troubles and gave me some fantastic ideas.

Prof. Kevin Parfitt, who always cracked the whip to make sure I was still on time when I was working on my graduate research.

Dr. Memari, who is awesome.

I would also like to extend a very big thanks to Stephen Pfund for providing endless support and a sounding board for every crazy idea. Thank you, Jonathan Minnick, for helping to keep me sane during the final stretches of this project and always encouraging me to keep going. The hours you two have spent on the phone with me mean more than you'll ever know!

Additionally, I would also like to thank my mom for never doubting me, and always being proud to have me as a daughter.

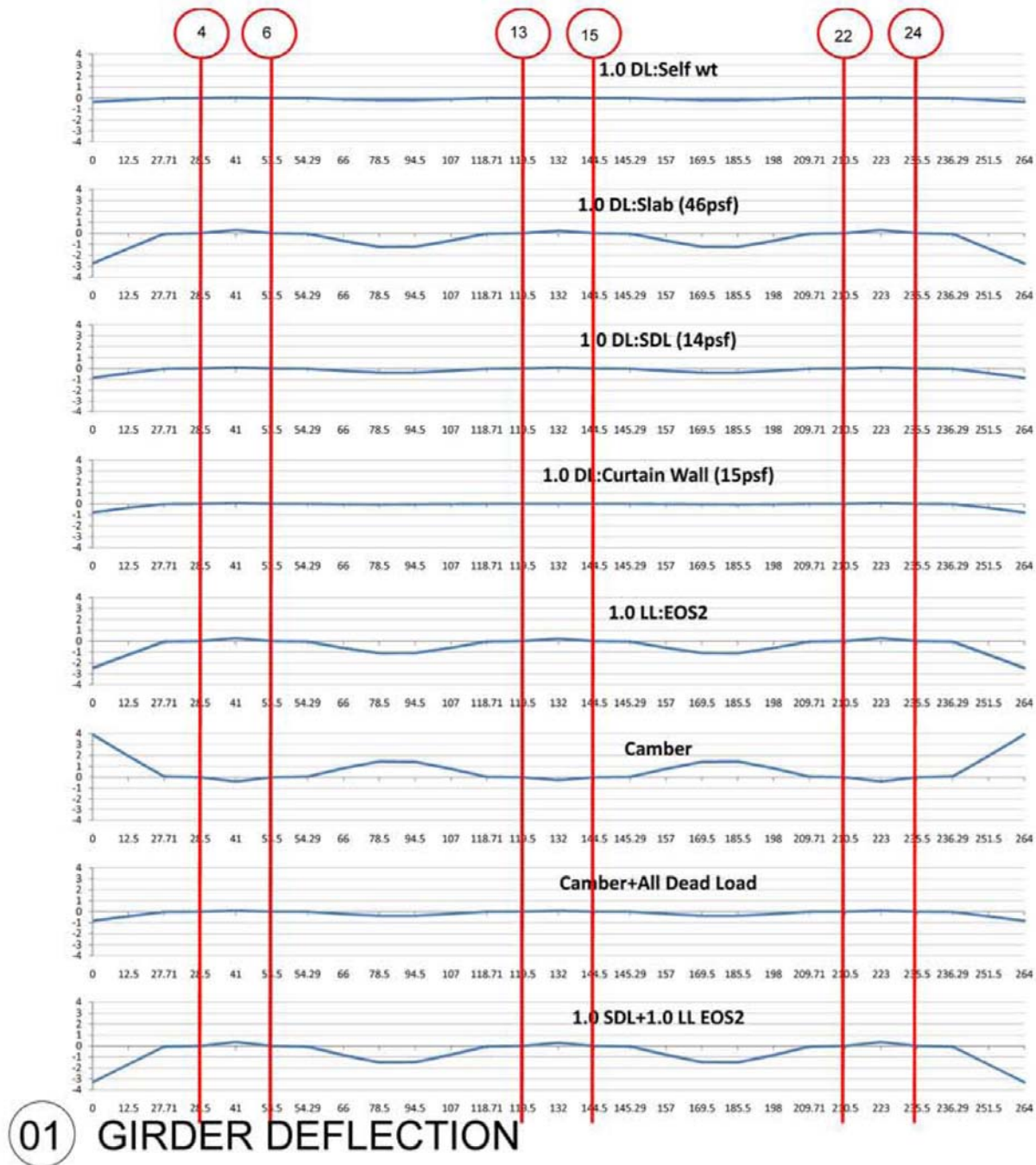
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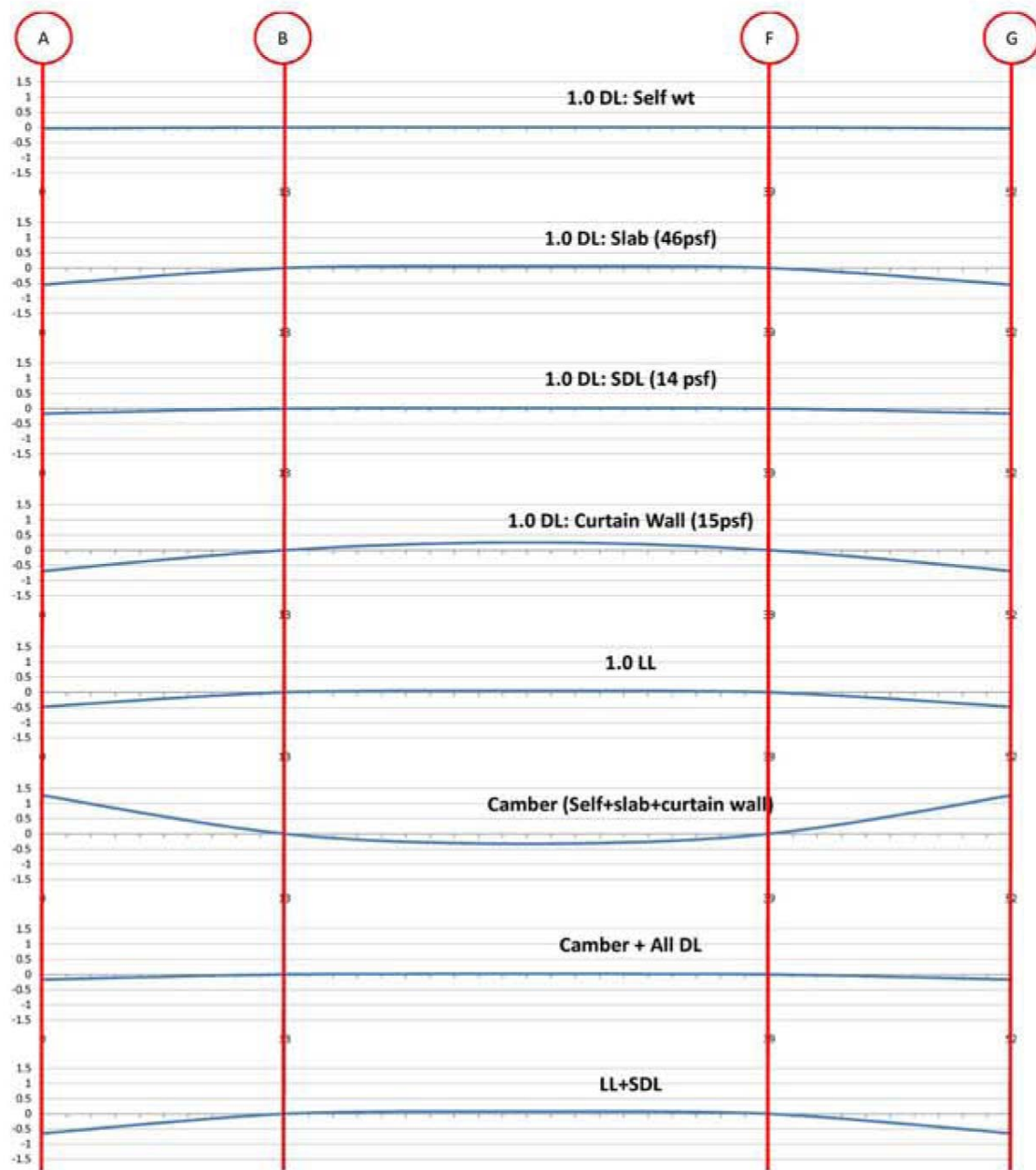
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Appendices

Appendix A –

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Final Summary Report**02 BEAM DEFLECTION**

APPENDICES - APPENDIX A
SOUTH WEST STUDENT HOUSING

TECH 1

BUILDING INFORMATION

- MODEL CODE: IBC 2006 AS AMENDED BY THE CITY OF TEMPE, ARIZONA
- DESIGN CODES: AISC "SPEC FOR STRUCTURAL STEEL BLDGS" AISC 360-05
ACI "BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONC" ACI 318-05
- STRUCTURAL STANDARDS: ASCE 7-05 WILL USE
ASCE 7-05, AISC 13TH ED, ACI 318-05, IBC 06

• DEFLECTION CRITERIA

↳ STRUCTURAL STEEL IS ALL CAMBERED TO DEAL W/ HIGH DEFLECTIONS.
OUT OF CURIOSITY - HOW DOES ONE ASSEMBLE A CAMBERED STRUCTURAL STEEL FLOOR
IF THE METHOD IS TO SLIDE BEAMS THROUGH PLASMA-CUT HOLES ON
HYDRAULIC ROLLERS?

CHARLIE'S NOTES: $\frac{1}{360}$ FOR LIVE $\frac{1}{240}$ FOR TOTAL $\frac{H}{400}$ FOR DRIFT

- STRUCTURAL OVERVIEW:

- FOUNDATIONS - ~~SPREAD~~ SPREAD FOOTING ACCORDING TO CHARLIE
ACCORDING TO SOILS REPORT RECOMMENDATIONS -
BLDG WILL EXERT SIGNIFICANT LOADS TO FOUNDATION ELEMENTS.
↳ EMPLOY DEEP FOUNDATION SYSTEM TO LIMIT SETTLEMENTS

ACCORDING TO TFG PACKAGE:

(3) 350 YD³ CORE MATS,
PERIMETER GRADE BEAMS,
SLAB ON GRADE ON TOP

• DRILLED PIERS + MAT

AXIAL CAPACITY = SKIN FRICTION OF SHAFT + END BEARING AT TIP

SOILS:

	DEPTH	FRICTION	END BEARING
I SILTY SAND + POORLY GRADED SAND	10-35'	0.4 KSF	—
II SAND GRAVEL COBBLE	35-100'	2.5 KSF	20 KSF

PIER SHAFTS SHOULD PENETRATE AT LEAST 2.5X PIER ϕ INTO II LAYER
& NO PIER $\phi < 12"$

MIN CLEAR SPACING = 3X BIGGEST ADJACENT PIER ϕ

PREDICTION: FOR ISOLATED PIER, $\phi < 60"$, SETTLEMENT $\leq 1"$

CHARLIE'S NOTES: BLDG DOESN'T HAVE SETTLING BIG SPANS ARE LONGER → NO DIFFERENTIAL SETTLING LOCAL?

- FLOOR SYSTEM - TYP. FOR ALL FLOORS - 3.25" LIGHTWEIGHT CONCRETE ON 3" DECK
OR HAND CALCS, CHARLIE ASSUMED 4.5" LIGHTWEIGHT CONCRETE
~~THINNING CORRUGATED SLAB ON LW.C~~
(P 14 IN PDF) 20 GAGE
GO W/ VULCRAFT

SUPPORTED BY STRUCTURAL STEEL FRAME

↳ TYP. FRAMING.

TYP. CORE BM = W14X50

TYP. BEAM = W14X40

TYP. INFILL BM = W14X22

TYP. GIRDER = W24X76

TYP. CONTINUOUS EDGE BEAM = W14X22

20 @ 12'-6"

15'

15'

SEC 1:33
SHOWS TYP. FRAMING PLAN

- FRAMING SYSTEM - SEE ABOVE FOR FLOOR FRAMING SYSTEM
SEE BELOW FOR GRAVITY FRAMING SYSTEM
- LATERAL SYSTEM - (3) CONCRETE CORES: 8" THICK, 25' X 25' ON CENTER (KIND OF)
SPACED 62'-6" APART (STARTING AT CENTER)

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APPENDIX	TEMPERATURE	22
TECH 1	SOUTHWEST STUDENT HOUSING	
* ROOF SYSTEM - 3" METAL DECK, 3.25" LWC, 2" RIGID INSULATION, EPDM MEMBRANE * MATERIALS: * STRUCTURAL STEEL - ALL ROLLED SHAPES - ASTM A36 OR A992 GRADE 60 - ALL PLATES & CONNECTION MATERIAL - ASTM A36 - ALL TUBULAR SECTIONS - ASTM A500 GRADE B - ALL PIPE SECTIONS - ASTM A53 GRADE B - ANCHOR BOLTS, U.B.N. - ASTM F1554 4" * CAST-IN-PLACE CONCRETE - FOUNDATIONS - 4 KSI N.W. U.O.N ON PLAN - S.O.G - 4 KSI N.W. - STRUCTURAL S.O.G - 5 KSI N.W. - FORMED SLABS - AS NOTED IN DWGS. (LIVE FLR 4 KSI) - WALLS (CORE) - 4-6 KSI * REINFORCEMENT - DEFORMED BARS - ASTM A615 GRADE 60 TOP, GRADE 70 FOR #9, #10, #11 - WELDED WIRE FABRIC - ASTM A195 * WELDING ELECTRODES - E7018 LOW HYDROGEN * BOLTING MATERIALS - ASTM A325 OR A490 U.O.N.		
LOADS - PRESCRIBED: DL 16 W/ = ACTUAL LL = 40 PSF PUBLIC AREAS = 100 CURTAIN WALL = 15 PSF SLAB DL = 40 PSF SNOW = 25 PSF GROUND 20 PSF PLATEAU WIND = 105 MPH 3-SEC GUST 1.0 = I C = EXPOSURE 1.0 = INTERNAL PRESSURE COEFFICIENT EQ = 1.0 = I II = OCCUPANCY TYPE 0.360 = S_s 0.456 = S_{ds} 0.070 = S₂ 0.191 = S_{ds2} 0.019 = C_s S = R F = SITE CLASS C = DESIGN CATEGORY CONCRETE SHEAR WALL = BASIC SEISMIC-FORCE-RESISTING SYSTEM 3550 K = DESIGN BASE SHEAR USED EQUIVALENT LATERAL FORCE METHOD		

TECH 1

APPENDICES- APPENDIX B
SOUTHWEST STUDENT HOUSING

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CALCULATED LOADS

*** GRAVITY**

- CONSTRUCTION DEAD LOAD

- DECK - 3.0 VLI (VULCRAST COMPOSITE DECK) 20 GAGE
3.25" LIGHTWEIGHT CONCRETE
DECK WEIGHT = 2.14 PSF
CONCRETE WEIGHT = 46 PSF
TOTAL = 48.14 PSF
- STRUCTURAL STEEL - ASSUME ≈ 8 PSF
CHECK CURRENT SIZES:

SIZE	LENGTH	#	WT (K)
W18x50	52'	6	15.6 K
W18x40	52'	12	25 K
W14x22	13	6	1.7 K
W24x176	262.5	2	42.4 K
W14x22	262.5	2	11.6 K
			146.3 K

TYPE FLOOR DIMENSIONS:
250' x 52' = 13,000 SF

APPROX. WEIGHT OF STRUCTURAL STL:
 $\frac{146.3 \times 10^3}{1300} = 11.25 \text{ PSF}$

STRUCTURAL STEEL - ASSUME 11 PSF ✓

→ TOTAL CONSTRUCTION DEAD LOAD = 48.14 + 11 = 59.14 → 59 PSF

- SUPERIMPOSED DEAD LOAD

→ ASSUME SDL OF 15 PSF ← ASSUMPTION BY ENGINEER. SHOULD HAVE CONFIRMED - PARTITIONS NOT INCLUDED.

- LIVE LOAD

- RESIDENTIAL = 40 PSF
- PARKING = 40 PSF
- CORRIDORS = 80 ON FLOORS ABOVE GROUND (PSF)
= 100 ON GROUND FLOOR (PSF)

(THERE IS A 6' WIDE CORRIDOR RUNNING THRU THE CENTER OF THE BUILDING IN THE LONG DIRECTION)
* ENGINEERS JUST TOOK ALL TO BE 40 PSF

LIVE LOAD REDUCTION:

81'-3" 87'-6" 81'-3" 52'

3

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TECH 1

APPENDICES-APPENDIX B
SOUTHWEST STUDENT HOUSING

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CALCULATED LOADS (CONTINUED)

REDUCTION FACTORS:

SECTION	AREA/FLOOR	# FLOORS	FACTOR
I	~9390 SF	1	0.5
		>1	0.41
II	4725 SF	1	0.5
		>1	0.4

} WILL JUST ROUND TO 0.4

$L = L_o \left(0.25 + \frac{15}{\sqrt{K_{LL} A T}} \right)$ $K_{LL} = 1$

≥ 0.5 (1 FLR)
 ≥ 0.4 (>1 FLR)

SAMPLE CALCULATIONS

Section I, 1 Floor:
 $0.25 + \frac{15}{\sqrt{1 \times 9390}} = 0.476 \rightarrow 0.5$

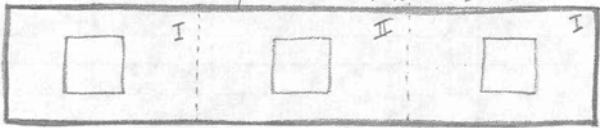
>1 Floor:
2? $0.25 + \frac{15}{\sqrt{1 \times 9390 \times 2}} = 0.41$
3? $0.25 + \frac{15}{\sqrt{1 \times 9390 \times 3}} = 0.38 \rightarrow 0.4$

$80 \text{ PSF} \times 0.5 = 40 \text{ PSF}$ (FOR CORE CALCULATIONS)

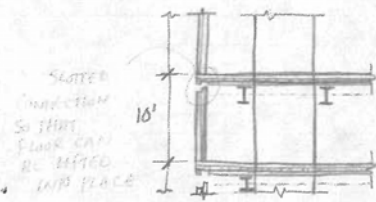
— MECHANICAL EQUIPMENT
ON GROUND FLOOR, w/ 2-PIPE SYSTEM FOR HEATING/COOLING ROOMS
CHILLER, PUMPS, ETC. ON GROUND FLOOR

— WALL LOADS

CURTAIN WALL - 15 PSF



EACH FLOOR SUPPORTS ITS RESPECTIVE "CURTAIN WALL"
 $\therefore$ FLOORS 2-19 HAVE 10' PERIMETER CURTAIN WALLS @ 15 PSF



— SNOW LOADS

$P_s = 0.7 C_e C_t I P_g \geq 0$
(FLAT ROOF)

$P_s = 0$

FIGURE 7-1 (ASCE 7-05)

(5000) 10
(4000) 5
(2500) ZERO

TEMPE, AZ IS @
AN ELEVATION OF...
1140-1495 FEET

$1495 < 3500 \rightarrow P_g = 0$

SHOULD TAKE INTO ACCOUNT RAIN LOADS?

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TECH 1	APPENDICES - APPENDIX C SOUTHWEST STUDENT HOUSING	28
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CALCULATED LOADS (CONTINUED)

*** LATERAL**

- WIND LOADS

ASSUME ENCLOSED BUILDING

FIGURE 6-1: 90 MPH NOMINAL S-SEC. JUST WIND SPEED
TABLE 6-1: IMPORTANCE CATEGORY II, FACTOR = 1.0
EXPOSURE CATEGORY B
(URBAN & SUBURBAN AREAS W/ NUMEROUS CLOSELY SPACED OBSTRUCTION ≥ SINGLE FAMILY DWELLINGS FOR 20 X BUILDING HEIGHT $T = 4/60 \text{ SEC}$)
→ JUSTIFIED BY SITE LOCATION IN CENTRAL TEMPE, AZ (SHOW PHOTO OF SITE?)

$G_{CPL} = 0.19$ (ENCLOSED)
 $L/B = \frac{(12.5' \times 0)}{52'} = 4.81 > 4$ (E-W) $= \frac{52}{250} = 0.209$ (N-S)

CP VALUES	E-W	N-S
WU WALL	0.8	0.7
LW WALL	-0.2	-0.5
SIDE WALL	-0.7	-0.7

ASSUME BUILDING IS RIGID (TO BE VERIFIED IN LATER REPORT)
TAKE $G = 0.85$

$q_z = 0.00256 (K_z) (K_{zt}) (K_d) (I) (PSF)$
 $15' \leq z \leq 1200'$
 $K_z = 2.01 \left(\frac{z}{1200} \right)^{2/7}$
 $K_{zt} = 1.0$
 $K_d = 0.85$
 $I = 1.0$ FOR MWFRS & COMPONENTS/CLADDING

$q_z = 0.00256 \left[2.01 \left(\frac{z}{1200} \right)^{2/7} \right] (0.85) (1.0) (1.0)$ PSF
FOR A FLOOR, HEIGHT z OFF THE GROUND

MWFRS: $P = q_z G C_p - q_i (GC_{pi})$ (PSF)

$P = \left[0.00256 \left\{ 2.01 \left(\frac{z}{1200} \right)^{2/7} \right\} 0.85 (90^2) \right] (0.85) C_p$ VARIES DEPENDING ON WALL & DIRECTION!

SO, 2ND FLOOR, WINDWARD WALL, EAST-WEST DIRECTION:
 $z = \frac{18}{2} = 9'$
 $P_w = \left[0.00256 \left\{ 2.01 \left(\frac{9}{1200} \right)^{2/7} \right\} 0.85 (90^2) \right] 0.85 (0.8) \approx 6 \text{ PSF}$

(SEE EXCEL TABLE FOR ALL CALCULATED VALUES, AS WELL AS TOTAL PRESSURE)
 TOTAL PRESSURE = $|WU| + |LW|$

TOTAL HEIGHT IS 208'

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TECH 1	APPENDICES APPENDIX D SOUTHWEST STUDENT HOUSING	31
AMPAD		
<u>CALCULATED LOADS (CONTINUED)</u> - SEISMIC LOADS FRAME SYSTEM: ORDINARY CONCRETE WALLS $R = 1.5$ $I = 1.0$ $C_s = \frac{S_{DS}}{(R/I)} \geq 0.01$ $C_s = \frac{S_{D1}}{T(R/I)}$ for $T \leq T_L$ (MAX) $T_a = 0.1N \sqrt{A} = 0.20$ OR $T_a = C_t h_n^{0.75} \approx 1.1 \text{ SEC}$ $S_s = 16\%g$ $S_1 = 8\%g$ $T_L = 6 \text{ SEC}$ } AS PER FIGURES FROM CH 22 OF ASCE 7-05 SITE CLASS = C (ACCORDING TO GEOTECH REPORT) $f_a = 1.2$ $f_v = 1.7$ $S_{MS} = f_a S_s$ $S_{M1} = f_v S_1$ $S_{DS} = \frac{2}{3} S_{MS}$ $S_{D1} = \frac{2}{3} S_{M1}$ $T_0 = 0.2 \frac{S_{D1}}{S_{DS}}$ $T_s = \frac{S_{D1}}{S_{DS}}$ $V = C_s W$ $S_{MS} = 1.2(0.16) = 0.192$ $S_{M1} = 1.7(0.08) = 0.136$ $S_{DS} = \frac{2}{3}(0.192) = 0.128$ $S_{D1} = \frac{2}{3}(0.136) = 0.0907$ $T_0 = 0.2 \frac{0.0907}{0.128} = 0.142$ $T_s = \frac{0.0907}{0.128} = 0.708$ $C_s = \frac{0.128}{1.5/1.0} = 0.0853 \leq \frac{0.0907}{(1.1)1.5/1.0} = 0.055$ $\rightarrow C_s = 0.055$ $W_{TOT} = \sum_{i=1}^{20} W_i$ $W_i = (C_{DL} + S_{DL}) A_{FLOOR} + \text{CURTAIN WALL (PERIMETER)} = (59+15)13000 + 15(604) = 9771.1 \text{ K}$ $V = (0.055)(1049 \times 20) = 1154 \text{ K}$ (SEE EXCEL TABLE FOR VERTICAL DISTRIBUTION)		

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APPENDICES - APPENDIX E

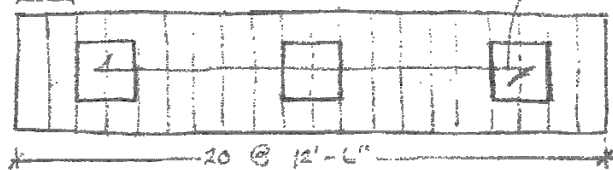
SOUTHWEST STUDENT HOUSING

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TECH 1

SPOT CHECKS*** METAL DECKING** (SEE PSS OF VULCRIFT STEEL DECK MANUAL) $3\frac{1}{4}"$ LIGHTWEIGHT CONCRETE (110 PCF)

3" 20 GAGE DECK

MAX UNSHORED SPAN:
1 SPAN = 10'-6"
2 SPAN = 12'-10"
3 SPAN = 13'-3"MIN # SPANS? $\rightarrow$ 2 $12'-6" \leq \text{MAX UNSHORED SPAN} \leq 13'-3"$ THP. SPAN = 12'-6" < 12'-10" OK

MAX SUPERIMPOSED LIVE LOAD: MAX SPAN = 12'-6"

$$\begin{aligned} &\checkmark \text{ MAX LIVE LOAD} = 75 \text{ PSF} \\ &\text{IMPROVABLE?} \rightarrow 20 \text{ PSF (6' WIDE CORRIDOR)} \\ &\quad + 15 \text{ PSF SUPERIMPOSED DEAD LOAD} \\ &\quad \hline 95 \text{ PSF} \quad 75 \text{ PSF} \quad \text{NG} \end{aligned}$$

SUPERSEDE IT:

$$\text{MAX 18 GAGE (3 VLL 18) SLL} = 169 \text{ PSF} \quad (3 \text{ VLL 18} = 84 \text{ PSF})$$

OK

USE 3" DECK (18 GAGES, MINIMUM) w/ $3\frac{1}{4}"$ LIGHTWEIGHT CONCRETE, 2 SPANS MINIMUM,
@ 12'-6" / SPAN

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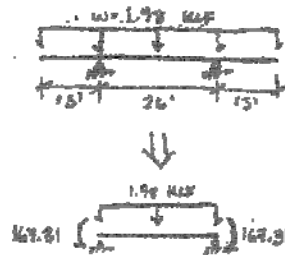
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APPENDICES - APPENDIX F

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TECH 1

SOUTHWEST STUDENT HOUSING

SPOT CHECKS (CONTINUED)*TYPICAL BEAM

$$205 = \text{FACTORED LOAD} = 1.2D + 1.6L$$

$$\text{TRIANGULAR WIDTH} = 12.5' \quad (\text{AREA})_{\text{EM}} = 650 \text{ SF}$$

REDUCIBLE LIVE LOAD

$$L = (80 \text{ PSF}) \left(0.25 + \frac{15}{\sqrt{4 \times 650}} \right) = 43.5 \text{ PSF}$$

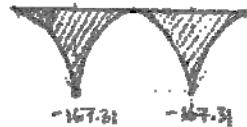
$$205 = 1.2(59 + 15) + 1.6(43.5) = 159.5 \text{ PSF}$$

$$W = 205 \times 12.5 = 1.98 \text{ KLF}$$

$$M = \frac{WL^2}{2} = \frac{(1.98)(26^2)}{2} = 167.31 \text{ FT-K}$$

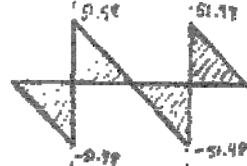
$$M @ \text{MIDSPAN} = \frac{WL^2}{8} - 167.31 = \frac{1.98(26^2)}{8} - 167.31 = 0$$

M in FT-K



$$M_R = 167.31 \text{ FT-K}$$

V in K



$$V_R = 51.98 \text{ K}$$

$$b_{\text{req}} = \begin{cases} \frac{1}{8} (\text{SPAN}) & = \frac{1}{8} (26') = 3.25' \\ \frac{1}{8} (\text{SPACING}) & = \frac{1}{8} (125') = 15.625' \\ \text{EDGE DISTANCE} & = 18' \end{cases}$$

$$\text{ASSUME } d = 1' \rightarrow \frac{1}{2} = 6.25' - \frac{1}{2} = 5.75'$$

TRY W10 X15

$$\phi M_n = 779 + \frac{192-174}{2} = 178 \text{ FT-K @ TFL}$$

L (INTERPOLATION)

$$a = \frac{221}{0.25(4)(3.25 \times 12)} = 1.67 > 1' \quad \text{NG}$$

TRY W10 X22

$$\phi M_n = 869 + \frac{179-169}{2} = 171.5 \text{ FT-K @ BFL}$$

$$a = \frac{117}{0.25(4)(3.25 \times 12)} = 0.882 < 1' \quad \text{OK}$$

$$\frac{117}{12.2} = 7 \text{ SHEAR STUDS } \times 2 = 14 \text{ STUDS}$$

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TECH 1

APPENDICES - APPENDIX F

SOUTHWEST STUDENT HOUSING

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SPOT CHECKS (CONTINUED)DEFLECTIONS? $\frac{L}{360}$ (LIVE) $\frac{L}{240}$ (TOTAL)

$\Delta = \frac{w(\frac{L}{2})^2}{24EI}$
SIMPLY
SUPPORTED
w/ END MOMENTS

$\Delta = \frac{wL^4}{8EI}$
CANTILEVERED

$$\left[\left(\frac{12}{2} \right)^3 - \left(\frac{24}{2} \right)^3 \right] + \left(\frac{12}{2} \right)^3 + \left(\frac{24}{2} \right)^3 = 1296 \frac{ft^3}{2}$$

$$I = 811 + \frac{329 \cdot 311}{2} = 320 \text{ in}^4$$

LIVE: 48.5 PSF $\rightarrow w = 0.584 \text{ kLF}$ TOTAL: 48.5 + 59 + 15 PSF $\rightarrow w = 1.469 \text{ kLF}$

