technical report 2:

pro-con structural study of alternate floor systems

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table of contents

executive summary

This Pro-Con Structural Study of Alternate Floor Systems Report describes the physical existing conditions of the structure of Granby Tower and addresses four alternative floor framing systems. Appropriate loadings and design assumptions were used to analyze each of the proposed floor framing systems to determine if the current system is the best option when considering cost, story height, lead time, constructability, and architectural impact.

The systems analyzed in this report were chosen for further investigation because they are proven systems for providing maximum floor to ceiling height or ease of construction. The systems chosen include:

1 • Post-Tensioned Two-Way Flat Plate Slab (Existing)

2 • Two-Way Reinforced Concrete Flat Plate Slab

3 • One-Way Reinforced Concrete Slab with Beams and Girders

4 • Non-Composite Steel Frame

5 • Precast Hollow-Core Girder-Slab

After a thorough analysis and comparison of systems, it was determined that the best system for Granby Tower is the existing post-tensioned flat plate slab. For reasons including slab depth, cost, architectural impact, and lead time, this system outperformed the rest. The two-way reinforced concrete flat plate is a viable alternative, and further study could provide insight to more benefits or drawbacks. A two-way flat plate system may take precedence over a post-tensioned system depending on the familiarity of the contractor or local practices, but in this application the larger floor-to-ceiling height and lesser weight of the post-tensioned system made this selection valuable.

The floor framing alternatives that proved inferior for this specific application were the one-way reinforced slab with beams and girders, the non-composite steel, and the girder slab systems. The one-way slab and non-composite alternatives resulted in a decrease in clear floor height by 1 foot and increased susceptibility to vibrations. Despite the minimal intrusion on architecture the overall cost of the systems was prohibitive especially when considering the additional height required to maintain a similar floor to ceiling height. The final system analyzed, the girder-slab system, also produced negative results since alterations on typical bays had to be made. Rearranging column grids would slightly interrupt the floor plans and decrease the value of Granby Tower's luxury apartments. Even though the construction process is expedited and the floor depth remains minimal, the negatives outweigh these benefits. The girder-slab system could possibly be implemented with further study and floor plan alteration, but a detailed cost analysis including construction scheduling would be needed. Since this analysis is out of the realm of this report, the girder-slab system is considered not feasible.

introduction

The Granby Tower (*fig 1*) is a proposed mixed-use, luxury, high rise located in the downtown historic district of Norfolk, Virginia. Historically Granby Street was the premier shopping, dining, gathering and theatre corridor, and these luxuries were supplemented by the direct connection to the Elizabeth River waterfront. The conveniences of Granby Street fell out of favor in the 1960's as suburban development between Norfolk and Virginia Beach promised bargain shopping malls. Due to the decline in popularity of a very important landmark and cultural center, city officials began reviving the city center in the 1970's and are still working to regain the prestige that Granby Street held in the early 1900's.

Granby Tower will be the tallest building in Norfolk upon completion and will provide roughly 300 luxury apartments with views of downtown Norfolk and the Elizabeth River, 6 stories of parking, a roof top fitness center and pool, leasable office space. It is becoming increasingly popular in the Norfolk and Virginia Beach areas to build above parking structures for a number of reasons. One of the most obvious reasons is that you must provide parking space, and since the site has little open space for a free standing garage, the best way to maximize your profit is to utilize the lower floors for parking. The second main reason for an above ground parking structure housed within the buildings structure is due to the sandy soil conditions and high ground water table that don't allow for deep foundations. Most designs, especially heavy concrete structures, require slab on grade with deep piles to penetrate the deep Yorktown Strata layer that is buried beneath layers of unstable sand and clay.

The lateral force resisting system at Granby Tower is designed as a concrete shear wall core which helps to maximize leasable space while keeping most views unobstructed. The floor framing system is a two-way flat-plate post-tensioned slab with minimal drop panels to capitalize on floor to ceiling height. The longest span seen by the slab is 30 feet with typical bays at 26' x 30'. These design features will allow spaces to feel spacious and elegant, and with a design focused on luxury, it is easy to see that Granby Tower will stand as a landmark for the city to celebrate a vibrant history and a promising future.

This report provides a detailed analysis of an investigation into the current floor framing system and four viable alternatives. Evaluation of each system for cost, story height, lead time, and constructability will prove which system is most feasible for Granby Tower.

fig 1 – rendering of Granby Tower

structural overview

foundation

To determine the soil bearing capacity, sixteen (16) 100 to 110-foot deep Standard Penetration Test borings were drilled within the proposed Granby Tower site. Borings were conducted in accordance with ASTM D 1586 standards and performed with rotary wash drilling procedures to analyze the soil types at 5 foot integrals. Soil tests determined that the first 20 feet of most samples consisted of silty fine sand (SM) or poorly graded fine sand (SP-SM). The next 25 feet of bore was composed of clay (CL) followed by 55 feet of poorly graded fine to coarse sand (SP-SM) and/or silty fine sand (SM). Due to the composition of the soil and location of the groundwater table (6 to 7 feet below grade), the geotechnical engineer recommended a deep pile foundation system with driven, precast, pre-stressed, concrete piles since shallow foundations would result in excessive settlements due to the extreme building weight.

To determine the feasibility and required depths of the piles, fifteen test piles were driven with and evaluated with a Pile Driving Analyzer. The analysis dictated the use of 12" square, precast, pre-stressed concrete piles (SPPC) at 80 feet deep with 100 ton capacity and 14" SPPC at 90 feet with 140 ton capacity. Roughly 1000 piles were driven throughout the site (*fig 2*) with 255-14" SPPC piles supporting the ordinary shear wall core. Due to the lateral forces seen by the shear walls, the outer 156 piles are designed for tension. The pile cap supporting the shear wall is 10 feet thick with a 28-day compressive strength (f'c) of 5000 psi and #10 and #11 reinforcing on top and bottom, while all other pile caps will be designed with an f'c of 4000 psi and # 7 and #8 reinforcing.

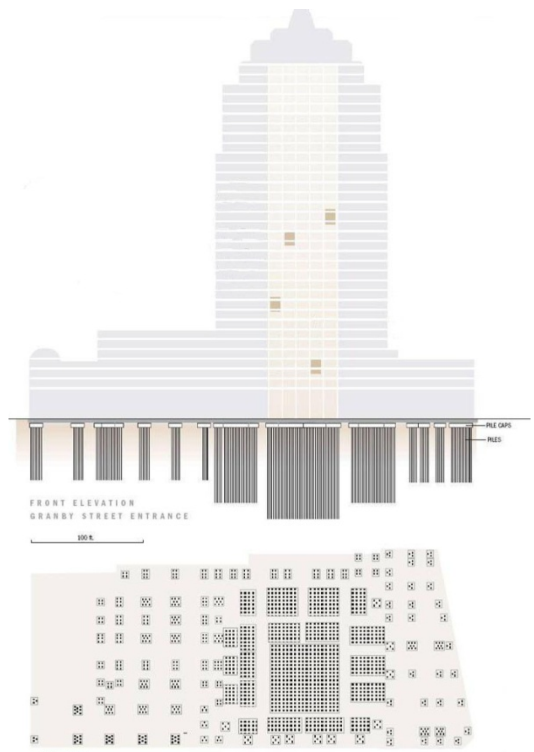

fig 2 – front elevation and plan of piles for Granby Tower. source: Abiouness, Cross and Bradshaw, Inc.

The slab on grade is 5" thick, reinforced with 6x6-W2.9xW2.9 welded wire fabric over a 10 mil polyethylene vapor barrier. The geotechnical engineer specified the slab to be placed over 4" porous fill with less than 5% passing the No. 200 sieve to act as a capillary barrier. The slab should also be "floating" in the sense that it is not rigidly connected to columns or foundations to reduce cracking.

floor system

The floor system for the Granby Tower consists of a two-way flat plate post tensioned slab designed in accordance with the 6th Edition Post-Tensioning Manual by the Post-Tensioning Institute and ACI 318-02. All slabs are designed with a 28-day compressive strength (f'c) of 5000 psi, and the first 7 levels of the tower require a 9" slab while the remaining levels are designed as an 8" slab. Tendons for post-tensioning will be ½" diameter (ø), 7-wire, low relaxation strand, fully encased in grease with a minimum sheathing thickness of 50mm. Maximum sag for tendons will be 5 ½" and supported by chairs or bolsters. Post-tensioning will occur when the concrete has reached 75% of its designed f'c, and all of the uniform tendons shall be stressed before banded tendons. Uniform tendons are evenly distributed through the north-south (long) direction with a maximum span of 26' while banded tendons run east-west (short direction) along column lines with a maximum span of 30'. (see *fig 3*)

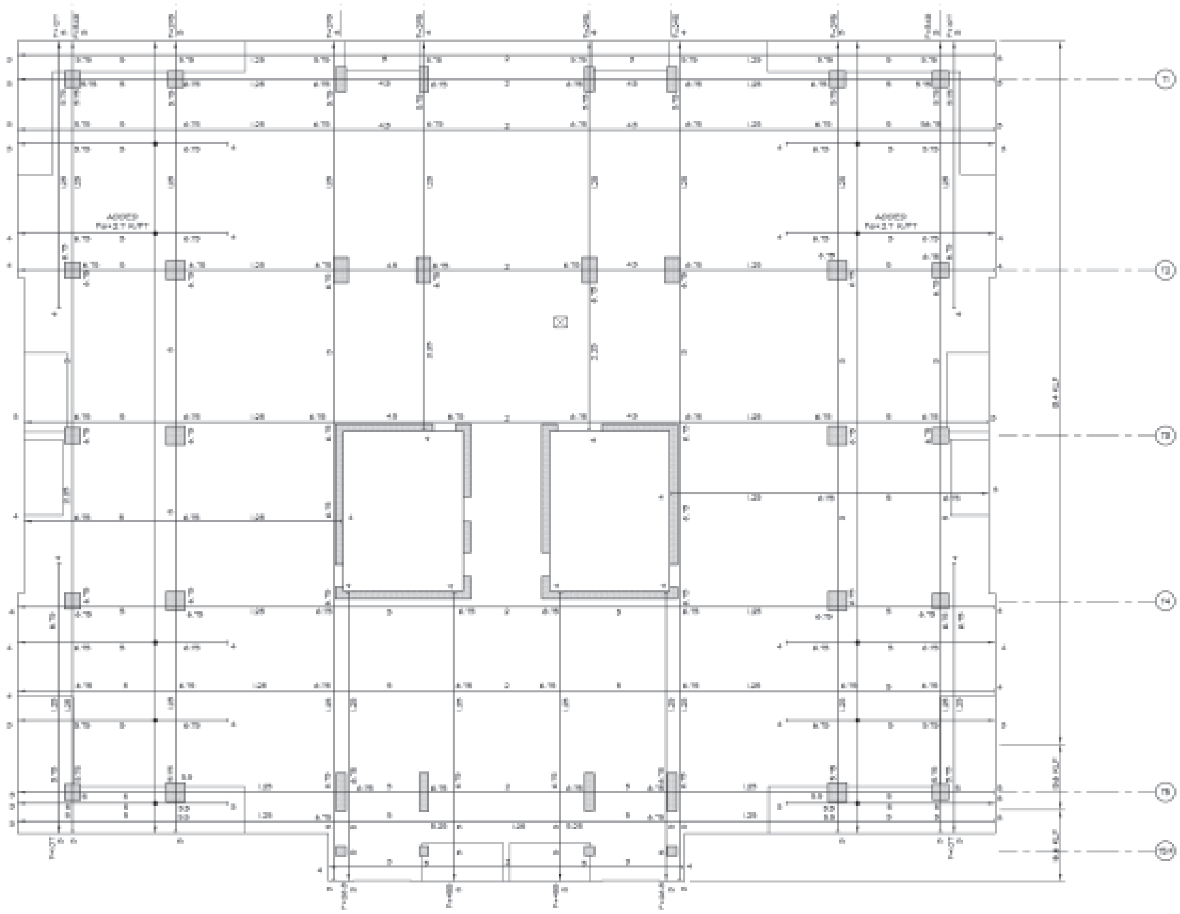

fig 3 – typical post-tensioning plan for levels 8 through 12. Plan and True North **→N**

columns

Gravity columns are laid out on a fairly regular grid with the largest bay at 26'x30'. Roughly 32 columns run the full building height with some of the exterior columns terminating at the buildings first significant set-back on the 29th floor. Most columns are square reinforced columns with rebar ranging from #7 to #10, but rectangular columns with the strong axis in the short building direction (east-west) are architecturally situated in central east and west apartments. Columns above the parking garage (Level 7) are designed with f'c = 5000 psi, and columns between Level 6 and the foundation are designed with f'c = 6500 psi. Banded tendons running through columns should be within 1.5 x T (thickness slab) of the column face and placed above other uniform tendons or rebar. Some drop panels are required on upper floors as column sizes decrease and slab edges become flush with exterior columns.

lateral system

The lateral load resisting system of Granby Tower consists of ordinary reinforced concrete shear walls (*fig 4*) that were designed in accordance to ACI 318-02. The two shear wall cores house the elevators, stairs, electrical and gas lines, and fire dampers. The first 6 levels consist of 24" thick reinforced shear walls with f'c = 8000 psi, while the remaining levels consist of 14" shear walls with 28-day compressive strengths of 6000 (Levels 7 through 23) and 5000 psi (Levels 24 through 34). Typical vertical reinforcement ranges in size and spacing from #10 @ 6" o.c. to #8 @ 12" o.c. while horizontal reinforcement ranges from #6 @ 6" o.c. to #5 @ 12" o.c. Typical end reinforcement consists of ten vertical rebar within a square section determined by the wall width and #4 ties @ 8" o.c vertical spacing from the foundation to Level 7 and #3 ties @ 8" o.c. vertical spacing from Level 7 to 34.

