Overview – Existing Conditions

Columns

All columns are 12”x24” with chamfered edges, where exposed. There are 32 columns which span from the foundation to the roof, over 115 feet, with number 4 ties spaced at 12 inches all the way up. Vertical reinforcing ranges from ten number 11 bars to six number 8 bars. In all cases, the vertical reinforcing is distributed along the 24” face of the column in two sheets, one on each side. In all cases, class B lap splices are required for vertical splicing. Concrete strength is normal weight 6000 psi from the foundation to the third floor, where it drops to 5000 psi until it reaches the roof. Typical floor to floor heights are close to 10’.

There is a double-height pool structure on the first floor that rests on grade. Because it intersects with two column lines, the two columns start at the second floor and proceed to the roof. They cannot continue down to the foundation, so their weight is picked up by a transfer beam that is 36” deep, 44” wide, and heavily reinforced with six number 8 bars on top, ten number 11 bars on the bottom with an additional row of six number 9 bars also on the bottom. The reinforcing is tied together with number 5 closed stirrups spaced at ten inches on center. This transfer beam also frames into to two similar girders, tied into columns, at either end.

The last two columns start at the roof and help hold up a mechanical screen wall. The roof of the screen wall consists of W14x22 curved steel members with 1-1/2” galvanized metal roof deck resting on top.

Floor Slabs

The floor slabs are usually 10-1/2” thick when not near columns. At each column there is a 2-1/2” drop panel to combine for a 13” slab thickness. A typical drop panel size is 5’-6”x6’-9” and accounts for 38 square feet. Steel reinforcing is laid out longitudinally and transversely on both the bottom and top. The slab reinforcing ranges from number 4 bars to number 6 bars spaced approximately 12 inches apart. Where not specified, number 5 bars spaced at 6” is the minimum required.

For slabs on level 3 and below, concrete strength is normal weight 6000 psi. Slabs resting on the fourth floor and up have a strength of 5000 psi. Minimum reinforcing protection for floor slabs is 3/4”.

The slabs on this project are considered to act as two way slabs, meaning that they carry load in both lateral directions. The three largest bays have dimensions of 29' x 26'-10". There are no beams spanning between columns in this case. In the largest bay, the drop panels cover roughly 6 feet of the span, or 20.7%.

Lateral System

The lateral components of this building are comprised of twelve shear walls of varying length. Five of the twelve are aligned with Plan North, while the other seven are aligned East-West. Each shear wall is one foot thick and is vertically reinforced with number 5 bars at 18" on center. They are each tied into the foundation by rebar that matches vertical reinforcing called out in the plans. All rebar is to have class B splices and extend one foot into the foundation with 90^{o} hooks. In most cases, two columns act as bookends for each shear wall. Where this occurs, the shear wall reinforcement of number 5 bars spaced at 18 inches is continued into the columns and hooked 90^{o}.

The longest shear walls are 21'-4" along grid lines B and C running North to South. Refer to figure 1 on the next page for a graphical shear wall layout. In the East-West direction, the longest shear wall is located along grid line 6, and is 21'-0" long. Nine of the twelve shear walls wrap around the two stair cases and lone elevator shaft that are spaced evenly throughout the building's long dimension.

The total length of the shear walls in the North-South direction is 99'-4", and 79'-0" in the East-West direction. Because the building is rectangular, forces acting on the wide side of the building have a much greater affect on the building's response than forces acting on the narrow side. Thus, more total shear wall length was provided to resist North-South loads.

With a total height of 130 feet, the shear walls travel the full height of the building and are in the same position, relative to each other, on every floor (although some individual floor layouts may vary).

To assist in the analysis of this structure, a RAM model was created following the building's floor plans. While the model has some limitations, and spot checks were made with some simplifying assumptions, the results were confirmed through hand calculations. However, the accuracy of the RAM output depends directly on the model generated, and there were some areas and conditions that were not feasible to model for this report.

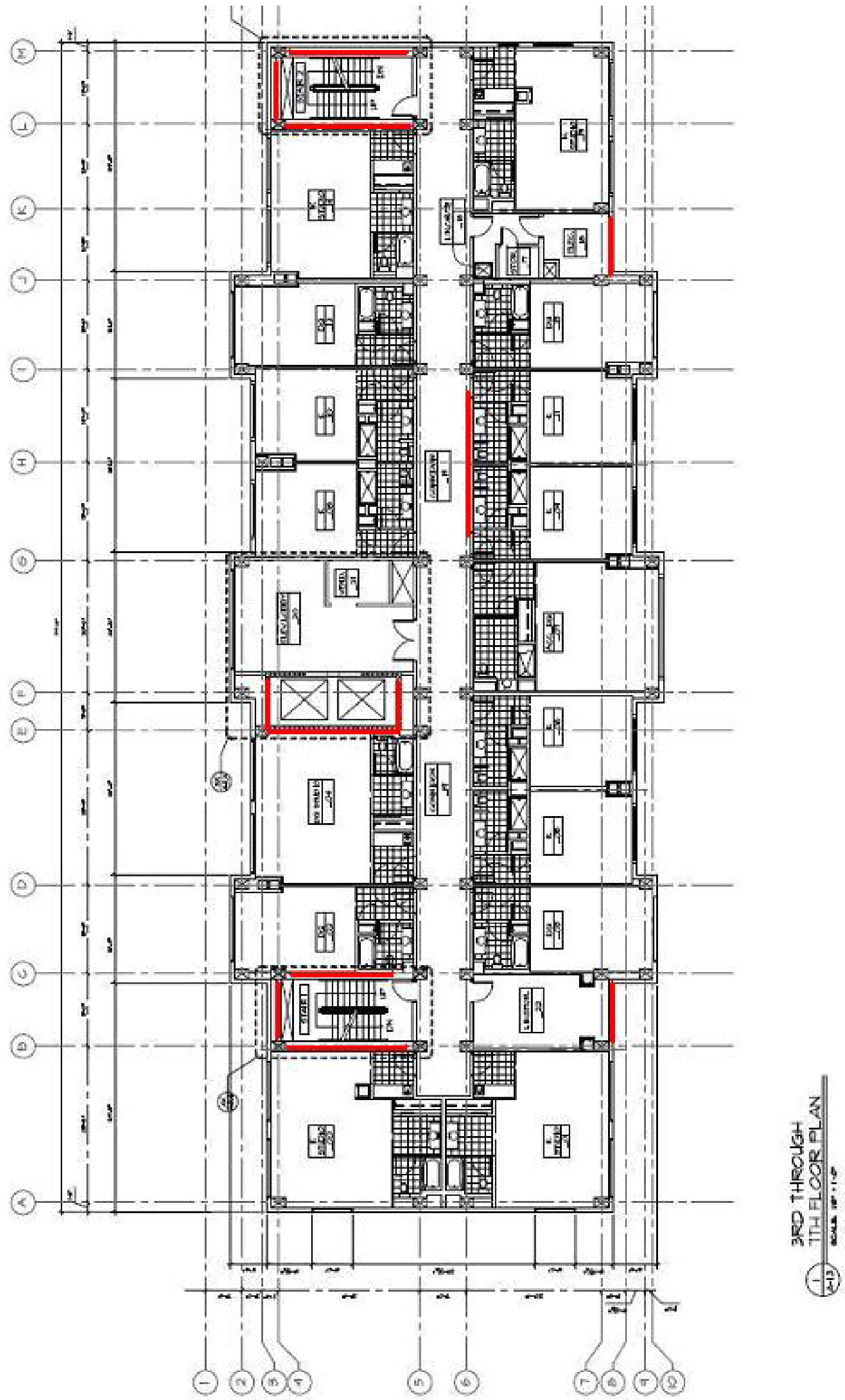

Fig. 1

Shown is the shear wall layout for a typical floor of the Hampton Inn & Suites in National Harbor, MD.

Code List

Building Code
Maryland Building Performance Standards (MBPS) – based on IBC 2003 and IRC

Structural Concrete Code
The American Concrete Institute (ACI) – sections 301, 318 and 315
Aggregate shall comply with ACI 304, and slump with 211.1
Reinforcing shall comply with ASTM A615, Grade 60

Masonry Code
ACI – section 530.1
Reinforcing shall comply with ASTM A615, Grade 60

Structural Steel Code
Load and Resistance Factor Design Specification (LRFD) conforming with the American Institute of Steel Construction (AISC) specification for structural steel for buildings, and AWS D1.1, latest edition
Connection bolts shall conform to ASTM A325

W shapes, columns	ASTM A992 or ASTM 572-50
S, M, and HP shapes	ASTM A36
column baseplates, web doubler plates	ASTM A992 or ASTM 572-50
channels, tees, bars, angles and plates	ASTM A36
HSS rectangular or square	ASTM A500 – GR. B (Fy=46ksi)
steel pipe	ASTM A500 – GR. B (Fy=42ksi)
anchor rods	ASTM A307, A449 where noted

Load Summary

	Corridor	Storage	Guest	Roof	Canopy
Slab	148	148	148	148	--
M/E/C/L	8	8	8	8	8
Roof	--	--	--	2	2
Insulation	--	--	--	8	8
Total Dead	156	156	156	166	16
Live	100	125	40	30	30
Partition	--	--	20	--	--
Total	256	281	216	196	48

New Proposal – Problem Statement

Concrete is the general material of choice for most hotel designers. It offers many unique advantages to other systems: it has a slim profile between floors, it is cost efficient, and it is easily cast on the job site. The problem with normally reinforced concrete flat plates is that, due to its own dead weight, it makes the structure weigh more than a more 'athletic' system would. At 10-1/2" thick, the flat plate weighs in at 131 pounds per square foot. This considerable dead weight dictates that the seismic base shear, according to ASCE 7-05, will be significantly greater than that of a lighter structure.