END SPAN:

$$\frac{L}{360} = \frac{13 \times 12}{360} = 0.43''$$

$$\Delta_{LIVE} = \frac{(0.584/2) (13)^4}{8 (29000) 320} \times 1296 = 0.03'' \quad \text{OK}$$

$$\frac{L}{240} = \frac{13 \times 12}{240} = 0.65''$$

$$\Delta_{TOTAL} = \frac{(1.469/2) (13)^4}{8 (29000) 320} \times 1296 = 0.091'' \quad \text{OK}$$

MIDDLE SPAN:

$$\frac{L}{360} = 0.86''$$

$$M_{LIVE} = 48.5 \text{ PSF} \times 12 \text{ ft} = 581.6 \text{ ft-k}$$

$$\Delta_{LIVE} = \frac{(0.584/2) (13)^4}{24 (29000) 320} \left[15^3 - \frac{312^3}{2} + 1972 \left(\frac{35 \times 12}{2} \right) + 312^3 - 1296 \left(\frac{55 \times 12}{2} \right) \right] = 0.84'' \quad \text{OK}$$

$$\frac{L}{240} = 1.3''$$

$$M_{TOTAL} = 496.522 \text{ ft-k} = 5958.5 \text{ in-k}$$

$$\Delta_{TOTAL} = 4.23'' \quad \text{N/A}$$

NOTE - THESE DEFLECTIONS DO NOT TAKE INTO ACCOUNT
GIRDER DEFLECTIONS

QUICK DEFLECTION CALC FOR SPECIFIED BM SIZE ON TYP. FRAMING PLAN:

W18 x 40 Assume $\alpha = 2'' \rightarrow y_2 = 5.25''$ $P_{RA} = 6$ $\phi M_n = 965 \text{ ft-k}$

$$\alpha = 1.584'' < 2'' \quad \text{OK} \quad I = 1190 \text{ in}^4$$

END SPANS:

$$\Delta_{LIVE} = 0.007'' \quad \text{OK}$$

$$\Delta_{TOTAL} = 0.022'' \quad \text{OK}$$

MIDDLE SPAN:

$$\Delta_{LIVE} = 0.226'' \quad \text{OK}$$

$$\Delta_{TOTAL} = 1.157'' \quad \text{OK}$$

(WILL CHECK OVERALL
DEFLECTION INCLUDING
GIRDER IN NEXT SPOT
CHECK)

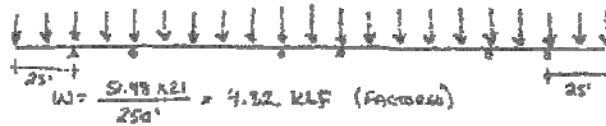
TECH 1

APPENDICES - APPENDIX G
SOUTHWEST STUDENT HOUSING

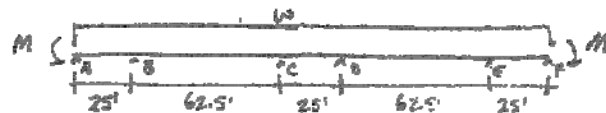
37

SEAT STACKS (CONTINUED)TYPICAL GIRDER (INTERIOR)

TRAIL WIDTH = 19.5'



Moment Distribution (See Attached Global Sheet)



$$M = \frac{WL^2}{2} = \frac{4.32 (25')^2}{2} = 1350 \text{ K-FT}$$

$$DF = \frac{1/L \text{ DIRECTION OF CHOICE}}{1/L_{LEFT} + 1/L_{RIGHT}}$$

(EI IS CONSTANT THROUGHOUT)

DISTRIBUTION FACTORS:

AB = 0.5
 BA = 0.714
 BC = 0.286
 CB = 0.286
 CD = 0.714
 DC = 0.714
 DE = 0.286
 ED = 0.286
 EF = 0.714
 FE = 0.5

SYMMETRIC

$$M_{UNE} = 245.75 \text{ K-FT}$$

$$M_{TOTAL} = 693.75 \text{ K-FT}$$

$$FEM = \frac{WL^2}{12}$$

3 EXCEL SHEETS INCLUDED REGARDING
MOMENT DISTRIBUTION:

- FACTORED LOADS
- LIVE LOAD ONLY (UNFACTORED)
- TOTAL LOAD

* SEE TYP. BM. SPOT CHECK FOR LOADS USED IN
TYP. GIRDER SPOT CHECK

FIXED END MOMENTS: (K-FT)			
JOINT	FACTORED	LIVE	TOTAL
AB	225+1350	41+244	116+694
BC	1406.25	254	723
CD	225	41	116
DE	1406.25	254	723
EF	225+1350	41+244	116+694

DESIGN LOADS:

$$M_{UE} = 1756 \text{ K-FT}$$

LIVE LOAD REDUCTIONS:

$$(80 \text{ PSF}) \left(0.25 + \frac{15}{\sqrt{4 \times 9075}} \right) = 0.36 (80 \text{ PSF}) \rightarrow L = 40 \text{ PSF}$$

Tech 1

APPENDICES-APPENDIX G
SOUTHWEST STUDENT HOUSING

35

SPOT CHECKS (CONTINUED)

$$\text{ASSUME } a = 3.25' \rightarrow 42 = 9.625'$$

$$b_{\text{ESS}} = \frac{1}{8} (\text{SPAN}) = \frac{1}{8} (162.5) = 20.3125' \leftarrow$$

$$\frac{1}{2} (\text{SPACING}) = \frac{1}{2} (25) = 12.5'$$

$$\text{EDGE DISTANCE} = 25'$$

TRY W27x102

$$\phi M_n = 1813 + (1840 - 1810) \left(\frac{9.625 - 4.5}{5 - 4.5} \right) = 1817.5 \text{ K} \cdot \text{ft} @ \text{PNA \#4}$$

$$a = \frac{879}{0.85(4)(93.75)} = 2.75 < 3.25' \text{ OK}$$

$$\frac{879}{17.2} = 51 \text{ SHEAR STUDS} \times 2 = 102 \text{ STUDS}$$

ENGINEER'S PRESCRIBED BEAM IS NOT IN COMPOSITE BM TABLE
IN AISC 13D STEEL CONSTRUCTION MANUAL (TABLE 3-19)
BUT IT'S A HEAVY W27x176, WHICH IS PROBABLY AS DEEP
OR MORE DEEP THAN MY W27x102

$$\text{DEFLECTIONS? } I = 7230 + 0.75(7430 - 7230) = 7280 \text{ in}^4$$

$$\begin{array}{ccccccc} & & & & & & (\text{SYMMETRIC}) \\ 1 & 2 & 3 & 4 & / & 3 & 2 & 1 \end{array}$$

SPAN 1:

$$\frac{L}{260} = 0.933$$

$$\Delta_{\text{LIVE}} = \frac{(0.75/12) 25^4}{7(29000) 7230} \times 1.728 = 0.526'' \text{ OK}$$

$$\frac{L}{240} = 1.25$$

$$\Delta_{\text{TOTAL}} = \frac{(8.23/12) 25^4}{7(29000) 7230} \times 1.728 = 0.674'' \text{ OK}$$

$$\Delta_x = \frac{w x}{24 E I} \left[x^3 - \left(2L + \frac{4M_1}{wL} - \frac{4M_2}{wL} \right) x^2 + \frac{12M_1}{w} x + L^3 - \frac{8M_1 L}{w} - \frac{4M_2 L}{w} \right]$$

(SINCE END MOMENTS ARE CORRELATED... TAKEN FROM CASE 32, AISC STEEL
CONSTRUCTION MANUAL, 2010, 14TH ED.)

$$\Delta_{\text{MAX}} @ x = \frac{L}{2} + \frac{M_1 - M_2}{wL}$$

SPAN 2:

$$x = 250.3''$$

$$\Delta_{\text{LIVE}} = 0.06''$$

$$\Delta_{\text{TOTAL}} = 0.17''$$

SPAN 3:

$$x = 415.1''$$

$$\Delta_{\text{LIVE}} = 0.83''$$

$$\Delta_{\text{TOTAL}} = 0.94'' \text{ OK}$$

SPAN 4:

$$x = 150''$$

$$\Delta_{\text{LIVE}} = 0.07''$$

$$\Delta_{\text{TOTAL}} = 0.19''$$

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AMFAD	<u>SPST CHECKS (CONTINUED)</u> NEW TYPICAL BEAM DEFLECTIONS: (W18x40) END SPANS: $\Delta_{LIVE} = 0.009 + 0.33 = 0.338" \quad \underline{OK}$ $\Delta_{TOTAL} = 0.022 + 0.94 = 0.962" \quad \underline{NG} \quad (\text{CAMBERED})$ MIDDLE SPAN: $\Delta_{LIVE} = 0.226 + 0.33 = 0.556" \quad \underline{OK}$ $\Delta_{TOTAL} = 1.137 + 0.94 = 2.077" \quad \underline{NG} \quad (\text{CAMBERED})$	12.

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TECH 1

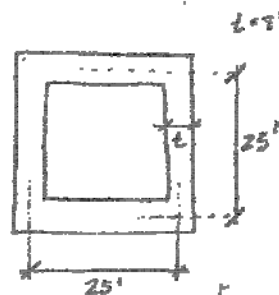
APPENDICES - APPENDIX H.

SOUTHWEST STUDENT HOUSING

9.

SPOT CHECKS (CONTINUED)

* CORES



$$\text{Area/CORE: } [(25 \times 12) \times 4] \times 9 = 9600 \text{ in}^2$$

$$\rightarrow \text{TOTAL CONCRETE AREA} = 3 \times 9600 = 28800 \text{ in}^2$$

$$P = \text{AXIAL LOAD} = L \times W \times 20$$

CURTAIN WALL WEIGHT (INCLUDED IN SEISMIC)

$$\rightarrow 15 \text{ PSF} \times 10' \times (250 \times 2 + 52 \times 2) = 90.6 \text{ k}$$

$$\text{ON 1ST FLOOR} = 90.6 \times \left(\frac{19'}{10'}\right) = 163.1 \text{ k}$$

$$P = 1.2 \left[(39 \times 15) (250 \times 2) \times 20 + 90.6 \times 19 + 163.1 \right] + 1.6 (90 \times 250 \times 2) \times 20$$

$$= 23890.3 + 16640 \approx 39730 \text{ KIIPS}$$

$$S_c = \frac{39730}{28800} = 1.38 \text{ KIIPS AXIAL STRESS}$$

$$S_c' = 4 \text{ KSI} \quad \text{OK}$$

WIND: MAX M W-E-W DIRECTION (WIDTH = 250')

 $\rightarrow$ TOTAL M = 68 855 k-ft (SEE WIND SPREADSHEET)

$$I_{\text{CORE}} = \frac{(12)(W)^3}{12} + 2 \left(t \cdot W \right) \left(\frac{W^2}{2} \right) = \frac{8(25 \times 12)^3}{12} + 2(8 \times 25 \times 12) \left(\frac{25 \times 12^2}{2} \right)$$

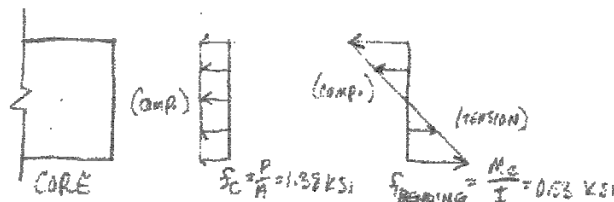
$$= 234000000 \text{ in}^4 = 11284.7 \text{ ft}^4$$

$$\text{BENDING STRESS } S = \frac{M \left(\frac{W}{2} \right)}{I} = \frac{68855 \left(\frac{25}{2} \right)}{11284.7} = 76.3 \text{ KEF} = 0.53 \text{ KSI}$$

$$\text{TOTAL MAX COMPRESSION} = S_c + S_{\text{BENDING}} = 1.91 \text{ KSI}$$

$$\text{TOTAL MIN COMPRESSION} = S_c - S_{\text{BENDING}} = 0.85 \text{ KSI}$$

OK
(CAN REINFORCE
w/ MINIMUM
REBAR REQUIRED
BY ACI 308)



$$S_c = \frac{P}{A} = 1.38 \text{ KSI}$$

$$S_{\text{BENDING}} = \frac{M \cdot c}{I} = 0.53 \text{ KSI}$$

SEISMIC:

$$S_{\text{BENDING}} = 76.3 \times \left(\frac{193600}{68855} \right) = 203.5 \text{ KEF} = 1.41 \text{ KSI}$$

$$\text{TOTAL MAX COMPRESSION} = 2.79 \text{ KSI}$$

$$\text{TOTAL MIN COMPRESSION} = -0.3 \text{ KSI} + \text{REBAR IN CORE WILL ALLEVIATE TENSION HERE}$$

OK

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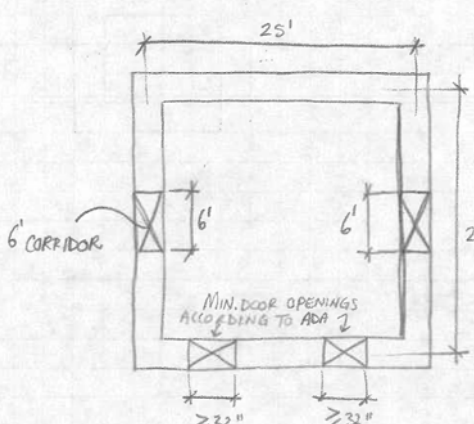
Team	APPENDICES - APPENDIX SOUTHWEST STUDENT HOUSING	43
Appendix	<u>FOUNDATION CALCULATIONS</u> TOTAL BUILDING WEIGHT (OBTAINED FROM SEISMIC SPREADSHEET) = 21125 KIPS $21125 \text{ KIPS} = \underbrace{\left(\frac{\pi}{4} d^2\right) (30 \text{ KSF}) \pi}_{\text{SAND GRAVEL CORE}} + \underbrace{\left(\pi d\right) (2.5 \text{ KSF}) (2.5') \pi}_{\text{SHAFT FRICTION}} + \underbrace{\left(\pi d\right) (0.4 \text{ KSF}) (35') \pi}_{\text{SURF FRICTION}}$ 81004 SAND + 200004 GRADED SAND TRY N=12 (GOAL? $d = 48"$) $21125 = (43.2 d^2 + 44 d) 12$ $d = \frac{(-44 \times 12) \pm \sqrt{(44 \times 12)^2 - 4(43.2 \times 12)(-21125)}}{2(43.2 \times 12)}$ $d = 6'$ NG TRY N=24 $21125 = (43.2 d^2 + 44 d) 24 \rightarrow d = 4' \text{ OK}$ $N=24 \rightarrow 8 \text{ PIERS / CORE MAT:}$ MIN CLEAR SPAN = $3 \times d = 12'$ NEED TO CHECK FOR PUNCHING SHEAR. MATS SIZED FOR PUNCHING SHEAR, $f'_c = 4000 \text{ PSI}$ PIER $\phi = 4'$, SPACING = 16" ON CENTER (12' CLEAR SPAN) CORE LOCATION BUILDING FOOTPRINT	

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TECH 1 (REVISION 10.11.2011) APPENDICES - APPENDIX H
SOUTHWEST STUDENT HOUSING

SPOT CHECKS (CONTINUED)
* CORES

WEAKEST SECTION:



ACTUAL CORE LAYOUT:



6' CORRIDOR

MIN. DOOR OPENINGS ACCORDING TO ADA 2

≥ 32"

AREAS. — OUTER CORES: $(25 \times 12)(8'') + (25 - 6)(2)(8'')(2) + (25 \times 12 - 32 \times 2)(8'') = 7936 \text{ in}^2$
 ↑ PER CORE

MIDDLE CORE: $(25 \times 12)(8'')(2) + (25 - 6)(12)(8'')(2) = 8448 \text{ in}^2$

TOTAL CROSS-SECTIONAL AREA @ WEAKEST POINT: $(7936)(2) + 8448 = 24320 \text{ in}^2$

TOTAL AREA ELSEWHERE: $(25 \times 12)(8'')(4) \times 3 = 28800 \text{ in}^2$

GRAVITY LOAD — $P = \text{AXIAL LOAD} \times L \times W \times 25$

CURTAIN WALL: $15 \text{ PSF} \times 10' \times (250 \times 2 + 52 \times 2) = 90.6 \text{ K}$ (FOR ALL FLOORS ABOVE 1ST FLOOR)

$90.6 \times \left(\frac{18'}{10'}\right) = 163.1 \text{ K}$ (ON 1ST FLOOR)

CORE SELF WEIGHT @ BASE:

ASSUME CORRIDOR & DOOR OPENINGS ARE 8' TALL

OVER 20 FLOORS, $20 \times 8 = 160'$ OF HEIGHT W/ WEAKEST SECTION

$\therefore SW = (150 \text{ PCF}) \left[160 \left(\frac{24320}{144} \right) + (208 - 160) \left(\frac{28800}{144} \right) \right] = 5493 \text{ KIPS}$

REINFORCED CONCRETE

$P = 1.2 \left[\frac{(59 + 15)(250 \times 52) \times 20}{1000} + 90.6 \times 19 + 163.1 + 5493 \right] + 1.6 \left[\frac{40 \times 250 \times 52 \times 20}{1000} \right]$

20 ELEVATED FLOOR SLABS.

$= 51941 + 16640$ FACTORED + REDUCED LIVE LOAD

↑ FACTORED DEAD LOAD

$P_u = 49581 \text{ KIPS}$

$f_c = \frac{49581}{24370} = 1.998 \approx 2 \text{ KSI}$

$P_{max} = 0.65(0.8) \left[0.85 f'_c A \right] = 0.442(4000)(24320) = 42998 \text{ K} < P_u = 49581 \text{ K}$

NG

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Tech 1 (REVISION 10.4.2011)

SPOT CHECKS (CONT'D)

* CORES (CONT'D)

MOMENT OF INERTIA :

UNCRACKED-

 I_x :

$$1. \quad S_{043} - \frac{bh^3}{12} = \frac{5' \left[(25'-6'') \times 12 \right]^3}{12} \times 9 = 7901568 \text{ in}^4$$

$$\text{Top} - \frac{bh^3}{12} = \frac{(25' \times 12)(9'')^3}{12} = 12900 \text{ in}^4$$

$$Ad^2 = (25' \times 12)(9'') \left(\frac{25'}{2} \times 12 \right)^2 = 54000000 \text{ in}^4$$

$$\text{Bottom} - \frac{bh^3}{12} = \frac{(25' \times 12 - 22' \times 12)(9'')^3}{12} = 10869 \text{ in}^4$$

$$Ad^2 = \left[(12' \times 12 - 22' \times 12)(9'') \right] \left(\frac{25'}{2} \times 12 \right)^2 = 42990000 \text{ in}^4$$

$$I_{x1} = 7901568 + 12900 + 54000000 + 10869 + 42990000 = 104404437 \text{ in}^4 = 5035 \text{ ft}^4$$

$$2. \quad S_{043} - \frac{bh^3}{12} = 7901568 \text{ in}^4$$

$$\text{Top/Bottom} - \frac{bh^3}{12} = 10869 \text{ in}^4 \quad Ad^2 = 54000000 \text{ in}^4$$

$$I_{x2} = 7901568 + 2 \times [10869 + 54000000] = 115921706 \text{ in}^4 = 5290 \text{ ft}^4$$

5294 ~ 5290

3. SEE CALCULATIONS FOR 1.

$$I_{x3} = 104404437 \text{ in}^4 = 5035 \text{ ft}^4$$

$$I_x = 5035 + 5040 + 5035 = 15660 \text{ ft}^4$$

$$E_{stl} = 29000 \text{ ksi} \quad E_{conc} = 57000 \sqrt{f'_{conc}} = 3605 \text{ ksi} \quad \eta = \frac{E_{stl}}{E_{conc}} = 8.0$$

$$\rho_{min} = 0.0012 \text{ FOR BARS } \leq \#5 \text{ (ACI 318-05 14.3.2 (a))}$$

$$= \frac{A_s}{b \cdot d}$$

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TECH 1 (REVISION 0.11.2011)

SPOT CHECKS (CONT'D)

* CORES (CONT'D)

$$M_{CR} = \frac{f_u I_{UNCR,TR}}{y_{BOT}} \quad y_{BOT} = h - \bar{y}$$

→ NEGLECTING STEEL FOR I.

BECAUSE OF ALMOST SYMMETRICAL LAYOUT OF CORES, $\bar{y}$ CAN BE ASSUMED @ 150" (ABOUT X-AXIS)
(THE CENTERLINE) $150" = 12.5'$

$$f_u = 7.5 \sqrt{f'_c} = 7.5 \sqrt{4000} = 474.34 \text{ PSI} = 68305 \text{ PSF} = 68.3 \text{ KSF}$$

$$M_{CR} = \frac{(68.3 \text{ KSF})(15660 \text{ FT}^2)}{12.5'} = 85566.24 \text{ K-FT}$$

$$M_{MIN} = 68355 \text{ K-FT @ BASE (UNFACTORED)} \text{ OR } 110169 \text{ K-FT (FACTORED)} \quad \underline{NG}$$

$$M_{MIN} = 57469 \text{ K-FT @ BASE} \quad \underline{OK}$$

→ SECTION IS CRACKED.

[SINCE POTENTIALLY UNCRACKED - NEGLECTING STEEL IN I_x CALCULATIONS LEADS TO A SMALLER I_x (SINCE THE STEEL IS EVENLY DISTRIBUTED THROUGHOUT THE WALLS)]

$$I_{x, UNCRACKED} = \frac{(110169 \text{ K-FT})(12.5')}{68.3 \text{ KSF}} = 20162.5 \text{ FT}^4$$

QUICK RECALL OF I_{TR} : d IS ALWAYS THE X-CENTERLINE FOR A CONCRETE SEGMENT.
(WITH MORE ACCURACY)

$$I_{TR} = \frac{bh^3}{12} + bh\left(\frac{h}{2} - \bar{y}\right)^2 + (n-1)A_s(d - \bar{y})^2$$

$$1. \text{ SIDES: } \rho = 0.0012 = \frac{A_s}{(b)(150'')} \quad A_s = 1.44 \text{ in}^2 \quad d = 150''$$

$$\frac{bh^3}{12} + bh\left(\frac{h}{2} - \bar{y}\right)^2 = \frac{(8'')(12.5-3'')^3}{12} + [(12.5-3'')(12'')(8'')]\left[\left(\frac{12.5-3}{2} + 3\right) \times 12\right]^2 = 3950784 \text{ in}^4$$

$$I_{x, TR} = 3950784 \times 4 + (8-1)(1.44)(150-150)^2 = 15803136 \text{ in}^4$$

$$\text{TOP: } A_s = 1.44 \text{ in}^2 \quad d = 4'' \quad (b = 300'')$$

$$\frac{bh^3}{12} + Ad^2 = 12960 + 54000000 = 54012960 \text{ in}^4$$

$$I_{x, TR} = 54012960 + (8-1)(1.44)(0-150)^2 = 5428960 \text{ in}^4$$

$$\text{BOTTOM: } A_s = 1.44 \text{ in}^2$$

$$\frac{bh^3}{12} + Ad^2 = 10069 + 42980000 = 42490069 \text{ in}^4$$

$$I_{x, TR} = 42490069 + (8-1)(1.44)(300-150)^2 = 42716969 \text{ in}^4$$

$$I_{x1, TR} = 15803136 + 54012960 + 42716969 = 112759605 \text{ in}^4 = 5438 \text{ FT}^4$$

↑ PER CHUNK
OF WALL, THERE
ARE 4 CHUNKS
TOTAL.↑ ALL AREA & I CALCS WERE DONE FROM
Q OF WALL, WHERE THE REINFORCEMENT
IS LOCATED

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TECH 1 (REVISION 10.11.2014)

SPOT CHECKS (CONT'D)

* GRES (CONT'D)

2. SIDES - 13803126 in⁴Top/BOTTOM - 54234600 in⁴

$$I_{x2\ TR} = 13803126 + 2 \times 54234600 = 124271326 \text{ in}^4 = 5994897$$

3. SAME AS 1.

$$I_{x\ TR} = 5438 \times 2 + 5994 = 16966 \text{ ft}^4 \leftarrow \text{INSUFFICIENT TO PREVENT CONCRETE FROM CRACKING!}$$

WHAT ABOUT $\epsilon = 10^\circ$?

$$I_x: 1. \text{ SIDES} - (245774) \left(\frac{1}{12} \right) = 992469 \text{ ft}^4$$

$$\text{TOP} - (11300) \frac{10^3}{12} + 54000 \times 100 \frac{10^3}{12} + (7)(1.44)(150)^2 = 3267.35 \text{ ft}^4$$

$$\text{BOTTOM} - (10060) \frac{10^3}{12} + 4248 \times 1000 \frac{10^3}{12} + (7)(0.44)(120)^2 = 2572.63 \text{ ft}^4$$

2. SIDES - 952.64 ft⁴Top/BOTTOM - 3267.35 ft⁴

3. SAME AS 1.

$$I_{x\ TR} = 952.64 \times 8 + 3267.35 \times 4 + 2572.63 \times 2 = 21042.63 \text{ ft}^4 > I_{x\ \text{needed}} \quad \text{OK}$$

$$M_{\text{wind}} < M_{\text{CR}} @ \epsilon = 10^\circ \rightarrow \text{CAN ASSUME UNCRACKED SECTION.}$$

THEORIES FOR DENATION: THE ACTUAL DESIGN MIGHT USE MORE REINFORCEMENT. THE DESIGNERS ALSO MAY HAVE CALCULATED THE WIND LOADS DIFFERENTLY.

FROM PREVIOUS CALCULATIONS, IT WAS SHOWN THAT THE SECTION WAS ASSUMED UNCRACKED, THUS THE CALCULATIONS TO FIND AN APPROPRIATE GEOMETRY THAT JUSTIFIES THIS ASSUMPTION.

SHEAR CHECK:

$$V_u = 22 \sqrt{3} \text{ E}' A \rightarrow \text{TO STEEL INT.} = 2(1.0) \sqrt{4000} \left[(12 \times 22) + 4 \times 300 + (25 \times 6) \times (2 \times 22) \right] = 916.81 \text{ kips}$$

$$\phi V_u = 0.75(1.0 V_u) = 687.6 \text{ kips / CORNER} \times 3 \text{ CORNERS}$$

$$\phi V_u = 2062.82 \text{ kips}$$

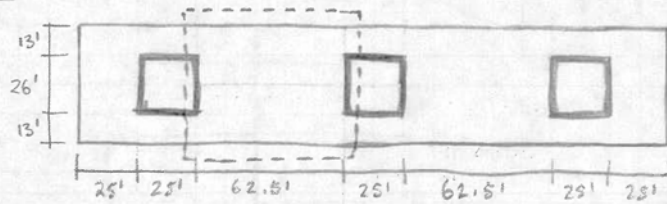
$$V_{\text{wind}} = 565 \text{ k (UNFACTORED)} & 704 \text{ k (FACTORED)} < \phi V_u \quad \text{OK}$$

$$V_{\text{seismic}} = 255 \text{ k} < V_{\text{wind}} \quad \text{OK}$$

$$\text{IF RUN = 0.5 V_u: } \phi V_u = 0.75(0.5 V_u) = 343.9 \text{ k / CORNER} \times 3 \text{ CORNERS} = 1031.4 \text{ k} > V_{\text{wind}} \quad \text{OK}$$

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TECH 11

FLOOR SYSTEM CALCULATIONS

DESIGNING FOR THE FLOOR SPAN ENCLOSED IN THE DOTTED LINE.

- ASSUMPTIONS: THE SUPPORTS PROVIDED BY EACH CORE (USE $M = \frac{wl^2}{11}$) ARE PARTIALLY FIXED

- LOADS: CONSTRUCTION LOADS NEED TO BE RECALCULATED
FIRE RATING FOR FLOOR - 2 HOURS, MIN

↑ AS PER RECOMMENDATION
OF DR. LEPAGE

SUPERIMPOSED DEAD LOAD = 15 PSF

LIVE LOAD (NOT REDUCED) = 80 PSF ← SMALL POTENTIAL FOR REDUCTION
ON CERTAIN BEAMS;
IT IS SIMPLER TO USE 80 PSF FOR EVERYTHING.LATERAL LOADS - WIND WOULD BE UNAFFECTED BY DIFFERENT FLOOR
SYSTEMS UNLESS THE FLOOR-TO-FLOOR HEIGHT CHANGED.SEISMIC LOADS WOULD NEED TO BE RECALCULATED TO DEAL
WITH DIFFERENT FLOOR ASSEMBLY WEIGHT.FLOOR SYSTEMS TO TEST:

- NONCOMPOSITE (SAME BEAM/GIRDER LAYOUT)
- LONG SPAN DECK (REMOVE ONE OF THE BEAMS FRAMING INTO THE GIRDER)
- HOLLOW CORE (SAME BEAM/GIRDER LAYOUT)
- POST-TENSIONED SLAB (WIDE GIRDERS + BEAMS, MOST LIKELY SAME LAYOUT)

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3" NW w/ 0.6C, 1.0C, 1.3C, 1.5C

MIN 2 SPANS (TO DEAL W/ CANTILEVER)

WILL GO W/ 2-1/2" NW $\rightarrow$ INITIAL SIZING: (BASED ON MAX UNSHORED CONDITION)

0.6C22 - 4' 7"	(2 @ 3 SPAN)	} NG
1.0C20 - 7' 3"	(2 @ 3 SPAN)	
1.3C20 - 8' 4"	(2 @ 3 SPAN)	
1.5C18 - 9' 9"	(2 SPAN) 10' 1" (3 SPAN)	
2C16 - 12' 6"	(2 SPAN) 12' 11" (3 SPAN)	OK

ASSUME FLOOR
WILL BE COVERED W/
A FINISH (AFFECTS
ALLOWABLE SLAB
LOAD)

2C16 CANNOT HOLD LOADS ON SUCH WIDE SPANS

3C16 w/ 3" NW?