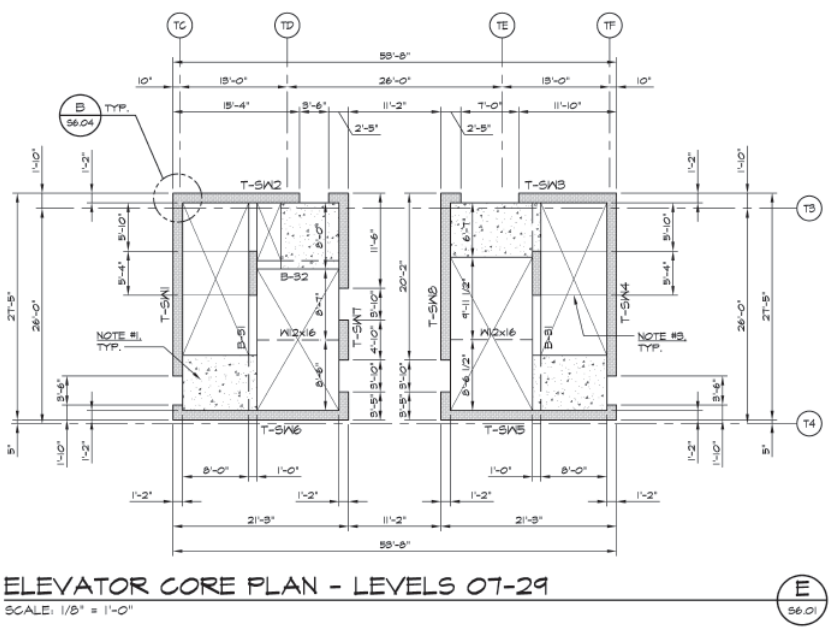

fig 4 – typical plan of shear wall core.

codes

codes and standards

At the time in which the Abiouness, Cross and Bradshaw began structural design of Granby Tower, the overarching permissible codes for design were the 2000 International Building Code (IBC), which references American Society of Civil Engineers (ASCE) 7-98, and Virginia Uniform Statewide Building Code 2000. Concrete was designed in accordance with American Concrete Institute (ACI) 318-99 and all masonry in accordance with ACI 530-99. Post-tensioning design references the 6th Edition Post-Tensioned Manual by the Post-Tensioned Institute, ACI 318-02, and IBC 2000. All steel design references the American Institute of Steel Construction (AISC) ASD 9th Edition, and cold-formed metal design references the 1996 American Iron and Steel Institute (AISI) Specification.

For my analysis of Granby Tower I utilized more recent building codes such as IBC 2006 and ASCE 7-05. All concrete design was based on ACI 318-05, and steel design on the Load and Resistance Factor Design portion of AISC Thirteenth Edition Steel Manual. For analysis of Granby Tower's existing post-tensioning system, I found the 6th Edition Post-Tensioned Manual by the Post-Tensioned Institute invaluable. Two-way reinforced flat plate and one-way reinforced slabs were designed in accordance with ACI 318-05, with reference from Nilson, Darwin, Dolan *Design of Concrete Structures 13th Edition* text, and verified with the Concrete Reinforcing Steel Institute Design Handbook 2002, 9th Edition. Non-composite steel framing was designed in accordance with AISC Steel Construction Manual, 13th Edition, and with reference from West/Geschwindner *Fundamentals of Structural Analysis 2nd Edition* text. Steel decking was designed in accordance with the United Steel Deck Design Manual and Catalogue of Products. Finally the girder-slab system was designed with assistance from Nitterhouse Concrete Products' design tables, and in accordance with Girder Slab Design Guide v1.3.

Cost analyses were carried out using RS Means Building Construction Cost Data 2008 Book, 66th Edition, RS Means Assemblies Cost Data 2008 Book, 33rd Edition, and RS Means Square Foot Costs 2008 Book, 29th Edition.

material properties

materials

Concrete: Normal Weight Concrete

Foundations	f'c = 4000 psi / 5000 psi
Shear Walls	f'c = 8000 psi / 6000psi / 5000 psi
Slab on Grade	f'c = 4000 psi
Elevated Slabs	f'c = 5000 psi
Columns	f'c = 6500 psi / 5000 psi

Reinforcing Steel

Reinforcing Bar	ASTM A615, Grade 60
Welded Wire Fabric	ASTM A185

Structural Steel

Structural Tubing (HSS)	ASTM A500, Grade B, Fy = 46ksi
W-shapes	ASTM A992, Grade 50, Fy = 50 ksi
Other rolled plates and shapes	ASTM A36, Fy = 36 ksi

loads

dead loads

The dead loads for materials used in design of Granby Tower were provided in drawings or sources as noted below.

Dead Loads

Normal Weight Concrete	150 pcf	ACI 318-05
Steel	per shape	AISC 13th Ed.
Steel Deck	2 psf	USD
Partition Wall	15 psf	ASCE 7-05
Miscellaneous	5 psf	

live loads

An extensive list of the live loads used in design of Granby Tower was provided with the structural general notes, but since my analysis was carried out with current codes, all assumed live loads were verified with ASCE 7-05.

Live Loads

Roofs	30 psf
Residential Floors	40 psf
Garage	50 psf
Balconies	100 psf
Public Rooms and Corridors	100 psf
Stairs	100 psf
Roof Garden	100 psf
Mechanical and Electrical Rooms	125 psf

floor systems

For this report the existing and four alternate floor systems were investigated for the appropriateness of installation in Granby Tower. Selection criteria for viable alternatives included minimal floor depth, ease of construction, and lead time. Once analyzed each criteria will be judged on weight, architectural impact, fire protection, vibration, and cost. The systems analyzed and further discussed in this section include:

1 • Post-Tensioned Two-Way Flat Plate Slab (Existing)

2 • Two-Way Reinforced Concrete Flat Plate Slab

3 • One-Way Reinforced Concrete Slab with Beams and Girders

4 • Non-Composite Steel Frame

5 • Precast Hollow-Core Girder-Slab

The bay chosen for analysis contained the largest spans in both directions and is outlined in *fig 5* on the next page. For simplicity of calculation, the selection was assumed to be an interior bay and all columns were assumed to be 36” x 36”. Design assumptions are noted on the calculations for each system that are located in the respective appendix.

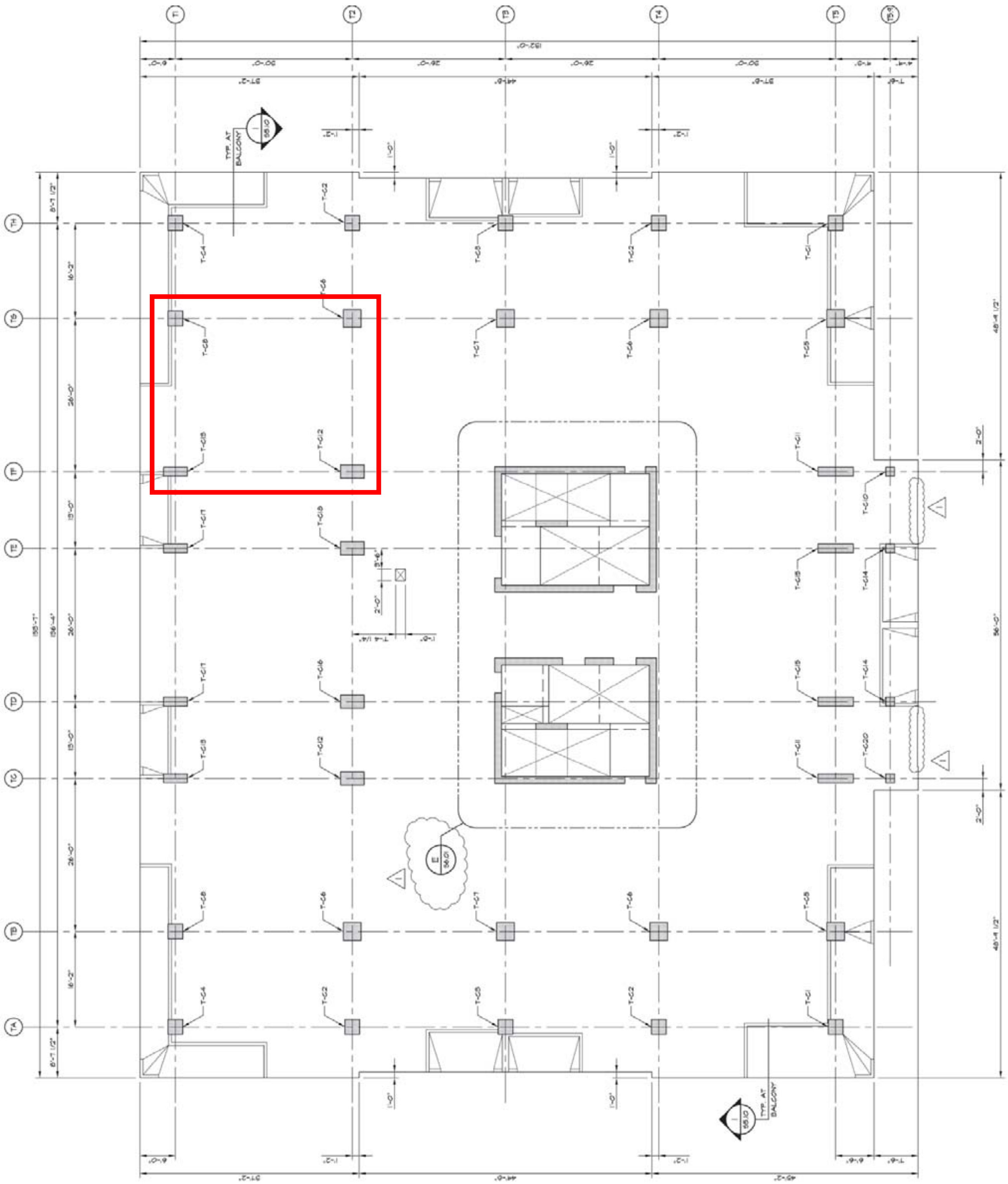

fig 5 – typical floor plan with 26' x 30' bay considered in analysis outlined as shown.

two-way post-tensioned flat plate slab (existing)

The Post-Tensioning Institute's 6th Edition Post-Tensioning Manual outlines a method of determining serviceability of two-way post-tensioned slabs through the equivalent frame method. This approach accounts for the primary moments due to loading and the secondary moments from tendon eccentricity in the columns. All slabs are designed with a 28-day compressive strength (f'c) of 5000 psi, and the first 7 levels of the tower require a 9" slab while the remaining levels are designed as an 8" slab. Tendons for post-tensioning will be ½" diameter (ø), 7-wire, low relaxation strand, fully encased in grease with a minimum sheathing thickness of 50mm. Maximum sag for tendons will be 5 ½" and supported by chairs or bolsters. A typical end detail for tendons, as shown in *fig 6*, displays the termination of tendons at mid-slab height. Post-tensioning will occur when the concrete has reached 75% of its designed f'c, and all of the uniform tendons shall be stressed before the banded tendons. Uniform tendons are evenly distributed through the north-south (long) direction with a maximum span of 26' while banded tendons run east-west (short direction) along column lines with a maximum span of 30'. (see *fig 7*)

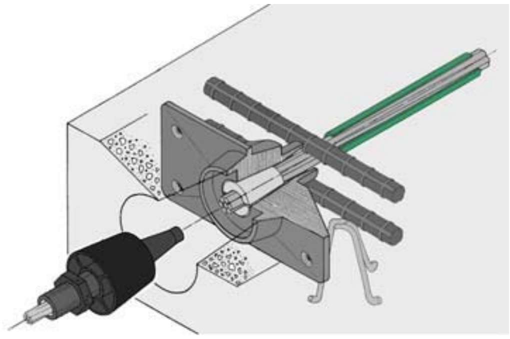

fig 6 – detail of tendon anchor. image provided by ptconcrete.com

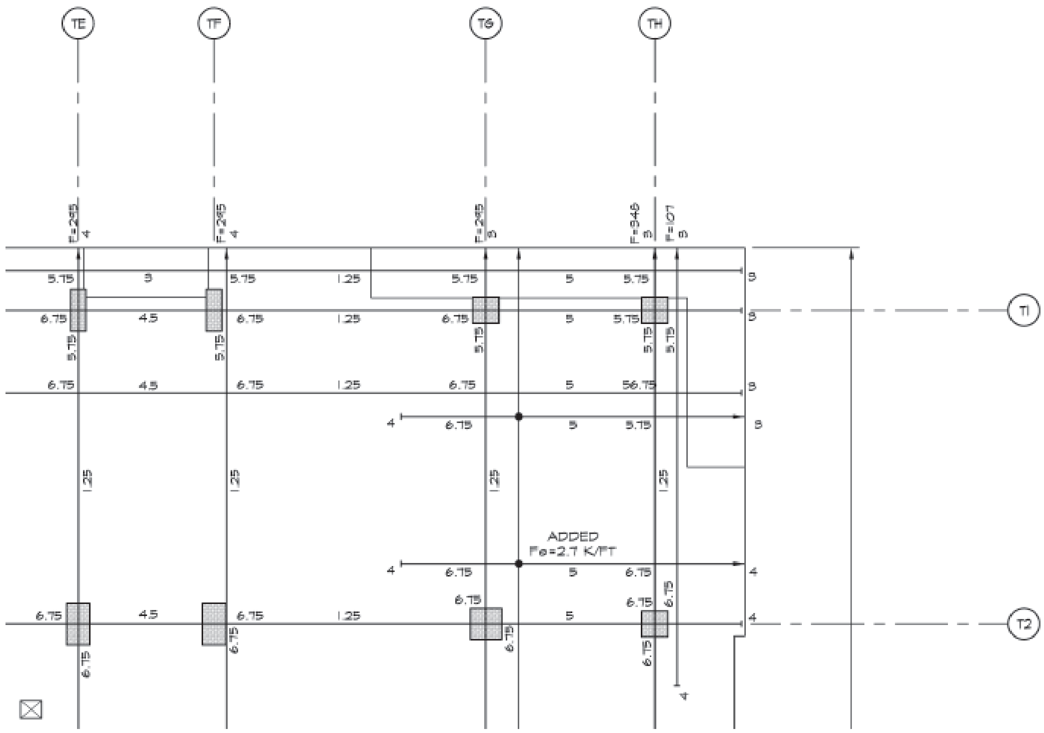

fig 7 – plan of bay analyzed in post-tension calculations

Pro

Post-tensioning is more advantageous than conventional concrete framing for a number of reasons but the main reason why this system is so beneficial stems from the physical properties of concrete. Since concrete fails in tension, post-tensioned members are stressed through cables to put the entire section in compression. This allows for longer spans, thinner slabs, and fewer beams since the sections perform more efficiently. Longer spans are beneficial because they allow for a more open floor plan with less, or more strategically placed columns. Thinner floor slabs mean more floor-to-ceiling height, more floor space if under height restrictions, and less building weight contributing to seismic base shear. Granby Tower benefits from these qualities of post-tensioning because the spacious feel within the apartments contributes to the impression of luxury. In high-rise construction like that of Granby Tower, minimization of floor depth (8 in. plus finish) allows for the possibility of additional floors while maintaining the same building height and incurring little to no additional building cladding costs.