Proposed Solution

To combat the high seismic base shear discovered in technical report three, a lighter alternative to a normal weight concrete flat plate will be investigated. While researching flooring systems for technical report two, it was estimated that an 8" post tensioned slab would be adequate for the applied loads. This decrease in the slab profile results in an approximate savings of 25% from the original weight of the slab. Additionally, because the slab will be lighter, the drop panels located at each column will be redesigned where necessary, or completely eliminated if deemed acceptable. Smaller column sections may be possible due to the decrease in punching shear experienced by the concrete slab. Overall, the switch to a post tensioned flooring system will reduce the weight of the building considerably. A possibility also arises that the number of shear walls could be decreased proportionally to the reduction of the seismic base shear. This gives the project team even more flexibility in the design of the structure.

All structural calculations will be carried out according to current building codes, such as ASCE 7-05, IBC 2003, ACI, and all other codes that apply. A RAM model of the building was created for technical assignment three, and this model can be adapted to model a lighter concrete floor section. This will indicate if the lateral system can be modified due to a smaller seismic base shear. Slab design will be in accordance to ACI provisions for post tensioned floor applications.

Breadth Studies

An obvious breadth study relates directly to the proposed change of flooring systems, and that is a study of constructability. Material costs as well as scheduling issues must be addressed as the construction method is reanalyzed. A comparative cost analysis between the existing conditions and the proposed solution will be conducted, as well as an in depth scheduling investigation.

The second breadth topic that will be explored is the curtain wall system on the façade of the building. A general building envelope study will determine the effectiveness of the current system. An alternative system will be compared to the existing envelope, and cost and schedule factors will also be evaluated.

Analysis Breakdown

Floor System

1. -Analyze superimposed dead loads per construction documents
   -Through ASCE 7-05, determine proper live loads

2. -Estimate slab thickness and post tension tendon profile
   -Refine calculations of slab thickness and tendon profile

3. -Determine reinforcement according to ACI

Lateral System

1. -Verify wind and seismic loadings through ASCE 7-05

2. -Distribute lateral loads to shear walls

3. -Design lateral resisting elements

Breadth Studies

1. -Analyze construction issues such as cost and schedule

2. -Examination of curtain wall system and building envelope
   -Cost analysis of proposed system

Structural Depth Study

Post Tensioned Floor System Analysis

When considering post tensioned analysis, the basic floor plan of a given building is a good indicator of whether or not post tensioning is a viable alternative. If the building has uniform bays in both directions with little variation, then post tensioning proves to be a very economical and simple approach to saving weight and money on the project. Alternatively, if the floor plan is not very uniform, this floor system becomes more complicated. As the irregularities in the floor plan increase in number, a point is reached where the post tensioned concrete slab and the cast in place slab balance each other out in terms of labor and cost.

The Hampton Inn & Suites lends itself considerably well to post tensioned analysis. Because the floor plan is orthogonal and relatively simple, finding a realistic layout for the tendon strands was intuitive, save for a few problem areas which will be discussed later. After reviewing the architectural drawings, it was found that some columns in the East-West direction were placed such that the tendons running in that direction could not reach them. Figure 2 on the next page demonstrates this potential problem, as the clouded areas indicate where a column must be shifted in order to reach the post tensioning tendons. After slightly shifting a few columns along the perimeter of the building, it was possible to reach every major column in the East-West direction. The geometry of the building dictated that the North-South tendons be distributed evenly, as the column layout became more irregular, and the column strips harder to define.

The original drawings called for a 10-1/2" normal weight cast in place concrete slab with mild steel reinforcing. Drop panels at each column added another 2-1/2" to the depth, for a total of 13". These drop panels, however, did more than just help with punching shear. On the original floor plans, there were cantilevers in the North-South direction that extended outward from the exterior columns as an architectural feature. In such cases, the drop panels on the exterior columns were rectangular and pointed towards the cantilever edge, almost like a supporting beam. By doing this, the slab was able to support the building façade with only mild steel reinforcing. This solution, though, presented other potential problems. One such problem lies within the fact that most concrete hotels use the painted floor slab above as a finished ceiling. By adding drop panels, the ceiling in some hotel rooms would have 2-1/2" protrusions into the space from the drop panels above – a significant architectural consideration. The new post tensioned system would aim to eliminate this issue as well.

Due to some irregularities in the floor plan, Ram Conce*pt* 2.0 was used as a finite element analysis model. The original floor plan was input into the program and

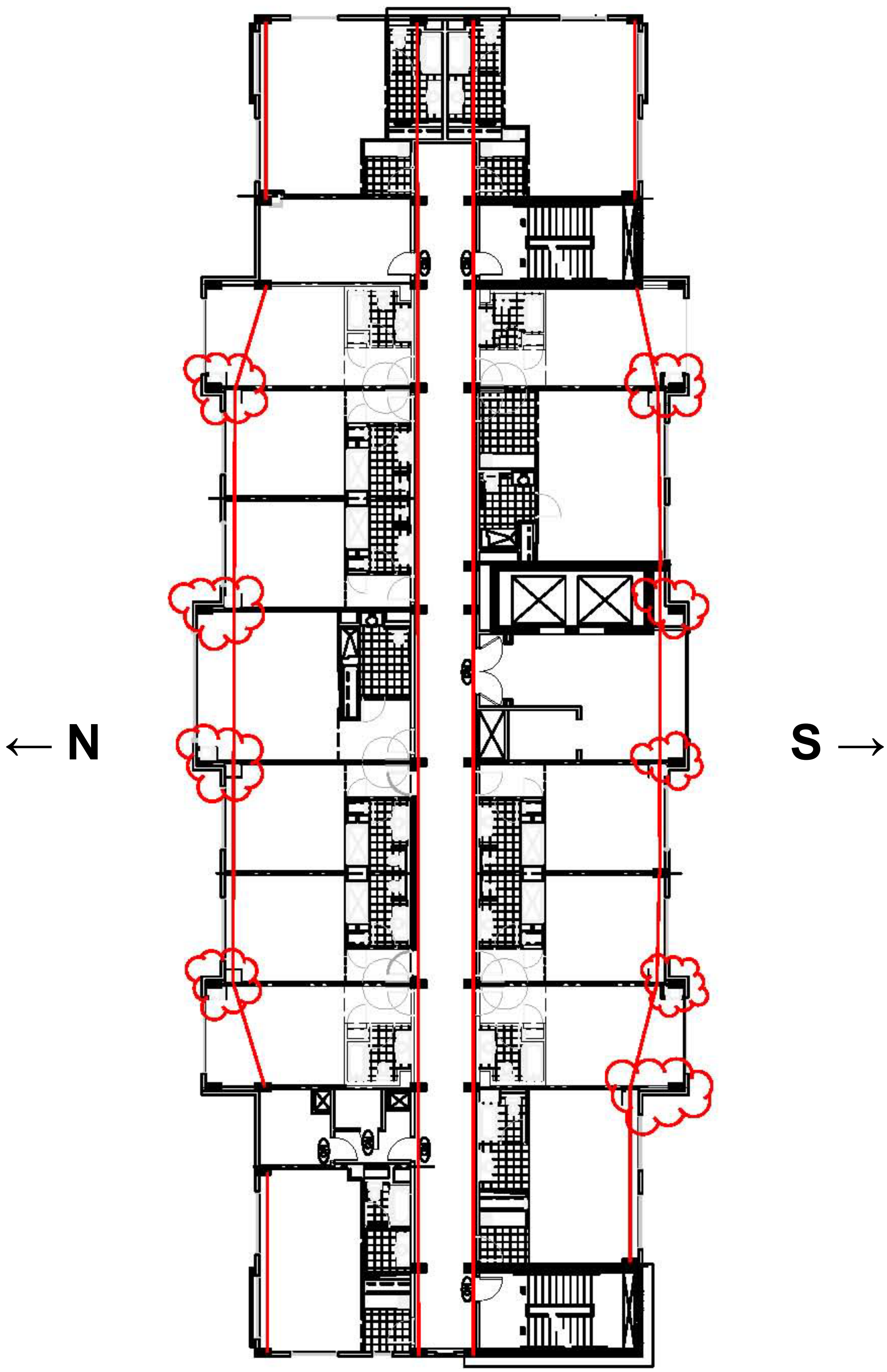

Fig. 2

evaluated in terms of the feasibility of adding post tensioning to the layout as it already existed.