MAX ALLOWABLE UNIFORM LOAD FOR $\leq \frac{8}{240} = 100 \text{ PSF}$

LOADS:

• CONSTRUCTION:

DECK WEIGHT = 59 PSF

ASSUMED STRUCTURAL SLAB = 15 PSF

TOTAL = 59 + 15 = 73 PSF

• SUPERIMPOSED: 1/8 PSF

TOTAL DEAD LOAD = 73 + 15 = 88 PSF

TOTAL LIVE LOAD = 90 PSF (COULD POTENTIALLY BE REDUCED BY ~7 PSF)

TOTAL SUPERIMPOSED UNIFORM LOAD = 15 PSF + 90 PSF = 95 PSF < 100 PSF **OK**

$\rightarrow$ USE 3" CONFORM DECK, 1/8 GAGE, WITH 3" N.W. CONCRETE
2 SPANS, MIN. TYP SPAN = 12'-6"

GIRDER DESIGN:TENSURE WIDTH = 26" $\rightarrow A = 26(62.5) = 1625 \text{ in}^2$

(ASSUMING THE EDGE GIRDER PROVIDES NO SUPPORT, THIS NO REDUCTION IN TENS.)

REDUCTIONS:

$$(90 \text{ PSF}) \left(1.25 + \frac{15}{\sqrt{9 \times 1625}} \right) = 0.44 (90)$$

$$\Rightarrow \text{LIVE LOAD} = 0.5(90) = 90 \text{ PSF}$$

$$W_{\text{FACTORED}} = [1.2(88) + 1.6(90)] \times 26' = 4.41 \text{ KLF}$$

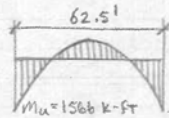
$$W_{\text{LIVE}} = (90 \text{ PSF}) \times 26' = 1.04 \text{ KLF}$$

$$W_{\text{TOTAL}} = [88 + 90] \times 26' = 3.33 \text{ KLF}$$

NONCOMPOSITE FLOOR SYSTEM (CONT'D)

GIRDER DESIGN (CONT'D)

$$\text{MAX MOMENT} \approx \frac{wl^2}{11} \rightarrow M_u = \frac{(4.41)(62.5)^2}{11} = 1566 \text{ K-FT}$$



→ MAINTAIN SAME FLOOR HEIGHTS ∴ W24 X —

FROM TABLE 3-2 OF AISC STEEL CONSTRUCTION MANUAL, 13TH ED.:

$$\phi_b M_{px} = 1570 \text{ K-FT FOR W24 X 146} > M_u = 1566 \text{ K-FT}$$

OK

DEFLECTIONS

$$I = 4590 \text{ in}^4$$

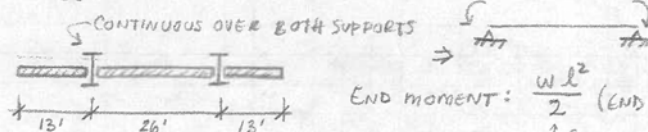
$$\delta_{\text{max}} = \frac{wl^4}{384EI} \left(\frac{12}{11} \right) = \frac{wl^4}{352EI}$$

$$\delta_{\text{LIVE}} = \frac{(1.04/12)(62.5 \times 12)^4}{352(29000)(4590)} = 0.587'' \quad \frac{l}{360} = 2.09'' \quad \text{OK}$$

$$\delta_{\text{TOTAL}} = \frac{(3.33/12)(62.5 \times 12)^4}{352(29000)(4590)} = 1.878'' \quad \frac{l}{240} = 3.125'' \quad \text{OK}$$

→ USE W24 X 146 FOR 62'-6" SPAN BETWEEN CORES (AKA GIRDER)

BEAM DESIGN:



$$\text{END MOMENT} = \frac{wl^2}{2} \quad (\text{END LOADED CANTILEVER})$$

↳ FOR UNIFORM LOAD.

TO INCLUDE CURTAIN WALL:

$$10' \times 12.5' \times 15 \text{ psf} = 1.875 \text{ K END LOAD} + PL$$

$$\text{TRIB. WIDTH} = 12.5'$$

$$\rightarrow 12.5' \times 52' = 650 \text{ SF}$$

$$(80) \left(0.25 + \frac{15}{\sqrt{4 \times 650}} \right) = 0.544(80) = 43.5 \text{ PSF}$$

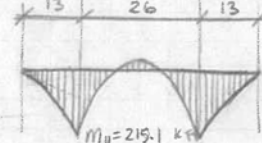
$$W_{\text{FACTORED}} = [1.2(89) + 1.6(43.5)] \times 12.5' = 2.2 \text{ KLF}$$

$$W_{\text{LIVE}} = 43.5 \times 12.5' = 0.544 \text{ KLF}$$

$$W_{\text{TOTAL}} = (89 + 43.5) \times 12.5' = 1.644 \text{ KLF}$$

$$\text{END MOMENTS} = \frac{(2.2)(13')^2}{2} + 1.875(13') = 215.1 \text{ K-FT}$$

$$\text{MIDSPAN MOMENT} = \frac{wl^2}{8} - \frac{2(M_{\text{END}})}{2} = \frac{(2.2)(26')^2}{8} - 215.1 = -29.8 \text{ K-FT}$$



$$M_u = 210.3 \text{ K-FT}$$

FOR EASE OF APPLYING DECK,

→ MAINTAIN SAME FLOOR HEIGHTS ∴ W18 X —

$$\phi_b M_{px} = 249 \text{ K-FT FOR W18 X 35} > M_u = 210.3 \text{ K-FT}$$

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3/3

NONCOMPOSITE FLOOR SYSTEM (CONT'D)

BEAM DESIGN (CONT'D)

DEFLECTIONS

$$I = 510 \text{ in}^4 \quad \delta_x = \frac{w x}{246E} \left[x^3 - \left(2x + \frac{4M_1}{wL} - \frac{4M_2}{wL} \right) x^2 + \frac{12M_1}{w} x + L^3 - \frac{8M_1 L}{w} - \frac{4M_2 L}{w} \right]$$

$$M_1 = M_2, \quad x = \frac{L}{2}$$

$$M_{LIVE} = \frac{(0.344)(13)^2}{2} = 14.6 \text{ k-ft}$$

$$\delta_x = \frac{w L}{48 E I} \left[\frac{L^3}{8} - \frac{L^3}{2} + \frac{16 M L}{w} + L^3 - \frac{12 M L}{w} \right]$$

$$M_{TOTAL} = 16.3 \text{ k-ft}$$

$$\delta_{LIVE} = \frac{(0.344/12)(26 \times 12)}{48(29000)(510)} \left[(26 \times 12)^3 - \frac{3(26 \times 12)^3}{8} - \frac{6(46 \times 12)}{(0.344/12)(26 \times 12)} \right] = -0.076''$$

$$\delta_{TOTAL} = \frac{(1.644/12)(26 \times 12)}{48(29000)(510)} \left[(26 \times 12)^3 - \frac{3(26 \times 12)^3}{8} - \frac{6(16.3 \times 12)}{(1.644/12)(26 \times 12)} \right] = -0.469''$$

$$\frac{L}{360} = 0.867'' > |-0.076''| \quad \text{OK}$$

(DEFLECTIONS ARE NEGATIVE BECAUSE THE END MOMENTS ARE LARGE ENOUGH TO CAUSE THE BEAM TO CURVE UP @ MIDSPAN)

$$\frac{L}{240} = 1.3'' > |-0.469''| \quad \text{OK}$$

$$\delta_{CANTILEVER} = \frac{w L^4}{8 E I} + \frac{P L^3}{24 E I}$$

CANTILEVER WALL NOT LIVE LOAD

← ENLARGED. MIGHT NEED TO CHECK CANTILEVER DEFLECTIONS

$$\delta_{LIVE, END} = \frac{(0.344/12)(13 \times 12)^4}{8(29000)(510)} + \frac{1.776(13 \times 12)^3}{24(29000)(510)} = 0.227'' < \frac{13 \times 12}{360} = 0.433'' \quad \text{OK}$$

$$\delta_{TOTAL, END} = \frac{(1.644/12)(13 \times 12)^4}{8(29000)(510)} + \frac{1.776(13 \times 12)^3}{24(29000)(510)} = 0.946'' > \frac{13 \times 12}{240} = 0.65'' \quad \text{NG}$$

$$0.18 \times 46? \quad I = 712$$

• MIDSPAN -

$$\delta_{LIVE} = (-0.076) \left(\frac{510}{712} \right) = -0.054'' < 0.867'' \quad \text{OK}$$

$$\delta_{TOTAL} = (-0.469) \left(\frac{510}{712} \right) = -0.336'' < 1.3'' \quad \text{OK}$$

• END SPAN -

$$\delta_{LIVE} = (0.227) \left(\frac{510}{712} \right) = 0.163'' < 0.433'' \quad \text{OK}$$

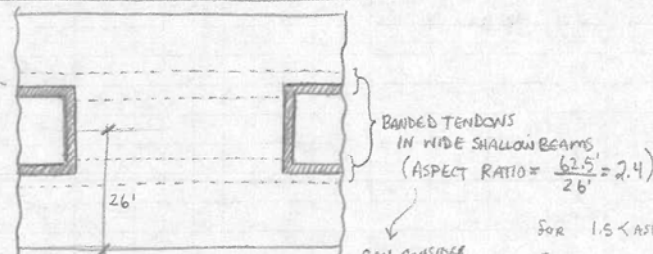
$$\delta_{TOTAL} = (0.946) \left(\frac{510}{712} \right) = 0.66'' < 0.65'' \quad \text{OK}$$

→ USE W18x46 FOR BEAMS @ 12'-6" SPACING.

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POST-TENSIONED CONCRETE SLAB (2-WAY)

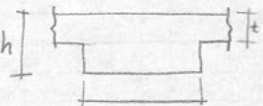


BANDED TENDONS
IN WIDE SHALLOW BEAMS
(ASPECT RATIO = $\frac{62.5}{26} = 2.4$)

FOR $1.5 < \text{ASPECT RATIO} < 2.5$,
RECOMMENDATION IS TO USE
WIDE SHALLOW BEAMS
(PTI TECH NOTE #3)

CAN CONSIDER
INITIAL SLAB SIZING
FROM SHORTER
DIRECTION, CAN USE
EQUIVALENT FRAME METHOD
FOR ANALYSIS

**DIMENSIONAL RESTRICTIONS
ON WIDE SHALLOW BEAMS:**



$h \leq 2t$ $h \leq \frac{b}{3}$

(SEE FIGURE 5.1-1 OF TECH NOTE)

MATERIALS:

CONCRETE - NW (150 PCF) $f'_c = 4000 \text{ PSI}$ $f'_{ci} = 3000 \text{ PSI}$
 REBAR - $F_y = 60000 \text{ PSI}$
 POST-TENSIONING - UNBONDED
 $\frac{1}{2}" \phi$ 7-WIRE $\rightarrow A = 0.153 \text{ in}^2$
 $f_{pu} = 270000 \text{ PSI}$ (ASSUME 30% PRESTRESS LOSS)
 $f_{py} = 243000 \text{ PSI}$
 $f_{pe} = 0.7 f_{pu} - \text{LOSSES} = 0.7(270) - 30 = 159000 \text{ PSI}$
 $\rightarrow P_{eff} = A f_{pe} = 24.327 \text{ K/TENDON}$

INITIAL SIZING:

FOR FULLY CONTINUOUS - $\frac{b}{t} \leq 45$ $t \geq \frac{b}{45}$ $t \geq 6.933 \text{ in} \rightarrow t = 7 \text{ in}$
 $\therefore h \leq 2t \rightarrow h \leq 2(7) = 14$, SAY $h = 14 \text{ in}$
 $h \leq \frac{b}{3} \rightarrow 14 \leq \frac{b}{3}$, SAY $b = 42 \text{ in}$

LOADS/LOADING:

DEAD - SELF WEIGHT = $\frac{BM \text{ SW}}{\text{TRIB}} + \text{SLAB SW} = \frac{150 \text{ PCF}}{12 \text{ in/ft}} \left[\frac{(14-7)(42)}{26 \times 12} + 7 \right] = 99.3 \text{ PSF}$
 SUPERIMPOSED = 15 PSF
 LIVE = 80 PSF $\rightarrow 26' \times 62.5' = 1625 \text{ SF}$ $(80) \left(0.25 + \frac{15}{\sqrt{1625 \times 1.0}} \right) = 0.622(80) \approx 50 \text{ PSF}$

$\frac{\text{LIVE}}{\text{DEAD}} = \frac{50}{114.3} = 0.44 < 0.75$ (NO PATTERN LOADING REQUIRED)

ASSUME GROSS SECTION PROPERTIES FOR CLASS II

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POST-TENSIONED CONCRETE (2-WAY) (CONT'D)

DESIGN PARAMETERS:

$$\text{ALLOWABLE STRESSES - } \textcircled{a} \text{ JACKING - } 0.6 f_{ci} = 0.6(3000) = 1800 \text{ PSI [COMPRESSION]}$$

$$3 \sqrt{f_{ci}} = 164 \text{ PSI [TENSION]}$$

$$\textcircled{b} \text{ SERVICE - } 0.45 f_c = 0.45(4000) = 1800 \text{ PSI [COMPRESSION]}$$

$$6 \sqrt{f_c} = 379.5 \text{ PSI [TENSION]}$$

$$\text{AVERAGE PRECOMPRESSION LIMITS - } \frac{P}{A} = 123 \text{ PSI (MIN.)}$$

$$= 300 \text{ PSI (MAX. FOR ECONOMICAL REASONS)}$$

$$\text{TARGET LOAD BALANCE - } 0.6 - 0.8 \rightarrow 0.8 \times (\text{SELF WEIGHT}) = 0.8(99.3) = 79.44 \text{ PSF}$$

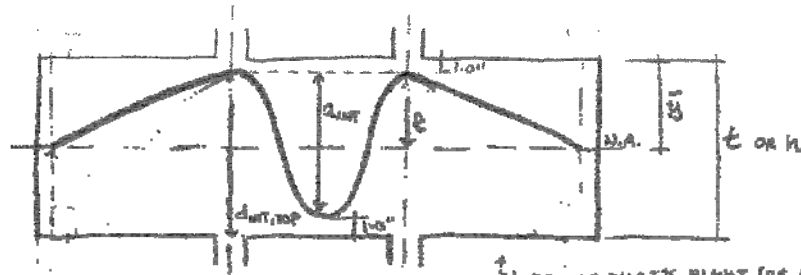
$$\text{COVER - TOP} = 0.75"$$

$$\text{BOTTOM} = 0.75" \text{ (RESTRAINED)}$$

$$= 1.5" \text{ (UNRESTRAINED)}$$

} Add $\frac{\phi}{2}$ to C_c

TENSION PROFILE (HEIGHTS FOR REFERENCE) -



1. ASSUME THAT'S RIGHT FOR A CANTILEVER?

EAST-WEST DIRECTION DESIGN:

PRESTRESS FORCE TO BALANCE - 80% S.W.

$$W_{BAL} = (79.44 \text{ PSF}) \times 26' = 2065.44 \text{ PLF}$$

FOR E-W DIRECTION, ONLY LOOKING @ INTERIOR BAY.

$$\hookrightarrow P = \frac{W_{BAL} L^2}{8 \Delta_{INT}}$$

$$A = 7' \times 31.2" + 7' \times 24.2" = 2478 \text{ in}^2$$

$$\bar{y} = \frac{(7 \times 31.2)(3.5) + (7 \times 24.2)(10.5)}{2112 + 7 \times 92} = 4.33" \text{ (from top)}$$

$$I = \left[\frac{(7 \times 31.2)^3}{12} + (7 \times 31.2)(3.5 - 4.33)^2 \right] + \left[\frac{(7 \times 24.2)^3}{12} + (7 \times 24.2)(10.5 - 4.33)^2 \right] = 22715.3 \text{ in}^4$$

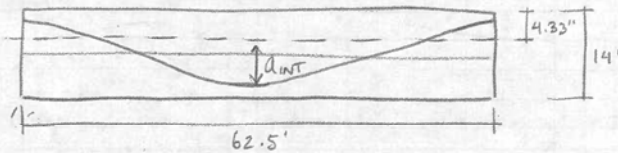
$$S_{TOP} = \frac{I}{\bar{y}_{TOP}} = \frac{22715.3}{14.75} = 1539.3 \text{ in}^3$$

$$S_{BOT} = \frac{I}{\bar{y}_{BOT}} = \frac{22715.3}{11.43} = 1986.4 \text{ in}^3$$

3/

POST-TENSIONED CONCRETE (2-WAY) (CONT'D)

E-W TENDON PROFILE IN WIDE SHALLOW BEAM:

SLAB CONTIGUES OFF ON BOTH ENDS, $\therefore$ TENDONS DON'T NEED TO RETURN TO THE NEUTRAL AXIS, AS THEY ARE CONTINUOUS

$$a_{INT} = 14 - 4.33 - 1.0 = 8.67''$$

$$SO \quad P = \frac{(2065.44)(62.5)^2}{8(8.67/12)} = 1395.9 \text{ KIPS}$$

$$\frac{1395.9}{24.327} = 57.4 \rightarrow \text{MAX ACCEPTABLE TENDON AREA: } \frac{P}{A} = 300 \text{ PSI} \quad P = 300(A)$$

$$P_{ACTUAL} = 30(24.327) = 729.8 \text{ K}$$

$$300(2478) = 743.4 \text{ K}$$

$$\hookrightarrow \frac{743.4}{24.327} = 30 \text{ TENDONS}$$

$$W_b = \frac{P_{ACTUAL}(8 \times a_{INT})}{l^2} = \frac{729.8(8 \times 8.67/12)}{62.5^2} = 1.08 \text{ KLF}$$

$$W_{S.W.} = 2.06 \text{ KLF}$$

$$\frac{W_b}{W_{S.W.}} = \frac{1.08}{2.06} = 0.523 < 1.0 \quad \underline{OK}$$

$$P_{EFF} = 729.8 \text{ K} \quad \frac{P}{A} = \frac{729.8}{2478} = 0.2945 \text{ KSI}$$

SLAB STRESSES:

$$\text{DEAD} - W_D = (99.3 + 15)(26') = 2.97 \text{ KLF}$$

IMMEDIATELY AFTER JACKING:

• MIDSPAN:

$$f_{TOP} = \frac{(-395.5 + 143.8)}{5269} - 0.2945 = -342 \text{ PSI}$$

$$f_{BOTT} = \frac{(395.5 - 143.8)}{2359} - 0.2945 = -188 \text{ PSI}$$

• SUPPORTS:

$$f_{TOP} = \frac{(1055 - 383.5)}{5269} - 0.2945 = -182 \text{ PSI}$$

$$f_{BOTT} = \frac{(1055 + 383.5)}{2359} - 0.2945 = -579 \text{ PSI}$$

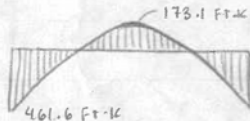
WITHIN CLASS U DESIGN
PARAMETERS, OK

$$\frac{W_D^2}{8} - \frac{W_L^2}{11} = 395.5 \text{ FT-K}$$

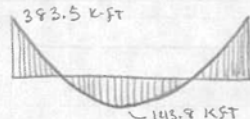


$$\frac{W_L^2}{11} = 1055 \text{ FT-K}$$

$$\text{LIVE} - W_L = (50)(26') = 1.3 \text{ KLF}$$



$$\text{BALANCING} - W_b = 1.08 \text{ KLF}$$



POST-TENSIONED CONCRETE (E-W 44) (CONT'D)

STRESSES AT SERVICE LOAD:

• MIDSPAN:

$$f_{TOP} = \frac{(-295.5 - 173.1 + 143.2)}{5269} - 0.2945 = -375 \text{ PSI}$$

$$f_{BOT} = \frac{(295.5 + 173.1 - 143.2)}{2359} - 0.2945 = -114 \text{ PSI}$$

• SUPPORTS:

$$f_{TOP} = \frac{(1055 + 461.6 - 383.5)}{5269} - 0.2945 = -79 \text{ PSI}$$

$$f_{BOT} = \frac{(-1055 - 461.6 + 383.5)}{2359} - 0.2945 = -774 \text{ PSI}$$

WITHIN CLASS U DESIGN PARAMETERS, OK

ULTIMATE STRENGTH:

$$M_1 = P_e = (729.8 \text{ K})(4.33 - 1.0) = 202.5 \text{ K-FT}$$

$$M_{SEC} = M_{BAL} - M_1 = 383.5 - \frac{202.5}{12} = 366.6 \text{ K-FT} \leftarrow \text{EQUAL @ BOTH SUPPORTS}$$

MINIMUM BONDED REINFORCEMENT:

OVERALL, NO STRESSES IN TENSION $\Rightarrow$ TENSILE REINFORCEMENT IS UNNECESSARY

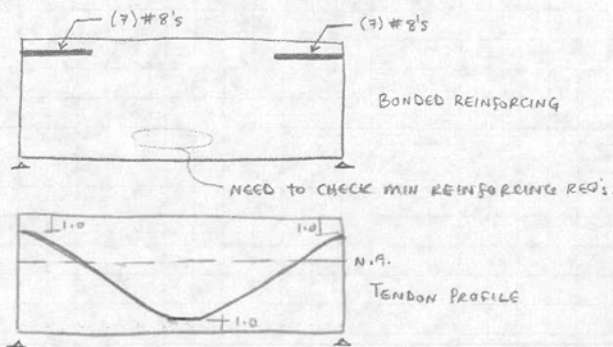
NEGATIVE MOMENT REGIONS:

$$A_{S, MIN} = 0.00075 A_{CS} \quad A_{CS} = \left\{ \frac{(25 + 62.5)}{2} = 43.75 \right\} \times 12' \times 14" = 7350 \text{ IN}^2$$

$$A_{S, MIN} = 0.00075(7350) = 5.5125 \text{ IN}^2$$

(7) #8's TOP (5.53 IN²)

AT BOTH SUPPORTS



5/1

POST-TENSIONED CONCRETE (2-WAY) (CONT'D)

$$A = 7' \times (62.5' \times 12) = 5250 \text{ in}^2$$

$$S = \frac{bh^2}{6} = \frac{(7'')(7'')^2}{6} = 6125 \text{ in}^3$$

NORTH-SOUTH DIRECTION DESIGN:

$$W_{BAL} = (79.44)(62.5') = 4965 \text{ PLF}$$

$$P = \frac{W_{BAL} l^2}{8 a_{INT}} = 3355.4 \text{ K} \quad \frac{3355.4}{24.327} = 137.9$$

$$P_{ACTUAL} = 64(24.327) = 1556.9 \text{ K}$$

$$W_D = \frac{1556.9 (9 \times 9.67 / 12)}{(62.5')^2} = 2.3 \text{ KLF}$$

$$W_{SW} = 4.965 \text{ KLF} \quad \left. \begin{array}{l} W_D \\ W_{SW} \end{array} \right\} \frac{W_D}{W_{SW}} < 1.0 \text{ OK}$$

$$\frac{P}{A} = \frac{1556.9}{5250} = 0.2965 \text{ KSI}$$

MAX ACCEPTABLE TENDONS:

$$\frac{P}{A} \geq 300 \text{ PSI} \quad 300(5250) = 1575 \text{ K}$$

$$\hookrightarrow \frac{1575}{24.327} = 64 \text{ TENDONS}$$

TENDON PROFILE:



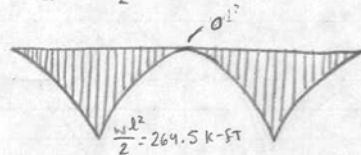
SLAB STRESSES:

$$\text{DEAD} - W_D = (99.3 + 15)(62.5') = 7.14 \text{ KLF} \quad P = (10' \times 62.5') \times 15 \text{ PSF} = 9.375 \text{ K}$$

$$\frac{W l^2}{8} - M_{\text{SUPPORT}} = 121.87 \text{ K-FT}$$



$$\text{LIVE} - W_L = (50)(62.5') = 3.13 \text{ KLF}$$



$$\text{BALANCING} - W_{BAL} = 2.3 \text{ KLF}$$



IMMEDIATELY AFTER JACKING:

• MIDSPAN

$$S_{\text{TOP}} = \frac{(-121.87 + 0)}{6125} - 0.2965 = -317 \text{ PSI}$$

$$S_{\text{BOTTOM}} = \frac{(121.87 + 0)}{6125} - 0.2965 = -277 \text{ PSI}$$

• SUPPORTS

$$S_{\text{TOP}} = \frac{(-725.2 + 194.35)}{6125} - 0.2965 = -383 \text{ PSI}$$

$$S_{\text{BUTT}} = \frac{(725.2 + 194.35)}{6125} - 0.2965 = -210 \text{ PSI}$$

WITHIN CLASS II DESIGN PARAMETERS. OK

POST-TENSIONED CONCRETE (2-WAY) (CONT'D)

STRESSES AT SERVICE LOAD:

• MIDSPAN

$$f_{TOP} = \frac{(-121.87 - 0 + 0)}{612.5} - 0.2965 = -317 \text{ PSI}$$

$$f_{BOTT} = \frac{(121.87 + 0 - 0)}{612.5} - 0.2965 = -277 \text{ PSI}$$

• SUPPORTS

$$f_{TOP} = \frac{(-725.2 - 264.5 + 194.35)}{612.5} - 0.2965 = -426 \text{ PSI}$$

$$f_{BOTT} = \frac{(725.2 + 264.5 - 194.35)}{612.5} - 0.2965 = -167 \text{ PSI}$$

WITHIN CLASS A PARAMETERS OK

ULTIMATE STRENGTH:

$$M_f = P_e = (1556.9)(2.5') = 3892.25 \text{ K-FT}$$

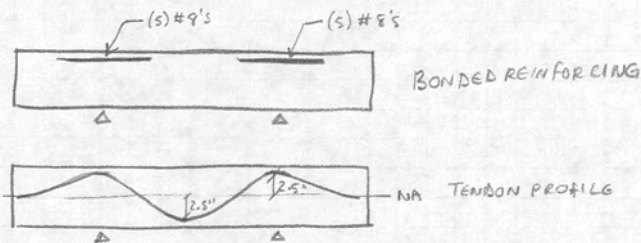
$$M_{sec} = 194.35 - \frac{3892.25}{12} = -130 \text{ K-FT} \leftarrow \text{EQUAL @ BOTH SUPPORTS}$$

MINIMUM BONDED REINFORCEMENT:

OVERALL, NO STRESSES IN TENSION $\Rightarrow$ TENSILE REINFORCEMENT IS UNNECESSARY
NEGATIVE MOMENT REGIONS:

$$A_{s,min} = 0.00075 A_{cs} \quad A_{cs} = \left\{ \frac{13+26}{2} = 19.5 \right\} \times 12' \times 7' = 5250 \text{ IN}^2$$

$$A_{s,min} = 3.94 \text{ IN}^2$$

(5) #8's TOP (3.95 IN²)
AT BOTH SUPPORTS

Southwest Student HousingTempe, Arizona
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MIN 2 SPANS (TO DCAL W/ (ANTI)LEVEL)



Goal: ELIMINATE A FEW BEAMS

LOOKED AT DIAMOND ENTERPRISES ROOF & FORM DECK

4 $\frac{1}{2}"$ DEEP DECK:

MAX SPAN = ?

$$S.W.: \left(\frac{3.65 \times 12}{144} + \frac{3 \times 4.5}{144} \right) / 1 \text{ FT} \times 110 \text{ PCF} = 40.1 \text{ PSF} \quad [\text{CONCRETE}]$$

X-SEA AREA

$$\text{MAX DECK S.W.} = 5.68 \text{ PSF} \Rightarrow 40.1 + 5.68 = 45.78 \approx 46 \text{ PSF}$$

LOADS:

• CONSTRUCTION -

DECK WEIGHT = 46 PSF

ALLOWANCE FOR STRUCTURAL STEEL = 15 PSF

• SUPERIMPOSED - 15 PSF

• LIVE - 80 PSF

FOR DECK, TOTAL (DL+LL) UNIFORM LOAD = 46 + 15 + 80 = 141 PSF

 $\rightarrow$ NO 4 $\frac{1}{2}"$ DECK GAUGE WORK FOR SINGLE SPAN $> 12'$ $\therefore$ 6" DECK?

$$\text{NEW S.W.:} \left(\frac{3.28 \times 12}{144} + \frac{3 \times 6}{144} \right) / 1 \text{ FT} \times 110 \text{ PCF} = 43.5 \text{ PSF} \quad [\text{CONCRETE}]$$

$$\text{MAX 6" DECK WEIGHT} = 6.61 \text{ PSF} \Rightarrow 43.5 + 6.61 = 50.2 \text{ PSF}$$

$$\text{NEW TOTAL (DL+LL) UNIFORM LOAD} = 50.2 + 15 + 80 = 145.2 \text{ PSF}$$

6" DEEP DECK, 14 GAUGE: FOR STRESS & DEFLECTION $\left(\frac{L}{240} \right)$, LOAD = 161 PSF FOR SINGLE 16' SPAN. $\rightarrow$ USE 6" DEEP DECK, 14 GAUGE, w/ 3 $\frac{1}{4}"$ L.W. CONCRETE.
TYP. SPAN $\leq 16'$ AS APPROPRIATE FOR BAY.

$$\frac{62.5}{16} \approx 4 \text{ SPANS (1 LESS BEAM PER 62.5' SOLUTION BETWEEN COLUMNS)}$$

 $\uparrow$ ABOVE DECK IS NON COMPOSITE (FORM).