This system has no negative effects on the architecture since the large spans allow for an open floor plan as previously discussed. The analysis of the post-tension floor system as conducted for Technical Assignment 1, proved 30” x 30” columns adequate for gravity loads and moments created after the tendons are stressed. The typical column size considered in each alternate floor framing system analysis was 36” x 36” so by inspection the columns are adequate.

Post-tensioned flat plate slabs are very rigid and dense which are two components for reducing vibration effects. While a thorough vibration study was not conducted, a good rule of thumb for analyzing a floor systems susceptibility to vibration is that heavy, rigid structures experience less vibration than lighter, flexible systems.

Concrete is always readily available so there is very little lead time associated with the construction process. The cost associated with post-tensioning varies per region and contractor experience as discussed below, but after some research I found post tensioning to be the most affordable option.

Con

The challenges associated with post-tensioning are generally closely tied to the contractor’s familiarity with the process. In regions of the country where post-tensioning is common practice due to height restrictions, the likelihood of a contractor having experience is high. The construction process is generally more intensive because tendons must be laid out in a very methodical fashion, shoring is required during concrete placing, and stressing must occur at planned integrals when the slab has reached the proper compressive strength. For these reasons the construction process will take longer than a precast system because you must wait for the concrete to reach 75% compressive strength. For inexperienced contractors, this may result in a higher bid to balance some of the learning curve, but after talking with some professionals in the Norfolk/Virginia Beach area, I’m confident that post-tensioning is common practice and will be conducted effectively under the supervision of Turner Construction Company.

two-way reinforced concrete flat plate slab

A two-way flat plate system (*fig 8*) rests directly on columns so the system must be primarily designed for shear since there are no column capitals or drop panels. The direct design method, as discussed in Nilson, Darwin, Dolan *Design of Concrete Structures 13th Edition* text, considers the strip of concrete along each column line as beams within the slab. The column strips in each direction are assumed to take more of the flat plate shear than the middle strips, so they are reinforced more thoroughly. The slab should be analyzed for punching shear since shear generally controls the design of two-way flat plate systems.

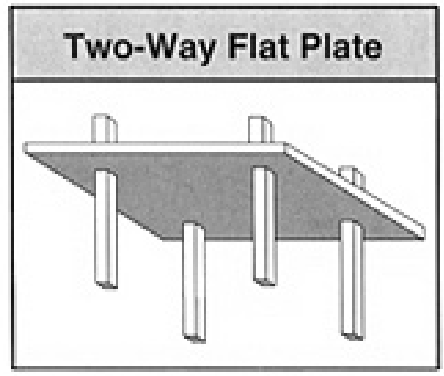

fig 8 – rendering of typical two-way flat plate construction. image provided by crsi.org.

A typical 26' x 30' bay was considered in design of a two-way reinforced concrete flat plate slab and an 11" normal weight concrete slab to be adequate. To determine an acceptable slab thickness for an L/360 deflection limit (ACI 318-05, Table 9.5(b)), ACI 318-05, Table 9.5(c) specifies minimum slab thicknesses per span. Reinforcement included # 7 bars running in both the column strips and middle strips. Direct design analysis and bar cut off reqirements in accordance with ACI 318-05, Fig. 13.3.8 are provided in appendix b.

Pro

One benefit of a two-way flat plate reinforced slab is the depth of the slab since there are no drop panels or supporting beams. Maintaining a minimal floor depth depends primarily on the span desired, so in the situation of Granby Tower, the larger spans require a slightly deeper slab to prevent excess deflection. Some of the benefits of a slightly larger slab include less risk of punching shear at the columns, less vibration of the floor slab due to a denser, heavier slab, and no additional fire protection. The deflection limit for design of a two-way flat plate slab is L/360, so the deflection of the system is less than an inch.

Con

As previously mentioned, the construction process is slightly simplified due to the flat plate, but there is a bit more work that goes into reinforcing a deep slab. Shoring is also neccesary during concrete placement, and as with all cast-in-place slabs, there is extra time factored into the schedule for formwork to be built and shored, concrete to cure, and formwork to be stripped; and then repeated. The slightly higher cost associated with this alternative is associated with the extra material required for the slab since much of the reinforcing is similar between post-tensioned and two-way flat plate slabs. Since this alternative is heavier than the existing system, additional investigation into the capacity of the foundation would need to be considered.

one-way reinforced concrete slab with beams and girders

One way slabs utilize reinforcement spanning in one direction while beams support the weak direction. A typical one-way concrete slab can span long distances with the aid of deep members, or conversely, slab and beam depth can be minimized with shorter spans. Because design requirements for Granby Tower include longer spans and less floor depths, I chose to analyze a one-way system with beams and girders (*fig 9*). By shortening the effective span between beams I was able to design a thinner slab, which in turn meant more, shallower beams and girders. The design calls for a 4 ½" slab reinforced with #3 reinforced bars @ 12" o.c. on top and bottom. Beams with a length of 26 feet and tributary area of 10' require a depth of 15" and width of 12". Girders with a length of 30', experiencing equal loading at beam intersections, are 18" deep and 14" wide. Other scenarios were analyzed using multiple 30' beams framing into 26' girders, but this required a thicker slab and resulted in deeper overall members. All element capacities, reinforcing, and deflections were verified with the CRSI Design Handbook.

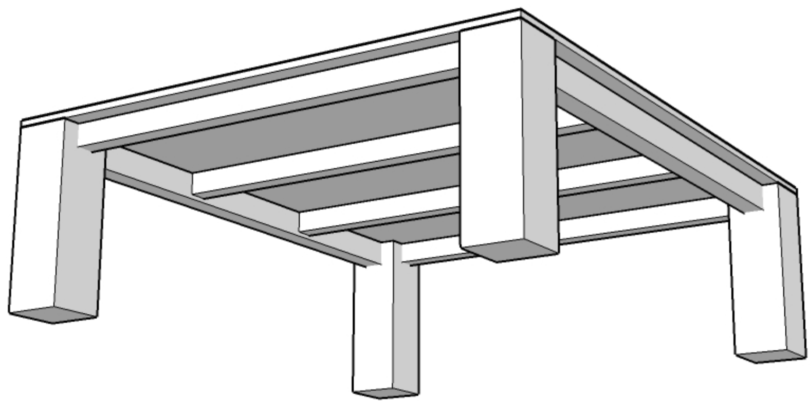

fig 9 – drawing of typical bay analyzed for one-way system with beams and girders.

Pro

An advantage of a one-way slab system includes most contractor's familiarity with the process of installation. Since the slab is able to perform more efficiently due to the strategically placed beams and girders, a lower building weight is associated with this system and therefore less strain is placed on the foundation and deflection of beams and girders is minimal ($\leq$ 0.75"). The lead time associated with concrete construction, as discussed earlier is minimal.

Con

The main drawback to this system is a floor depth of 18". This is 10" deeper than the existing post-tensioned slab and would most likely be considered unacceptable. The amount of leasable space lost due to a floor system of this depth would equate to roughly 3 floors. The cost associated with a one-way slab ($18.50/ft^2) is slightly higher than other concrete floor systems since more man hours are needed to prepare formwork, reinforce the beams and girders, and strip formwork when appropriate. For the purposes of Granby Tower, these negative qualities seem to outweigh the few, expected benefits.

non-composite steel framing

This basic steel floor framing system (*fig 10*) uses standard steel shapes with a ribbed steel deck that supports a thin concrete slab. Although this steel framing is not capable of delivering slender floor depths like concrete flat plates, I still felt it worthwile to analyze this alternative because the construction process is generally faster than concrete framing.

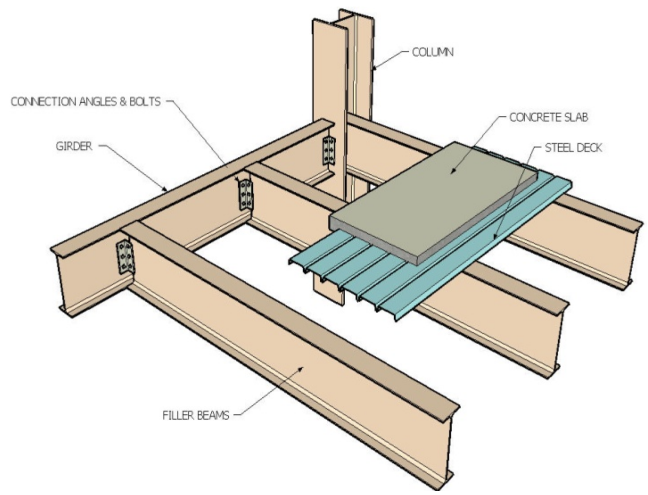

fig 10 – drawing of floor framing for non-composite steel floor. image provided by dehli.edu.

Design of this system was carried out with refence to West/Geschwindner *Fundamentals of Structural Analysis 2nd Edition* text and in accordance of AISC standards. The composite steel deck/ slab system chosen from United Steel Deck Design Manual and Catalogue of Products was a 4" concrete slab reinforced with 6x6 W1.4x1.4 WWF and interacting compositely with a 19 gauge, 1.5" LOK floor deck. The deck and slab chosen were sized so that no shoring would be required during concrete placement.

Initially beam selection ranged in sizes from A992 W10x17 through W12x16; all of which were able to develop composite action with the deck and slab. But, due to deflection limits, none of these sections had a large enough moment of inertia to interact compositely with the slab. Therefore, I choose W12x30 non-composite beams and non-composite W12x72 girders. A preliminary column analysis was also carried out to determine that W14x176 columns would be needed in place of the existing 36" x 36" reinforced concrete columns.

Pro

Steel framing systems' greatest advantage is the speed of construction. Once steel has been detailed and procured, assembly is accelerated since little time is spent preparing formwork, shoring, or waiting for concrete to cure. The slab depth is minimal which contributes to a lower system weight, and in my analysis I designed the slab and deck to require no shoring during concrete placing to expedite construction.

Con

While steel framing is generally a very cooperative building system, my analysis proved that it would not be very effective in Granby Tower. Firstly, switching from a concrete only building to a steel framed building would require some investigation into a possible alternative lateral system such as braced frames. If the designer chose to keep the shear walls, additional study would be required for the connection between the framing members and the shear wall.

The overall floor depth of a 4 inch slab (including flute and topping) resting on W12 members is roughly 16 inches, not including a necessary drop ceiling to conceil the structure. This added floor depth equates to roughly 2 floors worth of leaseable space. Architectually this outcome would most likely be unacceptable despite the low impact of the existing floor plan. Since this is a lighter system, as previously mentioned, floor vibrations would not be attenuated as easily as would be in a stiffer slab.

Unlike concrete only systems, steel framed systems must add fire protection since intense heat will sacrifice the strength properties of the members. While fire protection is a fairly easy spray-on product, the additional cost contributes to this flooring alternative's high cost. The material cost is higher than concrete since it must be detailed and manufactured off site, transported, and then lifted into place with a crane. So despite the east of construction associated with this system, the lead time is a major drawback.

girder-slab

The Girder-Slab System (*fig 11*) is a proprietary product developed by Girder-Slab Technologies LLC to develop composite action between hollow-core concrete planks and integrated steel girders. Interior girders called D-Beams (an open-web dissymetric beam) are connected to precast planks with cementitous grout. The advantage to a system such as this is a very shallow floor depth as would be possible with flat plate construction, but an expedited construction process due to precast products.

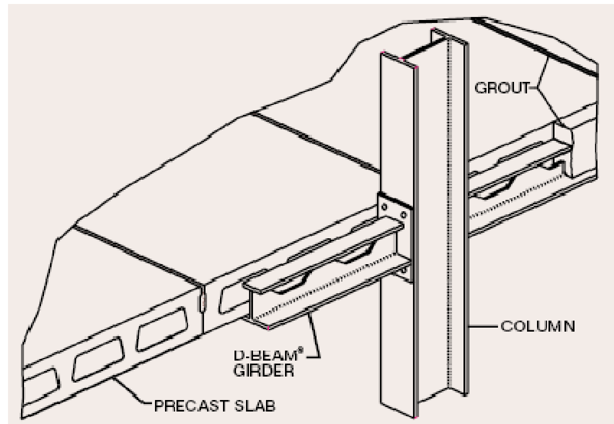

fig 11 – typical cut-away section of Girder-Slab construction including D-Beams and hollow core precast planks. Image provided by girder-slab.com

Precast panels were selected from the Nitterhouse Concrete Products design tables and chosen to span the 30' direction. The planks chosen were 8" x 4' hollow core plank, reinforced with (7) ½"Ø prestressing strands. This specific plank is topped with 2" of cast in place concrete to create a smooth finish. Refer to *fig 12*.

Selecting an appropriate D-Beam was aided with the Girder-Slab System D-Beam Calculator Reference Tool provided on the company's website. The spread sheet allowed me to analyze several scenarios to find the most advantageous layout. The resulting selection was DB 9 x 46; which is a transformed W14x61. The maximum achievable span with the Girder-Slab system was 16' so this involved adding several columns. Only 8 extra columns were needed since some bays are already 16' x 30' but nonetheless some of the additional columns would interfere with the floor plans. Preliminary column checks were also carried out for the Girder-Slab system and determined that W14x176 were required.

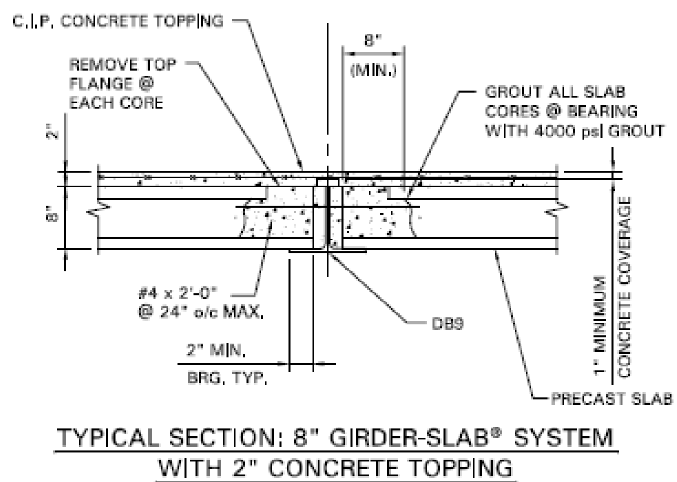

fig 12 – typical section provided by girder-slab.com

Pro

The Girder-Slab system was developed to address the floor depth issues associated with precast concrete planks and precast concrete girders. By integrating girder and plank systems to develop compositely, the floor depth remains minimal (10" plus finish). As with most precast products, the construction process is much faster since little time is spent preparing the framing members to receive a slab. The ease of construction is a huge benefit of this system because a speedier construction process will reduce the overall project costs.

