

Analysis III

“Project Acceleration”

3.1 Executive Summary

The objective of the third analysis was to reduce the schedule and display a breadth of knowledge in the structural field by changing the structural walls from CMU to another system, while also looking at how more detailed planning using a SIPS schedule can speed up a project.

The original thought for the structural part was that pre-cast concrete walls would be the best method, however this had to be eliminated as an alternative after talking to a precast manufacturer who said that they always use 8” panels for buildings under five stories. This would not have given the opportunity to meet the structural breadth requirements and had to be eliminated from the possible options. After studying several different options, it was found that the best structural system for this building uses a 6” tilt-up concrete panel.

The SIPS scheduling part of this analysis involved seven finishing trades which had previously only been planned by floor. The logic for this analysis is that more detailed scheduling would improve the work flow and shorten the duration necessary for the entire building. After creating the schedule it was found that the increased flow would not necessarily speed up the schedule, but would allow the fewest people to be needed on site and would also make it a lot easier to increase the rate of work. This is because the smallest crews possible were used in the SIPS scheduling, but if more people were necessary, there would be less overlapping of work areas because everybody would be assigned to their own area.

3.2 Structural Wall Design

Overview:

The structure for the building is CMU walls with precast plank floors. The logic behind this system is that it allows a shorter floor to floor height while maintaining its cost competitiveness by using masonry walls. In order to display breadth in the structural area, it was decided to look at alternative options for the structural system which could reduce the time required to construct the building without creating a substantial price increase.

Floor Comparison:

The first step was to look at the choices for alternative structural systems. The first comparison was floor systems. The options for floors are precast concrete plank, post tensioned cast in place concrete, and steel beam and girder framing with deck and 4 inch slab. Because of the 45 foot maximum height requirement of the building, and the need to maintain four floors, structural steel was quickly eliminated from the list of possible solutions. The logic behind this was that a W14 steel beam under a 4 inch concrete slab would add 10 inches per floor to the total height of the building, raising it 3'4" and breaking the zoning law. This left precast plank and post tensioned floor systems to be compared.

Using R.S. Means 2000 to price the options, a cast in place, post-tensioned, elevated flat slab, with four uses of forms, with 125 pounds of superimposed load and a span of 30 feet will cost \$259.60 per cubic yard and will be completed at a rate of 50.99 c.y. per day. This results in a cost of \$1,326,844 for a 6" slab over 92,000 s.f. In comparison an 8" thick precast hollow core plank, delivered and erected will cost 6.50

per square foot plus 1.14 per s.f. for topping. This gives a total cost to the project of 702,880. The precast can be erected at a rate of 5,600 s.f. per day, so it will take 16.5 days to erect. Due to the \$623,964 difference in cost before any factors were taken into account and the extra 3.5 days required for a post tensioned slab, the decision was made to keep the precast plank floor.

Wall Comparison:

The next decision to be made is the wall type. The options for walls were the existing CMU walls, steel columns with block infill, precast concrete walls, and tilt-up walls. See Table 1 for the daily output and cost of each of these systems. As the table shows, the fastest method is to use tilt-up panels, reducing the duration from 252 days to 58 days. Comparing costs, before factors are applied the tilt-up panels will reduce costs from \$320,033 to \$291,526, a savings of \$28,507. At this point all of the costs would be subject to the same location and inflation factors, so adding them would not change the outcome that a tilt-up panel is the least expensive choice. Because this is an estimate from a manual and not from actual subcontractor pricing, the level of accuracy achieved by the factors is not necessary.

In order to be sure nothing was missed in the cost for tilt-up construction, the cost breakdown was checked and found to include preparation of pouring surface, erect and strip forms, concrete in place, steel trowel finish and curing, reinforcing, inserts and misc. items, and panel erection and alignment.

Table 1 Structural Wall Comparison					
Wall Type	Daily output	Unit Cost	Unit	Duration (days)	Total Cost (\$)
CMU	365	5.95	s.f.	252	320033
Steel Column	600	21.50	l.f.(column)	4	4300
CMU Wall	375	5.70	s.f.	245	524400
Steel Girders	912	42.50	l.f.	1.1	43818
Steel Total				250.9	572518
Precast Concrete	768	13.45	s.f.	120	723435
Tilt-Up	1600	5.42	s.f.	58	291526

Structural calculation:

Now that the structural system to be sized was defined as tilt-up walls with precast plank floor, the structural sizing of the system was carried out. The first thing to check was whether the proposed 6" panel could support the vertical building loads after it is placed. An assumption was made at this point that #3 rebar at 12" o.c. would meet the needs for bending while the panel is being lifted into place, while two #3 rebar would be needed 1-1/2 inches from the top of the cantilevered sections around the window cutouts. The loads on the building were kept the same as for the CMU wall design. These loads are taken directly from the structural plans and shown as Table 2. The maximum plank span is thirty feet so this length will be used in all calculations.