WILL NEED TO ACCOMMODATE +3" OF DECK IN FLOOR HEIGHT OR BEAM SIZING.

2/

LONG SPAN DECK (CONT'D)GIRDER DESIGN:

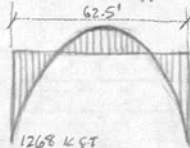
SPAN = 62.5' → LIVE LOAD REDUCTION TO 40 PSF (SEE NONCOMPOSITE CASES)

$$W_{\text{factored}} = \left[1.2(46+15+15) + 1.6(40) \right] \times 26' = 3.57 \text{ KLF}$$

$$W_{\text{LIVE}} = (40) \times 26 = 1.04 \text{ KLF}$$

$$W_{\text{TOTAL}} = (40+46+15+15) \times 26 = 3.02 \text{ KLF}$$

$$\text{MAX } M \approx \frac{Wl^2}{11} \rightarrow M_u = \frac{(3.57)(62.5)^2}{11} = 1268 \text{ K-FT}$$

→ MAINTAIN SAME FLOOR HEIGHTS ∴ W24 X
TABLE 3-2 OF AISC STEEL CONSTRUCTION MANUAL, 13TH ED.:

$$\phi_b M_{px} = 1390 \text{ FT-K FOR W24X131} > M_u = 1268 \text{ K-FT} \quad \text{OK}$$

DEFLECTIONS

$$I_x = 4020 \text{ in}^4 \quad \delta_{\text{max}} = \frac{wl^4}{384EI}$$

$$\delta_{\text{LIVE}} = \frac{(1.04/12)(62.5 \times 12)^4}{384(29000)(4020)} = 0.679'' \quad \frac{l}{360} = 2.08'' \quad \text{OK}$$

$$\delta_{\text{TOTAL}} = \frac{(3.02/12)(62.5 \times 12)^4}{384(29000)(4020)} = 1.94'' \quad \frac{l}{240} = 3.125'' \quad \text{OK}$$

BEAM DESIGN:

$$M \left(\text{---} \right) M = \frac{wl^2}{2} + Pl \quad P = 1.875$$

$$\text{TRUCK WIDTH} \approx 16' \rightarrow (16' \times 52') = 832 \text{ sq ft} \rightarrow (80) \left(0.25 + \frac{15}{\sqrt{4 \times 832}} \right) = 0.51(80) = 40.8 \text{ PSF}$$

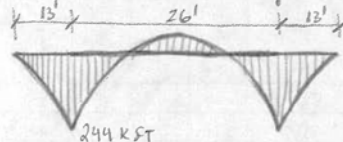
$$W_{\text{factored}} = \left[1.2(46+15+15) + 1.6(40.8) \right] \times 16' = 2.504 \text{ KLF}$$

$$W_{\text{LIVE}} = (40.8) \times 16 = 0.653 \text{ KLF}$$

$$W_{\text{TOTAL}} = (40.8+46+15+15) \times 16 = 1.969 \text{ KLF}$$

$$\text{END MOMENT } M_u = \frac{(2.504)(13')^2}{2} + (1.875 \times 1.2)(13') = 244 \text{ K-FT}$$

$$\text{MIDSPAN MOMENT} = \frac{wl^2}{8} - M_{\text{ENDS}} = \frac{(2.504)(26')^2}{8} - 244 = -32.4 \text{ K-FT}$$

→ MAINTAIN FLOOR HEIGHTS → W14 X
 $\phi_b M_{px} = 383 \text{ K-FT FOR W14X61} > M_u = 244 \text{ K-FT}$

$$M_{\text{LIVE}} = \frac{(0.653)(13')^2}{2} = 55.2 \text{ K-FT}$$

$$M_{\text{TOTAL}} = \frac{(1.969)(13')^2}{2} + 1.875(13) = 192.3 \text{ K-FT}$$

$$I_x = 640 \text{ in}^4$$

$$\delta_x = \frac{wl}{48EI} \left[\frac{5}{8}l^3 - \frac{6Ml}{W} \right] @ \text{MIDSPAN}$$

↑ SIMPLIFIED B/C END MOMENTS ARE SYMMETRIC
& MAX MIDSPAN δ IS IN THE VERY MIDDLE ($\frac{l}{2}$)

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LONG SPAN DECK (CONT'D)BEAM DESIGN (CONT'D)DEFLECTIONS

$$\begin{aligned} \delta_{LIVE} &= \frac{(0.453/12)(26 \times 12)}{48(29000)(640)} \left[\frac{5}{8}(26 \times 12)^3 - \frac{6(55.2 \times 12)}{0.683/12}(26 \times 12) \right] = -0.073'' < 0.867'' \quad \text{OK} \\ \delta_{TOTAL} &= \frac{(1.469/12)(26 \times 12)}{48(29000)(640)} \left[\frac{5}{8}(26 \times 12)^3 - \frac{6(122.3 \times 12)}{1.769/12}(26 \times 12) \right] = -0.399'' < 1.3'' \quad \text{OK} \end{aligned}$$

$$\delta_{LIVE} = \frac{wL^4}{384EI} + \frac{Pa^3}{3EI} \quad \text{(IS APPROPRIATE)}$$

$$\delta_{LIVE} = \frac{(0.653/12)(13 \times 12)^4}{8(29000)(640)} = 0.217'' < 0.433'' \quad \text{OK}$$

$$\delta_{TOTAL} = \frac{(0.659/12)(13 \times 12)^4}{8(29000)(640)} + \frac{1.875(13 \times 12)^3}{3(29000)(640)} = 0.749'' > 0.65'' \quad \text{NG}$$

W14x74? $I = 795 \text{ in}^4$

$$\begin{aligned} \delta_{LIVE} &= (-0.073) \left(\frac{640}{795} \right) = -0.059'' < 0.867'' \quad \text{OK} \\ \delta_{TOTAL} &= (-0.399) \left(\frac{640}{795} \right) = -0.321'' < 1.3'' \quad \text{OK} \end{aligned}$$

$$\begin{aligned} \delta_{LIVE} &= (0.217) \left(\frac{640}{795} \right) = 0.173'' < 0.433'' \quad \text{OK} \\ \delta_{TOTAL} &= (0.749) \left(\frac{640}{795} \right) = 0.603'' < 0.65'' \quad \text{OK} \end{aligned}$$

→ USE W14x74 FOR BEAMS @ $\frac{62.5}{4} \approx 15'-6"$ SPACING (4 BEAMS / 62.5' SPAN)

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TECH 1 (REVISION 10.11.2011)	APPENDICES - APPENDIX D SOUTHWEST STUDENT HOUSING
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CALCULATED LOADS (CONTINUED)

- SEISMIC LOADS

FRAME SYSTEM: REINFORCED CONCRETE SHEAR WALLS

$R=4.0$
 $I=1.0$

$C_s = \frac{S_{DS}}{(R/I)} \geq 0.01$ $C_s \leq \frac{S_{D1}}{C_u T_a (R/I)}$ FOR $T \leq T_L$

$T_a = (C_a) h_n^{0.75} \approx 1.1 \text{ SEC}$ $C_s = 16\%, g$
 $0.02 \rightarrow 208$ $S_1 = 9\%, g$
 $T_L = 6.24 \text{ SEC}$ } FIGURES FROM ASCE 7-05, CH 22

$F_x = C_{vx} V$ $C_{vx} = \frac{w_x h_x}{\sum_{i=1}^n w_i h_i}$

SITE CLASS C (FROM GEOTECH REPORT)

$f_a = 1.2$ $f_v = 1.7$

$S_{ms} = f_a S_s = 1.2(0.16) = 0.192$ $S_{DS} = \frac{2}{3} S_{ms} = 0.128$
 $S_{m1} = f_v S_1 = 1.7(0.09) = 0.153$ $S_{D1} = \frac{2}{3} S_{m1} = 0.0907 < 0.1$
 $T_0 = 0.2 \frac{S_{D1}}{S_{DS}} = 0.2 \frac{0.0907}{0.128} = 0.142$ $C_u = 1.7$
 $T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.0907}{0.128} = 0.708$

$C_s = \frac{0.128}{4/1.0} = 0.032 \leq \frac{0.0907}{1.7(9.1)(4/1.0)} = 0.0121 \Rightarrow C_s = 0.0121$

$W_{TOT} = \sum_{i=1}^{40} w_i$ $w_i = (C_{DL} + S_{DL}) A_{FLOOR} + \text{CURTAIN WALL (PERIMETER)} = (9+15)13000 + 15(604) = 971.1 \text{ K}$
 PER FLOOR

$V = 0.0121 (971.1 \times 20) = 235.5 \text{ KIPS}$ (SEE EXCEL TABLE FOR VERTICAL DISTRIBUTION)

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Appendix B –

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STRUCTURAL DEPTH	DESIGN LOADS	
	<u>DESIGN LOADS</u> - ASSUME SIMILAR GRAVITY LOADS TO EXISTING CONDITIONS → <u>SNOW LOADS</u>: $P_g = 20 \text{ PSF}$ (ASCE 7-05, FIG. 7-1) FLAT ROOF, $P_s = 0.7 C_e C_t I P_g \geq I P_g$ (ASCE 7-05, EQU. 7-1) SURFACE ROUGHNESS B (URBAN/SUBURBAN) EXPOSURE CATEGORY B - ASSUME ROOF IS PARTIALLY EXPOSED $\left\{ \begin{array}{l} C_e = 1.0 \end{array} \right.$ (ASCE 7-05, § 6.5.6, TABLE 7-2) $C_t = 0.85$ (CONTINUOUSLY HEATED BUILDING) (TABLE 7-3) $I = 1.0$ (OCCUPANCY II) (TABLE 7-4) $\Rightarrow P_s = 0.7 (1.0)(0.85)(1.0)(20) \geq (1.0)(20)$ $= 11.9 \geq 20 \text{ PSF} \quad \therefore P_s = 20 \text{ PSF}$ <u>EXISTING GRAVITY LOADS</u>: DEAD LOADS: CONSTRUCTION = 59 PSF SUPERIMPOSED = 15 PERIMETER WALL = 15 PSF LIVE LOADS = 80 PSF <u>LATERAL LOADS</u>: <u>WIND</u> SAME WIND SPEED, SAME EXPOSURE CATEGORY, ETC AS EXISTING $\therefore$ CAN USE PREVIOUSLY CALCULATED WIND LOADS (SEE TABLE 2 FROM ALL TECHNICAL REPORTS) BUILDING HEIGHT: 208', 19' FIRST STORY, 10' SUBSEQUENT STORIES <u>BASE SHEARS</u> PLAN ← 120 K ↓ 565 K <u>SEISMIC</u> - ASSUME STIFF SOIL $\therefore$ SITE CLASS D (ASCE 7-05, TABLE 20.3-1) $S_s = 60\%g \rightarrow F_a = 1.32$ (ASCE 7-05, TABLE 11.4.1) $S_1 = 15\%g \rightarrow F_v = 2.2$ (TABLE 11.4.2) $T_L = 12 \text{ SEC}$ (ASCE 7-05, FIGS 22-1, 22-2, 22-15) FRAME - ORDINARY REINFORCED CONCRETE SHEAR WALLS, $R = 4.0$ (ASCE 7-05, TABLE 12.2-1) ↑ NOT PERMITTED IN SDC D NEED TO USE SPECIAL REINFORCED CONCRETE SHEAR WALLS IN ACCORDANCE w/ ASCE 7-05 12.2.5.4: 1. NO EXTREME TORSIONAL IRREGULARITY (TYPE 1b) 2. SHEAR WALLS IN ANY ONE PLANE SHALL RESIST NO MORE THAN 60% OF TOTAL SEISMIC FORCES IN EACH DIRECTION, NEGLECTING ACCIDENTAL TORSION	Column TO NOTE WHERE ASSUMPTIONS ARE MADE.

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STRUCTURAL DEPTH	DESIGN LOADS	
ASSUME: MEETS ASCE 7-05, § 12.2.5.9 REQUIREMENTS (NEED TO CHECK!) ∴ SPECIAL REINFORCED CONCRETE SHEAR WALLS, $R=5.0$ (MAX HEIGHT = 240') (→ ASCE 7-05, TABLE 12.2-1) CHAPTER 9 FACTORS - $S_{M1} = 1.32 (0.6) = 0.792$ $S_{D1} = \frac{2}{3} (0.792) = 0.528$ $S_{M2} = 2.2 (0.15) = 0.33$ $S_{D2} = \frac{2}{3} (0.33) = 0.22$ $S_{D1} > 0.3$? SDC D FOR OCCUPANCY CATEGORY II $S_{D1} > 0.2$ (ASCE 7-05, TABLE 11.6-1) $T_0 = 0.2 \frac{0.22}{0.528} = 0.0833 \text{ sec}$ (ASCE 7-05 § 11.4.2) $T_1 = \frac{0.22}{0.528} = 0.4167 \text{ sec}$ CHAPTER 12 FACTORS - $C_u = 1.48$ (ASCE 7-05, TABLE 12.8-1) $C_t = 0.2$ FOR OTHER STRUCTURES (ASCE 7-05, TABLE 12.8-2) $\chi = 0.75$ → $T_a = (0.2) (208')^{0.95} \approx 1.1 \text{ sec}$ $T \leq C_u T_a = 1.48 (1.1) = 1.628 \text{ sec}$ $3.5 T_1 = 3.5 (0.4167) = 1.458 \text{ sec}$ → ASSUME $T < 3.5 T_1$ TO USE EQUIVALENT LATERAL FORCE METHOD (NEED TO CHECK!) IF USING ELF (ASCE 7-05, § 12.9) $C_s = \frac{S_{D1}}{R/I} = \frac{0.528}{5/1.0} = 0.1056 > 0.01 \checkmark$ $C_s \leq \frac{S_{D2}}{3.5 T_1 (5/1.0)} = \frac{0.22}{1.458 (5/1.0)} = 0.0302 \times$ USED IN PLACE OF C_{s1} BECAUSE $3.5 T_1 < C_u T_a$ ∴ $C_s = 0.0302$ $V = C_s W_{TOT}$ $C_0 \leq \frac{0.22}{1.628 (5/1.0)} = 0.027$		2/3 2/4

DON'T USE
THIS REQ.
 $C_s \geq 2.4\%$

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STRUCTURAL DEPTH $\frac{2}{3}$	DESIGN LOADS $\frac{2}{3}$	$\frac{2}{3}$
So $V = C_s W_{TOT}$ $W_{TOT} = W_{ROOF} + 19 W_{FLOOR TYP} + W_{FIRST FLOOR}$ $W_{ROOF} = (59 \text{ psf} + 15 \text{ psf})(250' \times 52') + \frac{16}{12} (2') (25' + 19') (2) (144 \text{ psf}) = 1046.5 \text{ K}$ $W_{FLOOR TYP} = (20 \text{ psf} + 59 \text{ psf} + 15 \text{ psf})(250' \times 52') + (15 \text{ psf})(10') (250' + 52') + 2 + \frac{16}{12} (10') (25' + 19') (2) (144 \text{ psf})$ $= 1481.6 \text{ K}$ $W_{FIRST FLOOR} = W_{FLOOR TYP} + \frac{16}{12} (9') (25' + 19') (2) (144 \text{ psf}) = 1623.6 \text{ K}$ $W_{TOT} = 1046.5 + 1481.6 \times 19 + 1623.6 = 29399 \text{ K}$ $V = (0.0302)(29399) = 896.34 \text{ K}$ (SEE TABLE @ END FOR DESIGNATION + ACCIDENTAL TORSION) ASSUME NO OPENINGS IN CORG EXCEPT CORRIDOR $\phi V_n = 0.75 (1.0 V_u)$ $V_u = 2.2 \sqrt{f_c'} A_{SHALL, CRITICAL DIRECTION}$ $A_{SHALL} = (25' - 6' CORRIDOR) (2' / 12) \times 6 \text{ WALLS} \times t_{WALL} = 1368 t_{WALL}$ $\phi V_n = (0.75)(1.0)(2 \times 1.0 \times \sqrt{4000} 1368 t) \geq 896.34 \text{ K}$ @ 4000 PSI: $t_{WALL} \geq 6.25"$ $\geq 796.34 \text{ K}$ @ 5000 PSI: $t_{WALL} \geq 4.82"$ MODEL TRIAL WALLS IN ETABS. USE NON-ITERATIVE, MASS-BASED P-Delta. MODEL MUST HAVE... Mode 1 $\leq 1.958 \text{ sec}$ $\delta_{MAX} \leq 4.16"$ (2%) $\delta_{MAX} = 0.029"$ WITH CRITICAL LOAD COMBINATION $1.2D + 1.0E + L + 0.2S$		

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MASSSES TO MODEL FOR 16" WALLS @ CRITICAL LOAD COMBO:

$$1.2D + 1.0E + 1.0L + 0.2S$$

$$M_{\text{ROOF}} = \left[\frac{1046.6 \text{ K}}{250 \times 52 \times 144} \times 1.2 + \frac{0.020}{144} \times 1.0 \right] \frac{1}{32.2} = 2.1696 \times 10^{-5}$$

$$M_{\text{FLR TYP}} = \left[\frac{1491.6 \text{ K}}{250 \times 52 \times 144} \times 1.2 + \frac{0.070}{144} \times 1.0 \right] \frac{1}{32.2} = 4.6748 \times 10^{-5}$$

$$M_{\text{FLR ONE}} = \left[\frac{1653.6}{250 \times 52 \times 144} \times 1.2 + \frac{0.080}{144} \times 1.0 \right] \frac{1}{32.2} = 9.977 \times 10^{-5}$$

16" @ 8000 PSI: MODE 1 = 1.2293 SEC.

$$2 = 1.2224 \text{ SEC}$$

$$3 = 1.0430 \text{ SEC}$$

16" @ 4000 PSI: MODE 1 = 1.4648 SEC

$$2 = 1.4565 \text{ SEC}$$

$$3 = 1.2422 \text{ SEC}$$

MASSSES TO MODEL FOR 8" WALLS @ CRITICAL LOAD COMBO:

$$W_{\text{ROOF}} = (59 + 15)(250 \times 52) + \frac{8}{12}(6')(25 \times 14)(2)(144 \text{ PCF}) = 1004.24 \text{ K}$$

$$M_{\text{ROOF}} = \left[\frac{W_{\text{ROOF}}}{250 \times 52 \times 144} \times 1.2 + \frac{0.020}{144} \times 0.2 \right] \frac{1}{32.2} = 2.0853 \times 10^{-5}$$

$$W_{\text{FLR TYP}} = (20 + 59 + 15)(250 \times 52) + (15)(10)(250 \times 52)(2) + \frac{8}{12}(10)(25 \times 14)(2)(144) = 1397.1 \text{ K}$$

$$M_{\text{FLR TYP}} = \left[\frac{W_{\text{FLR TYP}}}{250 \times 52 \times 144} \times 1.2 + \frac{0.070}{144} \times 1.0 \right] \frac{1}{32.2} = 4.507 \times 10^{-5}$$

$$W_{\text{FLR ONE}} = W_{\text{FLR TYP}} + \frac{8}{12}(9')(25 \times 14)(2)(144 \text{ PCF}) = 1473.1 \text{ K}$$

$$M_{\text{FLR ONE}} = \left[\frac{W_{\text{FLR ONE}}}{250 \times 52 \times 144} \times 1.2 + \frac{0.070}{144} \times 1.0 \right] \frac{1}{32.2} = 4.658 \times 10^{-5}$$

8" @ 8000 PSI: MODE 1 = 1.7470 SEC

$$2 = 1.7371 \text{ SEC}$$

$$3 = 1.4803 \text{ SEC}$$

8" @ 4000 PSI: MODE 1 = 2.0860 SEC

$$2 = 2.0741 \text{ SEC}$$

$$3 = 1.7658 \text{ SEC}$$

$$V_{\text{BASE}} = (0.080 + (0.062 \times 14)) = 0.854 \text{ IN}$$

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NEW MODEL ANALYSIS PARAMETERS:

- 18" WALLS
- 3' COUPLING BMS
- 6000 KSI CONCRETE
- $E_c = \frac{1}{2} 57000 \text{ PSI}$
- $M_{ROOF} = 2.206 \text{ E-5}$
- $M_{1ST} = 5.427 \text{ E-5}$
- $M_{2ND} = 4.752 \text{ E-5}$
- RIGID DIAPHRAGM

(NOT TAKING INTO ACCOUNT EV)

WEIGHTS:

$$W_{ROOF} = (59+15)(250 \times 52) + \left[\frac{18}{12}(5')(25+19)(2)(3) + \frac{18}{12}(3')(6')(6) \right] 150 = 1283.3^k$$

SHEAR WALLS COUPLING BM

→ 2142

$$W_{TYP FLR} = (59+15+20)(250 \times 52) + (15)(10)(250 \times 52)(2) + \left[2142 + \frac{18}{12}(5)(25+19)(2)(3) \right] 150 = 1930.9^k$$

→ 4122

$$W_{1ST FLR} = W_{TYP FLR} + \frac{18}{12}(8')(25+19)(2)(3)(150) = 2406.1^k$$

MASSES (BASED ON 1.2D + 1.0E + 1.0L + 0.25 LOAD COMBINATION):

$$M_{ROOF} = \left[\frac{W_{ROOF}}{250 \times 52 \times 144} \times 1.2 + \frac{0.020 \text{ KSF}}{144} \times 0.2 \right] \frac{1}{32.2} = \frac{1.641 \times 10^{-5}}{12} = 2.2008 \text{ E-6}$$

$$M_{TYP FLR} = \left[\frac{W_{TYP FLR}}{250 \times 52 \times 144} \times 1.2 + \frac{0.080 \text{ KSF}}{144} \times 1.0 \right] \frac{1}{32.2} = \frac{5.5693 \times 10^{-5}}{12} = 4.6411 \text{ E-6}$$

$$M_{1ST FLR} = \left[\frac{W_{1ST FLR}}{250 \times 52 \times 144} \times 1.2 + \frac{0.080 \text{ KSF}}{144} \times 1.0 \right] \frac{1}{32.2} = \frac{6.5153 \times 10^{-5}}{12} = 5.4279 \text{ E-6}$$

DESIGN LOADS:

$$X = C_{NS} W_{TOT} = 0.0294 \times 0.0302 (12 \times 3.3 + 18 \times 1930.9 + 2406.1) = 1.50^k$$

SEE 03-07-2012 SPREADSHEET FOR FORCE DISTRIBUTION + ACCIDENTAL MOMENT +
FLOOR DIAPHRAGM FORCE CALC

	STRUCTURAL DEPTH 6/31	CORE DESIGN 3/10	6/1
	NEW MODEL: - 16" WALLS - 2' CORNER WALLS - 4000 PSI CONCRETE - RIGID DIAPHRAGM	ASSUME CRACKED PROPERTIES (USE 0.5 I_g FOR WALLS, 0.25 I_g FOR BEAMS, AS ADVISED BY PREVIOUS ADVISEE)	
	WEIGHTS:	ROOF = $(59+15)(250 \times 52) + \left[\frac{16}{12}(5)(25+19) \times 2 \times 3 + \frac{16}{12}(3)(6) \times 6 \right] 150 = 1247.6 \text{ K}$ 1904 TYPICAL FLOOR = $(59+15+20)(250 \times 52) + (15)(10)(250+52)(2) + \left[1904 + \frac{16}{12}(5)(25+19) \times 2 \times 3 \right] 150 = 1862.2 \text{ K}$ 3664 FIRST FLOOR = TYPICAL FLOOR + $\frac{16}{12}(8)(25+19)(2)(3)(150) = 2284.6 \text{ K}$	
	MASSSES:	ROOF = $\frac{1247.6}{250 \times 52} \left(\frac{1}{32.2} \times \frac{1}{12^3} \right) = 1.725 \text{ E-6}$ TYPICAL FLOOR = $\frac{1862.2}{250 \times 52} \left(\frac{1}{32.2} \times \frac{1}{12^3} \right) = 2.574 \text{ E-6}$ FIRST FLOOR = $\frac{2284.6}{250 \times 52} \left(\frac{1}{32.2} \times \frac{1}{12^3} \right) = 3.158 \text{ E-6}$	
	DESIGN LOADS:	$V = C_e W_{TOT} = (0.027) \underbrace{(1247.6 + 1862.2 \times 18 + 2284.6)}_{37051.8 \text{ K}} = 1006.41$	
	MODE 1 =		

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STRUCTURAL DEPTH	CORE DESIGN	
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$V_{BASE} = \frac{116}{1001.1} K \rightarrow$ INITIAL CORE SIZING CHECK: $\phi V_n = 0.75 (1.0 V_c) \quad V_c = 2 \lambda \sqrt{f'_c} A$ $A = [25-6] \times 12 \times 2 t_{WALL} \times 3 CORES = 1368 t$ $V_c = 2 (1.0) \sqrt{4000} (1368 t) = 173.0 K t$ $\phi V_n = 0.75 (173.0 K t) = 129.78 K t \geq 1001.1 K \times 1.2$ $t \geq 9.26"$ $t = 16" \quad \underline{OK}$ $V_c = 17.3 K (16) = 276.8 K \leftarrow$ OVERALL PER CORE (IN WEAK DIRECTION), $V_c = 922.7 K$ $V_{BASE} \text{ PER CORE} = \frac{1.2 \times 1001}{3} = 400.56 K < V_c \text{ PER CORE}$ SHEAR RDNF: $V_u - \underbrace{2 \sqrt{f'_c} A_g}_{V_c} = V_s = 0 \therefore$ USE MIN. REINFORCEMENT FOR SHEAR MOMENT REINFORCEMENT: TOTAL CALCULATED MOMENT = $151215 \text{ FT-K} \times 1.2 \text{ (TORSIONAL EFFECTS)} = 181458 \text{ FT-K}$ $\rightarrow \frac{181458}{6 \text{ WEBS}} = 30243 \text{ FT-K/WEB}$ $\frac{30243}{25' \text{ LEVER ARM}} = 1209.72 K$ IN TENSION PRODUCED BY EQ. MOMENT AXIAL LOAD COUNTERACTS T. AXIAL LOAD = $\frac{W_{ELDG}}{A_{CROSSSECTIONAL \text{ TOTAL}}} \times A_{CROSSSECTIONAL, 1 \text{ WEB}}$ $= \frac{38445.6}{264 \times \frac{16}{12}} \times (25 \times \frac{16}{12}) = 3640.7 K \text{ PER WEB} > 1209.72 K$ WHOLE CORE IS IN COMPRESSION, NO NEED FOR T REINFORCEMENT $\therefore$ USE MIN. REINFORCEMENT FOR MOMENT WHAT FRACTION IS AXIAL LOAD? $3641 = x (4 \text{ WEBS}) (25 \times 12 \times 16) \quad x = 0.193$ AXIAL IS <u>$0.193 f'_c A_g$</u>		

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AMPAD	CORE REINFORCEMENT: 11.10.8 $V_u \leq 0.5 \phi V_c$? <u>NO</u> $\rho_t \geq 0.0025$ $s \leq \begin{cases} \frac{L_w}{5} = \frac{9.512}{5} = 23 \\ 3h = 3 \times 16 = 48 \\ 19" \end{cases}$ $\rho_1 \geq 0.0025$ $s \leq 19"$ 14.3.2 $\rho_2 \geq \begin{cases} 0.0012 \text{ FOR } \#5 \text{ OR LESS} \\ 0.0015 \text{ OT} \end{cases}$ $\rho_t \geq \begin{cases} 0.002 \text{ FOR } \#5 \text{ OR LESS} \\ 0.0025 \text{ OT} \end{cases}$ OR $\rho_2 \>\rho_t$ AS CALCULATED IN 11.10.8 & 11.10.9 $\Rightarrow \rho_t = \rho_2 \geq 0.0025$ IN 2 LAYERS (14.3.4) 14.3.6 - NO TIE IF $A_s \leq 0.01 A_g$ OR IS NOT NEEDED FOR COMPRESSION APPLICABLE? NOT FOR DESIGN 4. 21.7.4.1 $V_n = A_{cv}(\alpha_c \sqrt{f_c} + \rho_t f_c)$ $\alpha_c = \begin{cases} 3.0 \text{ } h_w/h_u \leq 1.5 \\ 2.0 \text{ } h_w/h_u > 2.0 \end{cases}$ $\downarrow$ ALREADY TOOK V_n AS $A_{cv}(\alpha_c \sqrt{f_c})$, $\alpha_c = 2.0$ 21.7.4.4 $V_n \leq 8 A_{cv} \sqrt{f_c}$ (OVERALL) $V_n \leq 10 A_{cv} \sqrt{f_c}$ (PER ACR) 21.7.6 - BOUNDARY ELEMENTS $\leftarrow$ EXTEND FROM CRITICAL SECTION A TO B $\begin{cases} L_w \\ h_u/4 \end{cases}$ $c \geq \frac{L_w}{600 (h_u/h_w)}$ LARGEST MOMENT AXIS DEPTH FOR FACED WALL + MOMENT @ S_u (DSR) $S_u/h_u \geq 0.004$	11.10.3 * $V_n \leq 10 \sqrt{f_c} h_d = 10 \sqrt{4000} (16) (0.8225712)$ $= 3429.6 \text{ k} \leftarrow \text{PER CORE}$ $V_n = 0.75 V_u = 0.75 (4227.2) < 3429.6$ <u>OK</u> 11.10.5 * $V_c = 2 \sqrt{f_c} h_d = 2 \sqrt{4000} (16) (0.8225712)$ $= 728.16 \text{ k} > 400.56 \text{ k}$ <u>OK</u> $0.75 V_c = 546.12 > 400.56 \text{ k}$ <u>OK</u> ALL AXIAL COMPRESSION $\rho = \frac{A_s}{b \cdot d} \rightarrow \text{PER LINEAL FOOT OF WALL} \dots$ $0.0025 = \frac{A_s}{16" \times 12"} \Rightarrow A_s = 0.48 \text{ in}^2$

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b) IN FLANGED, BOUNDARY ELEMENT EXTENDS 12" INTO FLANGE

CHOOSE TO INCLUDE BOUNDARY ELEMENTS FOR STRUCTURAL INTEGRITY WHEN HOLDING GRAVITY LOADS. THE BOUNDARY ELEMENTS ESSENTIALLY FUNCTION AS A COLUMN.

$$\rho_{min, COLUMN} = 1\% \text{ (BOUNDARY ELEMENT)}$$

$$\rho_{min, EVERYTHING ELSE} = 0.25\% \text{ (WALLS)}$$

REINFORCEMENT IS IN 2 LAYERS

-NORMAL LONGITUDINAL REINFORCEMENT:

$$\rho = 0.0025 \rightarrow A_s = 0.48 \text{ in}^2/\text{ft} \rightarrow (12) \#4S @ 6" \quad A_s = 0.795 \text{ in}^2$$

-BOUNDARY LONGITUDINAL REINFORCEMENT:

$$\rho = 1\% \rightarrow A_s = 6.4 \text{ in}^2 / 640 \text{ in}^2$$

$$6.4 + \left(\frac{16 + 12 \times 12}{12} \right) \times 0.48 = 8 \text{ in}^2$$

USE #8'S TO LIMIT CIRCLES

$$8 \text{ in}^2 = (4) \#8'S \text{ MINIMUM}$$

$$(16) \#8'S \rightarrow A_s = 12.57 \text{ in}^2$$

$$\rho = 1.96\%$$

-TRANSVERSE REINFORCEMENT

$$A_{tr} = \frac{0.09 S_b f_c}{f_y}$$

$$S_b = 16 - 1.5 \times 2 = 13"$$

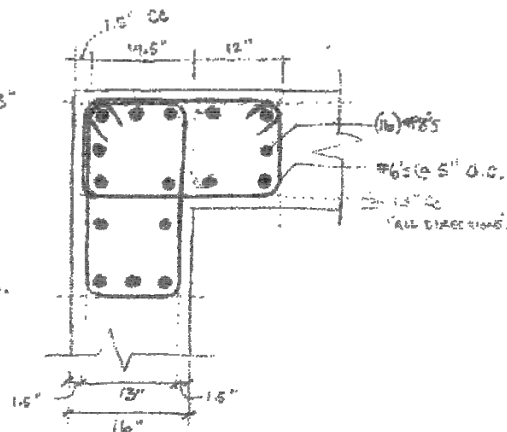
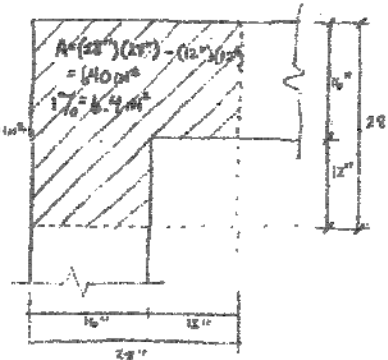
$$f_c = 4 \text{ ksi}$$

$$f_y = 60 \text{ ksi}$$

$$S = \frac{16}{3} = 5.33" \rightarrow S = 5"$$

$$A_{tr} = \frac{0.09(5)(13)(4)}{(60)} = 0.39 \text{ in}^2$$

USE #6'S ($A_s = 0.44$) @ 5" O.C.