Con

The main drawback to this system is the need to rearrange the column grid slightly to adapt to the span limitations of D-Beams (*fig 13)*. While minimal change is required, the architectural impact of stray columns will detract from the feeling of elegance. If desired, architectural study could be done to consider how to properly integrate this system with the existing floor plan, but the benefits of the other systems may deter one from considering further investigation.

Other negative aspects of this a precast girder-slab system include fire protection, vibration, and lead time. As with the non-composite framing system, fire protection is needed at all columns and results in additional cost. This system may be more susceptible to vibrations since the weight is relatively low, but more study could address this issue. Lastly, the lead time associated with this system would be much higher since two proprietary products are specified.

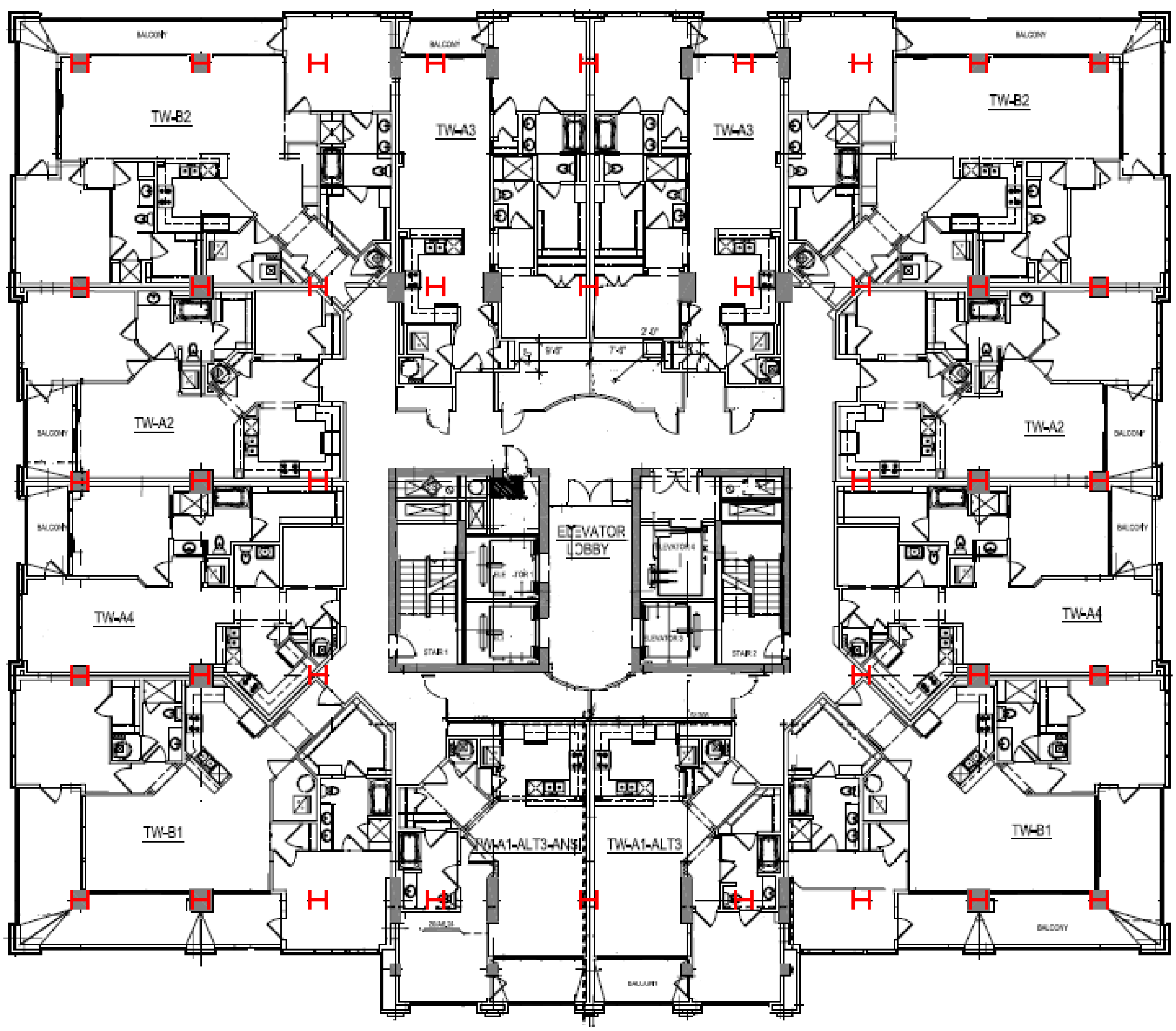

fig 13 – typical floor plan with possible alternate column arrangement. interface between slab and shear wall assumed integrated without columns.

system comparisons

Criterion	Existing Post-Tensioned	Two-Way Flat Plate	One-Way Beams & Girders	Composite Steel	Girder - Slab
System Weight *	100 psf	138 psf	77 psf	56 psf	82 psf
Slab Depth	8 in.	11 in.	4.5 in.	4 in.	10 in.
Total Depth	8 in.	11 in.	18 in.	16 in.	10.5 in.
Deflection	n/a	1 in.	0.75 in.	1.37 in.	0.96 in.
Bay Size	26' x 30'	26' x 30'	26' x 30'	26' x 30'	16' x 30'
Column Size	36" x 36"	36" x 36"	36" x 36"	W14 x 176	W14 x 176
Architectural Impact	none	none	Low ceiling	Low ceiling	Bay size
Fire Rating	2 hour	2 hour	2 hour	1.5 to 2 hour	2 to 3 hour
Fire Protection	none	none	none	Spray	Spray
Vibration	Great	Great	Average	Poor	Average
Lead Time	Short	Short	Short	Long	Long
Constructability	Hard	Medium	Medium	Easy	Easy
System Cost	$ 12.80/ft^2	$ 14.40/ft^2	$ 18.50/ft^2	$ 25.20/ft^2	$ 16.58/ft^2
Column Cost **	$ 4.11/ft^2	$ 4.52/ft^2	$ 4.31/ft^2	$ 5.66/ft^2	$ 6.92/ft^2
Total Cost	$ 16.91/ft	$ 18.93/ft^2	$ 22.81/ft^2	$ 30.86/ft^2	$ 23.50/ft^2
Feasibility	Very	Good	No	No	Moderate
Further Study	Yes	Yes	No	No	No

**System weight includes slab weight, deck material, and all beams and girders. Column weights not considered although concrete columns much heavier than the proposed W14 columns.*

***Additional column cost in cast in place concrete columns assumed for extra reinforcing since column size remains constant for all concrete systems.*

conclusion

This second technical report proves that the existing structural floor framing system, a two-way post-tensioned flat plate concrete slab, is the best option available for Granby Tower. For reasons including slab depth, cost, lead time, architectural impact, and vibration susceptibility, a post-tensioned slab possessed the qualities that warranted it more viable than any other alternative floor systems.

A two-way reinforced flat plate slab was the next best alternative to the existing system. The slab depth is designed to be 3” larger than the post-tensioned slab, and this occurs because the two-way flat plate was designed primarily for shear capacity. Designing a floor slab with stud rails would minimize the slab depth and possibly even out the benefits of the two-way flat plate and post tensioned systems. With all factors besides slab depth and cost being the same, this system can still be considered a viable option since post-tensioning effectiveness relies on the contractor’s experience. If the building were proposed for an area that post-tensioning was not common, a two-way reinforced flat plate slab could be used instead.

The Girder-Slab alternative would be the next best option due to the ease of construction and accelerated construction process, but the major draw-back to this system was the change in column spacing. This new column grid would cause some minor problems with apartment floor plans, and would require additional study to investigate a dual steel framed/shear wall system or an alternative lateral resisting system. Additional study would also be required to analyze the impact on the parking garage that is part of the lower 6 levels since close column spacing would not be ideal. If this system was considered during the design phase and column placement was taken into consideration as discussed, a Girder-Slab system would be a worthwhile alternative for upper floors.

The last two alternative floor framing systems analyzed, a one-way slab with beams and girders and a non-composite steel framing system, were both ruled out for the extreme floor thicknesses required. While neither option caused much negative architectural impact besides reduction in floor height, the overall cost of each system was incongruous with the resulting product. Therefore neither option should be considered for Granby Tower.

appendix a

two-way post-tensioned flat plate slab (existing)

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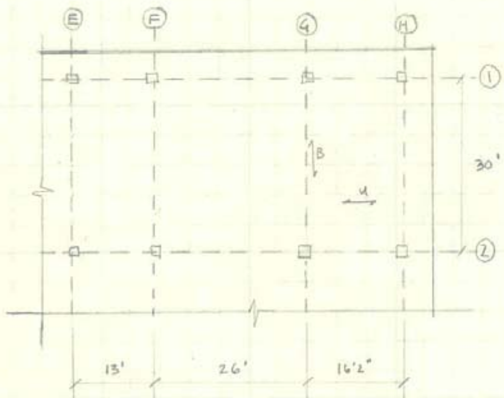
FLOOR SYSTEM ANALYSIS | GRANBY TOWER | POST TENSION | 1



BANDED TENDONS ALONG E,F,G,H

UNIFORM TENDONS IN 1,2... DIRECTION

ASSUME:

COLUMNS : 30 x 30 in
STORY HEIGHT : 10'3"

f'_c = 5000 psi
w = 150 PCF
f_y = 60000 psi
f_{pu} = 270000 psi

LL = 40 PSF (RESIDENTIAL FLOORS)
SIDL = 20 PSF

ANALYZE LEVEL 8, POST-TENSIONED FRAME (TYP.) [POST-TENSIONED MANUAL, 6th ED.]

→ SLAB

SLAB/DEPTH RATIO : $\frac{L}{45}$ [2-WAY SLAB, COL. ONLY, CONTINUOUS FLOOR SPAN]

LONGITUDINAL = $\frac{30(12)}{45}$ = 8 in ← USE 8 in SLAB [USED IN DESIGN]

TRANSVERSE = $\frac{26(12)}{45}$ = 6.9 in

→ LOAD BALANCING

DEAD LOAD (SLAB) : $\frac{8}{12}$(150 PCF) = 100 PSF

LIVE LOAD : 26x30 BAY → L = 40 PSF $\left(0.25 + 15/\sqrt{(1)(780\ SF)}\right)$ = 40 PSF(0.79) = 31 PSF

16x30 BAY → L = 40 PSF $\left(0.25 + 15/\sqrt{(1)(484\ SF)}\right)$ = 40 PSF(0.93) = 37 PSF

TOTAL FACTORED LOAD : 1.2(120 PSF) + 1.6(37 PSF) = $\boxed{203.2\ PSF}$

TENDONS TAKE 80% SLAB WEIGHT

w_{BAL} = 0.8(100 PSF) = 80 PSF

a = 8 in − 1.25 − 1.25 = 5.5 in (MAX TENDON SAG) [VALUES PER DESIGN]

Fe = $w_{BAL} L^2 / 8a$ = (80 PSF)$(30')^2$ / 8$\left(\frac{5.5}{12}\right)$ = 19.6 k/FT

FLOOR SYSTEM ANALYSIS | GRANBY TOWER | POST TENSION / FRAME ANALYSIS 2

→ ASSUME 14 KSI LONG TERM LOSSES AND 1/2" ϕ, 7-WIRE STRAND

$0.153\left((0.7)(270\ ksi) - 14\ ksi\right) = 26.8\ k\ /\text{TENDON}$

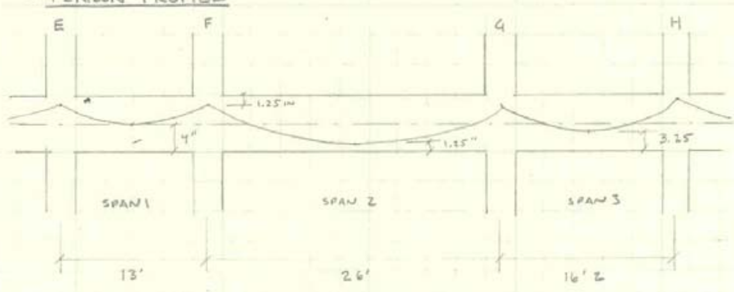
$\#\text{TENDONS} = (30')(19.6k/ft)/(26.8k/\text{TENDON}) = 21.9\ \text{TENDONS}$

USE 22 TENDONS

$F_e = 22(26.8\ k/\text{TDN})/30' = 19.65\ k/FT$

$F/A = 19.65/(8\times 12) = 0.205\ ksi$

→ TENDON PROFILE



ADJUST TENDON PROFILE

$W_{BAL} = \frac{8F_e a}{L^2} = 8(19.65)(5.5/12)/(26')^2 = 0.107\ ksf$

$\cdot\ a_1 = \frac{W_{BAL}L^2}{8F_e} = 0.107(13')^2/8(19.65) = 1.38\ in$

MIDSPAN CGS $= \frac{1}{2}(4+6.75) - 1.38\ in = \underline{4\ in}$

ACTUAL SAG, SPAN 1 $= \frac{1}{2}(4+6.75) - 4\ in = 1.375\ in$

ACTUAL BALANCED LOAD : $W_{BAL} = 8(19.65)(1.375/12)/(13')^2 = \underline{0.1066\ ksf}$

$\cdot\ a_3 = \frac{W_{BAL}L^2}{8F_e} = 0.107(16'2")^2/8(19.65) = 2.13\ in$

MIDSPAN CGS $= \frac{1}{2}(4+6.75) - 2.13\ in = \underline{3.25\ in}$

ACTUAL SAG, SPAN 3 $= \frac{1}{2}(4+6.75) - 3.25\ in = 2.125\ in$

ACTUAL BALANCED LOAD : $W_{BAL} = 8(19.65)(2.125/12)/(16'2")^2 = \underline{0.1065\ ksf}$

FLOOR SYSTEM ANALYSIS | GRANBY TOWER | POST TENSION 3

NET LOAD CAUSING BENDING :

SPAN 1 : $W_{NET} = 0.160 - 0.1066 = 0.0534$ KSF

SPAN 2 : $W_{NET} = 0.151 - 0.107 = 0.044$ KSF

SPAN 3 : $W_{NET} = 0.157 - 0.1065 = 0.0505$ KSF

→ EQUIVALENT FRAME

* ASSUME ALL COLUMNS 30 × 30 IN FOR SIMPLICITY , H = 10.25'