A preliminary thickness estimate yielded an 8" slab based on taking the maximum span length, 29 feet, divided by 45. The frame along column line 6 was analyzed via hand calculations attached in the appendix.

The general methodology used in RAM Conce*pt* 2.0 for a typical floor first involved finding the tributary loads on all column lines. Once the loads were determined, the next step was to decide how much load the post tensioned system would balance. After posting the question on the Structural Mentors discussion board, it was evident that the industry standard is to balance somewhere between 75 and 90 percent of the dead load. For this report, 90% of the dead load was chosen to be balanced based on the possibility that the initial slab thickness estimation erred on the thin side. The tendons used were ½" 270k wire strands with an effective tensile force of 25.7k per strand. One and a half inches of cover to the centroid of the tendon was observed, thus making the maximum drape 5-1/2". The tendon height over each column was held constant, and the mid span drape was adjusted to provide an upward line load approximately equal to 90% of the dead load seen by the bay, as stated above. The limitation of the drape meant that in some spans the desired balancing force could not be met. In such cases, the maximum drape was implemented to provide as much balancing as possible. On end spans and cantilevers, the profile at the terminating end was set to 4", or half of the slab thickness, to eliminate any eccentricity at the slab edge.

Latitudinal and longitudinal column strips were generated by the computer program and evaluated for consistency (see figs. 9 and 10). Where column strips coincided with shear walls, they were deleted as they were considered a redundancy. In some cases it wasn't clear as to where a particular column strip should go, or which orientation the analysis program should consider. Each of these occurrences was considered using engineering judgment, and a decision was ultimately made based on the geometry of the structure. Another consideration that was taken into account was the possibility of a punching shear failure. Because the slab was an additional 2-1/2" thicker before the redesign, punching shear did not become a controlling issue, or even an issue to question. After taking two and a half inches off of the slab thickness, punching shear was evaluated again using the same loads as before, and it was found that it did not cause a failure. In fact, the capacity of the slab was well above the factored shear force seen by most of the 12" x 24" columns. Not only was the punching shear capacity adequate, but it was sufficient even without the drop panels that existed previously, as the dead weight of the slab was decreased by roughly 25%. Thus, the finished ceiling would appear as a smooth continuous surface without "lumps" sticking downward into the space

RAM Conce*pt* 2.0 Model

Constructing the model was fairly straightforward. After laying out the floor plan and columns, post tensioning tendons were added where appropriate. In the East-West direction, it was calculated that to balance the appropriate load, 9 wires be banded together in exterior tendons and 12 wires in the interior tendons. The increase in number for the interior spans compared to the exterior spans is because the bays experience a larger tributary area per foot. A preliminary estimate of 15 and 12 tendons for the interior and exterior spans, respectively, resulted in an axial stress on the concrete slab that was just above industry standards of 150 to 250 psi. In order to stay consistent, the numbers were scaled down to reflect what the common practice dictated. In the transverse direction, tendons were placed uniformly in groups of 3 spaced evenly between column rows, typically at 3 or 4 feet center to center. Designing the transverse tendons took more effort than the longitudinal bands because the column strips were more irregular. Figures 3 and 4 on the next page show the tendon layout chosen for a typical floor.

Where noted in figure 2, columns had to be moved to allow the longitudinal tendons to pass through them. In the transverse direction, no modifications were needed because the importance of a single tendon greatly reduces when the load is distributed so much.

The presence of slab openings for mechanical purposes presented a few problems with the transverse tendon layout. In one area along column line A (see fig. 1), slab openings directly north and south of columns A5 and A6 were interfering with the desired spacing. The tendons were bowed out and around these openings where appropriate. In other areas, tendons had to be terminated on either side of the slab opening because the size was too great to bend around (fig. 4). When considering the transverse direction as a whole, uniformity was the desired characteristic as more redundancy meant greater load redistribution characteristics.

After the layouts and tendon profiles were set, the next step was to run a first analysis to see how the slab was reacting to the loads and support conditions. The first calculation showed positive signs, but a few of the column strips failed because they weren't adequately defined. Punching shear, as expected, did not cause any failures in the slab. One of the only areas of concern rested on column line 6 between columns D6 and E6. Because this area has the longest span in the building, the deflections were the greatest. Fixing the problem, however, proved to be as easy as adjusting the tendon profile at this location. By increasing the drape by a fraction of an inch, the upward thrust created by the tendon increased enough to significantly reduce deflection in this area.