BOUNDARY ELEMENT CAN BE DISCONTINUED WHEN COMPRESSIVE STRESS $< 0.15 f_c$ (ACI 318.05 21.7.6.3)

[SEE PG OF LDR DESIGN FOR CIRCLES]

① BASE, COMPRESSIVE STRESS $\approx 0.188 f_c$ (UNDER GRAVITY ONLY)

W/ GR MOMENT: 1209.72 k in C

$$+ \frac{3641 \text{ k}}{9850.72 \text{ in C TOTAL}} \rightarrow 0.253 f_c$$

$$M_{END} = 151157.1 \times 1.2 = 181387.52 \rightarrow \text{AXIAL DUE TO GRAVITY} = 1209.26 \text{ k}$$

$$M_{3RD} = 151010.1 \times 1.2 = 181212.12 \rightarrow 1209.07 \text{ k}$$

$$M_{4TH} = 150688.6 \times 1.2 = 180826.32 \rightarrow 1205.51 \text{ k}$$

$$M_{5TH} = 150103.2 \times 1.2 = 180123.84 \rightarrow 1200.83 \text{ k}$$

$$M_{6TH} = 149152.2 \times 1.2 = 178982.64 \rightarrow 1193.12 \text{ k}$$

$$M_{7TH} = 147722.4 \times 1.2 = 177266.98 \rightarrow 1181.73 \text{ k}$$

$$\text{AXIAL DUE TO GRAVITY} = 3412.93 \text{ k}$$

$$= 3229.98 \text{ k}$$

$$= 3097.13 \text{ k}$$

$$= 2864.27 \text{ k}$$

$$= 2681.42 \text{ k}$$

$$= 2498.56 \text{ k}$$

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STRUCTURAL DEPTH	CORE DESIGN	
AXIAL LOAD ON n^{th} FLOOR: 2ND = 4622.1 K $\rightarrow$ 0.241% 3RD = 4439.1 K $\rightarrow$ 0.231% 4TH = 4252.6 K $\rightarrow$ 0.222% 5TH = 4065.1 K $\rightarrow$ 0.212% 6TH = 3874.7 K $\rightarrow$ 0.202% 7TH = 3680.4 K $\rightarrow$ 0.192% AXIAL STRESS: 2ND = 4622.1 K $\rightarrow$ 0.241% 3RD = 4439.1 K $\rightarrow$ 0.231% 4TH = 4252.6 K $\rightarrow$ 0.222% 5TH = 4065.1 K $\rightarrow$ 0.212% 6TH = 3874.7 K $\rightarrow$ 0.202% 7TH = 3680.4 K $\rightarrow$ 0.192% M_{11TH} = 134578.8 X 1.2 = 161506.56 $\rightarrow$ 1076.7 K M_{12TH} = 128713.4 X 1.2 = 154456.1 $\rightarrow$ 1029.71 K M_{13TH} = 121435.4 X 1.2 = 145720.5 $\rightarrow$ 991.8 K AXIAL DUE TO GRAVITY = 1767.2 K AXIAL DUE TO GRAVITY = 1584.33 K AXIAL DUE TO GRAVITY = 1491.5 K AXIAL LOAD @ n^{th} FLOOR: 12TH = 2614.04 K $\rightarrow$ 0.136% 13TH = 2372.3 K $\rightarrow$ 0.124% 11TH = 2949.28 K $\rightarrow$ 0.149% AXIAL STRESS: 12TH = 2614.04 K $\rightarrow$ 0.136% 13TH = 2372.3 K $\rightarrow$ 0.124% 11TH = 2949.28 K $\rightarrow$ 0.149% BOO! FIRST FLOOR W/ MAX AXIAL COMPRESSIVE STRESS UNDER GRAVITY + EQ UNDER 0.15% $\rightarrow$ EXTEND BOUNDARY REINFORCEMENT THRU 11TH FLOOR (118 FT)	AXIAL STRESS: 2ND = 4622.1 K $\rightarrow$ 0.241% 3RD = 4439.1 K $\rightarrow$ 0.231% 4TH = 4252.6 K $\rightarrow$ 0.222% 5TH = 4065.1 K $\rightarrow$ 0.212% 6TH = 3874.7 K $\rightarrow$ 0.202% 7TH = 3680.4 K $\rightarrow$ 0.192% AXIAL DUE TO GRAVITY = 1767.2 K AXIAL DUE TO GRAVITY = 1584.33 K AXIAL DUE TO GRAVITY = 1491.5 K BOO! FIRST FLOOR W/ MAX AXIAL COMPRESSIVE STRESS UNDER GRAVITY + EQ UNDER 0.15% $\rightarrow$ EXTEND BOUNDARY REINFORCEMENT THRU 11TH FLOOR (118 FT)	AXIAL DUE TO GRAVITY = 1767.2 K AXIAL DUE TO GRAVITY = 1584.33 K AXIAL DUE TO GRAVITY = 1491.5 K
AXIAL LOAD @ n^{th} FLOOR: 12TH = 2614.04 K $\rightarrow$ 0.136% 13TH = 2372.3 K $\rightarrow$ 0.124% 11TH = 2949.28 K $\rightarrow$ 0.149% AXIAL STRESS: 12TH = 2614.04 K $\rightarrow$ 0.136% 13TH = 2372.3 K $\rightarrow$ 0.124% 11TH = 2949.28 K $\rightarrow$ 0.149% BOO! FIRST FLOOR W/ MAX AXIAL COMPRESSIVE STRESS UNDER GRAVITY + EQ UNDER 0.15% $\rightarrow$ EXTEND BOUNDARY REINFORCEMENT THRU 11TH FLOOR (118 FT)	AXIAL STRESS: 12TH = 2614.04 K $\rightarrow$ 0.136% 13TH = 2372.3 K $\rightarrow$ 0.124% 11TH = 2949.28 K $\rightarrow$ 0.149% BOO! FIRST FLOOR W/ MAX AXIAL COMPRESSIVE STRESS UNDER GRAVITY + EQ UNDER 0.15% $\rightarrow$ EXTEND BOUNDARY REINFORCEMENT THRU 11TH FLOOR (118 FT)	BOO! FIRST FLOOR W/ MAX AXIAL COMPRESSIVE STRESS UNDER GRAVITY + EQ UNDER 0.15% $\rightarrow$ EXTEND BOUNDARY REINFORCEMENT THRU 11TH FLOOR (118 FT)
DEFLECTION CHECK: (SEE NEXT PAGE FOR INITIAL CALC + ATTEMPTS TO REDUCE DEFLECTION)		

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STRUCTURAL DEPTH

CORE DESIGN

• PAUL - USE RUBBER AS SLEEVE?

• MCMASTER - CORRELATION OF TORQUE & AXIAL FORCE IN BOLTS
 ↳ CHECK BOLT STR TOO (LEPAGE THINKS THEN ARE ASTM A490, $f_y = 130 \text{ ksi}$)
 BOLTS ARE 7/8" ?
 [NEED TO VERIFY] OR A325

- TAKE A LOOK @ NISSEN

a) TURN ON AUTO EQUIV. LFT. FORCE METHOD → LOAD CASES EQ > AUTO

$$b) T = \frac{2\pi}{15\pi} \sqrt{\frac{11 \text{ in}^4}{E I}}$$

$$T = 1.9 \text{ sec} \pm 10\%$$

$$11. \text{ MASS / UNIT HEIGHT} = \frac{0.150 \text{ KIP} (250' \times 12')}{32.2 \left(\frac{208 \times 12^4}{2.0 \text{ STIFF}} \right)} = 0.04 \frac{\text{K} \cdot \text{sec}^2}{\text{ft}^2}$$

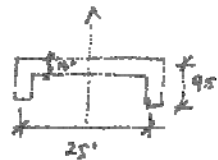
(UNIT = STIFF)

$$I = 5400 \text{ ft}^4 \rightarrow 74 \text{ in}^4$$

$$\omega = 8400$$

$$h = 208' \rightarrow 74 \text{ in}$$

$$\rightarrow \frac{0.150 \text{ KIP} (250' \times 12')}{32.2 \text{ IN/sec}^2 \left(\frac{208' \times 12^4}{2.0 \text{ STIFF}} \right)} = 0.0405$$



$$I = \frac{bh^3}{12} + Ad^2$$

$$= \frac{16(95 \times 12)^3}{12} + (16 \times 95 \times 12) (12.5 \times 12)^2 = 2089 \text{ ft}^4$$

$$I_{2 \text{ SIDES}} = 9148.9 \text{ ft}^4$$

$$I_{2 \text{ SIDES}} = \frac{(2 \text{ SIDES}) (12)^3}{12} = 9.99 \text{ ft}^4$$

$$I_c = 9158.9 \text{ ft}^4$$

GET I

VERIFY #S IN OTHER SHEET

GET Δ IN MODEL & VERIFY STIFFNESS

$$\Delta = \frac{PL^3}{3EI}$$

(CANTILEVER - 56)

$$\Delta = \frac{1000 \text{ K} (208 \times 12)^3}{3(3600) (9158.9 \times 12^4 \times 6)} = 2.79 \text{ IN}$$

APPROX DEFLECTION w/ 1000 K LOAD IN Y-DIR
@ ROOFTOP.

$$T = \frac{2\pi}{3.52} \sqrt{\frac{0.0405 \times (208 \times 12)^4}{(3600) (9158.9 \times 12^4 \times 6)}} = 1.64 \text{ SEC} \pm 10\%$$

↳ CON'S DERIVED FROM
 C.M. HARRIS, "SHOCK AND VIBRATION HANDBOOK"

LOOK @ "DYNAMIC ANALYSIS OF STRUCTURES"
 MASS / FLR
 MASS - INertia "UNIT 43 HEIGHT"

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12/31 | 9/10 | 12/1

STRUCTURAL DEPTH | CORE DESIGN

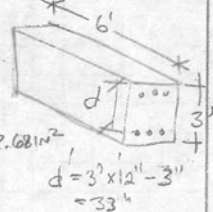
COUPLING BEAMS:

FLOOR	SHEAR FORCE (K)	0.8 V _{MAX, GROUP} (K)
1	103.56	104.6
2	123.11	
3	130.7	
4	130.57	
5	135.87	
6	116.92	
7	106.51	
8	95.05	
9	83.23	
10	71.58	
11	60.50	
12	50.32	40.256
13	41.29	
14	33.64	
15	27.51	
16	22.99	19.392
17	20.10	
18	18.7	
19	18.5	
20	9.73	

INITIAL SIZING:
LOWER BEAMS —
 $V = 104.6 \text{ K} \rightarrow M = 104.6 \times \frac{6 \times 12}{2} = 313.8 \text{ K-ft}$
@ MIDDLE OF BEAM

$A_s = \frac{M_u}{4d}$ FOR $f'_c = 4000 \text{ PSI}$
 $f_y = 60 \text{ KSI}$
 $\phi = 1.0$
 $\phi = 0.9$
 $A_s = \frac{0.9 M_u}{4d} = \frac{M_u}{4.44d}$

$M_u = M_{PR}$
 $M_{PR} = M \times 1.25$
 $A_s = \frac{M \times 1.25}{4.44d} = \frac{314(1.25)}{4.44(33)} = 2.68 \text{ in}^2$
SAY (3) #9's?
 $A_s = 2.98 \text{ in}^2$



→ SHEAR REINFORCEMENT: $\frac{V}{4.44(33)} = 3.16$ $M = \frac{477 \text{ K-ft} \times 12 \text{ ft}}{6 \times 12 \text{ ft} / 2} = 159 \text{ K}$

DESIGNED w/ $V_c = 0$

$V_s = V_u / \phi \leq 8 \sqrt{f'_c} b_w d$
 $= \frac{159}{0.75} \leq 8 \sqrt{4000} 16(34) \Rightarrow 212 \text{ K} \leq 275.2 \text{ K}$

IF $V_s \leq 4 \sqrt{f'_c} b_w d$, $S_{MAX} = \frac{d}{2} = 12" \leftarrow$
 $\downarrow$
 137.6 NG $\therefore S_{MAX} = 12" \leftarrow$ (4 LEGS) $\leftarrow$ (MIN)

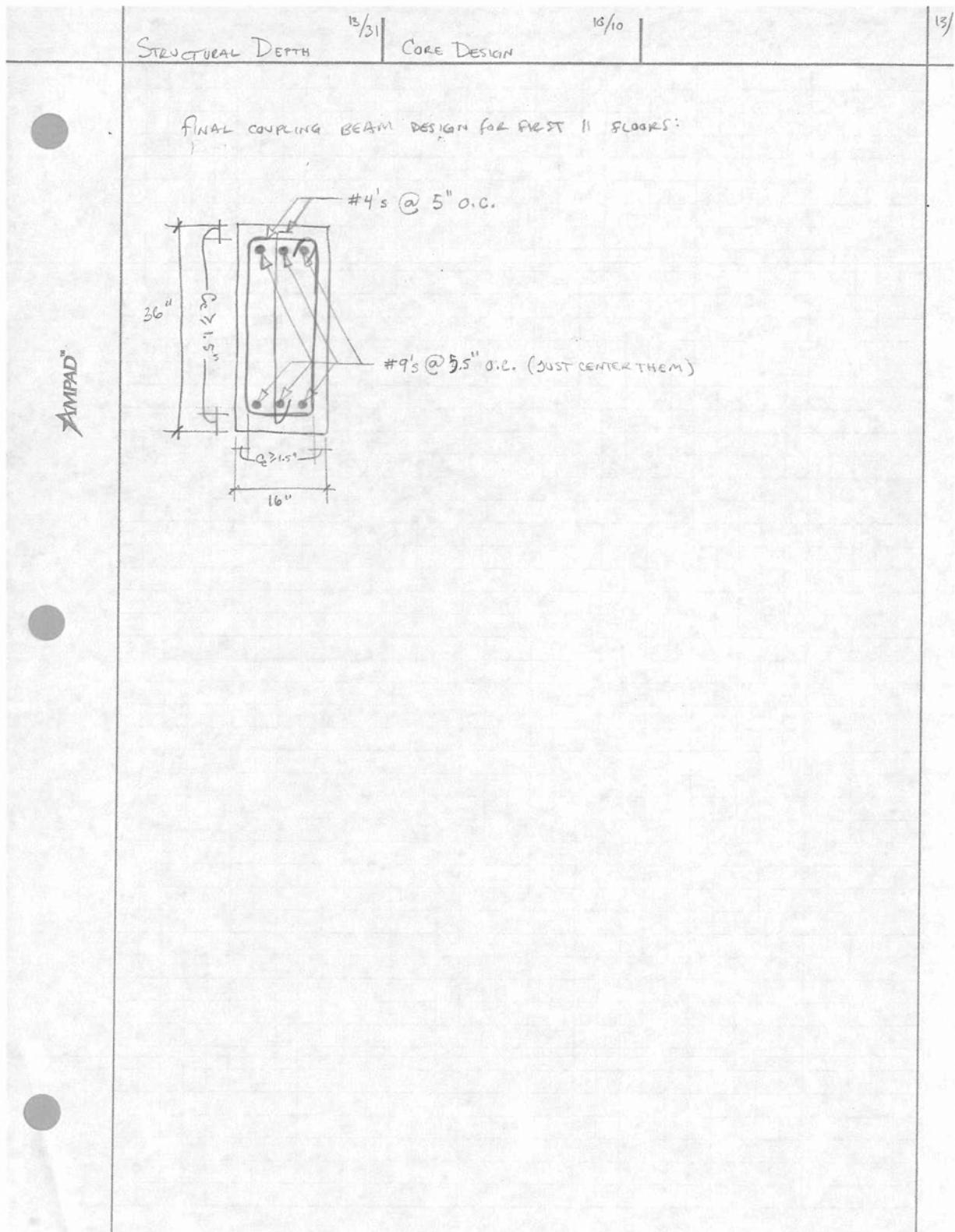
LET $S = 6"$

$A_{V_{MIN}} = \frac{0.75 \sqrt{f'_c} b_w S}{f_{yt}} = 0.036 \text{ in}^2$
 $\frac{50 b_w S}{f_{yt}} = 0.08 \text{ in}^2$ MAX

$V_s = \frac{A_v f_y d}{S}$
 $212 \leq \frac{A_v (60)(33)}{6"} \quad A_v \geq 0.642 \text{ in}^2$

$S = 5"$
 $212 \leq \frac{A_v (60)(33)}{5"} \quad A_v \geq 0.535 \text{ in}^2 \Rightarrow \text{use (3) #9's}$

$A_s = 3(0.2) = 0.6 \text{ in}^2 @ 5"$

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04.04.2012

Ksenia Tretiakova, Structural Option
AE Consultant: Dr. Andres Lepage

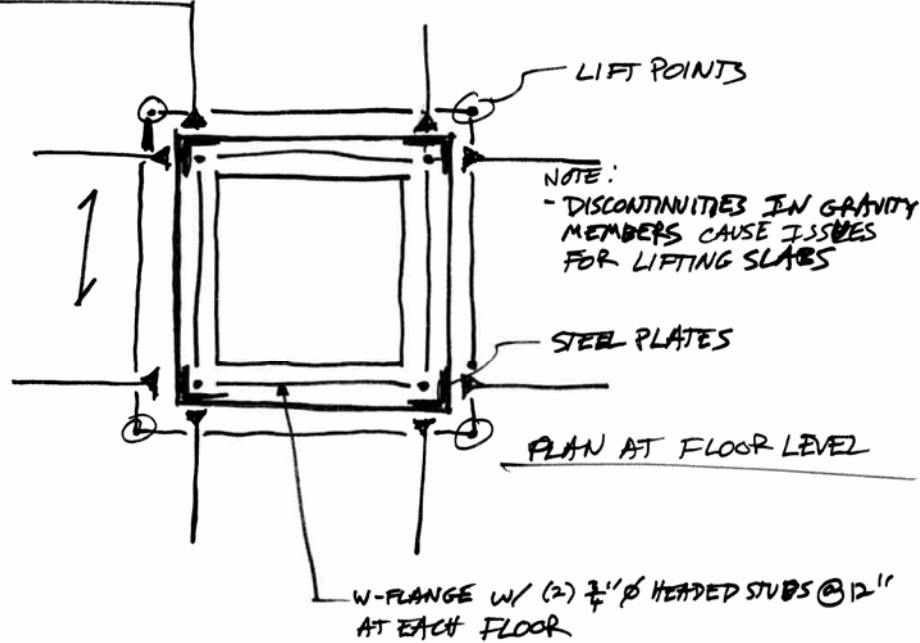
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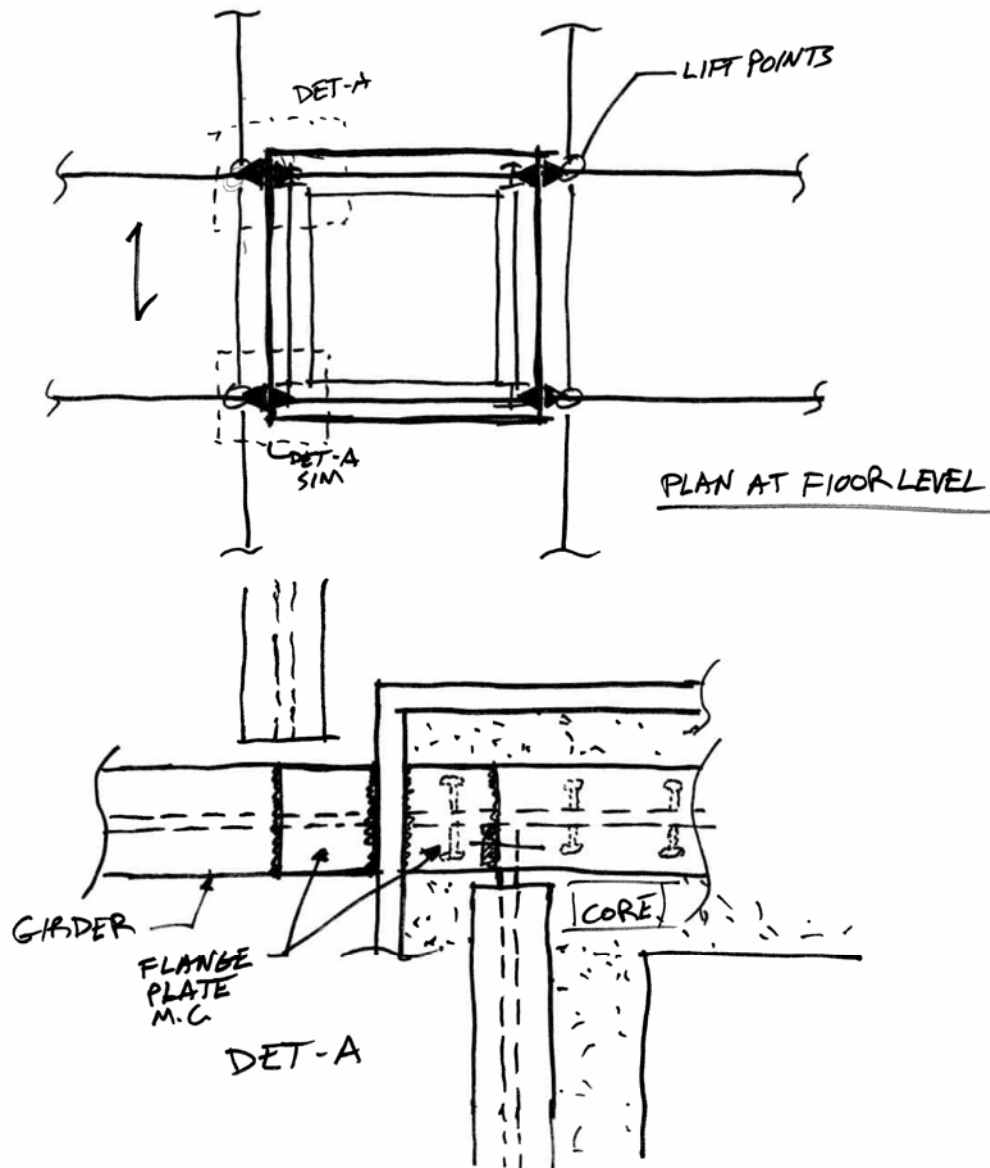
Appendix D –

See following pages for scans

REV.	LERA LESLIE E. ROBERTSON ASSOCIATES, RLLP CONSULTING STRUCTURAL ENGINEERS	P	DRAWING NO.
		DRAFT	
		DESIGN	
		CHECK	

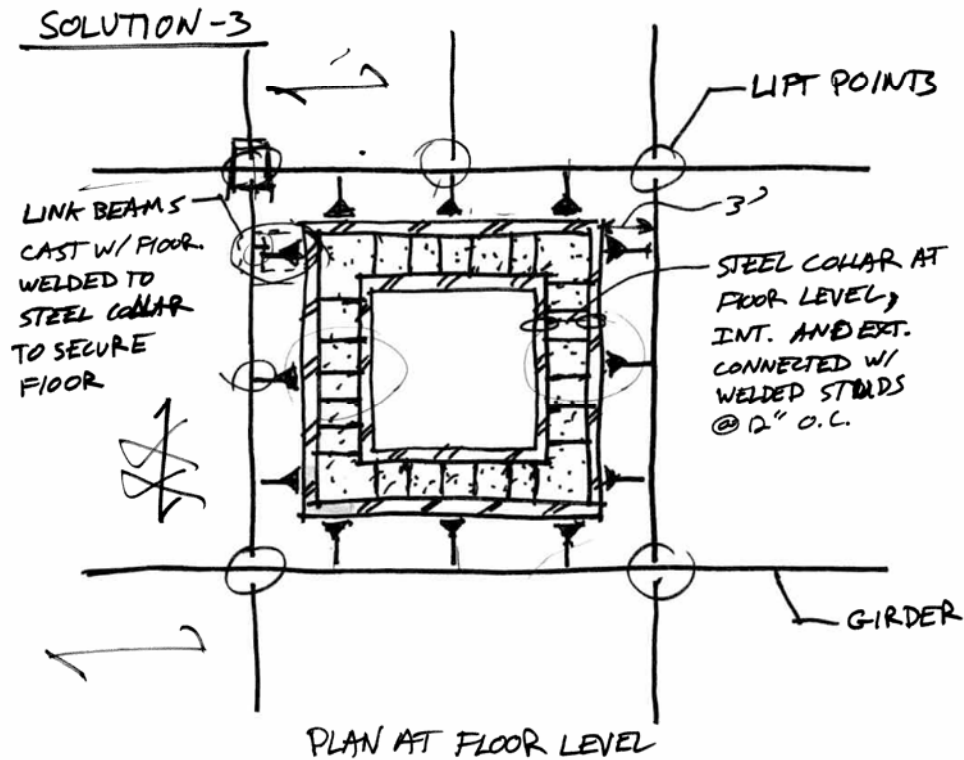
SOLUTION-1

REV.	LERA LESLIE E. ROBERTSON ASSOCIATES, RLLP CONSULTING STRUCTURAL ENGINEERS	P	DRAWING NO.
		DRAFT	
		DESIGN	
		CHECK	

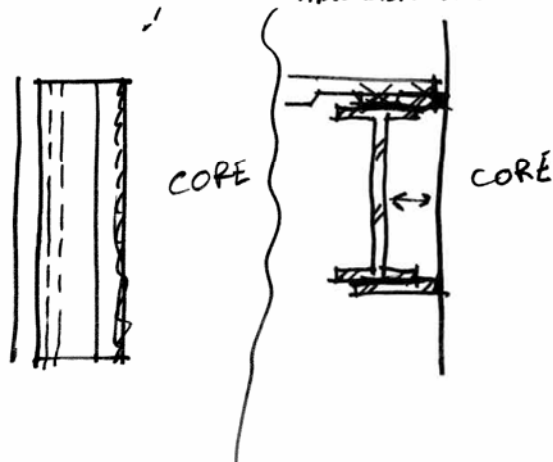
SOLUTION-2

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REV.	LERA LESLIE E. ROBERTSON ASSOCIATES, RLLP CONSULTING STRUCTURAL ENGINEERS	P	DRAWING NO.
		DRAFT	
		DESIGN	
		CHECK	