COLUMN STIFFNESS :

$I = \frac{1}{12}(30)(30^3) = 67500 \text{ in}^4$

$E = E_{COL} / E_{SLAB} = 1.0$

$k_c = \frac{4EI}{L-2h} = 4(67500)(1.0) / (10.25(12) - 2(8\text{ in})) = 2523.4 \text{ in}^3$

TORSIONAL STIFFNESS :

$C = \left[1 - 0.63\frac{x}{y}\right]\frac{x^3 y}{3} = \left[1 - 0.63\left(\frac{8}{30}\right)\right]\left(\frac{(8)^3(30)}{3}\right) = 4260 \text{ in}^4$

$K_t = \frac{9CE}{L_2(1 - c_2/L_2)^3} = (9(4260)(1.0)) / ((30 \times 12)(1 - 1.33/30')^3) = 122 \text{ in}^3$

$K_{EC} = \left[\frac{1}{\Sigma k_c} + \frac{1}{\Sigma K_t}\right]^{-1} = \left[\frac{1}{2(2523.4)} + \frac{1}{2(122)}\right]^{-1} = 232.7 \text{ in}^3$

SLAB STIFFNESS :

$K_{S1} = [4(1.0)(30')(8")^3] / [12(13') - (30/2)] = 435.8 \text{ in}^3$

$K_{S2} = [4(1.0)(30')(8")^3] / [12(26') - (30/2)] = 206.9 \text{ in}^3$

$K_{S3} = [4(1.0)(30')(8)^3] / [12(16'2") - (30/2)] = 343.2 \text{ in}^3$

FIXED END MOMENTS

SPAN 1 : $0.0534(13)^2/12 = 0.752$ ft k

SPAN 2 : $0.044(26)^2/12 = 2.479$ ft k

SPAN 3 : $0.0505(16'2")^2/12 = 1.10$ ft k

FLOOR SYSTEM ANALYSIS | GRANBY TOWER | POST TENSION | 4

DISTRIBUTION FACTORS

SPAN 1, EXT : $435.8/(435.8 + 232.7) = 0.65$

SPAN 3, EXT : $343.2/(343.2 + 232.7) = 0.60$

SPAN 1, INT : $435.8/(435.8 + 232.7 + 206.9) = 0.50$

SPAN 2, COL F : $206.9/(435.8 + 232.7 + 206.9) = 0.24$

SPAN 2, COL G : $206.9/(206.9 + 232.7 + 343.2) = 0.26$

SPAN 3, INT : $343.2/(206.9 + 232.7 + 343.2) = 0.44$

	E	F		G		H
DF	0.65	0.50	0.24	0.26	0.44	0.60
FEM	-0.752	0.752	-2.479	2.479	-1.1	1.1
DIST	0.489	0.864	0.414	-0.359	-0.607	-0.66
CO	0.432	0.245	-0.180	0.207	-0.330	-0.304
DIST	-0.281	-0.033	-0.016	0.032	0.054	0.182
	-0.112	1.828	-2.261	2.359	-1.983	0.318

→ NET TENSILE STRESSES

$-M_{MAX}$ @ FACE OF COL F

$$-M_{max} = -2.261 + \frac{1}{3}\left(\frac{0.044(26)}{2}\right)\left(\frac{30}{12}\right) = -1.784 \text{ ft k}$$

$$S = bh^2/6 = 12(8)^2/6 = 128 \text{ in}^3$$

$$f_{t,b} = -f_{pc} \pm \frac{M_{NET}}{S_{t,b}} = -0.205 \pm \frac{12(1.784)}{128}$$

$$= -0.038,\ -0.372 \text{ ksi} \quad \text{(NO TENSION)}$$

ALLOWABLE COMPRESSION

$0.6 f'_c$ (AT TRANSFER, $f'_{ci} = 0.75 f'_c$) $= 0.6(0.75)(5 \text{ ksi}) = 2.25 \text{ ksi} > f_{t,b}$ OK ✓

$0.45 f'_c$ (AT SERVICE LOAD) $= 0.45(5 \text{ ksi}) = 2.25 \text{ ksi} > f_{t,b}$ OK ✓

FLOOR SYSTEM ANALYSIS | GRANBY TOWER | POST TENSION | 5

+MMAX @ MIDSPAN OF SPAN 2

$+M_{MAX} = \left(\frac{(0.044)(26)^2}{8}\right) - 2.479 = 1.239$ ft k

$f_{t,b} = -f_{pc} + \frac{M_{net}}{S} = -0.205 + \frac{12(1.239)}{128}$

$= -0.089, -0.32$ ksi (NO TENSION)

ALLOWABLE COMPRESSION : 2.25 ksi $> f_{t,b}$ OK ✓

→ FLEXURAL CAPACITY

$FEM_1 = (0.1066)(13)^2/12 = 1.5$ ft k ✓

$FEM_2 = (0.107)(26)^2/12 = 6.03$ ft k

$FEM_3 = (0.1065)(16'2'')^2/12 = 2.32$ ft k

	E		F		G		H
DF	0.65	0.50	0.24	0.26	0.44	0.60	
FEM	-1.5	1.5	-6.03	6.03	-2.32	2.32	
DIST	0.975	2.265	1.087	-0.965	-1.632	-1.392	
CO	1.133	0.488	-0.482	0.544	-0.696	-0.816	
DIST	-0.736	-0.003	-0.001	0.040	0.067	0.49	
	-0.128	4.25	-5.426	5.649	-4.581	0.602	

→ SECONDARY MOMENTS

$M_{1,EXT} = 0.128 - 19.6(4-1.25)/12 = -4.36$ ft k

$M_{3,EXT} = 0.602 - 19.6(2.75)/12 = -3.8$ ft k

$M_{1,INT} = 4.25 - 19.6(2.75)/12 = -0.24$ ft k

$M_{2,F} = 5.426 - 4.492 = 0.93$ ft k

$M_{2,G} = 5.649 - 4.492 = 1.16$ ft k

$M_{3,INT} = 4.581 - 4.492 = 0.09$ ft k

FLOOR SYSTEM ANALYSIS | GRANBY TOWER | POST TENSION | 6

→ FACTORED LOAD MOMENTS

$FEM_1 = 0.208\,(13)^2/12 = 2.93$ ft k

$FEM_2 = 0.194\,(26)^2/12 = 10.93$ ft k

$FEM_3 = 0.203\,(16'2'')^2/12 = 4.42$ ft k

	E	F	F	G	G	H
DF	0.65	0.50	0.24	0.26	0.44	0.60
FEM	-2.93	2.93	-10.93	10.93	-4.42	4.42
DIST	1.905	4.00	1.92	-1.693	-2.86	-2.652
CO	-2.00	0.953	-0.846	0.96	-1.326	-1.432
DIST	-1.3	-0.054	-0.026	0.095	0.161	0.859
SUM	-0.325	7.829	-9.882	10.292	-8.439	1.195
2ND MOM	-4.36	-0.24	0.93	-0.93	0.09	-3.8
@ COL	-4.685	7.589	-8.952	9.362	-8.349	-2.605

→ DESIGN MOMENTS @ MIDSPAN

SPAN 1:

$$V_{EXT} = \left[\frac{0.208\,(13)}{2} - \frac{7.589 - 4.685}{13}\right] = 1.13 \text{ k/ft}$$

$V_{INT} = 1.43$ k/ft

POINT OF ZERO SHEAR AND MAX MOMENT:

$x = 1.13/(0.208) = 5.43$ FT FROM COL E ℄

POSITIVE MOMENT

$M_{MAX} = 0.5\,(1.13)(5.43) - 4.685 = -1.62$ ft k/ft

SPAN 2:

$$V = \left[\frac{0.208\,(26)}{2} - \frac{9.362 - 8.952}{26}\right] = 2.69 \text{ k/ft}$$

$x = 2.69/(0.208) = 12.92$ FT

$M_{MAX} = 0.5\,(2.69)(12.92 \text{ ft}) - 9.362 = 8.02$ ft k/ft

SPAN 3: $V = \left[\frac{0.208\,(16.16)}{2} - \left(\frac{8.349 - 2.605}{16.16}\right)\right] = 1.33$ k/ft ← V_{EXT}

$x = 1.33/(0.208) = 6.39$ FT

$M_{MAX} = 0.5\,(1.33)(6.39 \text{ ft}) - 2.605 = 1.64$ ft k/ft

FLOOR SYSTEM ANALYSIS | GRANBY TOWER | POST TENSION | 7

→ FLEXURAL STRENGTH

CAPACITY CHECK AT INT. SUPPORT

$A_s = 0.00075 A_{cf} = 0.00075(30')(12)(8'') = 2.16 \text{ in}^2$

→ TRY (7) #5 @ 6IN O.C

BAR LENGTH $= 2(26 - 30/12)/6 + 30/12 = 10'4''$

$A_s = \frac{7(0.31)}{30'} = 0.072 \text{ in}^2/\text{ft}$

→ CALC DESIGN STRESS IN TENDON

$f_{ps} = f_{pe} + 10000 + \frac{f'_c}{300\rho_p}$

$\rho_p = \frac{A_{ps}}{bd} = 0.153(22 \text{ TENDONS}) / (30)(12)(8 - 1.25\text{in}) = 0.00139$

$f_{pe} = (0.7(270) - 14) = 175 \text{ ksi}$

$f_{ps} = 175 + 10 + \frac{5}{300(0.00139)} = 197 \text{ ksi}$

$f_{ps} < 0.85 f_{pu} = 0.85(270) = 230$ OK

$f_{ps} < f_{se} + 30 = 175 + 30 = 205$ OK

$F_{su} = 197(0.153)(22)/30' = 22.1 \text{ k/ft}$ (TENDONS)

$F_u = 60 \text{ ksi}(0.072 \text{ in}^2/\text{ft}) = 4.32 \text{ k/ft}$ (REBAR)

$F_{TOT} = 26.42 \text{ k/FT}$

→ DEPTH OF COMPRESSION BLOCK

$a = \frac{F}{0.85 b f'_c} = 26.42/((0.85)(12)(5\text{ksi})) = 0.52 \text{ in}$

$\varepsilon_t = (6.75 - 0.48)(0.003)/(0.52/0.85) = 0.0307$

$d - a/2 = (6.75 - 0.52/2)/12 = 0.54 \text{ ft}$

* ASSUME REBAR + TENDONS IN SAME PLANE

FLOOR SYSTEM ANALYSIS | GRANBY TOWER | POST - TENSION | 8

→ MOMENT CAPACITY @ COLUMN ℄

$\phi M_n = 0.9(0.54)(26.42) = 12.84$ ft k/ft > 9.32 ft k/ft ok ✓

PERMISSIBLE CHANGE IN NEGATIVE MOMENT

$1000\,\varepsilon_t = 1000(0.0307) = 30.7\% > 20\%$ MAX

AVAILABLE INCREASE : $0.2(9.32) = 1.86$ ft k/ft

ACTUAL INCREASE : $12.84 - 9.32 = 3.52 \nleq 1.86$ ft k/ft NO GOOD

→ MOMENT CAPACITY @ MIDSPAN OF SPAN 2

$8.02 - 1.86$ ft k/ft $= 6.16$ ft k/ft

$A_{ps} f_{ps} = 22.1$ k/ft

$a = 22.1/(0.85)(12)(5) = 0.433$ in

$\phi M_n = 0.9(22.1)(\frac{1}{12})(5.5 - 0.433/2) = 8.76$ ft k/ft > 6.16 ft k/ft ok ✓

→ MOMENT CAPACITY @ MIDSPAN OF SPAN 1

$\phi M_n = 0.9(22.1)(\frac{1}{12})(2.75 - 0.433/2) = 4.2$ ft k/ft > 1.62 ft k/ft ok ✓

→ MOMENT CAPACITY @ MIDSPAN OF SPAN 3

$\phi M_n = 0.9(22.1)(\frac{1}{12})(3.25 - 0.433/2) = 5.03$ ft k/ft > 1.64 ft k/ft ok ✓

→ EXTERIOR COLUMNS [SIMILAR TO INTERIOR SINCE TENDONS ANCHORED AT SLAB EDGE]

$A_{s\,min} = 0.00075(30')(12)(8\text{ in}) = 2.16\text{ in}^2$ → TRY (7) #5 @ 6" o.c., $A_s = 2.17\text{ in}^2$

$A_s = 7(0.31)/(30') = 0.072\text{ in}^2/\text{ft}$

$\rho_p = A_{ps}/bd = 22(0.153)/(30)(12)(6.75) = 0.00139$

$f_{ps} = 175 + 10 + \frac{5}{300(0.00139)} = 197$ ksi [CHECK SAME AS INT. COLUMN ok ✓]

$A_{ps} f_{ps} = 22(0.153)(197)/30 = 22.1$ k/ft

$a = (22.1 + 60(0.072))/(0.85)(12)(5) = 0.52$ in

FLOOR SYSTEM ANALYSIS | GRANBY TOWER | POST TENSION | 9

TENDONS: $d - a/2 = 6.75 - 0.52/2 = 6.49\ in/12 = 0.54\ ft$

REBAR: SAME PLANE AS TENDONS SINCE TENDONS ANCHORED AT ℄ OF SLAB DEPTH AT EDGE OF CANTILEVER SLAB

$\phi M_n = 0.9(22.1 + 4.32)(0.54) = 12.84\ ft\,k/ft > 3.8\ ft\,k/ft$ ok ✓

→ SHEAR CAPACITY @ EXTERIOR COLUMN

$V_{u\,H} = 1.33\ k/ft\,(30\ ft) = 40\ k$

$M_{TRANS} = 2.605\ ft\,k/ft\,(30\,ft) = 78.15\ ft\,k$

COMBINED SHEAR STRESS @ INSIDE FACE:

$d = 0.8(8\,in) = 6.4\ in$

$c_1 = 30\,in$ $b_1 = 30\,in + 3.2 = 33.2\ in$

$c_2 = 30\,in$ $b_2 = 30\,in + 6.4 = 36.4\ in$

$A_c = ((2)(33.2) + 36.4))(6.4) = 657.9\ in^2$

$J_c/c = \left[2(33.2)(6.4)(33.2 + 2(36.4)) + (6.4)^3(2(33.2) + 36.4)/33.2\right]/(6.4\,in) = 7165\ in^3$