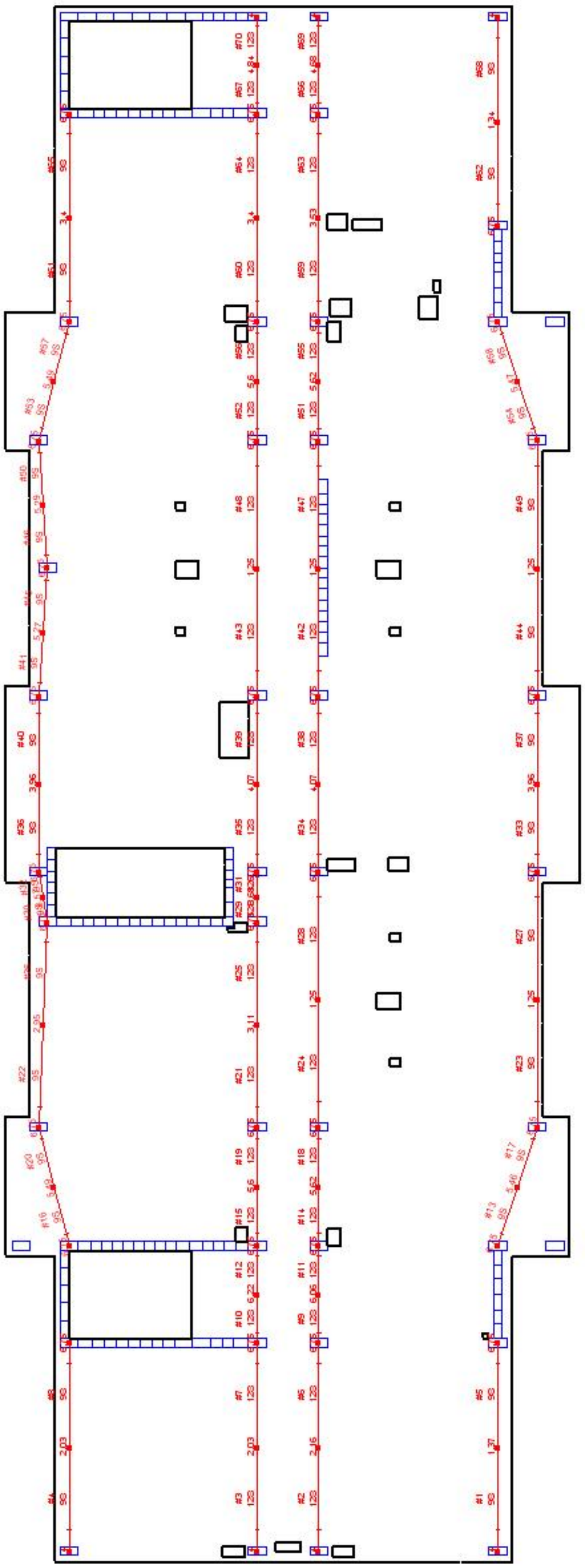

Fig. 3

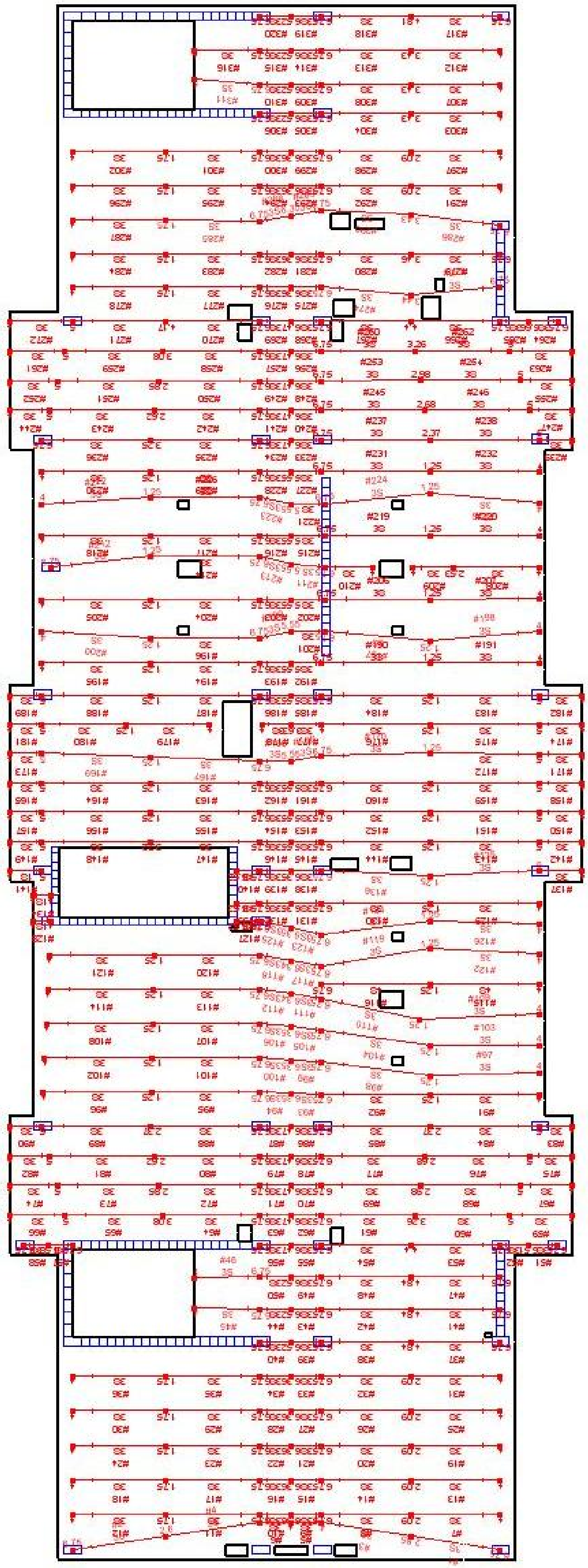

Fig. 4

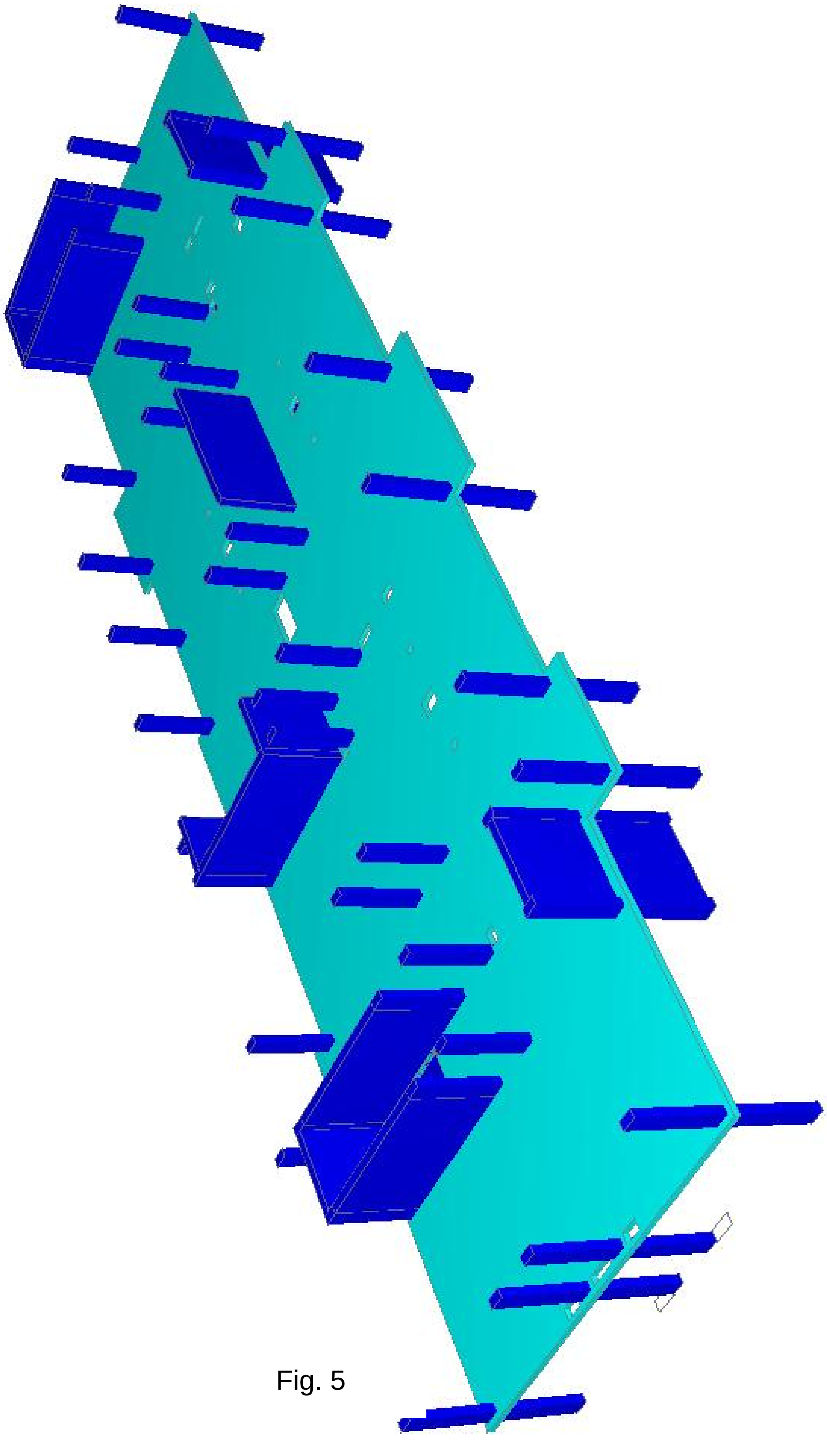

Fig. 5

After some refinement, a similar solution was sought for each instance where the deflection could be improved. The deflection reduction was accomplished without having a significant camber on the slab in the absence of loading. In fact, the maximum camber was found to be less than one tenth of an inch for the slab alone.

In all, the post tensioned system performed admirably given the building geometry. Excessive deflections were not found anywhere on the floor plan, and the largest deflection occurring at the longest span was only 0.6", or approximately L/580. Punching shear did not control with the thinner slab as was initially thought. The system has more than adequate strength to resist the factored loads applied thanks to the perimeter length of the columns. A weight savings of approximately 25% on each floor slab meant that the weight of two and a half slabs, as they presently exist, could be completely eliminated from the overall building dead weight. This results in a weight savings of 3438 kips for the entire structure, and a possible reduction of the total building height by 27-1/2".

Figure 7 on the next page shows the sustained long term deflection of the slab. Note that the red portion has a maximum deflection of 0.6". On the right, the precompression plan is shown as figure 8. As was desired, the precompression is very uniform across the entire plan.