NOTE: CONNECTION OF STEEL COLLAR TO CONCRETE
CORE TO RESIST GRAVITY AND LATERAL LOADS
AND DISTRIBUTE



04.04.2012

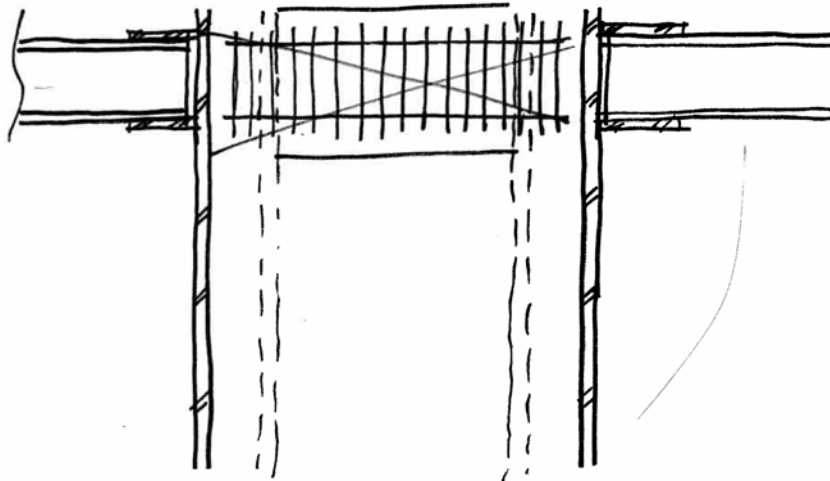
Ksenia Tretiakova, Structural Option
AE Consultant: Dr. Andres Lepage

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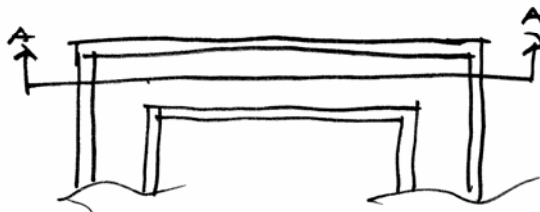
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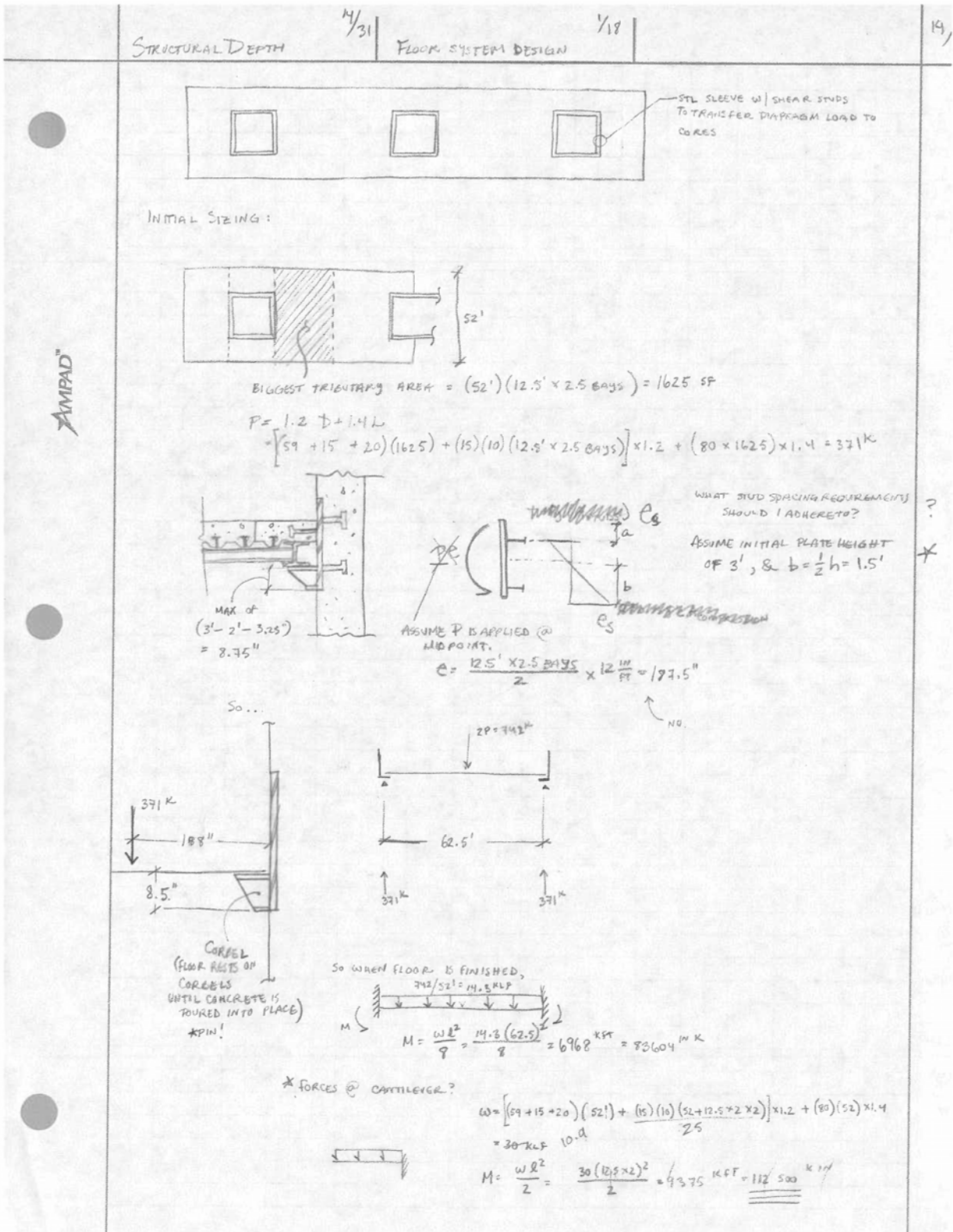
REV.	LERA LESLIE E. ROBERTSON ASSOCIATES, RLLP CONSULTING STRUCTURAL ENGINEERS	P	DRAWING NO.
		DRAFT	
		DESIGN	
		CHECK	

SOLUTION-3 CONT'D



SECTION-A



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15/31 | 3/18 | 15/

STRUCTURAL DEPTH | FLOOR SYSTEM DESIGN

∴ USE CANTILEVERED ENDS TO DESIGN 3FL SLEEVE CONNECTION

$M_u = 112,500 \text{ K IN}$

ASSUME:

STUDS @ 6" O.C.

3'

TOP STUDS ARE YIELDING

$E_{ST} \geq 0.002$

FINDING N.A.

$\frac{0.002}{(3-C-0.5)} = \frac{0.003}{C}$

$0.002 C = 0.003 (3-C-0.5)$

$0.005 C = 0.0075 \quad C = 1.5'$

If $E_{ST} = 0.002$,
 $E_{S2} = 0.001$

$T = \sum A_{si} f_{si} (1.5' - d_i)$ $A_{ST} > A_{S2}$

$= \frac{A_S}{2} (60) (1.5 \times 12'' - 6'') + \frac{A_S}{2} (60) (1.5 \times 12'' - 12'') = 540 A_S \text{ K}$

$112,500 \leq (0.9) \left[540 A_S \left(1.5' \times 12'' - \frac{6'' + 12''}{2} \right) + 19,278 \left(1.5' \times 12'' - \frac{12.6''}{2} \right) \right]$

$112,500 \leq 20,300 + 4374 A_S$ (COMPRESSION TAKES CARE OF ALL MOMENT IN THIS CASE)

$\rightarrow T < C \quad 19,278 \text{ K} = 540 A_S$ $A_S = 35.7 \text{ IN}^2 = 17.85 \text{ IN}^2$ $= 41 \text{ SHEAR STUDS OVER } 28'$

$(1) \frac{3}{4}" \text{ SHEAR STUD } A_S = 0.44 \text{ IN}^2$ $(\times 2 \text{ ROWS})$ $\text{SO TOTAL OF } 82$

∴ STUDS @ 6" O.C BOTH WAYS

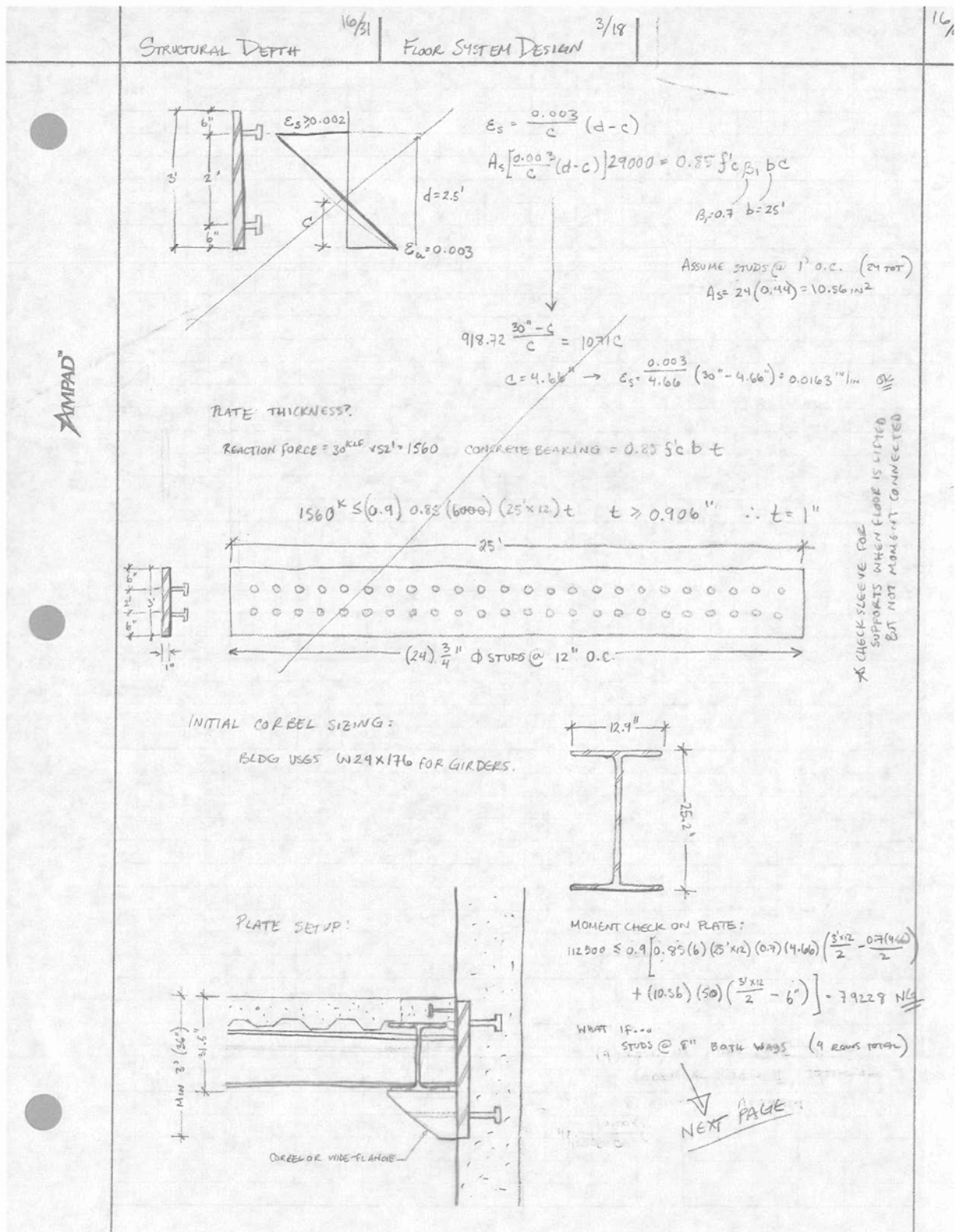
GROSSLY OVERDESIGNED!

BUT, THE SLEEVE HAS TO BE 3' TALL TO ACCOMMODATE FLOOR THICKNESS.

WHAT IF ONE ROW OF SHEAR STUDS?

$T = A_S (60) (1.5' \times 12'' - 6'') = 720 A_S = 19,278 \text{ K}$ $A_S = 26.78 \text{ IN}^2 = 61 \text{ STUDS}$ $\text{STUDS @ } 4" \text{ O.C.}$

SO ONE SHEAR STUD

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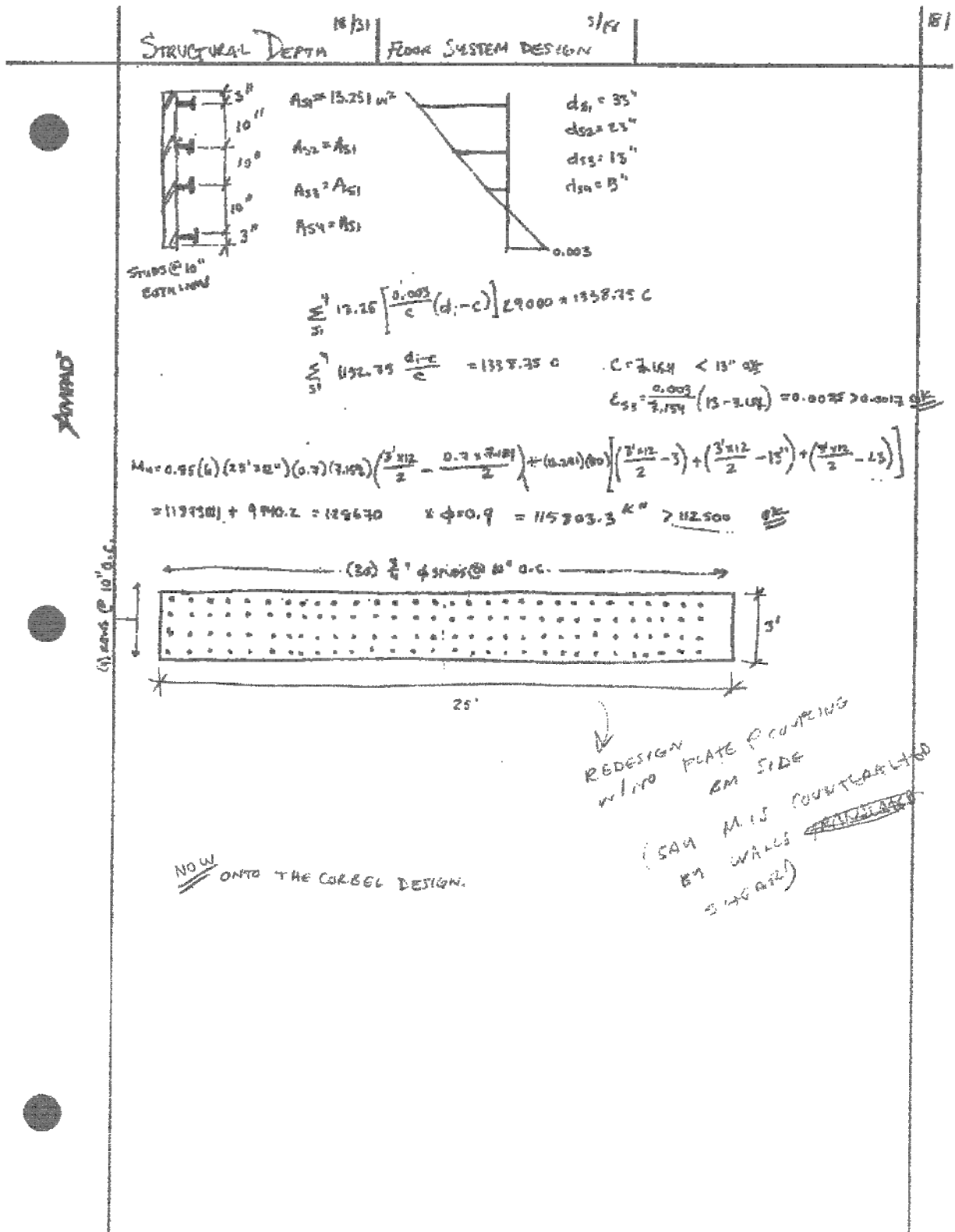
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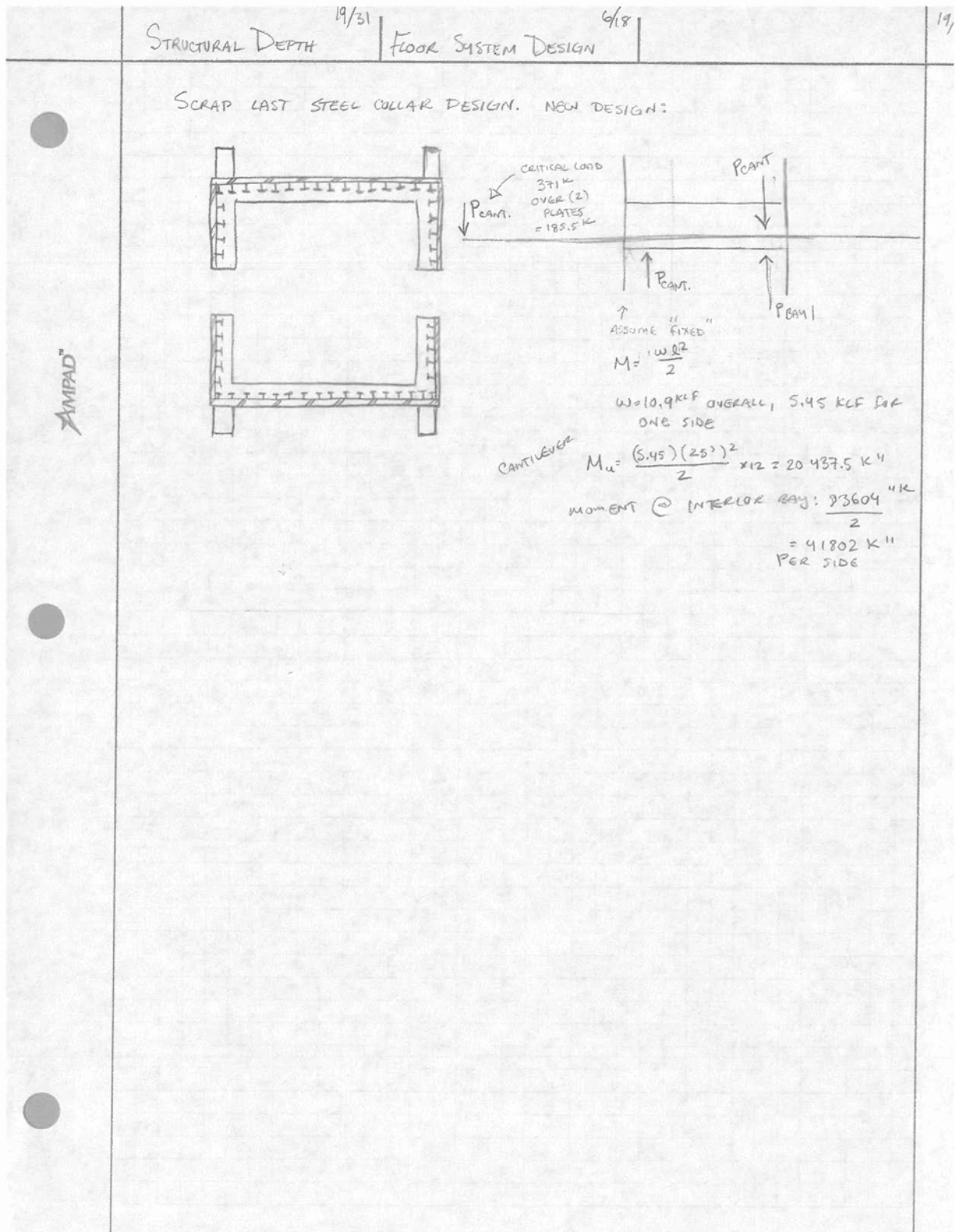
17/21	4/19	17/1
STRUCTURAL DEPTH	FLOOR SYSTEM DESIGN	
 $A_{s1} = 16.35 \text{ m}^2$ $A_{s2} = A_{s1}$ $A_{s3} = A_{s1}$ $A_{s4} = A_{s1}$	 $E_u = 0.003$	Assume $E_{s1, s2, s3} > 0.003$ $d_{s1} = 34" \quad d_{s2} = 26" \quad d_{s3} = 18" \quad d_{s4} = 10"$
$\sum_{s1} 16.35 \left[\frac{0.003}{c} (d_f c) \right] 29000 = 0.85 f'_c \beta_1 b c$ $\downarrow$ $\downarrow$ $\downarrow$ 16.35 $d_f = c$ $b = 0.7$ 25×12		
$1422.45 \left[\frac{34-c}{c} + \frac{26-c}{c} + \frac{18-c}{c} + \frac{10-c}{c} \right]$		
$1422.45 \left(\frac{88-4c}{c} \right) = 1339.75 c \quad c = 7.775" < 10" \quad \underline{OK}$		
$E_{s1} = \frac{0.003}{7.775} (10 - 7.775) = 0.0008 \quad \underline{OK}$		
$E_{s2} = \frac{0.003}{7.775} (18 - 7.775) = 0.004 \quad \underline{OK}$		
Assume s_4 is in comp. $\therefore E_{s4} = 0.003 > 0.003$		
$1422.45 \left[\frac{34-c}{c} + \frac{26-c}{c} + \frac{18-c}{c} \right] = 0.85 (6) (0.7) (25 \times 12) c$ $c = 7.65" < 10" \quad \underline{OK}$		
s_4 is in T but NOT YIELDING.		
$c = 7.775"$		
$M_u = 0.85 (6) (25 \times 12) (0.7) (7.775) \left(\frac{3 \times 12}{2} - \frac{0.7 \times 7.775}{2} \right) +$ $(16.35) (50) \left[\left(\frac{3 \times 12}{2} - 2 \right) + \left(\frac{3 \times 12}{2} - 10 \right) + \left(\frac{3 \times 12}{2} - 18 \right) + \left(\frac{3 \times 12}{2} - 26 \right) \right]$ $= 127276.5 + \dots$		
$\downarrow$ <u>OVERDESIGNED</u>		

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STRUCTURAL DEPTH 20/31	FLOOR SYSTEM DESIGN 7/19	20
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PARTIAL STEEL COLLAR

CENTER COLLAR HAS MOST LOAD

STUDS FOR GRAVITY: CHECK SHEAR FRICTION (ACI 318-05 §11.7)

$V_u = A_{ve} f_y \mu$
 $\mu = 1.0$
 $f_y = 50 \text{ ksi}$
 $\leq \begin{cases} 0.2 S'_c A_c \\ 800 A_c \\ \text{MIN} \end{cases}$

$\phi = 0.75?$

TRIBUTARY AREA FOR INITIAL SIZING:

CENTER COLLAR HAS MOST LOAD. $A_{\text{ONE COLLAR}} = \frac{4550}{2} = 2275 \text{ SF}$

FLOOR LOAD:

$$[(59 + 20 + 15)(2275) + (15)(10')(87.5')] \times 1.2 + [(80)(2275)] \times 1.4 = 527.17 \text{ K}$$

$V_u = 528 \text{ K}$

$528 \leq (0.75) A_{\text{SHEAR STUDS}} (50) (1.0)$
 $A_{\text{SHEAR STUDS}} \geq 14.08 \text{ IN}^2$

START OUT W/ 6" O.C. IN BOTH DIRECTIONS, $\frac{3}{4}" \phi$ 50 KSI SHEAR STUDS
DOUBLE ROW TOP + BOTTOM

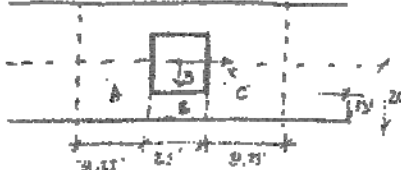
$A_{\text{ONE}} = 0.442$

$A_{\text{STL}} = 86.57 \text{ IN}^2 > A_{\text{MIN}} \text{ OK}$

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STRUCTURAL DEPTH 2/31	KENNA SYSTEM DESIGN 8/8	21,
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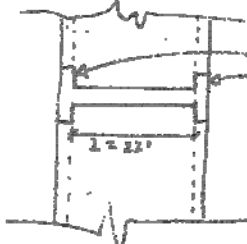


CENTROID $X = 0$ BY OBSERVATION

$$y = \frac{(26 \times 31.25) \times 2 \times 13 + (25 \times 15) \times \frac{15}{2}}{26 \times 31.25 \times 2 + 25 \times 15} = 11.92'$$

$V_{u1} = 329^k$

$M_{u1} = 329^k \times 11.92' \times 12 = 755.04^k$



ADDITIONAL TIES FOR STUDS IF NECESSARY

$\frac{M_u}{L} = 3432^k$ OF FORCE TO RESIST

+ V_{u1}

$V_u = 3960^k$

$3960 \leq (0.45) A_{STUDS} (50) (1.0) \quad A_{STUDS} \geq 105.6 \text{ in}^2$

$\frac{3}{4}" \phi$ STUDS @ 6" O.C. 106 ROW = NG

$\rightarrow \frac{3}{4}" \phi$ STUDS @ 4" O.C. 106 ROW? (TOP + BOT)

$A_{STL} = 130.7 \text{ in}^2$ OK

(TOP) $\frac{3}{4}" \phi$ STUDS @ 4" O.C.
FOR ROWS: 4 ROWS @ 6" O.C. @ 2' CUB.
 $\rightarrow$ 2 ROWS TOP, 2 ROWS BOT.