$\gamma_v = 1 - \left(\frac{1}{1 + 2/3\sqrt{33.2/36.4}}\right) = 0.39$

$v_u = 40000/657.9\,in^2 + 0.39(12.84)(1000)(12)/7165 = 69.18\ psi$

$V_c = 4\sqrt{5000} = 283\ psi \rightarrow \phi V_n = 0.75(283) = 212.25\ psi$

$\phi V_n = 212.25\ psi > V_u = 69.18\ psi$ ok ✓

→ SHEAR CAPACITY @ INTERIOR COLUMN

$V_u = (1.33 + 2.69)(30') = 120.6\ k$

$M_{TRANS} = (30')(9.362 - 8.349\ ft\,k/ft) = 30.4\ ft\,k$

$V_u = 120600/657.9 + 0.39(30.4)(1000)(12)/7165 = 203.2\ psi$

$\phi V_n = 212.25\ psi > V_u = 203.2\ psi$ ok ✓

→ $\boxed{\text{SHEAR AND FLEXURE CAPACITY ADEQUATE}}$

USE 8" FLAT PLATE SLAB w/ 22 TENDONS UNIFORMLY DISTRIBUTED IN NORTH-SOUTH DIRECTION AND BANDED OVER THE COLUMN LINES IN THE EAST-WEST DIRECTION. USE (7) #5 @ 6" o.c. IN BOTH DIRECTIONS AROUND COLUMN IN SAME PLANE AS TENDONS.
→ ALL CRITERIA CHECKS WITH DESIGN AND COLUMN ASSUMPTION MORE CONSERVATIVE.

appendix b

two-way reinforced concrete flat plate slab

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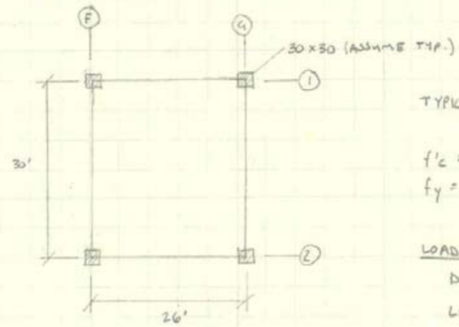
ALTERNATE FLOOR SYSTEMS | GRANBY TOWER | 2-WAY FLAT PLATE, REINFORCED

2 WAY FLAT-PLATE REINFORCED CONCRETE SLAB



TYPICAL (LONGEST SPAN) BAY, LEVEL 8

$f'_c = 5000$ psi ASSUME #7 BARS
$f_y = 60000$ psi

LOADING
DEAD = 20 PSF
LIVE = 40 PSF

SLAB THICKNESS: BASED ON SHEAR → NO DROP PANEL

$t = l_n/30 \rightarrow l_n = 30' - (30/12) = 27.5'$ [DEFLECTION LIMIT: $L/360$]

$t_{min} = (27.5')(12 in/ft)/30 = 11$ in

TOTAL FACTORED LOAD

$TL = 1.2\,(150\,PCF(11/12) + 20\,PSF) + 1.6\,(40\,PSF) = 253$ PSF

TOTAL STATIC MOMENT

$M_{o2} = \frac{1}{8}(0.253\,KSF)(30')(26' - 30/12)^2 = 524$ ft k

$M_{oF} = \frac{1}{8}(0.253\,KSF)(26')(30' - 30/12)^2 = 622$ ft k

INT/EXT SPAN MOMENT DISTRIBUTION

$M_{o2}+ = 0.35\,(524\,ft\,k) = 183$ ft k
$M_{o2}- = 0.65\,(524\,ft\,k) = 341$ ft k
INT SPAN

$M_{oF}+ = 0.52\,(622\,ft\,k) = 323$ ft k
$M_{oF}-_{INT} = 0.7\,(622\,ft\,k) = 435$ ft k
$M_{oF}-_{EXT} = 0.26\,(622\,ft\,k) = 162$ ft k
SLAB w/o BEAMS/EDGE BEAMS

MOMENT DISTRIBUTION

0.75 M- TO C.S ; 0.25 M- TO M.S

0.6 M+ TO C.S ; 0.4 M+ TO M.S

ALTERNATE FLOOR SYSTEMS | GRANBY TOWER | 2 WAY FLAT PLATE, REINFORCED | 2

LINE 2

M_{TOT}	-341	183	-341
M_{CS}	-256	110	-256
M_{MS}	-85	73	-85

LINE F

M_{TOT}	-435	323	-162
M_{CS}	-326	194	122
M_{MS}	-109	129	40

COLUMN STRIP = 13 ft , MIDDLE STRIP 2 = 17 ft , MIDDLE STRIP F = 13 ft

	2		F		
COLUMN STRIP	M^-	M^+	M^-_{INT}	M^+	M^-_{EXT}
TOTAL STATIC MOMENT, ft k	341	183	435	323	162
COLUMN STRIP MOMENT, ft k	256	110	326	194	122
STRIP WIDTH, b, IN	156	156	156	156	156
EFFECTIVE DEPTH, d, IN	8.94	8.94	9.81	9.81	9.81
M_N, ft k (M_u/ϕ)	284	122	362	216	134
R, psi (M_N/bd^2)	273.	117	289	173	107
ρ	0.0047	0.0015	0.005	0.003	0.0023
A_s (ρbd)	6.55	2.09	7.65	4.59	3.52
$A_{s\,min}$	3.43	3.43	3.43	3.43	3.43
N	11	4	13	8	6
N_{min}	6	6	6	6	6

	2		F		
MIDDLE STRIP	M^-	M^+	M^-_{INT}	M^+	M^-_{EXT}
TOTAL STATIC MOMENT, ft k	341	183	435	323	162
MIDDLE STRIP MOMENT, ft k	85	73	109	129	40
STRIP WIDTH, b, IN	204	204	156	156	156
EFFECTIVE DEPTH, d, IN	7.19	7.19	8.07	8.07	8.07
M_N, ft k (M_u/ϕ)	94	81	121	143	44
R, psi (M_n/bd^2)	107	92	143	152	52
ρ	0.0023	0.0021	0.0025	0.0026	0.0009
A_s (ρbd)	3.37	3.08	3.15	3.27	1.13
$A_{s\,min}$ ✓	4.5	4.5	3.43	3.43	3.43
N	6	6	6	6	2
N_{min}	8	8	6	6	6

BEAM ACTION SHEAR

$$V_u = wA = 0.253\ ksf\left(30/2 - 30/24 - 9.81/12\right)(1') = 3.27\ k$$

$$\phi V_c = 0.75(2)\sqrt{5000}\,(9.81)(12)/1000 = 12.5\ k$$

$\phi V_c > V_u$ ok ✓

ALTERNATE FLOOR SYSTEMS | GRANBY TOWER | 2-WAY FLAT PLATE, REINFORCED | 3

PUNCHING SHEAR

$b_o = 4(30) = 120$ in
$\alpha_c = 40$
$\beta = 1$

$V_u = wA = 0.253\left((30)(26) - (30)(30)/144\right) = 196$ k

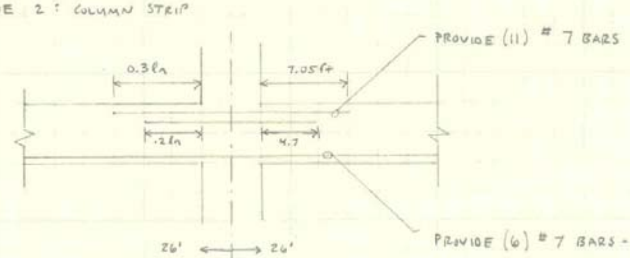
$$V_c = \min\begin{cases} 4\sqrt{f'_c}\, b_o d = 4(\sqrt{5000})(120)(9.81) = 332 \text{ k} \\ 6\sqrt{f'_c}\, b_o d \\ \left(\frac{\alpha_c}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o d = \left(\frac{40}{120/9.81} + 2\right)\sqrt{5000}\,(120)(9.81) = 439 \text{ k} \end{cases}$$

$\phi V_c = 0.75(332 \text{ k}) = 249$ k

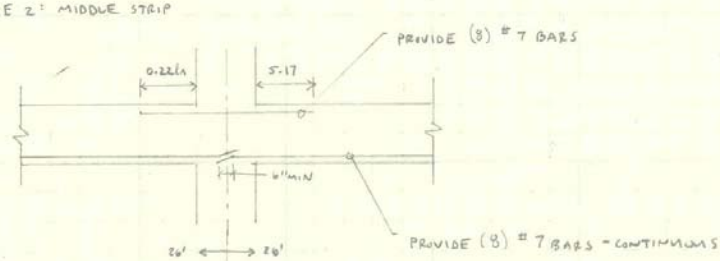
$\phi V_c > V_u$ ok

BAR CUT OFFS

LINE 2: COLUMN STRIP

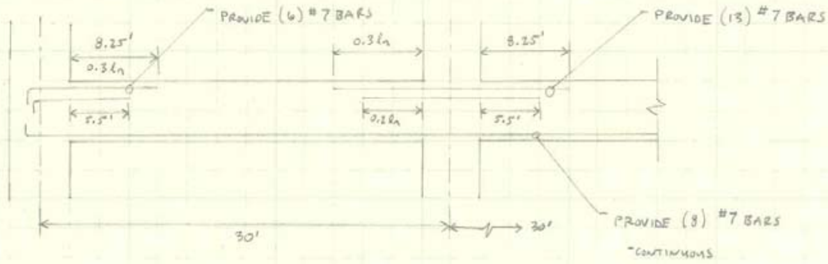


LINE 2: MIDDLE STRIP

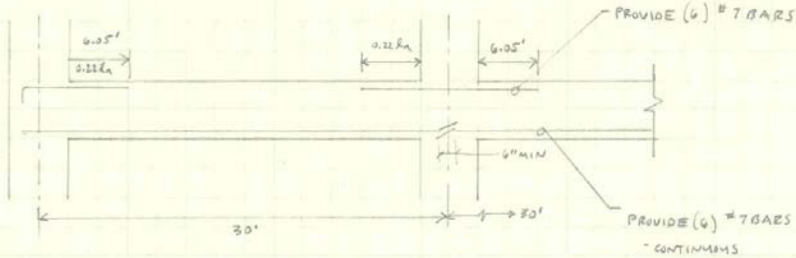


ALTERNATE FLOOR SYSTEMS | GRANBY TOWER | 2 WAY FLAT PLATE, REINFORCED | 4

LINE F: COLUMN STRIP



LINE F: MIDDLE STRIP



ANALYSIS: TWO-WAY FLAT PLATE REINFORCED CONCRETE SLAB ADEQUATE WHEN DESIGNED AS AN 11" NORMAL WEIGHT CONCRETE SLAB WITH NO DROP PANELS OR COLUMN CAPITALS. ALL REINFORCING SHOULD BE #7 REINFORCING BAR WITH CUT OFFS AS SPECIFIED PER COLUMN STRIP OR MIDDLE STRIP.

appendix c

one-way reinforced concrete slab with beams and girders

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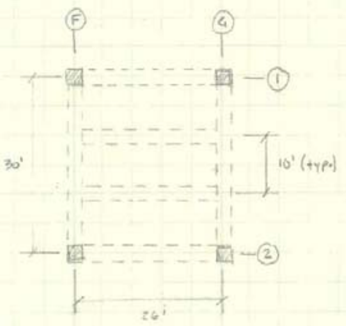
ALTERNATE FLOOR SYSTEMS | GRANBY TOWER | ONE WAY CONCRETE SLAB | 1

ONE-WAY REINFORCED CONCRETE SLAB w/ BEAMS AND GIRDERS



ASSUME INTERIOR SPANS

$f'_c = 5000$ psi
$f_y = 60000$ psi

LOADING
DEAD (PARTITION) = 20 PSF
LIVE = 40 PSF

MEMBER DEPTH BASED ON DEFLECTION [ACI 318-05, TABLE 9.5(a)]

SLAB = $\ell/28 = (10)(12)/28 = 4.3$ IN $\rightarrow$ 4.5 IN

BEAMS = $\ell/21 = (26)(12)/21 = 14.8 \rightarrow$ 15 IN

GIRDERS = $\ell/21 = (30)(12)/21 = 17.1 \rightarrow$ 18 IN

ACI MOMENT COEFFICIENTS [ACI 318-05, § 8.3.3]

$M^+ = \left(\frac{1}{16}\right) w_u \ell_n^2$ $\qquad M^- = \left(\frac{1}{11}\right) w_u \ell_n^2$

$w_u = 1.2\left(20\text{ PSF} + \frac{4.5}{12}(150\text{ PCF})\right) + 1.6(40\text{ PSF}) = 155.5$ PSF

SLAB DESIGN (ASSUME INTERIOR BAY)

$M^+ = \frac{1}{16}(0.155)(10')^2 = 0.969$ k-ft

$M^- = \frac{1}{11}(0.155)(10')^2 = 1.41$ k-ft

$\rho_{max} = (0.85)^2\left(\frac{5}{60}\right)\frac{0.003}{0.003+0.004} = 0.026$

$d^2 = M_u / \phi\rho f_y b(1 - 0.59\rho f_y / f'_c)$
$= 1.41(12) / \left((0.9(0.026)(60)(12)(1 - 0.59(0.026)(60/5)\right)$
$d = 1.11$ IN (USE CODE MINIMUM)

$\rightarrow d = 4.5 - 1'' = 3.5$ IN

ASSUME $a = 1$

$A_s = M_u / \phi f_y (d - a/2) = 1.41(12) / 0.9(60)(3.5 - 1/2) = 0.104$ in^2

ALTERNATE FLOOR SYSTEMS | GRANBY TOWER | ONE WAY CONCRETE SLAB | 2

CHECK a

$a = A_s f_y / 0.85 f'_c b = (0.104)(60) / 0.85(5)(12) = 0.123$ in

$A_s = 1.41(12) / 0.9(60)(3.5 - \frac{0.123}{2}) = 0.091 \text{ in}^2 < 0.0018bt = \underline{0.097 \text{ in}^2}$

TEMP + SHRINK CONTROLS

$a = (0.091)(60)/0.85(5)(12) = 0.107$

$A_{s\,\text{MIDSPAN}} = 0.969(12) / 0.9(60)(3.5 - \frac{0.107}{2}) = 0.062 < \underline{0.097 \text{ in}^2}$

$\boxed{\rightarrow \text{USE \#3 BARS THROUGHOUT } (A_s = 0.11 \text{ in}^2) \text{ @ } 12" \text{ o.c}}$