Roof Load (fourth floor walls):

$$\text{Total Dead} = 70 \text{ psf}$$

$$\text{Total Live Load} = 30 \text{ psf}$$

$$1.2D + 1.6L = 1.2(70) + 1.6(30) = 132 \text{ psf}$$

$$132 \times 30/2 = 1980 \text{ plf}$$

Component \ Area	1st Floor	Typical Floor	Lobby	Roof
Collateral		4	4	4
Partitions		10		
4" Concrete	50			
5" Concrete				
8" Hollow P/C		55	55	35
3/4" Gyp. Top		6	6	
Roofs & Insul.				11
Total Dead Load	50	75	65	70
Total Live Load	100	40	100	30
Total Load	150	115	165	100

Third Floor Walls:

$$\text{Fourth Floor panels} + \text{Transferred Load} = 960 + 1980 = 2940 \text{ plf}$$

$$\text{Dead Load} = 75 \times 1.2 = 90 \text{ psf} \times 30/2 = 1350 \text{ plf}$$

$$\text{Live Load} = 40 \times 1.6 = 64 \text{ psf} \times 30/2 = 960 \text{ plf}$$

$$\text{Total Load} = 2940 + 1350 + 960 = 5250 \text{ plf}$$

Second Floor Walls:

$$\text{Third Floor Walls} + \text{Transferred Load} = 960 + 5250 = 6210 \text{ plf}$$

$$\text{Dead Load} = 75 \times 1.2 \times 30/2 = 1350 \text{ plf}$$

$$\text{Live Load} = 40 \times 1.6 \times 30/2 = 960 \text{ plf}$$

$$\text{Total Load} = 6210 + 1350 + 960 = 8520 \text{ plf}$$

First Floor Walls:

$$\text{Second Floor Walls} + \text{Transferred Load} = 960 + 8520 = 9480$$

$$\text{Dead Load} = 75 \times 1.2 \times 30/2 = 1350 \text{ plf}$$

$$\text{Live Load} = 40 \times 1.6 \times 30/2 = 960 \text{ plf}$$

$$\text{Total Load} = 9480 + 1350 + 960 = 11790 \text{ plf}$$

Panel Axial Strength:

$$P_o = .85 f'_c A_c + A_s f_y = .85(4)(12 \times 6 \times .11) + .11(60) = 251 \text{ Kips}$$

$$251 > 11.8 \rightarrow \underline{\text{OK}}$$

The next step in designing the panel is to find the optimum lifting points. The worst position for a panel is in the horizontal position because after the panel begins to rotate the axial load of its own weight will reduce the area in tension, the same as post tensioning or a beam column. This means that if the panel can resist bending when it is horizontal, it can resist bending at any point during the rotation. Because the panel was being designed as a four point lift, the quarters of the panel were found by splitting the panel at its axis of symmetry and finding the centroid of the remaining half panel in the x and y directions, then the lifting hook was placed in the middle of the quarter. The typical panel is represented as Figure 1. The calculation of these points is shown below.