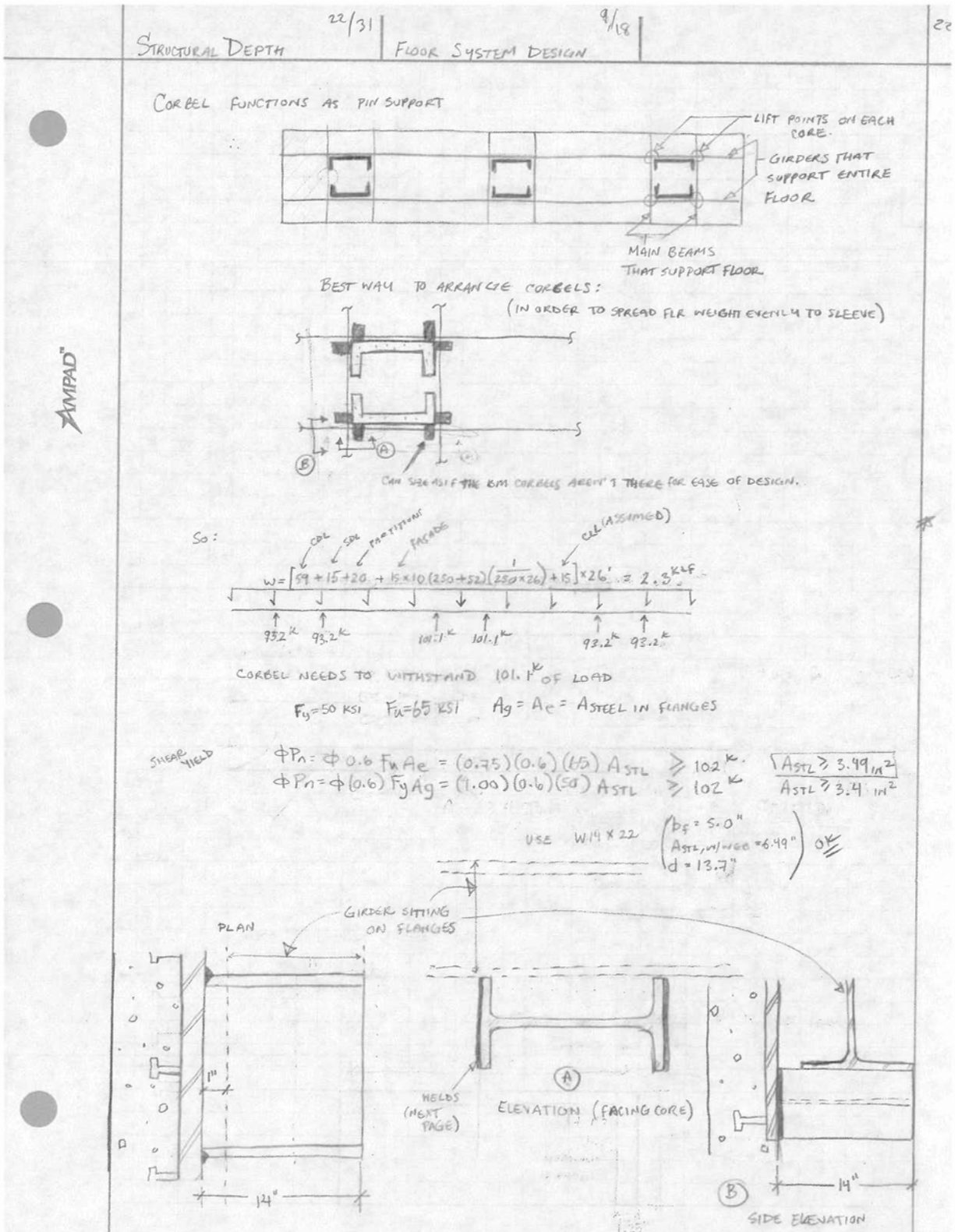
$M_{u1} = 755.04^k$

$\phi M_n = (0.9) A_s f_y = (0.9) (t) (w) (60 \text{ ksi})$

$755.04 \leq (0.9) (25 \times 12) (60) t \quad t \geq 4.661" \rightarrow \boxed{t = 5"} \leftarrow$

CHECK BUCKLING:

$329 \leq 0.95 f_c' A_g = 0.95 (6) (25 \times 12 \times 5) = 7650^k \quad \text{OK}$



23/31 | FLOOR SYSTEM DESIGN | 10/19 | 23,

STRUCTURAL DEPTH

WELD DESIGN:

THE WELDS ON THIS CORBEL ARE ECCENTRICALLY LOADED.
USE TABLE 8-4 IN AISC STEEL CONSTRUCTION MANUAL

PLAN VIEW AGAIN:

W24x176 GIRDER
CORBEL

P
 $K=0$
 $l=5"$ (FULL HEIGHT OF CORBEL)
 $e_x=7.45"$

$7.45 = a(5")$ $a=1.49$

C	
a	K=0
1.40	0.927
1.60	0.815

$\therefore C=0.8766$

$\phi R_n = \phi C C_1 D l$
 $C_1 = \text{ELECTRODE STRENGTH}$
(ASSUME E70XX, $C_1=1.0$) *
 $\phi=0.75$

$(0.75)(0.8766)(1.0) D (5") \geq 102 K$

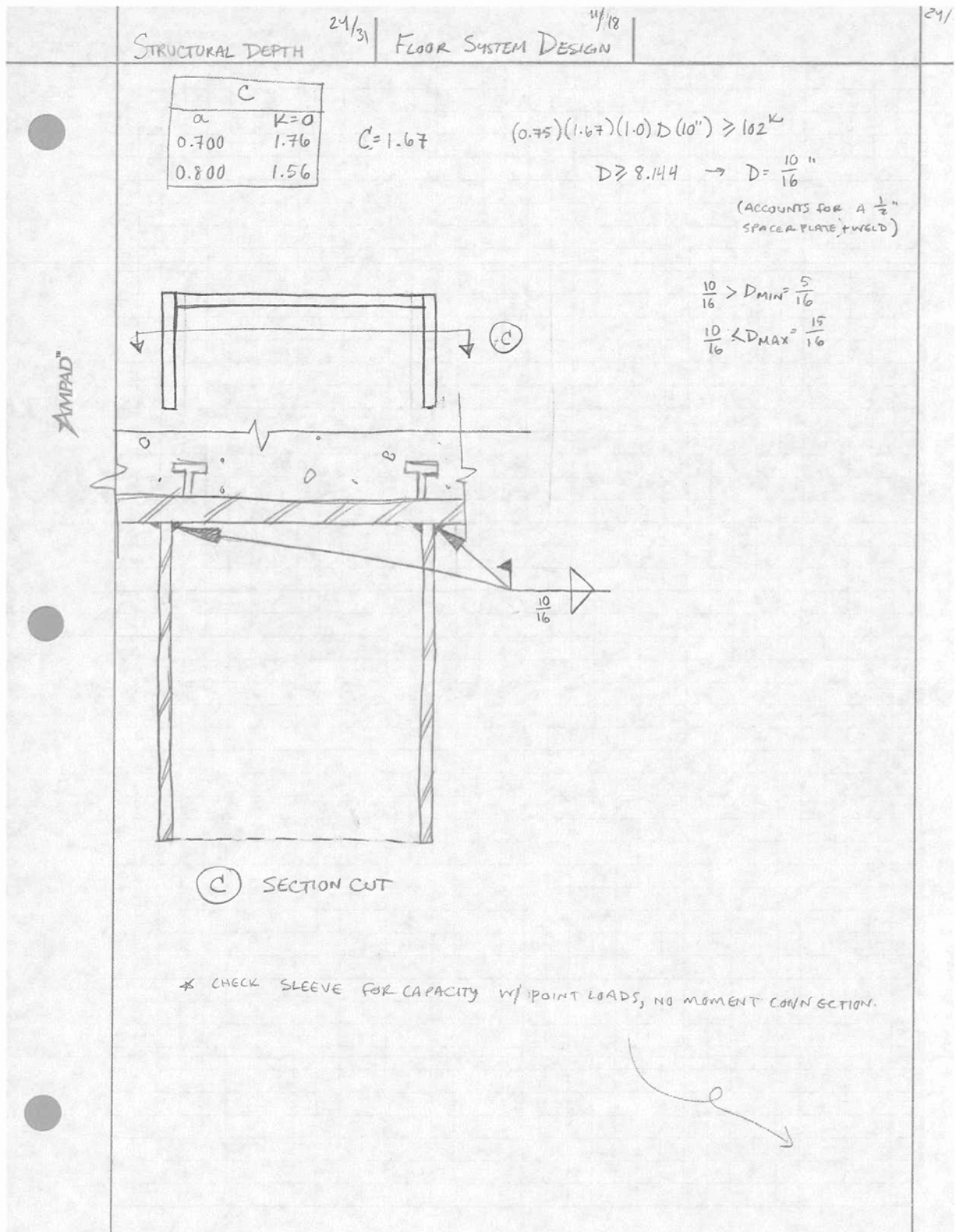
$D \geq 31.03 \rightarrow \frac{32}{16} "$ NG

$\text{MIN } D = \frac{5}{16} "$
 $\text{MAX } D = (1" - \frac{1}{16}) = \frac{15}{16} "$

→ USE BUILT UP MEMBER & WELD ON BOTH SIDES OF EACH PLATE TO DOUBLE L.

$A_{STL} \geq 3.49 \text{ in}^2 \rightarrow t_{\text{PLATE}} = 0.5" \quad (A_{STL} = 5" \times 0.5" \times 2 = 5 \text{ in}^2)$
W/ SPACER PLATE @ TOP PREATTACHED

CAN WELD BOTH SIDES OF PLATES
 $\therefore l = 5" \times 2 = 10"$
 $e_x = 7.45 = a(10")$ $a=0.745$

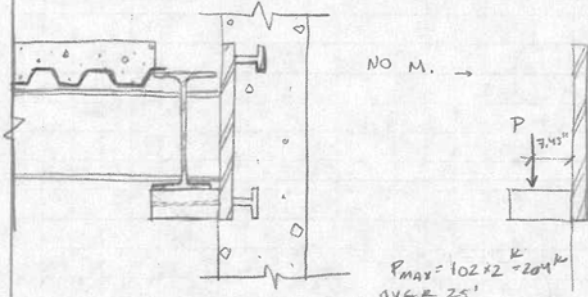
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25/31 | FLOOR SYSTEM DESIGN | 25

STRUCTURAL DEPTH

INITIAL LOADING CONDITIONS (WHEN FLOOR IS LIFTED INTO PLACE):



NO M. →

$P_{MAX} = 102 \times 2 = 204 \text{ K}$
(OVER 25' (SEE PREV CALCS))

$204 \text{ K} (7.45'') = 1520 \text{ '' K}$

→ PLATE IS SIZED FOR 112500 '' K OK

DIAPHRAGM FORCES:

- CHECK VERTICAL CAPACITY FOR LOAD + EV
- DESIGN MOMENT CONNECTION TO CORE

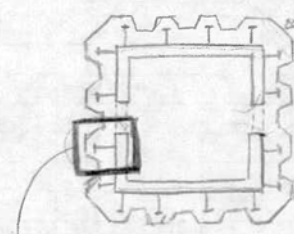
$F_{PX} = \frac{\sum_{i=1}^n F_i}{\sum_{i=1}^n W_i} W_{PX} \rightarrow \text{SPREADSHEET, MAX } F_{PX} = F_{P17} = 179 \text{ K} \leq 0.4 \phi_{DS} I_{WPX} = 296.3 \text{ K}$

$\phi = 1.0$

$0.522 \quad 1481.6$

$\geq 0.2 \phi_{DS} I_{WPX} = 148.16 \text{ K}$

SO...



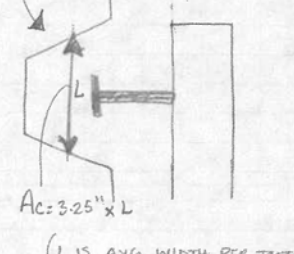
CONTOUR OF POURED CONCRETE WHEN FLOOR IS LIFTED INTO PLACE

x 3 FOR

↑ 179 KIPS x $\Omega = 2.0$

↓ 60 K FOR ONE WALL SIDE

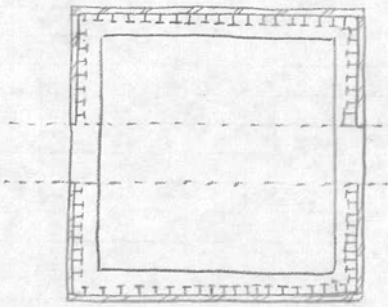
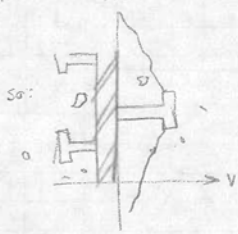
STEEL PLATE @ SIDES



$A_c = 3-25'' \times L$

(L IS AVG WIDTH PER TOOTH)

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STRUCTURAL DEPTH	26/31	FLOOR SYSTEM DESIGN	13/18	26,
	$\phi V_n = 0.75 (1.0 V_c)$ $V_c = 2 \lambda \sqrt{f'_c} A_g \quad \lambda = 0.75 \text{ FOR LIGHTWEIGHT}$ $V_c = 2 (0.7) \sqrt{4000} (3.25') L_{\text{CHC}} = 287.77 L_c$ $\phi V_n = (0.75) 287.77 L_c = 215.93 L_c \geq 179,000 \text{ lb} \quad \frac{L_c \geq 829.37 \text{ IN}}{6 \text{ SIDES}} \quad L_{\text{ONE SIDE}} \geq 138.23 \text{ IN}$ $\downarrow$ USE HALF OF WALL LENGTH: 150 IN (USE AVG LENGTH OF SHEAR TAB)			
	SHEAR STUD STRENGTH Q_n FOR $f'_c = 3 \text{ KSI}$, LW CONCRETE $\phi = \frac{1}{2}''$ $\rightarrow = 7.6 \text{ K}$ SO $\sum Q_n \geq V_u \quad (7.6)n \geq 179 \quad \frac{n \geq 23.6}{6 \text{ SIDES}} \quad n_{\text{ONE SIDE}} \geq 3.93 \text{ STUDS}$ SO MIN 4 STUDS / SIDE ($@ \frac{1}{2}''$) $\rightarrow$ move to $\frac{3}{4}''$ (EASIER TO FIND) MIN $S = 6\phi$ MAX $S = 36''$ OR 8" SLAB t (FOR STUDS IN DECK) } FROM L. GESCHWINDER WHAT ABOUT STUDS NOT IN DECK? "UNIFIED STEEL CONSTRUCTION" MAY AS WELL ADHERE TO MAXIMUM			
	* STL COLLAR WILL ENCASE CORE LIKE SO:			
	 SAY $\frac{3}{4}''$ STUDS @ 12" O.C. ALONG STL PLATE			
	IN ADDITION TO SHEAR, NEED TO DESIGN STUDS FOR: - D.4.1. c) CONCRETE BREAKOUT STR IN TENSION (D.5.2) d) CONCRETE BREAKOUT STR IN SHEAR (D.6.2) (AMONG OTHERS)			
	 3.25" TOPPING TIME PERMITTING			

29/31 STRUCTURAL DEPTH	4/18 FLOOR SYSTEM DESIGN	29
----------------------------------	------------------------------------	----

CONCRETE BEAM-HOOK STRENGTH IN TENSION - SINGLE ANCHOR

$$N_b \leq \frac{A_s}{A_{sc}} \psi_{e,h} \psi_{e,m} \psi_{e,n} N_b$$

$$N_b = \phi_s \sqrt{f_c'} h_{ef}^{1.5} = 17 \sqrt{4000} (7.5)^{1.5} = 22.64 \text{ k}$$

17 for POST-TENSIONED ANCHORS

AND PROJECTED CONCRETE FAILURE AREA

A_{sc} PROJECTED CONCRETE FAILURE → 4000 ≤ 1.3 h_{ef}

$$A_{sc} = 9 h_{ef}^2 = 523 \text{ in}^2$$

$$\frac{3}{4} \text{ STUDS, ADDITIONALLY, TOLER. A } L_{d,dev} = 8 \frac{s}{16} \left(\text{HERE } h = \frac{s}{8} \right)$$

$$h_{ef} = 7.5 - \frac{s}{8} - \frac{s}{16} = 7 \frac{5}{8}$$

$$\psi_{e,h} = 0.7 + 0.5 \frac{C_{min}}{1.5 h_{ef}} \text{ FOR } C_{min} < h_{ef} \times 1.5$$

$$= 0.798$$

$\psi_{e,m} = 1.0$ UNSEPARATED
(NO CRACKING IS EXPECTED IN SLAB @ BEAM CATCH BUT NOT ABSOLUTELY)

$$\psi_{e,n} = \frac{1.5 h_{ef}}{C_{min}} \text{ FOR } C_{min} < C_{sc}$$

$$\text{IN SLAB } C_{min} = 1.0$$

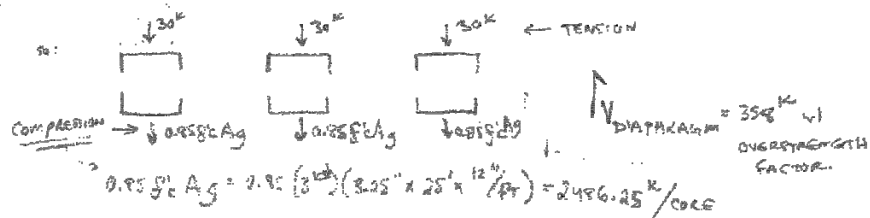
12" O.C. SPACING < 3 h_{ef} ∴

$$A_{sc} = (3.25") (12") = 39 \text{ in}^2$$

* SPACING MIGHT WORK AGAINST ME ∴ ADJUSTED A_{sc} IN ACCOUNT FOR PROJECTED END OF FAILURE OVERLAP

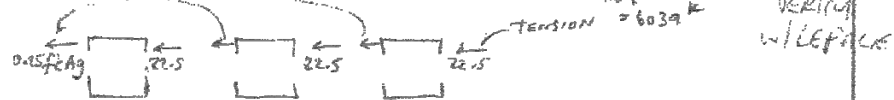
$$N_b = 22.64 \leq \frac{39}{523} (0.798)(1.0)(1.0)(22.64) = 1.25 \text{ k}$$

$$\text{FOR (24) STUDS @ 12" O.C., } N_b = 24(1.25) = 30 \text{ k}$$



$$\text{TOTAL RESISTING FORCE} = 30 \text{ k} \times 3 + 2486.25 \text{ k} \times 3 = 7549.75 \text{ k} >> 358 \text{ k}$$

(COMPRESSION) X.O.P. = 6039 k



$$V_{DAMAGE} = 358 \text{ k}$$

$$9 \text{ STUDS PER WEB} \times \text{LOWESS} = 1.25 (9) (6) = 67.5 \text{ k}$$

$$0.85 f_c' A_g = 0.85 (3 \text{ k}) (h' \times \frac{12}{16} \times 3.25" \times 2 \text{ SIDES}) = 663 \text{ k/core}$$

$$\text{TOTAL RESISTING FORCE} = 67.5 \text{ k} + 663 \text{ k} \times 3 = 2056.3 \text{ k} >> 358 \text{ k}$$

X.O.P. = 1645 k

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CONCRETE BREAKOUT STRENGTH IN SHEAR

D6.2.1 (2) SINGLE ANCHOR, PERPENDICULAR TO EDGE

$$V_{cb} = \frac{A_{vc}}{A_{vc0}} \psi_{ed,v} \psi_{c,v} V_b$$

(3) PARALLEL TO EDGE

$$V_{cb, \text{PARALLEL}} = 2 V_{cb} ; \psi_{ed,v} = 1.0$$

$A_{vc0} = 4.5 (C_{a1})^2 = 162 \text{ in}^2$
 C from prism edge to anchor axis = 6"

$V_b = 8 \left(\frac{l_e}{d_o} \right)^{0.2} \sqrt{f_c} \sqrt{f_c'} (C_{a1})^{1.5}$ [FOR HEADED BOLTS CONTINUOUSLY WELDED IN STL ATTACHMENTS $\geq \frac{3}{4} l_e$]

$l_e = \text{LOAD BEARING LENGTH OF ANCHOR}$
 $= l_{ef} = 7.625"$
 $8 d_o = 8 \left(\frac{3}{4} \right) \cdot 6" \leftarrow l_e$
 MIN

$= 8 \left(\frac{6}{3.75} \right)^{0.2} \sqrt{3/4} \sqrt{3000} (1.125)^{1.5} = 1.19 \text{ K} \times 2 \text{ (D6.2.1.1.C)}$

$\psi_{ed,v} = 1.0$ (NO EDGE PERPENDICULAR TO C_{a1})

$\psi_{c,v} = 1.0$ (CONSERVATIVELY SPEAKING, THERE WON'T BE CRACKING MOST LIKELY)

$A_{vc} = 1.5 C_{a1} (C_{a2} \times 2) = 29.25 \text{ in}^2$

$V_b = 1.19 \text{ K} \leq \frac{29.25}{162} (1.0)(1.0)(1.19) = 0.215 \text{ K} / \text{ANCHOR} \times 2 \text{ (D6.2.1.1.C)}$

for...

9 STUDS/WEB X 12 WEBS = 108 STUDS RESISTING
 $= 23.22 \text{ K CAPACITY}$
 (ADD TO NOM. CAPACITY OF STUDS IN T & FLOOR DIAPHRAGM IN C)
 $\rightarrow 6036.4 \text{ K} = \text{TOTAL SHEAR CAPACITY IN } \phi 5 > 358 \text{ K}$

24 STUDS/WALL X 6 WALLS = 144
 $= 30.96$
 TOTAL CAPACITY = 1666 K / 77358 K

$V = 358 \text{ K}$

USE $\frac{3}{4} \phi$ SHEAR STUD CONNECTORS @ 12" O.C., CENTERED IN ENCL TOPPING ON ALL SIDES IN STL PLATE AREA (SET STUDS 6" FROM CORNER EDGES)

COLUMN? DEAG STUD

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29/3 | 16/18 | 29

STRUCTURAL DEPTH | FLOOR SYSTEM DESIGN

2 IDEAS TO TRANSFER FLOOR DIAPHRAGM FORCES TO CORE:

- USE SHEAR STUDS IN CONCRETE TOPPING THAT ARE ATTACHED TO STEEL COLLAR ON CORE
- USE STEEL BEAMS RUNNING BESIDE THE CORE AS COLLECTOR ELEMENTS ("DRAG STRUTS") TO TRANSFER SHEAR INTO CORE → PUT STUDS INTO BEAM TO PROMOTE SHEAR TRANSFER/COMPOSITE ACTION

A RECHECK OF APPENDIX D CALCS: (ACI 318-05)

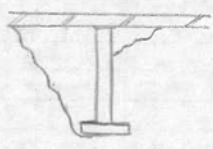
SO $V_u = 179 \text{ K}$ @ 17TH FLOOR $\Omega_b = 2.5$ $C_d = 5$

$V_{\text{PER WALL}} = \frac{179 \times 2.5}{6} = 74.58 \text{ K} \rightarrow V_{\text{PER WGB}} = \frac{74.58}{2} = 37.3 \text{ K}$
(PER 9.5' PORTION OF WALL)

LOOKS LIKE I SHOULD CHECK CONCRETE PRISM IN SHEAR. (FOR ANCHORS FAR FROM A FREE EDGE)

I THINK INITIAL ADVICE WAS NOT ENTIRE. (CONCRETE BREAKOUT NEEDS MORE FREE EDGES OR ACCURATE CALC)

g) PRISM D.6.3



$V_{cp} \leq K_{cp} N_{cp}$ $K_{cp} = 2.0$ for $h_{ef} \geq 2.5"$

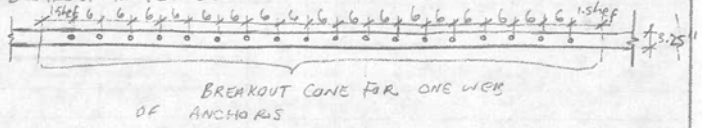
$V_{cp} \leq 2(1.25) = 2.5 \text{ K}$ $\rightarrow 1.25 \text{ K}$ (SEE P 14 CALCS)

@ 9 STUDS/WGB, $V = 22.5 \text{ K} < 37.3 \text{ K}$ NG

STUDS @ 6" SPACING? USE MULTIPLE ANCHORS THIS TIME

LET'S START W/ CONCRETE BREAKOUT IN TENSION:

c) BREAKOUT, TENSION D.5.2



$N_{cbg} = \frac{A_{nc}}{A_{nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$

$A_{nc} \leq n A_{nco}$ $n = 18$

$A_{nc} = (1.5 \times 7.625 \times 2 + 17 \times 6) \times 3.25 = 405.844 \text{ in}^2$

$A_{nco} = 9 h_{ef}^2 = 9(7.625)^2 = 523.3 \text{ in}^2$

$406 \text{ in}^2 \leq (18)(523) = 9414 \text{ in}^2$ OK

FOR $\frac{3}{4}" \phi$, $8 \frac{3}{16}"$ STUDS.

USE $3 \frac{3}{16}"$ STUDS. $h_{ef} = 2.625 \text{ in}$

$A_{nc} = [1.5 \times 2.625 \times 2 + 17 \times 6] \times 3.25 = 109.9 \text{ in}^2$

$A_{nco} = 9(2.625)^2 = 62.4 \text{ in}^2$ $\times n = 18 = 1116.3 \text{ in}^2$

$\frac{A_{nc}}{A_{nco}} = 1.773$

$N_b = K_c \sqrt{f'_c} h_{ef}^{1.5} = 24(\sqrt{3000})(2.625)^{1.5} = 5.6 \text{ K/STUD}$

$\psi_{ec,N} = 1.0$ (NO ECCENTRIC LOADING)

$\psi_{cp,N} = 1.0$

$\psi_{ed,N} = 0.7 + 0.3 \frac{1.625}{1.5(2.625)} = 0.924$

$\psi_{c,N} = 1.25$ (NO CRACKING, CAST-IN-PLACE)

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c) CONTINUED

$$N_b = 5.6 \text{ K/STUD} \leq (1.77)(1.0)(0.824)(1.25)(5.6) = 10.224 \text{ K/NSB} \underline{\underline{NG}} \quad (NEED \geq 38 \text{ K})$$

↓
WHAT WOULD IT TAKE TO WORK?

→ INCREASE TOPPING THICKNESS TO ALLOW FOR FULL CONE OF FAILURE TO FORM.

→ NEW $t = (2.625) \text{ (hef)} (1.5) \times 2 = 7.875 \rightarrow 8''$ OF TOPPING @ CORE (A CURE)

$$A_{NC} = [1.5 \times 2.625 \times 2 + 17 \times 6''] \times (7.875'') = 865.3 \text{ in}^2$$

$$\frac{A_{NC}}{A_{NCO}} = \frac{865.3}{62.0} = 13.96 \quad N_b = 5.6 \text{ K/STUD}$$

$\phi_{FEN} = 1.0 \quad \phi_{FEN} = 1.0 \quad (C_{a, \min} = 4'' > 1.5 \times 2.625 = 3.94'')$ $\phi_{CIN} = 1.25$

$$N_b = 18 \times 5.6 \text{ K} \leq (13.96)(1.0)(1.0)(1.25)(5.6) = 97.7 \text{ K} > 74.6 \text{ K} \underline{\underline{OK}}$$

100.8 K

→ CAN REDUCE CURE SIZE IF NECESSARY

d) BREAKOUT, SHEAR D.6.2



$$V_u = \left[\frac{A_{NC}}{A_{NCO}} \phi_{FEN} \phi_{FEN} \phi_{CIN} V_b \right] \times 2 \quad (D.6.2.1 c)$$

$C_{a1} = 6''$ (DISTANCE FROM ANCHOR TO EDGE OF FAILURE PRISM IN DIR. OF V)

$$V_b = \left(\frac{2.625}{2.14} \right)^{0.2} \sqrt{3/4} \sqrt{3000} (6'')^{1.5} = 7.165 \text{ K/STUD} \times 2 = 14.3 \text{ K/STUD}$$

$$A_{NCO} = 4.5 (6'')^2 = 162 \text{ in}^2$$

$$A_{NC} = 8' (1.5 \times 6'') = 72 \text{ in}^2 \quad \frac{A_{NC}}{A_{NCO}} = 0.4444$$

$\phi_{FEN} = 1.0$ (NO EDGE EFFECT)

$\phi_{CIN} = 1.4$ (NO CRACKING)

IN CASE C_{a1} WAS ESTIMATED INCORRECTLY, MAX CALC FOR $C_{a1} = 2.625''$

$$V_b = 4.15 \text{ K} \leq 2.58 \text{ K/STUD} \times 18 \text{ STUDS} = 46.44 \text{ K} > 37.3 \text{ K} \underline{\underline{OK}}$$

$$V_b = 14.3 \text{ K/STUD} \leq (0.4444)(1.0)(1.4)(14.3) = 8.9 \text{ K/STUD} \times 18 \text{ STUDS} = 160.16 \text{ K/NSB} > 37.3 \text{ K} \underline{\underline{OK}}$$

e) PRYOUT, SHEAR D.6.3

$$V_{cpb} = K_{cp} N_{cbg} \quad K_{cp} = 2.0 \text{ FOR } h_{ef} \geq 2.5'' \quad h_{ef} = 2.625''$$

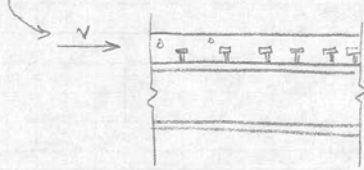
$$N_{cbg} = 97.7 \text{ K}$$

$$V_{cpb} = (2)(97.7) = 195.4 \text{ K/NSB} >> 37.3 \text{ K} \underline{\underline{OK}}$$

SOLUTION 1: PARTIAL STEEL COLLAR, w/ LWC TOPPING CAST OVER SHEAR STUDS THAT CONNECT INTO THE COLLAR. LWC TOPPING WILL BE INCREASED FROM 3.25" TO 8" IN THE CORE PERIMETER TO ACCOMMODATE NEW SHEAR TRANSFER STRESSES.

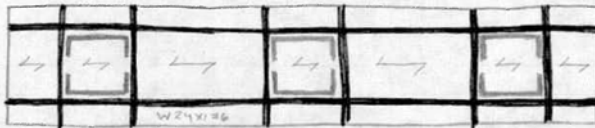
31/31 | STRUCTURAL DEPTH | FLOOR SYSTEM DESIGN | 18/19 | 31/

SOLUTION 2: DRAG STRUT AS SHEAR COLLECTOR?



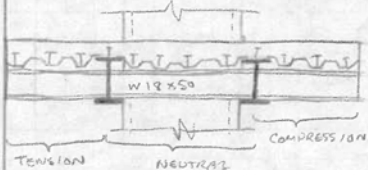
SHEAR TRANSFER TO BEAMS & GIRDERS, THEN FROM GIRDERS TO CORES.

BEAMS & GIRDERS WOULD NEED TO BE DESIGNED AS BEAM-COLUMNS.



FIRST: CONNECTION BETWEEN SLAB & STRUCTURAL STEEL FRAMING -
 Q_n FOR $F'_c = 3000$ PSI, LW CONCRETE, STUD $\phi = 1/2"$ IS 7.6 K

$V_u = 179 \times 2.5 = 447.5$ K
 $\frac{447.5}{6 \text{ BEAMS IN THIS DIR}} = 74.6$ K PER BEAM COLLECTOR ELEMENT



TENSION NEUTRAL COMPRESSION

DIAPHRAGM

$\frac{75 \text{ K}}{52 \text{ FT}} = 1.5 \text{ K/FT}$

1 STUD/FT WOULD BE FINE IN BOTH DIRECTIONS

$\Sigma Q_n = 7.6 (51 \text{ STUDS}) = 387.6 \text{ K/B EAM}$
 $\Sigma Q_n = 7.6 (249 \text{ STUDS}) = 1892.4 \text{ K/G I R D E R}$

BEAM DESIGN?