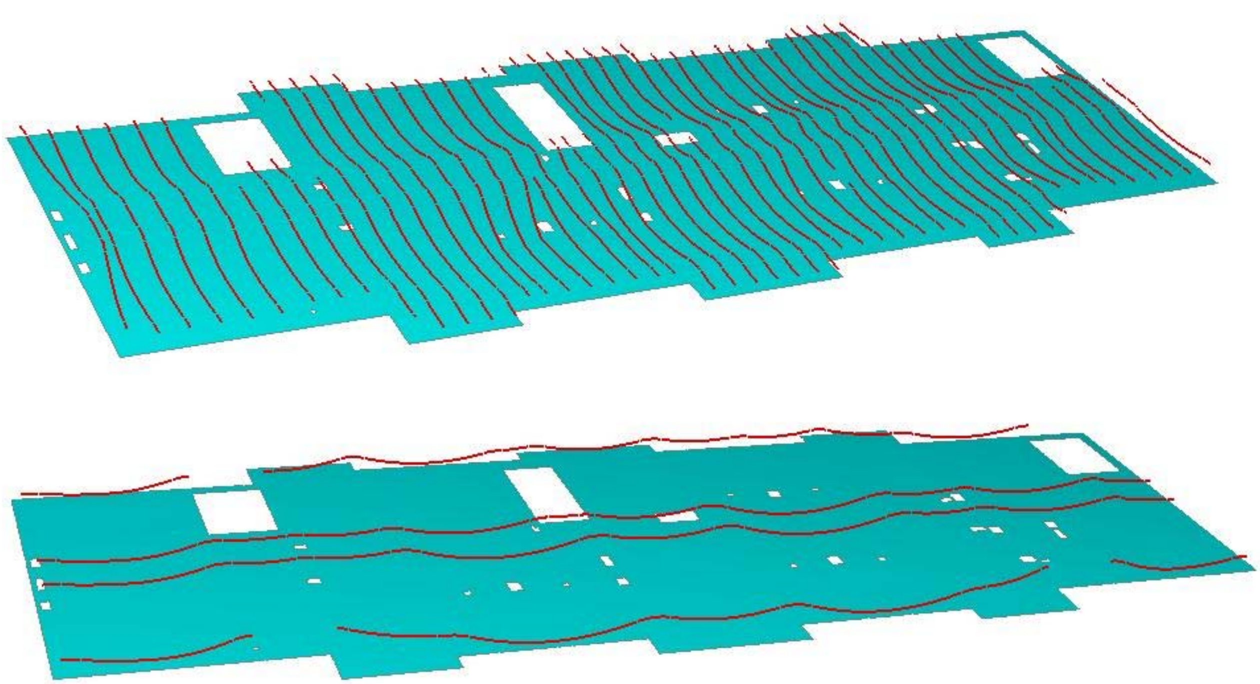

Fig. 6

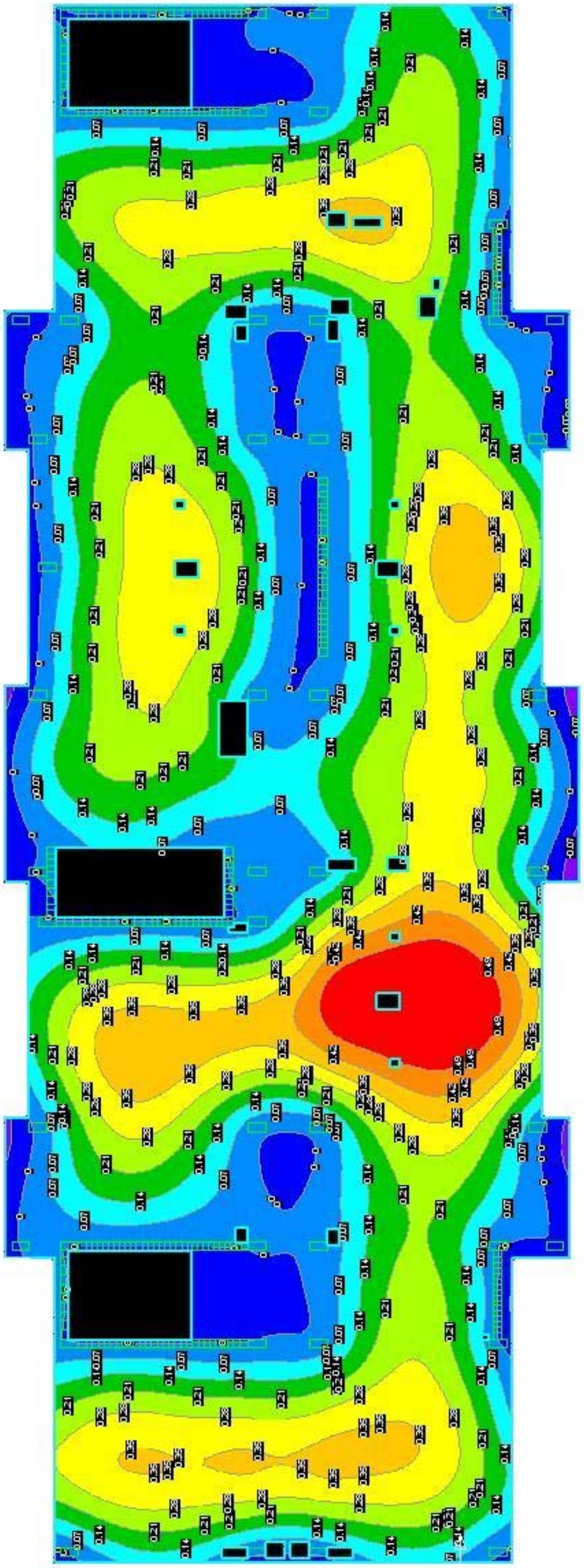

Fig. 7

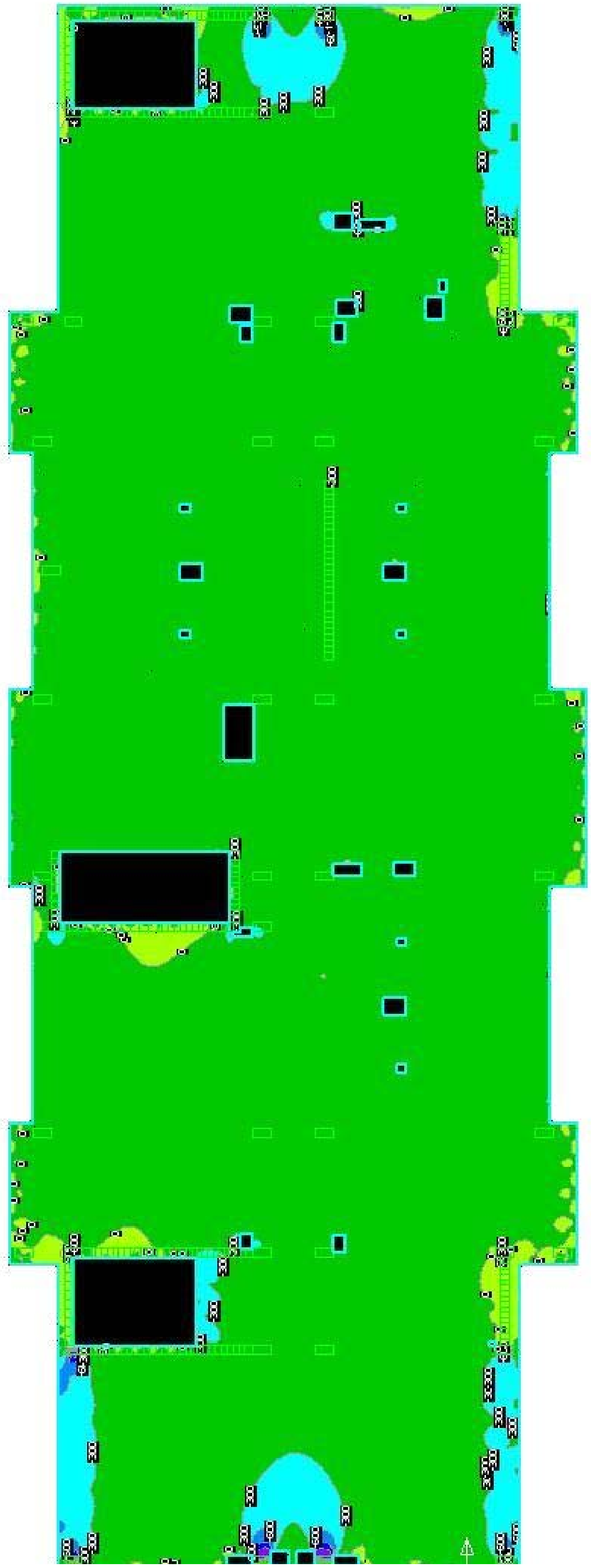

Fig. 8

One highly scrutinized part of the floor slab along column line A proved to be the only instance where a mechanical opening had to be modified for the slab to meet code requirements. Because the mechanical openings occurred directly beside the columns, RAM Conce*pt* 2.0 gave a failure status to the column strip between columns A5 and A6. After many attempts to change the tendon layout, a different approach was considered. Deleting the slab penetrations solved the problem, but posed a new obstacle to the plumbing contractor, as the area to run pipe through the slab was cut in half. A simple redesign of the bathroom unit would have solved the problem as the slab openings were simply in a needed space. The uniform tendons were bent around the openings in the slab and, when next to the openings, the profile was held constant at 4" to minimize any positive moment absorbed by the slab in this weak location. Deflection was not a concern because the area in question was directly next to two columns, thus not allowing any sag in the slab.

<u>Lateral System Analysis</u>

The distribution of loads depends directly on each member's relative stiffness. Because each shear wall is the same thickness, relative stiffnesses can be closely approximated by each member's length. In this case, all shear walls are orthogonal to each other and do not need to be broken down into components. Each shear wall's relative stiffness value is listed in the column on the far right in Appendix A under the 'Force Distribution Calculations' spreadsheet.

The more efficient the load path, the more lateral force a building's frame is able to transfer to the foundation. In the case of wind, the largest forces are transferred from the very top of the building all the way down. The wind hits the façade, is transferred to intermediate elements, and then to the columns and shear walls. Once the load reaches these lateral elements, they are transferred down to the foundation. Due to design simplicity, the shear wall and column layout does not change as the floors go up, there is no diminishing of strength towards the top of the building. Because seismic forces control in this study, larger forces need to be resisted, but the maximum applied force occurs at story level 10, not the top. The load path is the same as before, and rigid diaphragm action from the floor slab helps keep drift to a minimum.

A potential weakness of the system is the fact that only one shear wall lies completely within the floor slab. The shear wall between columns G6 and I6 has rigid diaphragm action and bracing in all directions, but the others do not. Because they either lie on the exterior of the building or along an elevator or stairwell shaft, all the other shear walls have at least one side without bracing. Consideration must be taken into account to adequately tie the floor slab into each shear wall, where possible.