SHEAR CHECK

$V_u = 1.15(\frac{1}{2})(156 \text{ PSF})(10') - 156(\frac{3.5}{12}) = 741$ lb

$\phi V_c = \phi 2\sqrt{f'_c}\, b d = 0.75(2)\sqrt{5000}(12)(3.5) = 4455$ lb

$\phi V_c > V_u$ ok

$\boxed{\rightarrow \text{USE 4.5" NWC SLAB WITH \#3 REINFORCING @ 12" o.c THROUGHOUT}}$

BEAM DESIGN (INTERIOR BEAMS)

ASSUME $h = 15$ in, $d = 15 - 1.5 - .5 - \frac{.75}{2} = 12.625$, $b_w = 12$ in

$b_{eff} =$ min of:
- $\frac{1}{4}$ SPAN $= \frac{1}{4}(26)(12) = 78$
- $b_w + 16h_f = 12 + 16(4.5) = 84$
- $b_w + \frac{1}{2}$ CLR SPAN $= 12 + \frac{1}{2}(120") = 72$ in

CHECK T-BEAM BEHAVIOR

$w_u = 1.2\left[\left(20 + \frac{4.5}{12}(150)\right)(10') + \frac{10.5(12)}{144}(150)\right] + 1.6\left[(40 \text{ PSF})(10')\right] = 1.7$ KLF

$M_u = \frac{1}{8}(1.7\text{KLF})(26)^2(12) = 1736.5$ in-k

$M_{nTBM} = \phi 0.85 f'_c b h_f (d - h_f/2) = 0.9(0.85)(5\text{ksi})(72\text{in})(4.5\text{in})(12.6 - \frac{4.5}{2})$

$= 12858$ in-k

$M_u \not> M_{nTBM} \rightarrow$ NO T-BEAM BEHAVIOR

ALTERNATE FLOOR SYSTEMS | GRANBY TOWER | ONE WAY CONCRETE SLAB | 3

$$M^{+}_{midspan} = \tfrac{1}{16} w_u l_n^2 = \tfrac{1}{16}(1.7\,\text{klf})(26')^2 = 71.8\ \text{kft}$$

$$M^{-}_{support} = \tfrac{1}{11} w_u l_n^2 = \tfrac{1}{11}(1.7\,\text{klf})(26')^2 = 104.5\ \text{kft}$$

NEGATIVE MOMENT

$$M_u = 104.5\ \text{kft} = 1254\ \text{k in}$$

$$a = A_s f_y / 0.85 f'_c b$$

$$M_u = \phi A_s f_y \left(d - \tfrac{a}{2}\right) \rightarrow M_u = \phi A_s f_y \left[d - \left(\frac{A_s f_y}{1.7 f'_c b}\right)\right]$$

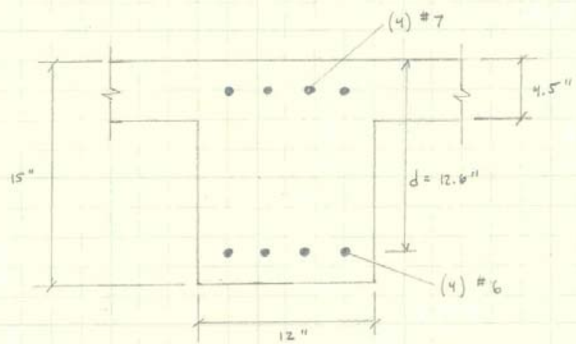
$$0 = 0.9 A_s (60)\left[13.5 - \left(\frac{A_s(60)}{1.7(5)(12)}\right)\right] - 1254\ \text{k in}$$

$$A_s = 2.03\ \text{in}^2 \rightarrow \text{USE } (4)\ \#7 \quad (A_s = 2.4\ \text{in}^2)$$

POSITIVE MOMENT

$$M_u = 71.8\ \text{kft} = 861.6\ \text{k in}$$

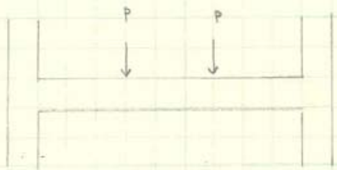
$$A_s = 1.35\ \text{in}^2 \rightarrow \text{USE } (4)\ \#6 \quad (A_s = 1.76\ \text{in}^2)$$



→ USE 15" x 12" CONCRETE BEAM w/ (4) #6 REINFORCEMENT AT MIDSPAN AND THROUGHOUT, AND (4) #7 REINFORCEMENT AT SUPPORTS

ALTERNATE FLOOR SYSTEMS	GRANBY TOWER	ONE WAY CONCRETE SLAB	4

GIRDER DESIGN



$P = 1.7\ \text{klf}\ (26') = 44.2\ \text{k}$

$M^+ = 0.111\ PL$

$M^- = 0.222\ PL$

#8

$h = 18\ \text{in}$ $\quad d = 18 - 1.5 - 0.5 - \frac{1}{2} = 15.5$

$b_w = 14\ \text{in}$

NEGATIVE MOMENT

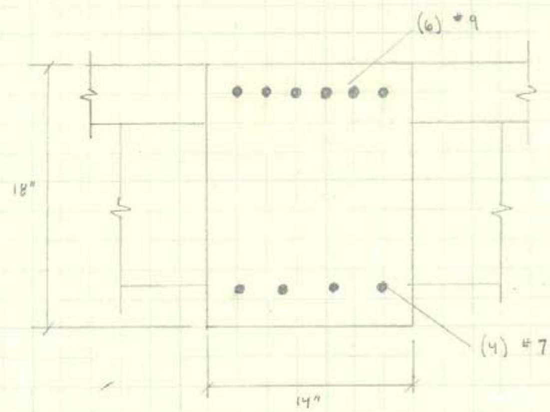
$M^- = 0.222\,(44.2)(30') = 294.4\ \text{ft k} = 3532.5\ \text{in k}$

$A_s = 5.1\ \text{in}^2 \rightarrow$ USE (6) #9 $(A_s = 6\ \text{in}^2)$

POSITIVE MOMENT

$M^+ = 0.111\,(44.2)(30') = 147.2\ \text{ft k} = 1766.2\ \text{in k}$

$A_s = 2.3\ \text{in}^2 \rightarrow$ USE (4) #7 $(A_s = 2.4\ \text{in}^2)$



→ USE 18" x 14" CONCRETE GIRDER w/(4) #7 REINFORCEMENT AT MIDSPAN AND THROUGHOUT, AND (6) #9 REINFORCEMENT AT SUPPORTS.

CHECK DESIGN WITH CRSI HANDBOOK

NOTE: CRSI ASSUMES $f'_c = 3000$ psi FOR SLABS + $f'_c = 4000$ psi FOR BEAMS / GIRDERS

SLAB: 4 1/2" THICK AND 10' SPAN, $w = 56$ PSF

FACTORED USABLE SUPERIMPOSED LOAD = 194 psf $> w_u = 156$ psf

REINFORCING: TOP = #4 @ 12 IN O.C
BOTTOM = #3 @ 12 IN O.C
TEMP + SHRINK = #3 @ 18 IN O.C

BEAM: $h = 15$, $w_u\,(1.2 + 1.6) = 1.7$ KLF $\rightarrow$ $w_u\,(1.4 + 1.7) = 1.9$ KLF

$\rightarrow h = 14$, $b = 12$, $w_u = 2.1$ KLF

MOM CAPACITY = $\phi M_n = 126$ ft k $> M_u = 105$ ft k

REINFORCING: TOP = (2) #11
BOTTOM = (2) #9
STIRRUPS = (30) #4, 29 @ 5 IN, 1 @ 2 IN EACH END

GIRDER: $h = 18$, ASSUME DISTRIBUTED LOAD $\rightarrow w_u = [1.4(20 + 56 + 5) + 1.7(40)](26) = 4.7$ KLF

ASSUME 20' OUT OF THE 30' EXPERIENCE LOAD DUE TO BEAMS ON COLUMN LINE

$\rightarrow h = 20$, $b = 14$, $\left(w_u = 4.7\text{ KLF}\left(\frac{2}{3}\right) = 3.1\text{ KLF}\right)$ $w_u = 3.7$ KLF

MOMENT CAPACITY: $\phi M_n = 302$ k ft $> M_u = 294$ k ft

REINFORCING: TOP = (3) #11
BOTTOM = (2) #11
STIRRUPS = (26) #5 @ 7 IN O.C

DEFLECTION

BEAM: $C = 1851 \times 10^{-9}$ IN

$$\delta = C\left(\frac{w}{1.6}\right)l_n^4 = (1851 \times 10^{-9})\left(\frac{1.7\text{ KLF}}{1.6}\right)(26' - 14/12)^4 = 0.75\text{ IN} < \frac{L}{240} = 1.3\text{ IN} \quad \underline{\text{OK}}$$

GIRDER: $C = 500 \times 10^{-9}$ in

$$\delta = C\left(\frac{w}{1.6}\right)l_n^4 = (500 \times 10^{-9})\left(\frac{3.1\text{ KLF}}{1.6}\right)\left(30 - \frac{36}{12}\right)^4 = 0.51\text{ IN} < \frac{L}{240} = 1.35\text{ IN} \quad \underline{\text{OK}}$$

appendix d

non-composite steel framing

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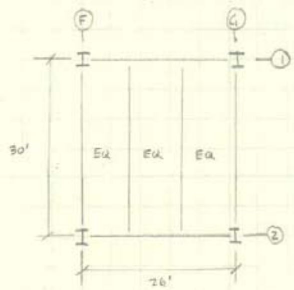
ALTERNATE FLOOR SYSTEMS | GRANBY TOWER | COMPOSITE STEEL

COMPOSITE STEEL



TYPICAL (LARGEST SPAN) BAY, LEVEL 8

$f'_c = 3000$ psi

$f_y = 50$ ksi

LOADING

DEAD : 20 PSF

LIVE : 40 PSF → REDUCABLE TO 16PSF

COLUMN DESIGN (COLUMN G2 → ASSUME TYPICAL BAY LAYOUT)

ACCUMULATED COLUMN LOAD (ASSUME 4", NWC SLAB)

$P = 1.2\left((20\text{PSF} + 150\text{PCF}\left(\frac{4}{12}\right)\right)(26)(30)(24\text{ FLOORS}) + 1.6\,(16\text{PSF})(26)(30)(24\text{ FLOORS})$

$= 2052$ K

BEAM UNBALANCED LOADING

$L = L_0\left(0.25 + 15\sqrt{2(26)(30)}\right) = 0.63$

$w_{uLL} = 0.63(40\text{PSF})(26)'(1.6) = 1.05$ KLF

$w_{uDL} = 1.2(150\text{PCF})\left(\frac{4}{12}\right)(26') = 1.56$ KLF

$FEM_1 = \frac{1}{12}(2.61\text{ KLF})(30)^2 = 196$ ft k

$FEM_2 = \frac{1}{12}(1.56\text{ KLF})(30)^2 = 117$ ft k

$\frac{1}{2}(FEM_1 - FEM_2) \rightarrow$ COLUMN

$P_{EFF} \approx 2052\text{k} + \left(\frac{24}{14}\right)(40\text{ ft k}) = 2120$ k

(ASSUME $k=1$, $L = 10.25$ → $KL = 10.25'$)

USE W14 × 176

$\phi P_N = 2150$ k (@ KL = 11)

PRELIMINARY COLUMN CHECK VERIFIES USE OF W14 X176 FOR TYPICAL BAY

ALTERNATE FLOOR SYSTEMS | GRANBY TOWER | COMPOSITE STEEL 2

SLAB DESIGN

→ TRY 4" SLAB

$w_u = 1.2\left(20\text{ PSF} + 150\left(\tfrac{4}{12}\right)\right) + 1.6(40) = 148\text{ PSF}$

→ TRY 19 GA. 1.5" LOK FLOOR DECK w/ 4" SLAB

ALLOWABLE LOAD: 160 PSF > 148 PSF ✓

MAX UNSHORED SPAN: 8.77 ft > 26/3 = 8.67 ft ✓

$\boxed{\text{USE 4" SLAB w/ 19 GA, 1.5" LOK FLOOR DECK AND 6x6 W1.4xW1.4 WWF}}$

BEAM DESIGN

$M_u = \frac{1}{8}(0.148\text{ KSF})(8.67)(30')^2 = 144\text{ ft k}$

ASSUME $a = 1.0$

$Y_2 = 4 - \frac{1.0}{2} = 3.5$

$b_{eff} = \min \begin{cases} \text{SIDE SPACING} = 8.67\text{ ft} = 104\text{ IN} \\ 1/4\text{ SPAN} = \frac{30}{4} = 7.5\text{ ft} = 90\text{ IN} \end{cases}$

TRY W12x19 → $\phi M_n = 150\text{ ft k} > M_u = 144\text{ ft k}$

$\Sigma Q_n = 104$

$a = \Sigma Q_n / 0.85 f'_c b = 104 / 0.85(3)(90)$

$= 0.45\text{ IN}$ ← CONSERVATIVE

SHAPE	PNA	ΣQ_n	ϕM_n	# STUDS	WT STL	WT STUD	TOTAL
W12x16	4	156	146	15	480	150	630
W12x14	TFL	208	147	20	420	200	620
W10x22	6	99.2	146	10	660	100	760
W10x19	4	162	149	16	570	160	760
W10x17	2	217	151	21	510	210	720
W10x19	6	104	150	10	570	100	620

CHECK DEFLECTION

WT OF CONC $= \left(\frac{3.25}{12}\right) 150\text{ PCF} = 40.6\text{ PSF}$ (TRIB WIDTH = 8.67')

$\delta_{DL} = 5(0.352)(30)^4(1728) / 384(29E^3) I \leq 1"$ $I \geq 221\text{ in}^4$

→ NONE OF THE ABOVE SHAPES MEET THE MOMENT OF INERTIA REQUIREMENT SO THE DECK WILL WORK COMPOSITELY WHILE THE BEAM WILL NOT.