$$y = [(5'6'')(10'8'')(5'4'') + (1')(3'6'')(10'2'') + (2'8'')(3'6'')(1'4'')] /$$

$$[(5'6'')(10'8'') + (1')(3'6'') + (2'8'')(3'6'')]$$

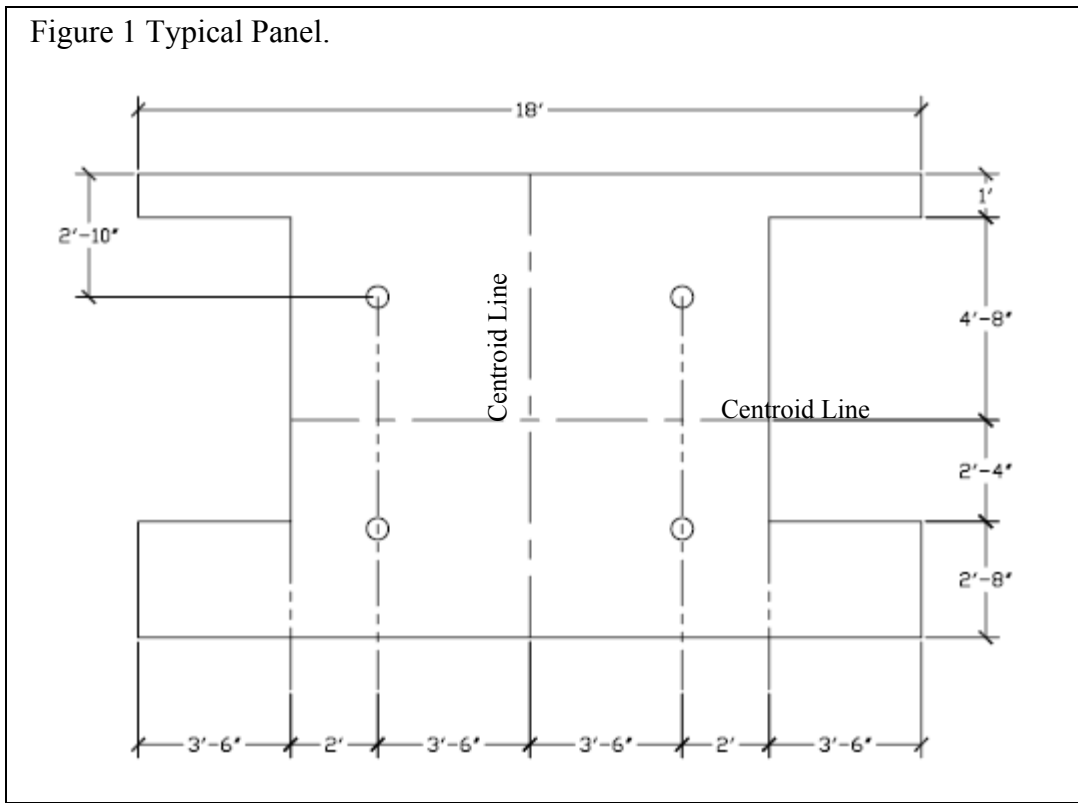
$$= 360.8 / 71.63 = 5.04', \text{ approximately } 5'$$

$$x = [(5'6'')(10'8'')(5'6''/2) + (1')(3'6'')(5'6'' + 3'6''/2) + (2'8'')(3'6'')(5'6'' + 3'6''/2)] /$$

$$[(5'6'')(10'8'') + (1')(3'6'') + (2'8'')(3'6'')]$$

$$= 254.43 / 71.63 = 3.55', \text{ approximately } 3'6''$$

Figure 1 Typical Panel.



After finding the lifting points, the weight on each lifting point was calculated by finding the weight of the entire panel and dividing it by four. To make sure the load is balanced evenly, the weight of each quarter, as defined by the approximate centroid lines was also found. This calculation is shown below.

$$W = 2 [(5'6'')(10'8'') + (1')(3'6'') + (2'8'')(3'6'')] (6'')(150 \text{ pcf}) = 10728 \text{ pounds}$$

$$10728 / 4 = 2682 \text{ pounds for each lifting point}$$

By Centroids:

$$\text{Lower Quarters} = [(5'6'')(5') + (2'8'')(3'6'')] (6'')(150) = 2763.4 \text{ pounds}$$

$$\text{Upper Quarters} = [(5'6')(5'8'') + (1')(3'6'')] (6'')(150) = 2601.4 \text{ pounds}$$

As you can see, the lifting points for the lower quarters appear to hold slightly more weight than the upper quarters. This is due to the rounding of the exact centroid locations. To be conservative, the higher 2,763.4 pounds will be used in checking the shear at the lifting points to ensure they do not tear out of the panel. This is similar to punching shear of a concrete slab around columns and was calculated as follows.

$$\Phi V = .75(f'_c)^{1/2}(4)(A_c)$$

A_c = perimeter x depth

Depth = 6"

Assuming shear failure at 45 degree angle, perimeter = $2(\pi)(r) = 2(3.14)(6) = 37.7$ inches

$$A_c = 6 \times 37.7 = 226.2 \text{ in}^2$$

$$\Phi V = .75(4000)^{1/2}(4)(226.2) = 42,918 \text{ pounds}$$

42,918 > 2763.4 → OK

The next load calculated was the moment due to the weight of the panel. This was calculated as a simply supported beam between the lifting points and a cantilevered beam from the lifting points out to the edge of the panel. The moment calculations are:

$$\text{Middle: } wl^2/8 = (150/2)(7)^2 / 8 = 460 \text{ ft. lb.}$$

$$\text{Ends: longer} = wl^2/2 = (150/2)(5'6'')^2 / 2 = 1134 \text{ ft. lb.}$$

$$\text{Shorter} = wl^2/2 = (150/2)(2)^2 / 2 = 150 \text{ ft. lb.}$$

Calculating the moment capacity as a 1 foot wide flat slab with a #3 60 ksi rebar spaced ever 12 inches, 1-1/2 inches from the bottom (exterior) of the panel gives the following results.