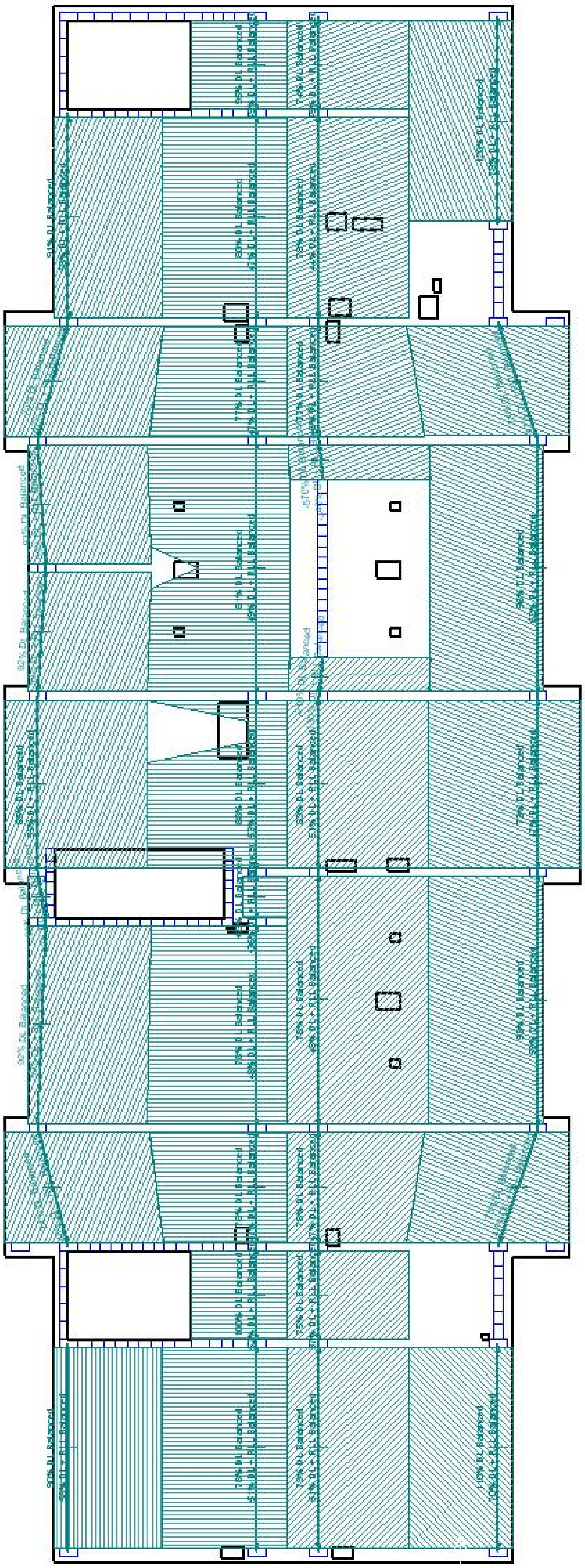

Fig. 9

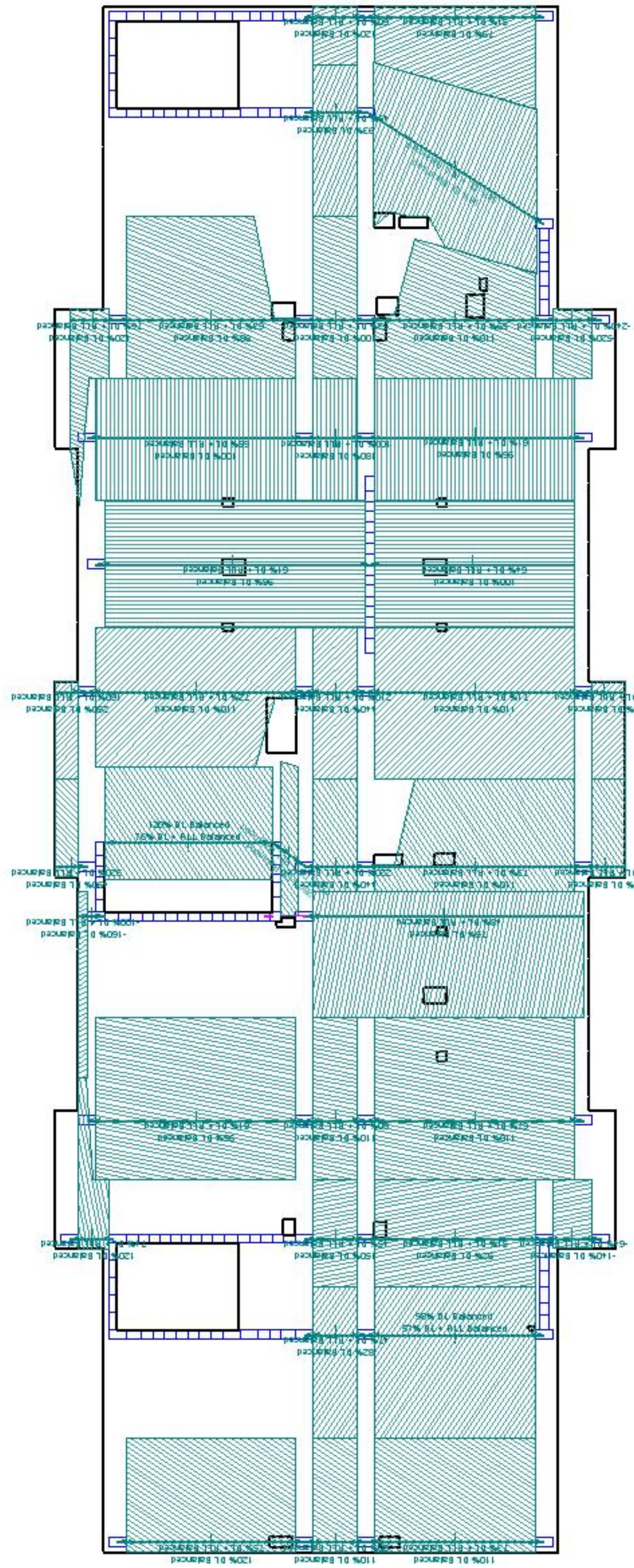

Fig. 10

One of the accompanying goals of the post tensioned analysis, in addition to reducing the dead weight of the structure, was to possibly redesign the lateral system based on a smaller base shear value. The calculated base shear from Technical Report Three yielded a controlling seismic shear of 594 kips in both directions. Perpendicular to the long direction of the building, the wind created a base shear of 491 kips. Thus, even if the seismic forces reduced drastically due to the total weight savings, the shear could never be less than what the wind forces dictated.

To help with tedious analytical procedures, RAM Structural System was used to create yet another model of the structure. This model, however, simulated the effects of lateral forces applied to the building. By modifying the original model used for Technical Reports One and Three, a new behavior was studied under the premise of a thinner slab.

Assuming that changing the slab thickness from 10-1/2" to 8" did not significantly change the overall building stiffness, the reduction of the story drift can be attributed to two factors. By code, the building weight directly affects the equivalent lateral force that is used to predict behavior under seismic conditions. The second factor has to do with the drift itself. Secondary effects, known as P-delta effects, also contribute to story drift. As the building sways, the eccentricity created by the resultant force of each story compounds the issue of lateral movement. By reducing the weight of each floor, the P-delta moment is also reduced, thus not contributing as much to the total drift of the structure.

The new equivalent lateral force was calculated to be 530 kips, a reduction of 64 kips when compared to the thicker slab, or about 11%. Because the base shear depends directly on building weight, the overall building weight must have also decreased by 11%. The difference between the 25% slab weight reduction and 11% building weight reduction is due to the presence of columns, shear walls, and building facades, as they did not decrease in weight or size.

Analyzing the shear force distribution for two loads that differ by only 11% was a moot endeavor. Because none of the shear walls could be completely eliminated, a redesign of the system was considered. The placement of the walls, however, was not an easy element to change. Spectacular views were afforded to each hotel room with much thought and consideration by the architect, and moving any of the shear walls placed along the exterior of the building would have blocked some of these views. As the plans exist, both shear walls along the building façade occur at linen storage rooms – a convenient placement for the two shear walls; repositioning them was out of the question. Nine other shear walls were placed around stairways and elevator cores – another convenient location. That left a single shear wall in the entire building, the only one that garnered consideration to move. Because the torsional affects of the shear wall layout rendered favorable results, the last wall in question was aptly placed near the center of rigidity of the building, absorbing direct shear with little rotational influence.

In summary, the lateral redistribution of controlling shear forces did not change the lateral resisting system. The new seismic base shear, 530 kips, was only 11% smaller than the original value of 594 kips.

Foundation Considerations

Shallow foundations were used for this project with typical foundation thicknesses anywhere between two and five feet. Columns that carried the heaviest loads were compressed by about 700 kips of axial force. In these cases, the typical spread footing dimensions were 15’-0 x 15’-0 x 3’-0. The footings were designed through RAM Structural System for both slab thicknesses and came out to be the same each time. This suggests that gravity loads are not the controlling factor, but rather uplift or overturning is governing the design. In light of this, the sizes of the footings need not be changed for this analysis.

Structural Depth Summary and Conclusion

As a conclusion to the structural depth topic, the proposed post tensioned system is indeed a feasible alternative to the cast in place concrete floor slab. In this report it has been shown that the proposed system meets code requirements while being 25% lighter and able to resist punching shear without the need for drop panels. As a direct result of this, the ceiling as seen from inside the individual hotel rooms will be without drop panel “lumps”. The maximum deflection was found to be 0.6” for a 29 foot span, a value of L/580. Another benefit of the proposed system is a reduced drift response to imposed lateral forces thanks to a less severe P-delta factor. Wind and seismic controlling drifts decreased from 2” and 5.3” to 1.2” and 3.88”, respectively. Switching to the post tensioned system does not necessarily mean that the lateral shear wall system or foundations be redesigned. As a matter of fact, the increased performance of the building was because the shear walls were slightly over designed, albeit at a slightly higher overall cost than an optimized system.