ALTERNATE FLOOR SYSTEMS | GRANBY TOWER | COMPOSITE STEEL 3

$$\delta_{LL} = \frac{5(0.35)(30)^4(1728)}{384(29000)\,I} \le \frac{30(12)}{360} \qquad I \ge 220\ \text{in}^4$$

→ TRY W12 x 30 (I = 238)

$$\delta_{TL} = \frac{5(0.52)(30)^4(1728)}{384(29000)\,I} \le \frac{30(12)}{240} \qquad I \ge 218\ \text{in}^4$$

→ W12 x 30 OK FOR DEFLECTION

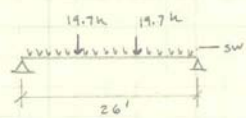
CHECK CAPACITY

$$M_u = \tfrac{1}{8}\left(148(8.67) + 30\ \text{PLF}\right)(30)^2 = 148\ \text{ft k} < \phi M_n = 162\ \text{ft k} \quad ✓$$

$$V_u = \tfrac{1}{2}\left(148(8.67) + 30\ \text{PLF}\right)(30) = 19.7\ \text{k} < \phi V_n = 96\ \text{k} \quad ✓$$

$\boxed{\text{USE W 12 x 30 NON-COMPOSITE BEAMS}}$

GIRDER DESIGN



$$\delta_T = \frac{(19.7\text{k})(26)^3(1728)}{28(29000)\,I} + \frac{5(0.08)(26)^4(1728)}{384(29000)\,I} \le \frac{26(12)}{240} \rightarrow I \ge 589\ \text{in}^4$$

→ TRY W12 x 72 (I = 597)

$$M_u = 19.7\text{k}(8.67') + \tfrac{1}{8}(0.072)(26)^2 = 177\ \text{ft k} < \phi M_n = 405\ \text{ft k}$$

$$V_u = 19.7\ \text{k} + 0.072(13') = 20.6\ \text{k} < \phi V_n = 158\ \text{k}$$

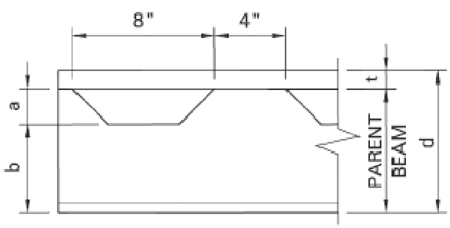
$\boxed{\text{USE W12 x 72 NON-COMPOSITE GIRDERS}}$

→ ANALYSIS: DUE TO SERVICEABILITY REQUIREMENTS, A COMPOSITE SYSTEM COULD NOT BE DEVELOPED WITH THE BEST CASE SLAB AND DECK. FOR THE NON-COMPOSITE SYSTEM, USE A 4" REINFORCED (6 x 6 W1.4 x 1.4 WWF) SLAB w/ 19 GA 1.5" LOK FLOOR DECK, SUPPORTED BY W12 x 30 BEAMS AND W12 x 72 GIRDERS. DEVELOPING COMPOSITE ACTION WOULD RESULT IN ADDED SHORING COSTS AND THEREFORE LESS FEASIBILITY.

appendix e

girder-slab

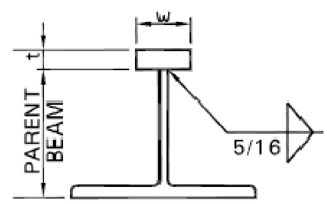
D-BEAM® DIMENSIONS TABLE



Designation	Web Included		Depth	Web	Parent Beam			Top Bar
	Weight	AVG AREA	d	Thickness t_w	Size	a	b	w x t
	lb./ft.	In.^{2}	In.	In.		In.	In.	In. x In.
DB 8 x 35	34.7	10.2	8	.340	W10 x 49	4	3	3 x 1
DB 8 x 37	36.7	10.8	8	.345	W12 x 53	2	5	3 x 1
DB 8 x 40	39.8	11.7	8	.340	W10 x 49	3	3.5	3 x 1.5
DB 8 x 42	41.8	12.3	8	.345	W12 x 53	1	5.5	3 x 1.5
DB 9 x 41	40.7	11.9	9.645	.375	W14 x 61	3.375	5.25	3 x 1
DB 9 x 46	45.8	13.4	9.645	.375	W14 x 61	2.375	5.75	3 x 1.5

D-BEAM® PROPERTIES TABLE

Designation	Steel Only Web Ignored						Transformed Section Web Ignored				
	Ix	C bot	C top	S bot	S top	Allowable Moment Fy=50 KSI f_b= 0.6Fy	Ix	C bot	C top	S bot	S top
	In.^{4}	In.	In.	In.^{3}	In.^{3}	kft	In.^{4}	In.	In.	In.^{3}	In.^{3}
DB 8 x 35	102	2.80	5.20	36.5	19.7	49	279	4.16	4.40	67.1	63.5
DB 8 x 37	103	2.76	5.24	37.3	19.7	49	282	4.16	4.42	67.7	63.8
DB 8 x 40	122	3.39	4.61	36.1	26.5	66	289	4.26	4.30	67.9	67.2
DB 8 x 42	123	3.35	4.65	36.9	26.5	66	291	4.26	4.32	68.4	67.5
DB 9 x 41	159	3.12	6.51	51.0	24.4	61	332	4.27	5.35	77.7	62.1
DB 9 x 46	195	3.84	5.79	50.8	33.7	84	356	4.43	5.20	80.6	68.6

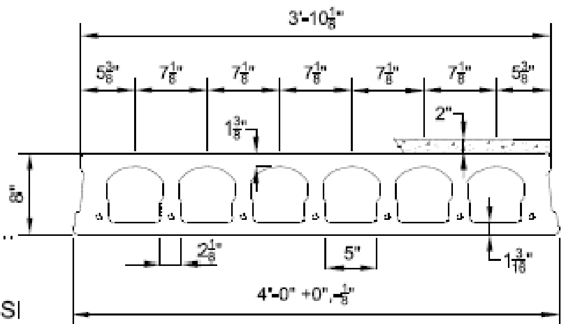


Prestressed Concrete
8"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

$A_c = 278$ in.^{2} Precast $S_{bc} = 583$ in.^{3}
$I_c = 2967$ in.^{4} Topping $S_{tc} = 854$ in.^{3}
$Y_{bc} = 5.09$ in. Precast $S_{tc} = 1019$ in.^{3}
$Y_{tc} = 2.91$ in. Wt.= 221 PLF
Wt.= 55.25 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI.
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   4-1/2"Ø, 270K = 92.4 k-ft
   7-1/2"Ø, 270K = 148.4 k-ft
7. Maximum bottom tensile stress is $7.5\sqrt{f'c} = 580$ PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS		IBC 2003 & ACI 318-02 (1.2 D + 1.6 L)																		
Strand Pattern		SPAN (FEET)																		
		17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	281	242	209	181	156	135	117	101	87	74	63	53	44						
7 - 1/2"ø	LOAD (PSF)	479	447	403	356	315	280	249	222	199	177	159	142	127	113	101	90	80	70	62

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

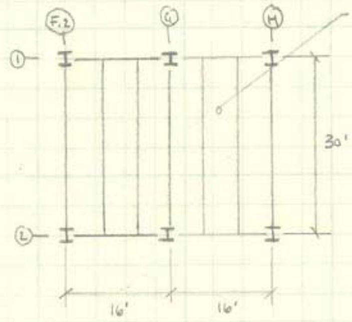
This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

05/14/07

8F2.0T

ALTERNATE FLOOR SYSTEMS | GRANBY TOWER | GIRDER SLAB | 1

HOLLOW CORE PLANKS ON TRANSFORMED W-SHAPE GIRDERS



8" PLANK w/ 2" NWC TOPPING (2HR RATING)

* CHANGE TYPICAL BAY TO 16' x 30'

LOADING

PARTITION = 20 PSF

LIVE = 40 PSF

HOLLOW CORE PLANK DESIGN

→ TRY 8" x 4' HOLLOW CORE PLANK w/ 2" NWC TOPPING + (7) 1/2" ϕ LO-RELAX STRAND

SPAN 30'

$w_T = 1.2(20\text{ PSF}) + 1.6(40\text{ PSF}) = 88\text{ PSF}$

→ SAFE SUPERIMPOSED SERVICE LOAD = 113 PSF > 88 PSF OK

CHECK PLANK CAPACITY

$f'_c = 6000\text{ psi}$ $\quad w_{D\,PLANK} = 55.25\text{ PSF}$

$\phi M_n = 178.4\text{ ft-k}$ $\quad w_{D\,TOPPING} = 25\text{ PSF}$

$w_u = 1.2(55.25 + 25 + 20) + 1.6(40\text{ PSF}) = 184.3\text{ PSF}$

$w_{u,\,PLANK} = (4')(184.3\text{ PSF}) = 0.737\text{ kLF}$

$M_u = \left(\frac{1}{8}\right)(0.737\text{ kLF})(30')^2 = 83\text{ ft-k} < \phi M_n = 178.4\text{ ft-k}$ OK

CHECK DEFLECTION

$E = w_c^{1.5} \cdot 33 f'_c{}^{1/2} = (150)^{1.5}\, 33(6000)^{0.5} = 4696\text{ ksi}$

$\Delta = \dfrac{5(0.737\text{ kLF})(30)^4(1728)}{384(4696)(2967\text{ in}^4)} = 0.96\text{ in} < L/360 = 30(12)/360 = 1\text{ in}$ OK

USE 8" x 4' HOLLOW CORE PLANK w/ (7) 1/2" ϕ LO RELAX STRAND AND 2" NWC TOPPING

ALTERNATE FLOOR SYSTEM | GRANBY TOWER | GIRDER SLAB 2

GIRDER-SLAB D-BEAM DESIGN

DESIGN INFORMATION

LOADING
- DEAD (PLANK) = 55.75 PSF
- PARTITION = 20 PSF
- CONC TOPPING = 25 PSF
- LIVE = 40 PSF

SPAN LENGTHS
- DB = 16 FT
- PLANK = 30 FT

COMPRESSIVE STRENGTH
- GRMT, $f'c$ = 4000 psi
- CONC PLANK, $f'c$ = 6000 psi

$\Delta_{LL} = L/360 = 0.53$ in

DB PROPERTIES (DB 9 X 46)

STEEL SECTION
- $I_S = 195 \text{ in}^4$
- $S_t = 33.7 \text{ in}^3$
- $S_b = 50.8 \text{ in}^3$
- $M_{SCAP} = 84$ ft k
- $t_w = 0.375$ in
- $b = 5.75$ in

TRANSFORMED SECTION
- $I_t = 356 \text{ in}^4$
- $S_t = 68.6 \text{ in}^3$
- $S_b = 90.6 \text{ in}^3$

INITIAL LOAD - PRECOMPOSITE

$M_{DL} = \frac{1}{8}(55.25 \text{ PSF})(16^2)(30) = 53 \text{ ft k} < M_{SCAP} = 84 \text{ ft k}$ ok ✓

$$\Delta_{DL} = \frac{5(30 \text{ ft})(0.055 \text{ ksf})(16')^4(1728)}{384(195 \text{ in}^4)(29000)} = 0.43 \text{ in}$$

$\Delta = L/[(16)(12)/0.43] = L/446$

TOTAL LOAD - COMPOSITE

$M_{SUP} = \frac{1}{8}(20+25+40 \text{ PSF})(16^2)(30) = 81.6 \text{ ft k}$

$M_{TOT} = M_{DL} + M_{SUP} = 134.6 \text{ ft k}$

$S_{REQ} \geq (134.6 \text{ k ft})(12 \text{ in/ft}) / (0.6)(50 \text{ k/in}^2) = 53.9 \text{ in}^3 < S_t = 68.6 \text{ in}^3$ ok ✓

$$\Delta_{SUP} = \frac{5(30 \text{ ft})(0.085 \text{ ksf})(16')^4(1728)}{384(356 \text{ in}^4)(29000)} = 0.36 \text{ in} < \Delta_{LL} = 0.53$$ ok ✓

$\Delta_{TOT} = 0.8 \text{ in} = L/[(16)(12)/0.8] = L/240$ ok

CHECK COMPRESSIVE STRESS ON CONCRETE

$N = E_{STL}/E_{CONC} = 29000 \text{ ksi}/[57000(4000)^{0.5}] = 8.04$

$S_{tc} = 8.04(68.6) = 552 \text{ in}^3$

ALTERNATE FLOOR SYSTEM | GRANBY TOWER | GIRDER SLAB | 3

$f_c = (81.6\ \text{k-ft})(12) / 552\ \text{in}^3 = 1.77\ \text{ksi}$

$F_c = 0.45(4\ \text{ksi}) = 1.8\ \text{ksi} > f_c = 1.77\ \text{ksi}$ OK ✓

CHECK BOTTOM FLANGE TENSILE STRESS

$f_b = (53\ \text{ft-k})(12)/(50.8) + (81.6\ \text{ft-k})(12)/(80.6\ \text{in}^3) = 24.7\ \text{ksi}$

$F_b = 0.9(50\ \text{ksi}) = 45\ \text{ksi} > f_b = 24.7\ \text{ksi}$ OK ✓

CHECK SHEAR

TOTAL LOAD = 140 PSF

$w = 0.14\ \text{ksf}\ (30') = 4.2\ \text{klf}$

$R = (4.2\ \text{klf})(16/2') = 33.6\ \text{k}$

$f_v = (33.6\ \text{k}) / (0.375)(5.75\ \text{in}) = 15.6\ \text{ksi}$

$F_v = 0.4(50\ \text{ksi}) = 20\ \text{ksi} > f_v = 15.6\ \text{ksi}$ OK ✓

ANALYSIS

USE (16') DB 9x46 (OPEN WEB DISSYMETRIC BEAM) TO SUPPORT 8" x 4' HOLLOWCORE PLANKS REINFORCED w/ (7) 1/2" ϕ LO-RELAXATION STRAND AND TOPPED WITH 2" NORMAL WEIGHT CONCRETE.

COLUMN SPACING MUST BE 16' MAXIMUM BETWEEN COLUMN LINES 1, 2, ETC. TO ALLOW FOR DBEAM GIRDERS TO MEET CAPACITY REQUIREMENTS

COLUMN CHECK

$L = L_o\left(0.25 + 15/\sqrt{4(16)(30)}\right) = 0.59$

$P = [1.2(20\ \text{psf} + 80\ \text{psf}) + 1.7(0.59)(40)](16)(30)(24\ \text{floors}) = 1845\ \text{k}$

$L = L_o\left(0.25 + 15/\sqrt{2(16)30}\right) = 0.73$

$FEM_1 = \frac{1}{12}(1.2(100) + 1.6(40))(26)(30)^2\left(\frac{1}{1000}\right) = 358.9\ \text{ft k}$

$FEM_2 = \frac{1}{12}(1.2(100))(26)(30)^2\left(\frac{1}{1000}\right) = 234\ \text{ft k}$

$P_{EFF} = 1845\ \text{k} + \left(\frac{24}{14}\right)(62.4\ \text{ft k}) = 1952\ \text{k}$

→ **USE W14x176** $\phi P_n = 2150 > P_{EFF} = 1952\ \text{k}$