$$\text{Reinforcement} = .11 \text{ in}^2 \text{ per foot}$$

$$D = 6 - 1.5 - 3/16 = 4.31 \text{ in}$$

$$a = [(.11)(60)] / [(.85)(4)(12)] = .162$$

$$c = .162 / .85 = .190 < .375(d_t) = 1.62 \rightarrow \Phi = .9$$

$$\Phi M_n = .9[(.11)(60)(4.31 - .162/2)] = 25.11 \text{ in. kips.}$$

$$25.11 / 12 = 2.09 \text{ ft. kips.}$$

$$2090 \text{ ft. lbs.} > 1134 \text{ ft. lbs.} \rightarrow \text{OK}$$

The last thing checked was whether the cantilevered sections on the end of the panel would support their own weight when the panel is in place.

For the 1' x 3'6" section over the window:

$$\text{Moment} = wl^2 / 2 = (150/2)(3'6")^2 / 2 = 459 \text{ ft. lb.}$$

Capacity =

$$\text{Reinforcement} = 2 \times .11 \text{ in}^2 = .22 \text{ in}^2$$

$$D = 12 - 1.5 - 3/16 = 10.31 \text{ in}$$

$$a = [(.22)(60)] / [(.85)(4)(12)] = .324$$

$$c = .324 / .85 = .381 < .375(d_t) = 3.87 \rightarrow \Phi = .9$$

$$\Phi M_n = .9[(.22)(60)(10.31 - .324/2)] = 120.56 \text{ in. kips.}$$

$$120.56 / 12 = 10,049 \text{ ft. lbs.} > 459 \text{ ft. lbs.} \rightarrow \text{OK}$$

For the 2'8" x 3'6" section under the window:

$$\text{Moment} = wl^2 / 2 = (200)(3'6")^2 / 2 = 12259 \text{ ft. lb.}$$

Capacity =

$$\text{Reinforcement} = 2 \times .11 \text{ in}^2 = .22 \text{ in}^2$$

$$D = 32 - 1.5 - 3/16 = 30.3 \text{ in}$$

$$a = [(.22)(60)] / [(.85)(4)(12)] = .324$$

$$c = .324 / .85 = .381 < .375(d_t) = 11.36 \rightarrow \Phi = .9$$

$$\Phi M_n = .9[(.22)(60)(30.3 - .324/2)] = 397 \text{ in. kips.}$$

$$397 / 12 = 33,200 \text{ ft. lbs.} > 1225 \text{ ft. lbs.} \rightarrow \text{OK}$$

Recommendation:

From the calculations shown above, a 6" tilt-up panel, with a #3, 60 ksi reinforcing bar placed in a grid every foot, at 1-1/2 inches from the top of the panel, and two of the same rebars placed 1-1/2 inches from the top of the cantilevered ends, around the windows, will support the panel while it is being lifted into place as well as while it is supporting the building loads.

The cost and schedule estimates show that using tilt-up wall panels should be both faster, and less expensive than the current design of CMU walls. Because of this it is recommended that the tilt-up panels be used.

SIPS Scheduling

Overview:

Short Interval Production Scheduling (SIPS) is a fairly new tool developed by Hensel Phelps Construction Company and first documented by Alvin Burkhart as part of the 1989 Construction Congress. The goal of the schedule is to plan activities to a much higher level of detail than is allowed by the typical Critical Path Method schedule in order to allow better coordination among trades and a smoother flow of work. By creating a “Parade of Trades”¹ for the work flow a reduction in the need for built in buffers is achieved. This creates a shorter overall project duration; which also reduces costs.

In this project a SIPS schedule was created for the finishing trades. The trades had previously been simply scheduled by floor; however, it was believed that the typical apartment floor plan of the residence hall would make it possible to achieve a better flow of work by scheduling the trades to the detail of a SIPS schedule.

Trade Identification:

For the purposes of this research, finishing trades were defined as interior trades which must be completed after drywalling and taping is complete. On this project this definition included seven activities in the apartments. Once these activities were identified the interrelationships were studied and the most effective order was decided to be: wall painting, ceiling, electrical trim and lights, casework, plumbing fixtures, doors and hardware, then flooring.