Mechanical Breadth Study

Existing Conditions

For the building façade, STV Inc. chose to use precast concrete panels with a glazed 4-1/2" window system. The windows are supported by wood blocking provided by the general contractor, and do not rest on the precast panels. The panels themselves attach to finished floor slabs with embedded bolts and angles provided by the precast manufacturer. Refer to figure 11 for a section of the building façade.

Building envelopes can be divided into three categories – cavity walls, barrier walls, and mass walls. In this case, the wall can be classified as a barrier wall system. That is, the exterior assembly relies principally on the weather-tight integrity of the outermost exterior wall surface to resist bulk rainwater penetration and moisture ingress. As a result of the chosen envelope design, the construction joints between the architectural precast panels and the glazed windows is considered as integral a part of the moisture resisting system as the components themselves. It is because a barrier wall lacks redundancy in its design that the actual construction of the façade is critical to its performance.

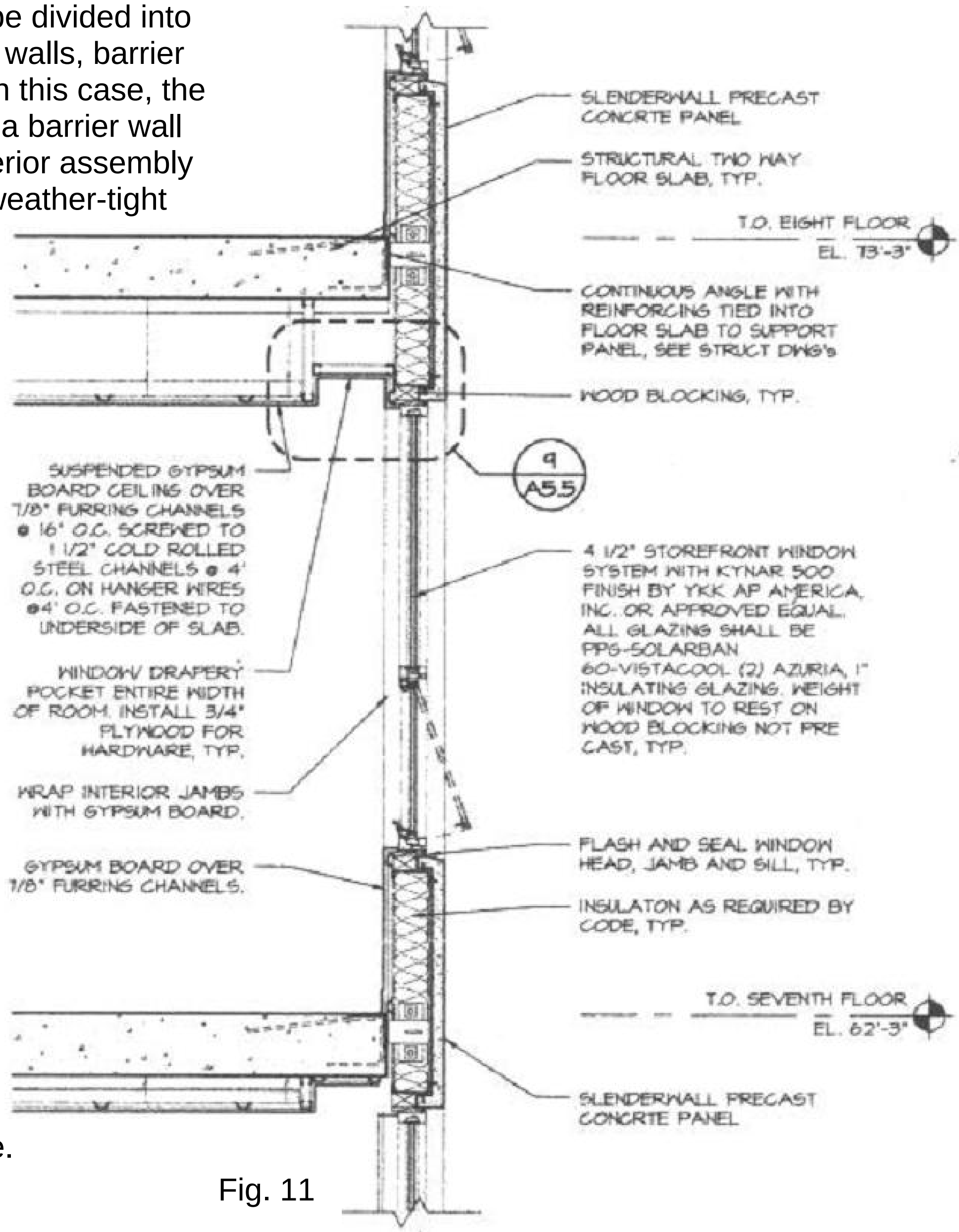

Fig. 11

Even today, uncontrolled rainwater penetration and moisture ingress are the most common threats to the integrity and performance of the building envelope. Research suggests that miscommunications and errors in the design and installation of the façade components are responsible for most of the problems encountered with exterior walls, and not the materials themselves. A rush to put forth a finished product under budget and ahead of schedule, unfortunately, is the driving force behind most project team members instead of demonstrating the proper attention to detail that these assemblies require. Also consider the misinterpretation by some engineers that "value engineering" is an appropriate excuse to eliminate some key elements in terms of the overall performance of a system, and it is easy to see why there is such a discrepancy between what *should* be built and what actually *is* built.

As is the case with any barrier wall system, the communication between the mechanical engineer, architect, and possibly the owner representative, is crucial. Regularly scheduled meetings are recommended to ensure that coordination continues throughout the entire building process.

Possibilities for Redesign

As stated in the initial conditions portion of this breadth study, the weight of the windows is not to rest on the precast panel, but rather on wood blocking provided by the general contractor. This small but important detail means that the precast concrete must only support itself, meaning that the connection to the existing slab need be only as large as required to support its own dead weight.

A way to possibly shed more weight from the structure would be to replace the existing precast panels with a much lighter EIFS system. EIFS stands for Exterior Insulation and Finish Systems, and consists of layers of impermeable membranes that not only stop water penetration, but also serve to insulate the building (see figure 12). But, because it is also a barrier protection system, the same dangers listed above exist with EIFS.

To implement a non-structural envelope system, structural framing would have to be constructed around the perimeter of each floor slab. This work could easily be accomplished by the general contractor, as wood studs and blocking can generally meet the design requirements of components and cladding systems.

Some major advantages of using EIFS relate to the weight savings that can be had due to the impermeable nature of the thin layers. EIFS is significantly lighter than conventional stucco façades, and drastically lighter than architectural precast panels. Some other reasons to use EIFS include the fact that it can be customized to fit any surface, it can be applied in any color, and it does not require time to cure.

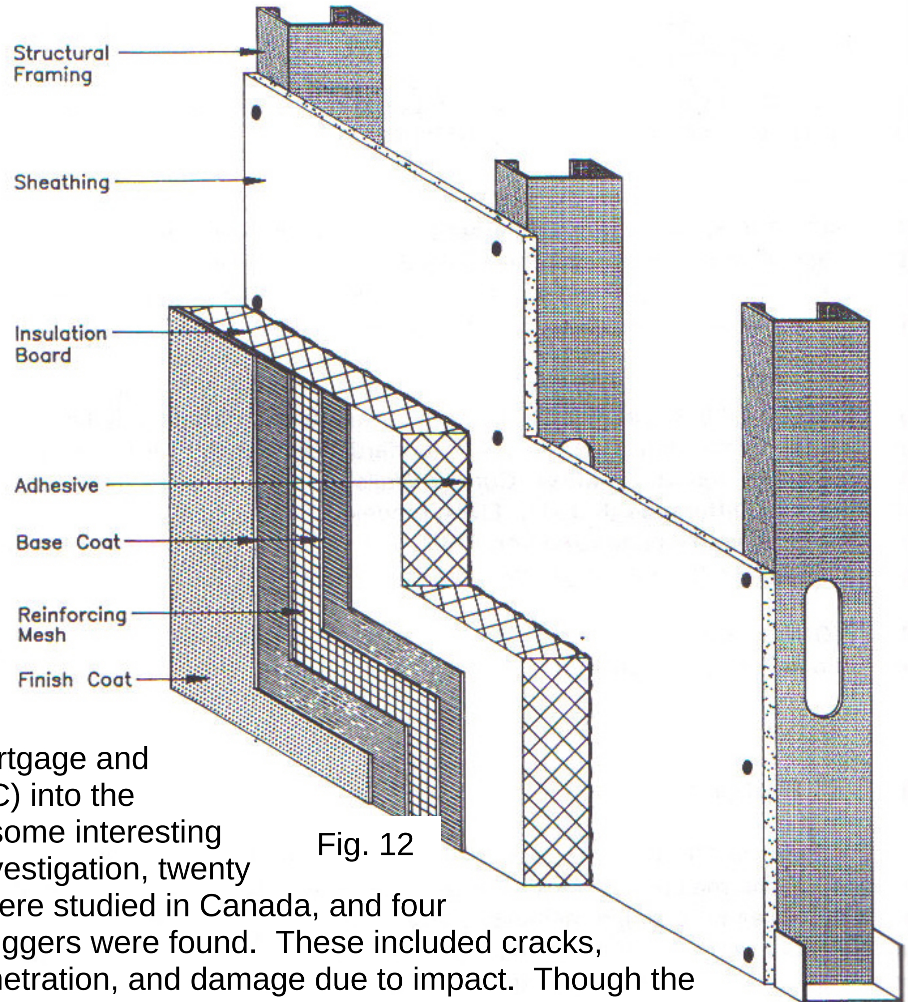

Fig. 12

Upon its introduction to the building industry in the 1970s, EIFS seemed to be an inexpensive solution to many problems encountered in building projects. However, as professionals and owners alike would soon learn, implementing the system would not be as easy as originally thought.