04.04.2012

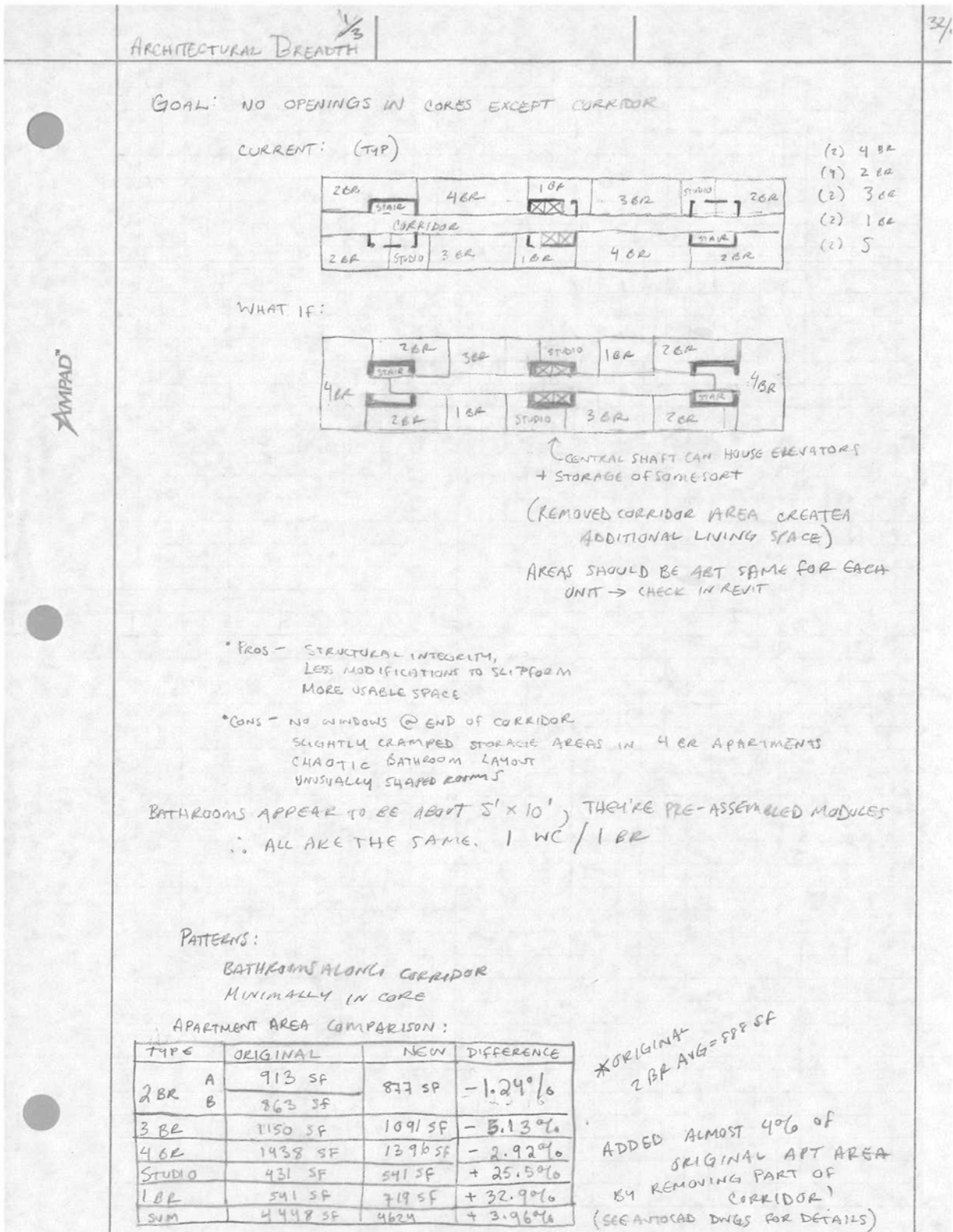
Ksenia Tretiakova, Structural Option
AE Consultant: Dr. Andres Lepage

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Appendix E –

See following pages for scans



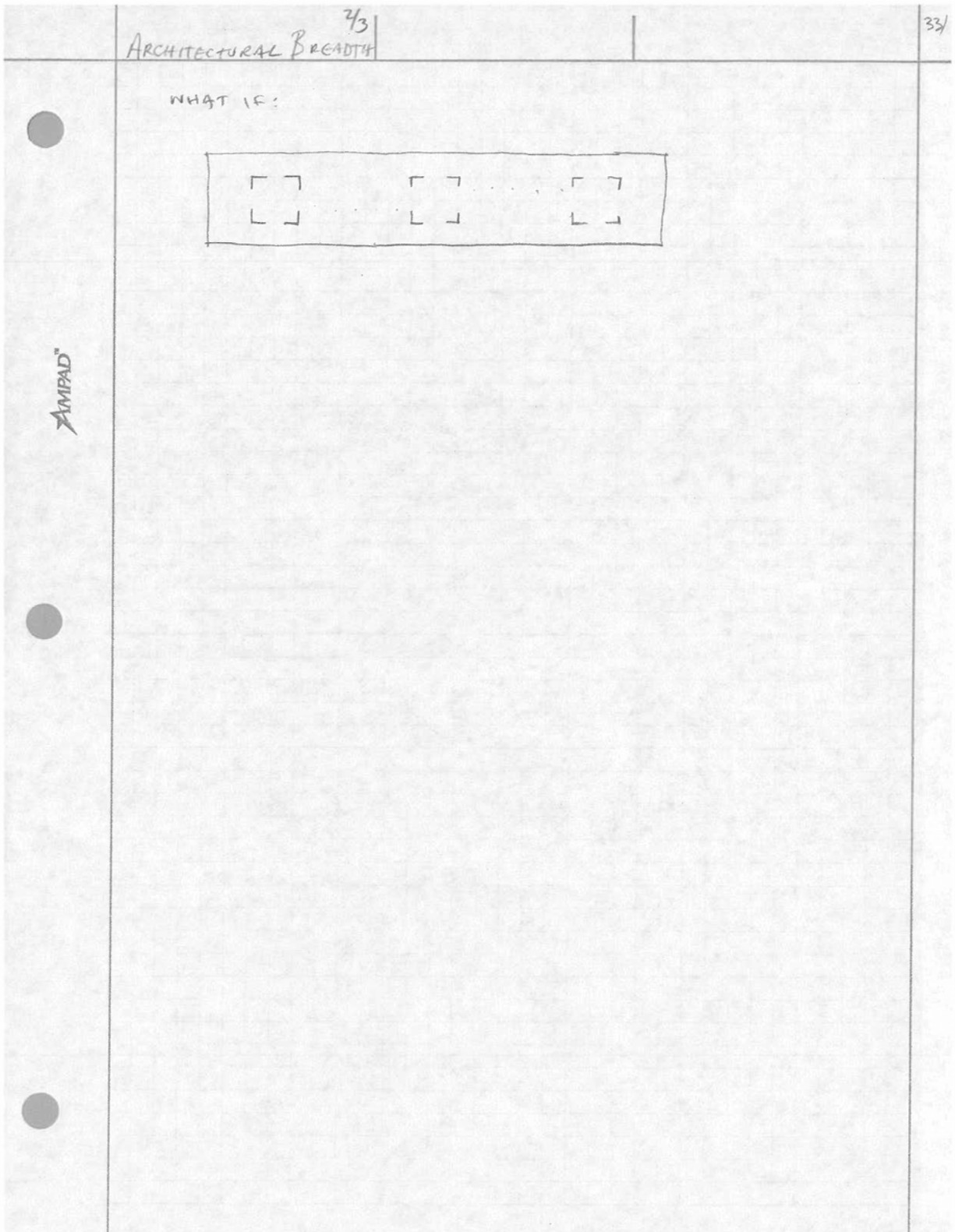
04.04.2012

Ksenia Tretiakova, Structural Option
AE Consultant: Dr. Andres Lepage

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	ARCHITECTURAL BREADTH 3/	34
	ORIGINAL: - BATHROOMS ARE ALL LOCATED ALONG CENTRAL CORRIDOR (FLOOR PLANS CAN VARY, & THE PLUMBING WILL JUST RUN THRU CORE & CORRIDOR WALLS) - CAN MODULARIZE FLOOR PLAN, SO THAT THE INTERIOR PARTITIONS CAN COME PREASSEMBLED.	

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	SUSTAINABILITY BREADTH ^{1/3}	351
	CREDITS TO WORK IN: SUSTAINABLE SITES CREDIT 4.2 - COVERED STORAGE FACILITIES FOR SECURING (1) BICYCLES FOR 15% OR MORE OF BLDG OCCUPANTS → 528 RESIDENTS, 15% = 79 → DOABLE. * CAN HAVE CANTILEVERED ROOF AS COVER FOR BIKE RACKS @ FRONT & BACK OF BUILDING SUSTAINABLE SITES CREDIT 4.4 - PROVIDE NO NEW PARKING (2) SUSTAINABLE SITES CREDIT 5.1 - FOR PREVIOUSLY DEVELOPED AREAS (CHECK), RESTORE OR PROTECT A MIN. OF 50% OF SITE (EXCLUDING BLDG FOOTPRINT) OR 20% OF TOTAL SITE AREA (WHICHEVER IS BIGGER) (1) CAN GO HARD ON LAND SUSTAINABLE SITES CREDIT 7.1 - SHADE 50% OF HARDSCAPE W/ NATIVE VEGETATION (1) SUSTAINABLE SITES CREDIT 7.2 - ROOFING SRI ≥ 79 → USE WHITE - ON - BLACK (1) EPDM ROOF, INSTEAD OF CONVENTIONAL BLACK. WATER EFFICIENCY PREREQUISITE - USE WATER-EFFICIENT FIXTURES IN PREPARED BATHROOMS MATERIALS+RESOURCES CREDIT 2 - PLAN TO RECYCLE OR SALVAGE 50% OF CONSTRUCTION WASTE (1) MATERIALS+RESOURCES CREDIT 3 - 10% OF MATERIALS, BY COST, ARE RECYCLED (2) → CAN USE RECYCLED CONCRETE AS AGGREGATE, + RECYCLED STEEL MATERIALS+RESOURCES CREDIT 4 - 20% OF MATERIALS, BY COST, ARE RECYCLED (2) → SAME. + CAN USE RECYCLED FLOORING MATERIALS REUSE CREDIT 6 - 2.5% OF VALUE OF BLDG MATS IS RECYCLED MATS (1) → FLOOR FINISHES, PARTITION WALLS, DOORS, FURNITURE, (THE MAJORITY OF THE INTERIOR FINISHES) (Axiom) INDOOR ENVIRONMENT QUALITY CREDIT 4.1 - LOW E ADHESIVES + SEALANTS (1) JUST NEED PLANNING ↓ DITTO 4.2 - PAINTS & COATINGS (1) ↓ DITTO 4.4 - COMPOSITE WOOD & AGG FIBER PRODUCTS (1) HAVE NO ADDED UREA-FORMALDEHYDE RESINS (NO LAMINATES) ↓ DITTO - FOCUS ON CASE WORK, DOORS & FINISHES 4.3 - LOW E FLOORING ↓ DITTO. USE TILE IN KITCHENS, NOT LINOLEUM, & MEET CARPET REQ'S. (DON'T USE OTHER HARD SURFACE FLOORING) INDOOR ENVIRONMENT QUALITY CREDIT 7.2 - SEND OUT SURVEYS TO RESIDENCES TO SURVEY (1) INNOVATION IN DESIGN CREDIT 2 - LEED ACCREDITED PROFESSIONAL (1)	

★ ASSUME BLDG MEETS LEED PREREQUISITE

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SUSTAINABILITY BREADTH		2/3	36/
CLOSER EVALUATION OF ATTAINABLE CREDITS.			
<u>SUSTAINABLE SITES</u>			
4.2 (1 CREDIT) - \$			
RESIDENTIAL: PROVIDE COVERED BIKE STORAGE FOR $\geq 15\%$ OF OCCUPANTS.			
15% OF 528 IS 79 RESIDENTS			
USE ... 13342370010 (ALUMINUM FRAME, ACRYLIC GLAZING, 9' X 12' X 8' SHELTER)			
DAILY OUTPUT = 0.73			
BARE MATERIAL = 4950			
BARE LABOR = 946			
05521650002 (INDUSTRIAL RAILINGS, SHOP FABRIC, UNITS = 2 L.F.)			
DAILY OUTPUT = 255			
BARE MATERIAL = 27.03			
BARE LABOR = 5.17			
03305340360 (ON-GRADE, DIRECT CHUTE, 500 PSI, NO FORMS, REIN, FINISH, C.I.)			
DAILY OUTPUT = 150			
BARE MATERIAL = 86.90			
BARE LABOR = 9.20			
IF 1 BIKE IS $\sim 45' \times 1'$ $\rightarrow 79 \times 4.5 \text{ SF} = 356 \text{ SF} \rightarrow$ WOULD NEED ~ 4 SHELTERS			
$\downarrow$ ESTIMATE @ $\sim \$70.00/\text{SF}$ X 400 SF = \$28,000.00			
HALF OF RAILING + 400 SF OF CONCRETE \$26,000 $\rightarrow$ 65 / SF TOO LOW?			
4.4 (2 CREDITS) - FREE			
PROVIDE NO NEW PARKING			
5.1 (1 CREDIT) } - \$\$			
7.1 (1 CREDIT) }			
LANDSCAPE + PROTECT MIN 50% OF HARDSCAPE, SHADE W/ NATIVE VEGETATION			
SO, A TREE COSTS ABOUT \$250 (LABOR, MATERIAL, EQUIPMENT) ON AVERAGE			
SAY A TREE PER 5' X 5' SQUARE WOULD WORK.			
LOTS ABOUT 150' X 250', BUILDING + VARIOUS STUFF WILL PROBABLY TAKE UP			
$85' \times 250' \rightarrow \frac{(150-85) \times 250}{(5 \times 5)} = 650 \text{ "TREES"} \rightarrow \sim \$170,000.00$			
IN BASE LANDSCAPING COSTS			
7.2 (1 CREDIT) - FREE			
USE WHITE-ON-BLACK EPDM ON ROOF			
<u>WATER EFFICIENCY</u>			
PREREQUISITE - FREE?			
CHOOSE MORE EFFICIENT FIXTURES FOR PREFABRICATED BATHROOMS			
(INITIAL INVESTMENT IS ALREADY MADE, JUST PLAN FOR IT)			

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SUSTAINABILITY BREADTH ^{3/3}		37/
2 (1 CREDIT) - FREE HAVE BINS AVAILABLE & PLAN AHEAD. ASU ADVOCATES FOR THIS CREDIT & WILL PROVIDE HELP. 3 (2 CREDITS) } FREE 4 (2 CREDITS) } 20% OF MATERIALS, BY COST, ARE RECYCLED 6 (1 CREDIT) } USE OF RECYCLED STEEL & CONCRETE AGGREGATE WILL EASILY MEET THIS CREDIT (CONCRETE + STEEL ARE ... ~30-35% OF THE COST, IF NOT MORE? CAN ALSO USE RECYCLED FLOORING, CASEWORK, POTENTIALLY GLAZING, DOORS, PARTITION BARRIERS, MAYBE FURNITURE. <u>INDOOR ENVIRONMENT QUALITY</u> 4.1 (1 CREDIT) } FREE 4.2 (1 CREDIT) } WILL EXPEND THE \$ ANYWAY, SO CAN PLAN AHEAD & LOOK INTO ADHERING TO THESE CREDITS. 4.3 (1 CREDIT) } 4.4 (1 CREDIT) } SHOULDN'T HAVE TO PAY EXTRA. 7.2 (1 CREDIT) - FREE-ISH SEND OUT EMAILS OR FLYERS TO RESIDENCES TO EVALUATE THEIR CONTENTEDNESS WITH THE THERMAL COMFORT LEVEL <u>INNOVATION IN DESIGN</u> 2 (1 CREDIT) - FREE HIRE A LEED AP OR USE ONE YOU HAVE ON STAFF.		

04.04.2012

Ksenia Tretiakova, Structural Option
 AE Consultant: Dr. Andres Lepage

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Appendix F –

Bare Material					Labor Costs		Total Costs		Daily Output	
Properties	Existing Tempe, AZ Material Cost	St Louis, MO Material Cost	Units		AZ	MO	AZ	MO	AZ	MO
Welding structural steel in field										
01902200 Continuous fillet weld, 3/4" thick, type 6011	2.7	2.96	ft		26.83	40.49	39.42	54.23	12	12
Shear Stud Connectors										
050523851100 3/4" dia x 8-3/16" L	1.04	1.13	ea.		0.72	1.08	2.18	2.67	890	890
Steel Plates (Used: Structural Steel Projects - 051223.77; Apartments over 15 stories)										
051223770500 Apartments over 15 stories	2605.5	2818.8	ton		346.95	536.85	3078.1	4193.21	13.9	13.9
051223650400 1/2" thick plate	24.61	26.62	SF		-	-	24.61	26.62	-	-
Reinforcing Steel										
032110600700 Walls, #3 to #7	964.41	863.97	ton		448.88	581.18	1413.29	1445.15	3	3
032110600750 Walls, #8 to #18	964.41	863.97	ton		337.73	437.27	1302.14	1301.24	4	4
032110601050 10-50 ton job, #3 to #7, add	10%	10%	-		-	-	-	-	-	-
032110601150 Over 100 ton job, #3 to #7, deduct	10%	10%	-		-	-	-	-	-	-
032110601160 Over 100 ton job, #8 to #18, deduct	15%	15%	-		-	-	-	-	-	-
032110602000 Unloading and sorting, add	-	-	ton		23.09	29.89	29.86	37.27	100	100
032110602210 Crane cost for handling, avg, add	-	-	ton		25.22	32.66	32.62	40.73	92	59.79
032110501020 Shop extra, bending of limited % of bars	33.18	29.73	ton		-	-	33.18	29.73	-	-
032110502200 Shop extra for quantity, 20-49 tons	24.89	22.3	ton		-	-	24.89	22.3	-	-
032110502300 Shop extra for quantity, 100-300 tons	9.96	8.92	ton		-	-	9.96	8.92	-	-
032110502550 Shop extra for size #4	76.74	68.75	ton		-	-	76.74	68.75	-	-
032110502650 Shop extra for size #6	34.74	31.12	ton		-	-	34.74	31.12	-	-
032110502700 Shop extra for size #7-11	46.15	41.34	ton		-	-	46.15	41.34	-	-
Concrete										
Normal Weight										
033105350300 4000 psi	87.76	92.8	yd ³		-	-	87.76	92.8	-	-
033105350412 8000 psi	172.1	182	yd ³		-	-	172.1	182	-	-

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Building Height = 208 ft			
	Existing Design	New Design	
		1	2
Concrete			
Concrete Shear Walls			
f'c (psi)	4000	4000	4000
thickness (in)	8	16	16
LF of wall (ft)	240	264	216
volume (yd3)	1233	2712	2219
bare material cost (\$)	108172	237979	194710
Concrete Coupling Beams			
f'c (psi)	4000	4000	4000
depth (in)	24	36	36
LF of beam (ft)	60	36	84
volume (yd3)	59	107	249
bare material cost (\$)	5201	9361	21842
Steel Reinforcing Bars			
Ordinary Wall Reinforcement (Vertical)			
pmin	0.0012	0.0025	0.0025
Ag (in2)	23040	50688	41472
As min (in2)	27.648	126.72	103.68
bar size (#)	4	4	4
bar size (in)	0.5	0.5	0.5
min # bars	141	645	528
# layers	1	2	2
spacing (in)	20.5	9.8	9.8
real # bars	160.0	792	648
weight (tons)	11.12	55.02	45.02
bare material cost (\$)	10720	53064	43416
Ordinary Wall Reinforcement (Horizontal)			
pmin	0.002	0.0025	0.0025
Ag (in2)	96	192	192
As min (in2/ft height)	0.192	0.48	0.48
bar size (#)	4	4	4
bar size (in)	0.5	0.5	0.5
min # bars	0.98	2.44	2.44
# layers	1.00	2.00	2.00
spacing (in)	12.27	9.82	9.82
new spacing (in)	12	6	6
weight (tons)	16.67	36.68	30.01
bare material cost (\$)	16080	35376	28944
Special Wall Reinforcement (Vertical)			
bar size (#)	-	8	8
bar size (in)	0	1	1
real # bars/element	0	16	16
# elements	0	12	12
height of element (ft)	-	118	118
weight (tons)	0	30.25	30.25
bare material cost (\$)	0	29169	29169

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Special Wall Reinforcement (Horizontal)			
bar size (#)	-	6	6
bar size (in)	0	0.75	0.75
extension into wall flange (in)	-	12	12
spacing (in)	-	5	5
weight (tons)	0	35.54	35.54
bare material cost (\$)	0	34274	34274
Coupling Beam Reinforcement (Longitudinal)			
bar size (#)	4	9	9
bar size (in)	0.5	1.13	1.13
real # bars	4	6	6
weight (tons)	0.96192	6	13
bare material cost (\$)	927.68527	6249	12499
Coupling Beam Reinforcement (Transverse)			
bar size (#)	3	4	4
bar size (in)	0.375	0.5	0.5
bars/ft	1	3.6	3.6
weight (tons)	15.6	80	159
bare material cost (\$)	15044.796	76812	153625
Additional Costs			
weight of #4 bars (tons)	27.8	91.7	75.0
weight of #6 bars (tons)	0.0	35.5	35.5
weight of #8 bars (tons)	0.0	30.2	30.2
total job weight (tons)	27.8	157.5	140.8
% bars bent	-	22.6	25.2
shop extras (\$)	31612	146083	130331
labor costs (\$)	13816	74939	66650
Other Steel			
Shear Stud Connectors			
quantity	0	67840	6120
bare material cost (\$)	0	70553.6	6364.8
Welds			
LF of continuous fillet welds (ft)	0	400	400
bare material cost (\$)	0	1080	1080
labor costs (\$)	0	10732	10732
Steel Plates			
1/2" thick plate area (ft2)	0	794.4	794.4
weight of other steel (tons)	0	1615.7	1615.7
bare material cost (\$)	0	2069809	2069809
labor costs (\$)	0	560567	560567
OVERALL COST (\$)		185601	3262433.3 3191524.7

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CONSTRUCTION BREADTH	MATERIAL COST	36/
ST. LOUIS PRICES VS TEMPE PRICES (+ REDDIGN COSTS)		
CONCRETE VOLUME:		
ORIGINAL • 8" WALLS, 2' DEEP COUPLING BMS		
VOLUME = $\frac{8''}{12} (208') (25' + 19') (6 \text{ C-SHAPED WALLS}) = 36608 \text{ FT}^3 \text{ (WALLS)}$		
$= \frac{8''}{12} (2') (6') (6 \text{ COUPLING BMS PER FLOOR}) (20 \text{ STORIES}) = 960 \text{ FT}^3 \text{ (BEAMS)}$		
TOTAL = 37568 FT ³		
NEW • 16" WALLS, 3' DEEP COUPLING BMS		
VOLUME = $\frac{16''}{12} (208') (25' + 19') (6 \text{ C-SHAPED WALLS}) = 73216 \text{ FT}^3 \text{ (WALLS)}$		
$= \frac{16''}{12} (3') (6') (6 \text{ COUPLING BMS PER FLOOR}) (20 \text{ STORIES}) = 2880 \text{ FT}^3$		
TOTAL = 76096 FT ³		
REINFORCEMENT BARS:		
ORIGINAL • $\rho_{\text{MIN, CORES VERTICAL}} = 0.0012$ FOR BAR $\phi \leq \#5$ [ACI 318-05 14.3.2 a]		
$0.0012 = \frac{A_s}{A_g}$		
$A_g = [(25'-6') \times 6 + (25'-3' \times 2) \times 2 + (25'-3') \times 2 + (25') \times 12 \frac{16}{16} \times 8 \text{ IN}]$ $= 23616 \text{ IN}^2$		
$A_s = 28.24 \text{ IN}^2 @ \#5\text{'S OR LESS}$		
CHECK: $\frac{28.24}{144 \text{ BARS}} = 20.5" > 19" \text{ USE } 163 \text{ BARS}$		
(MAINTAIN CONSISTENCY - USE #4'S 144 BARS OVER ENTIRE CROSS-SECTION (208' UP))		
TONNAGE = $\frac{(163 \text{ BARS}) (0.668 \frac{16}{16} \text{ BAR-FT}) (208 \text{ FT})}{2000 \frac{16}{16} \text{ TON}} = 11.32 \text{ TONS}$		
$\rho_{\text{MIN, CORES HORIZONTAL}} = 0.0020$ FOR BAR $\phi \leq \#4$ [ACI 318-05 14.3.3 a]		
$0.002 = \frac{A_s}{8' \times 12'}$		
$A_s = 0.192 \text{ IN}^2/\text{FT OF HEIGHT} \rightarrow \#4\text{'S @ } 12"$		
LF PER FT HEIGHT = $\left[\begin{array}{l} \text{OUTER CORES} \\ 25' + (25'-3') \times 2 + (25'-6') \times 2 \end{array} \right] \times 2$ $+ \left[\begin{array}{l} \text{INNER CORE} \\ (25'-3') \times 2 + (25'-6') \times 2 \end{array} \right] \times 2 = 246 \text{ LF / FT H}$		
TONNAGE = $\frac{(246 \frac{\text{LF OF BAR}}{\text{FT H}}) (208 \text{ FT H}) (0.668 \frac{16}{16} \text{ LF BAR})}{2000} = 17.1 \text{ TONS}$		
TOTAL TONNAGE = $(11.32 + 17.1) \times 1.1 = 31.3 \text{ TONS}$ ASSUME 10% WASTE		

* MINIMAL MT OF #4'S: 0.668 16/16 FT (ASTM A615 / A706 CHART)

CONSTRUCTION BREADTH	MATERIAL COST	39,
NEW • BOUNDARY ELEMENTS: 4 CORNERS/CORE X 3 CORE = 12 BOUNDARY ELEMENTS GOING UP 118 FT VERTICAL REINFORCEMENT - (16) #8'S PER ELEMENT $\text{TONNAGE} = \frac{(16 \text{ BARS/ELEMENT})(12 \text{ ELEMENTS})(118 \text{ LF})(2.670 \text{ LB/BAR-LF})}{2000} = 30.25 \text{ TONS}$ HORIZONTAL REINFORCEMENT - #6'S @ 5" O.C. → LF OF HOOP: $\frac{13" \times 2 + (14.5" + 12") \times 2}{12} \times 2 \text{ HOOPS} / 5" \text{ OF HEIGHT}$ $= 13.2 \text{ LF OF BAR} / 5" \text{ OF HEIGHT}$ $\text{TONNAGE} = \frac{(13.2 \text{ LF BAR/ELEMENT})}{5" \text{ HEIGHT}} \left(\frac{12" \text{ HEIGHT}}{5" \text{ HEIGHT}} \right) (118 \text{ FT HEIGHT})(12 \text{ ELEMENTS})(1.502 \text{ LB/BAR-LF})$ $= 33.69 \text{ TONS}$ ORDINARY WALL REINFORCEMENT: VERTICAL - (2) #4'S @ 6" O.C. FOR WALL SECTIONS NOT IN BOUNDARY ELEMENT AREA → LF WALL NOT IN ELEMENT AREA = $\left[(25' - \frac{25"}{12} \times 2) \times 2 + (25' - 6' - \frac{25"}{12} \times 2) \times 2 \right] \times 3 \text{ CORES}$ $= 208 \text{ LF OF WALL}$ $\text{TONNAGE} = \frac{(2 \text{ BARS})}{(6" \text{ LENGTH})} \left(\frac{12"}{12"} \right) (208 \text{ FT L}) (208 \text{ FT HEIGHT})(0.668 \text{ LB/BAR-LF})$ $= 115.6 \text{ TONS}$ HORIZONTAL - $\rho_{\min} = 0.0025 \rightarrow (2) \#4'S @ 6" \text{ O.C. (SAME AS VERTICAL)}$ HORIZONTAL REINF GOES ALL THE WAY THRU THE WALLS ∴ LF OF BAR / FT HEIGHT = $\left[(25' - 6') \times 2 + (25') \right] \times 3 \text{ CORES} \times \frac{12"}{6"} = 528 \text{ LF OF BAR / FT HEIGHT}$ $\text{TONNAGE} = \frac{(528 \text{ BAR-LF})}{(12 \text{ FT H})} (208 \text{ FT H}) (0.668 \text{ LB/BAR-LF}) \times \frac{2 \text{ BARS}}{1 \text{ AREA}} = 73.36 \text{ TONS}$ COUPLING BEAMS: $\frac{l_n}{h} = \frac{6'}{3'} = 2$ ∴ MIGHT BE REDUCED TO 2 ∴ RATIO = 3 → USE § 21.3.1 PROVISIONS (ACI 717-05) DIAGONAL REINF. NOT REQ. ASSUME IT IS UNNEEDED.		

	CONSTRUCTION BREADTH ^{3/}	MATERIAL COST ^{3/}	40/
	<u>STEEL COLLARS:</u>		
	ORIGINAL • NONE		
	NEW • STEEL PLATE: UNIT WEIGHT = 490 PCF		
	FOR 5" PLATE: $(490 \frac{lb}{ft^3}) (\frac{5 in}{12 in/ft}) = 204 PSF$		
	TOTAL AREA OF PLATE PER CORE PER FLOOR -		
	LF LENGTH = $(25') \times 2 + (25' - 6') \times 2 = 88 ft \times 3 CORES = 264 ft$		
	HEIGHT = 3 ft		
	AREA/FLOOR = $264 (3) = 792 SF$		
	TOTAL PLATE WEIGHT = $(204 PSF) (792 \frac{SF}{FLR}) (20 FLRS) = \underline{1615.7 TONS OF STEEL PLATE}$		
	<u>SHEAR STUD CONNECTORS:</u>		
	(4) ROWS OF (94) STUDS @ 4" O.C. W/ 3" CLR FROM EDGE ON 25' PLATES.		
	(4) ROWS OF (28) STUDS @ 4" O.C. W/ 4" CLR FROM EDGE ON 10' PLATES		
	+ (1) ROW OF STUDS @ 12" O.C. ON OTHER SIDE OF BOTH PLATES		
	↳ 24 STUDS FOR 25' PLATE		
	9 STUDS FOR 9' PLATE		
	<u>882 STUDS/COLLAR/CORE/FLOOR</u>		
	↳ 2646 STUDS/FLOOR		
	↳ <u>52920 STUDS IN BLDG</u> (ALL $\frac{3}{4}" \Phi$, 8 $\frac{3}{16}" L$)		
	<u>CORBELS:</u>		
	14" LONG 0.5" THICK 5" HIGH + 12" LONG SPACER PLATE (14" WIDE, 0.5" THICK)		
	NEED SF		
	SF PLATE/CORBEL: $\frac{14 \times 5}{144} \times 2 + \frac{12 \times 14}{144} \times 2 = 3.31 ft^2$		
	(1) CORBELS		
	↳ <u>4 CORBELS</u> FLOOR-CORE		
	3 CORES X 20 FLOORS X 3.31 = <u>794.4 ft²</u>		
	WELDS - 5" X 2 SIDES/PLATE X 2 PLATES/CORBEL X 12 CORBELS/FLR X 20 FLRS/12" X 200'		

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CONSTRUCTION BREADTH

SCHEDULING

1/1

4/1/4