¹ Tommelein, I.D., Riley, D., and Howell, G.A. (1998). "[Parade Game: Impact of Work Flow Variability on Succeeding Trade Performance.](#)" *Proc. Sixth Annual Conference of the International Group for Lean Construction*, IGLC-6, 13-15 August held in Guaruja, Brazil, 14 pp.

Trade Durations:

After determining the trades to be included in the finish scheduling, the amount of time necessary to complete work in a typical area was needed. For this, the productivity rates from R.S.

Hours/Unit	Unit	Activity
0.006	s.f.	Paint Walls
0.011	s.f.	Paint Ceiling
0.50	ea.	Elec. Trim & Light Fixt.
0.40	l.f.	Casework
1.6	ea.	Plumbing Fixt.
1.14	ea.	Doors / Hardware
0.016	s.f.	Flooring

Means were used. The rates found are shown in Table 3. After taking off the amount of work for each trade in the typical Unit A apartment arrangement the durations shown in

Table 4 were found.

crew size	Preferred Order	Durations	# Crews Needed	Resulting Duration
1	Paint Walls	2 days	2	1
1	Paint Ceiling	2.25 days	2	1.125
1	Elec. Trim & Light Fixt.	2 days	2	1
2	Casework	1 day	1	1
1	Plumbing Fixt.	2 days	2	1
1	Doors / Hardware	1 day	1	1
1	Flooring	2.5 days	2*	1.25

At this point it is realized that not all unit types will result in exactly the same amount of work. In

order to create the proper flow, it will be necessary for the subcontractors to accept that on some days they may have to work late in order to finish their unit, but on other days they will get to go home early or be sent to other jobs.

Creating the SIPS Schedule:

In order to balance the durations, the fastest trades were examined first. These were casework and doors/hardware. Because doors/hardware is a one person activity, slowing it down can only be accomplished by having the carpenter stop working. This would not be beneficial to the project. Next the slower activities were looked at and

found that by adding another person to wall painting, ceiling painting, plumbing fixtures, and flooring each trade could finish a typical unit in one day. The slowest activity, flooring, would need 1.25 days, but because this activity is the last one through the space, they will be able to catch up on Saturday or by working ten hour days. The final schedule is shown in APPENDIX C.

4D Model:

To display the flow of work through the building, a 4D model was created. 4D models are created by merging a 3D model with a schedule. Software for this task is currently available in the form of Common Point or NavisWorks. For this project, the decision was made to work with NavisWorks because it seemed to be more user friendly.

The first step was to create the 3D model in AutoCAD. This was accomplished by drafting the basic geometries in 2D and extruding them to create blocks. In order to provide the necessary link to the schedule, the model was then exported to Autodesk VIZ which allowed saving it as a 3DS file which can be read by NavisWorks. Next the Microsoft Project schedule was linked to NavisWorks. Then different types of activities were set up and given individual colors as shown in Table 5 so it would be clear who was in each space. This was followed by attaching the necessary objects from the model to each activity and running a simulation to test everything.

During simulating, several inconsistencies were found in the plan. They were fixed in the schedule and the model was updated to show the improved plan. The mistakes included missing

Table 5. Activity Colors	
Activity	Color
Paint Walls	Red
Paint Ceiling	Green
Elec. Trim & Light Fixt.	Yellow
Casework	Purple
Plumbing Fixt.	Blue
Doors / Hardware	Orange
Flooring	Brown
Finished Area	Green Tint

activities and steps performed out of sequence. While these mistakes were fairly easy to spot in the model they had been overlooked several times in the schedule. This is the truly valuable part of a 4D model because it is easy to overlook a mistake when it is buried in several pages of a schedule than when the objects are obviously out of sequence in a model. For screenshots of the finished model, see APPENDIX C.

Schedule Comparison:

In the original plan for the building the total time for the finishing trades to complete a floor was 34 working days. In the schedule developed using SIPS methods, the work required 33 working days. While this only saves one working day, the bare minimum of crews are used in a much more orderly working pattern. Should it become necessary to accelerate the finishing trades to offset delays incurred on the project, it seems likely that the SIPS schedule could be accelerated with much less loss in productivity because the work could still follow the pattern created, possibly by simply linking adjacent areas and doubling the manpower to double the rate of work. In the original schedule where crews are only told what floor to be on, adding manpower is more likely to create losses in productivity due to congestion.

Recommendation:

Using SIPS scheduling is a good way to organize a project so that trades can flow in a more methodical pattern. In this project it yields the fewest number of people required to complete the finished trades. This would reduce the cost of the work and make it easier to shorten the total duration of this part of the project if it were necessary to do so. It is recommended that increased planning be put into parts of a project where there are several crews working in the same area or within close proximity to each other.