Through the wide usage of EIFS, many problems have been discovered about its ability to successfully seal a building envelope. An investigation by Canada Mortgage and Housing Corporation (CMHC) into the installation of EIFS yielded some interesting results. According to the investigation, twenty three installations of EIFS were studied in Canada, and four major performance failure triggers were found. These included cracks, joint problems, moisture penetration, and damage due to impact. Though the finish was in good condition, not one of the twenty three buildings was free from defect. Approximately 30% of the installations had defects serious enough to consider replacement[1]. A separate investigation by Richard Lampo and Jonathan Trovillion, two materials engineers for the US Army Construction Engineering Research Laboratories (USACERL) yielded similar results[2]. In their report, the observed problem areas for EIFS included cracking, impact damage, and faulty workmanship which led to a catastrophic system delamination (see figure 13). This occurred when an entire section of the insulation system lost adhesion to the building cladding, causing the layered insulation to fall off the face of the building.

Indeed, the general industry opinion on the cladding reflects the findings of these two independent studies. EIFS is intended to completely seal the exterior of the building and not let any water in at all, which is called a face sealed system. This contrasts the other main envelope philosophy that allows water penetration past the outermost layer, but offers a pathway for that moisture to escape. Because buildings shift, shrink, creep, and generally move, the proper installation of EIFS is crucial to its performance. Unfortunately, this matter is one of the most difficult issues to enforce.

Once water or water vapor penetrates the EIFS layer, there is nowhere it can go to escape. For very long periods of time, it remains in contact with the sheathing that covers the building. This is ultimately the biggest drawback to using EIFS as a waterproofing layer. Instances where some mould and mildew are present occur virtually everywhere. In some cases, areas of advanced mould or moss can grow, leading to a complete destruction of the system in order to repair.

Delamination also occurs, as mentioned above, when the EIFS loses its attachment to the building itself. Once a problem is identified, there is no way, other than destructive testing, to ensure that the damage has not spread.

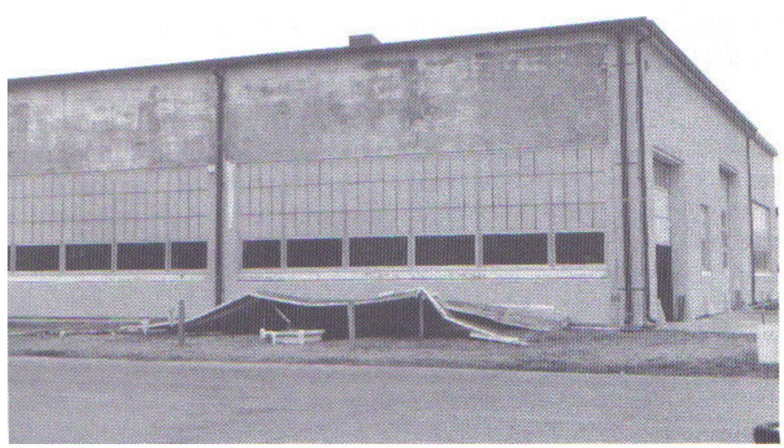

Fig. 13

Some possible solutions to the EIFS water penetration problem have been circulating ever since the issue came up. An article written by Tony Tufariello of Stamford proposes solutions that would help to limit moisture problems[3]. Among the proposed solutions is an incorporation of a “mesh weep” water drainage system to allow water to escape from inside the EIFS layer. Another solution could be to use a pressure equalization system that equals the pressure on the outside and the inside of the synthetic stucco. This would significantly reduce the amount of water penetration in the first place.

An excerpt from an article written by Gary L. Zwayer titled “EIFS: When It Works, When It Does Not” demonstrates the author’s confidence in the system when it is implemented correctly[4]:

> “The findings from the EIFS clad projects I have investigated since joining Wiss, Janney, Elstner Associates, Inc. all indicate that the problems had occurred because the designer or the applicator failed to understand the system or follow the manufacturer’s instructions.”

The author further goes on to describe how he believes all problems associated with EIFS can be avoided through proper attention to the design of the system. Over the years, EIFS has been continually improved upon and has evolved considerably since the first applications about 30 years ago. Not only have the design considerations changed, but the quality of construction has improved as well. There are two main aspects to ensure that EIFS succeeds. It is the responsibility of the manufacturer to design a quality product, and the owner must choose a reputable installer in order for EIFS to prevent water penetration.

A study by William F. Egan and Jason W. Iacovelli into the projected life cycle costs of EIFS compared to other systems found that EIFS has a great cost advantage[5]. At an estimated initial cost of $11.43 per square foot, the EIFS system was the second lowest among the popular cladding assemblies studied. If the EIFS needs to be completely replaced, as is required in some cases, the overall cost doubles to $22.86 which would make it the second most expensive system, trailing only stone veneer. Consumers and producers alike know the risks and rewards of using EIFS. When it is done successfully, it is a hard alternative to beat.

Using EIFS has inherent risks that must be weighed prior to making the decision to use it. The synthetic stucco has had a bad reputation due to contractor, manufacturer, and applicator errors. Due to these errors, it is suggested that the contractor work closely with the manufacturer in each project if possible. The system has now bee around long enough that it is becoming efficient and more effective as a face sealant. In all, EIFS is an inexpensive way to achieve both insulation and water repulsion; it just has to be installed correctly.

Mechanical Breadth Summary and Conclusion

The use of architectural precast panels is a popular strategy to build barrier wall systems. As with any barrier wall system, the coordination of all parties involved is crucial to the performance of the envelope. Failures in these walls usually result from an error or omission on the part of the detailer or construction crew. Field checks should be made on a regular basis to ensure proper craftsmanship. If the proper steps are taken to ensure that the barrier wall is sound, either system will work to satisfaction. The ultimate decision comes down to the architect and his or her vision for the project. The recommendations of this report would be to use the EIFS system based on its versatility. It is much lighter than the concrete panels and can be painted to match them exactly.

Construction Management Breadth Study

Existing Conditions

Switching from a cast in place slab to a post tensioned slab raised questions about any impact the switch would have on the budget and schedule for the project. Construction on the site started in November of 2006 and the project open date is set for March 1, 2008, a time period of about 16 months.

In order to see what potential impact the structural redesign would have on the construction management aspects, the exact initial schedule and cost data were not needed. Instead, only the differential was calculated based on the volume of materials saved and the labor included with the savings.

As discussed previously, the shear walls, columns, and foundations did not need to be redesigned in the structural depth study. Because the schedule is so similar between the post tensioned slab and the normally reinforced cast in place slab, the majority of the work done is identical except for a few discrepancies in the amount of steel and concrete to be placed.

Approach

RS Means 2002 was used as a guide for unit prices of concrete, steel, post tensioning strands and material placement. By calculating a savings of 27-1/2" off the entire height of the structure, the volume of the columns and shear walls for a 27-1/2" height was multiplied by the average cost of the materials and labor and subtracted from an initial value of zero to constitute saving money. The total savings in slab volume were calculated in a similar way and subtracted identically. From the RAM Conce*pt* 2.0 computer output, the difference (in tons) of normal 60ksi reinforcing was multiplied by its unit cost and subtracted alongside the other savings. This constituted a savings of $386,425 not including overhead and profit. The post tensioning tendons, however, still had to be added to the null value because they did not exist previously. This yielded a total approximate savings for the structural redesign of $108,000 after including multiplication factors of location and inflation.

To calculate the change in schedule, the unit values determined in the cost estimation were multiplied by the average daily output of the crews who assemble them. This resulted in an average schedule increase of about 1 day per floor, or eleven days for the whole building. The largest scheduling setback came from the time it takes to place the post tensioning strands. At an extra six days per floor when compared to the normally reinforced system, it adds up to a significant amount of time. The reduced amount of rebar that needed to be placed was the biggest time saver for the post tensioned system. By shortening the placement time a full four days, it helped to offset the previous lengthening of the schedule.

Construction Management Breadth Summary and Conclusion

After calculating the adjusted schedule and cost data, it is clear that the structural redesign did not have a major impact on the schedule, but yielded a savings of $172,100 after inflation, profit, and overhead were factored in. This is a significant change to the project budget and suggests that, all else being equal, the structural redesign was indeed a viable alternative to the existing system. When coupled with the results from the structural depth analysis, the post tensioned system makes a strong case for itself.

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