Hampton Inn & Suites – National Harbor, MD

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Structural Option
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Senior Thesis Final Report

Spring 2007

General
- 11 story hotel with basement
- 151 rooms
- 600 ft² swimming pool area
- 650 ft² exercise room
- 128 feet to top of tower roof

Construction
- Cast in place concrete floors and columns
- 10/20/06 Construction Start Date
- 3/1/08 Project Open date

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National Harbor, MD

Project Team
- Owner - OTO Development
- Architect - STV, Inc.
- Structural Engineer - Hope Furrer Associates
- MEP - Schlenger Pitz & Associates
- Civil - Loiderman Soltesz Associates

Architectural
- Unique SlimWall curtain wall design
- Precast architectural panels
- 11th floor balcony
- Single ply TPO ballasted roof membrane

Electrical
- 350 kVA emergency generator with 370 gallon sub-base tank

Structural
- Reinforced 10-1/2″ flat slab with 2-1/2″ drop panels
- 12 shear walls for rigidity
- 24″x36″ transfer girder over pool structure
- Spread footings located 14′ below grade

Mechanical
- Window/wall ratio of 0.34
- 120A, 9500 cfm, 6800 lb. rooftop heating/cooling unit concealed by mechanical screenwall

Table of Contents

Appendices

Executive Summary

Structural Depth Study

After proposing to investigate a change in flooring systems from a normally reinforced cast in place concrete slab to a post tensioned flat slab, it was the intention of this report to fully carry out an investigation looking into the structural feasibility, design, refinement, and, ultimately, recommendation for the alternate system.

Upon the completion of the examination, it was found that the alternative flooring system was indeed a viable alternative to the existing system. A finite analysis computer model was generated in RAM Conce*pt* 2.0 to predict the behavior of a thinner slab under loading conditions. The new system passed all code issues and was easily adapted to the Hampton Inn & Suites' geometry; hence this section of the report concludes that the post tensioned system has significant advantages over the cast in place slab.

Mechanical Breadth Study

A comparative building façade evaluation was conducted, and it was found that the use of EIFS when compared to architectural precast panels offers only slight advantages. Because both systems are barrier wall envelopes, the same constructability precautions exist for both methods. The biggest advantage EIFS possesses is a weight savings over the precast panels, but because the difference is small when compared to other dead loads, either system will perform adequately.

Construction Management Breadth Study

The impact that the proposed floor system change had on the schedule of the project was minimal at a mere 11 days over the original timeline. The budget, however, was greatly affected, as a savings of $172,100 could be observed if the post tensioned system was implemented. The amount of savings found by this analysis is more than enough to seriously consider which system would have been best.

Conclusions

It is the ultimate recommendation of this report, in fact, that, had the building not yet begun construction, the post tensioned system be used in place of the existing system.

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.

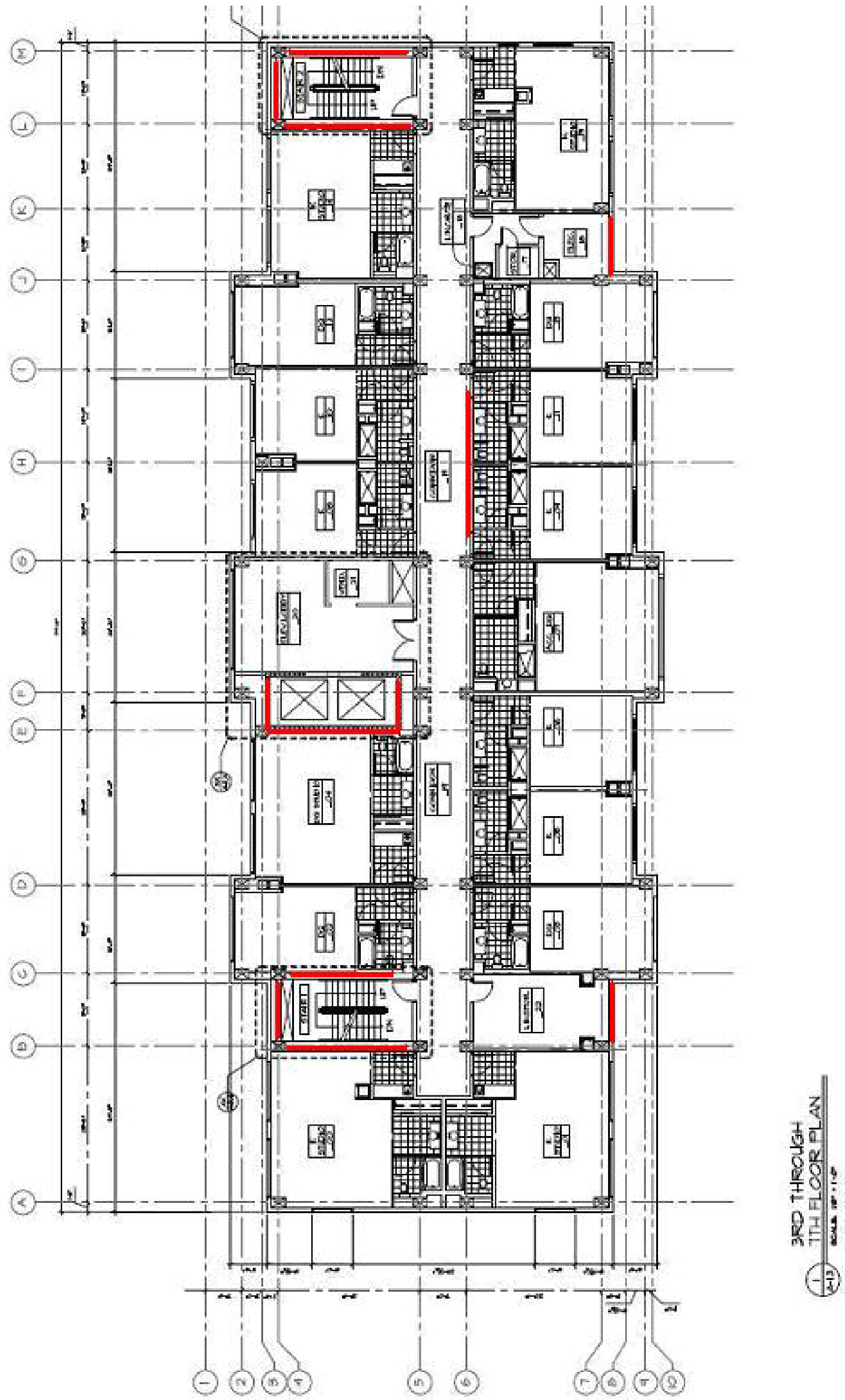

Fig. 1

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

<u>Floor System</u>

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

<u>Lateral System</u>

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

2. -Distribute lateral loads to shear walls

3. -Design lateral resisting elements

<u>Breadth Studies</u>

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

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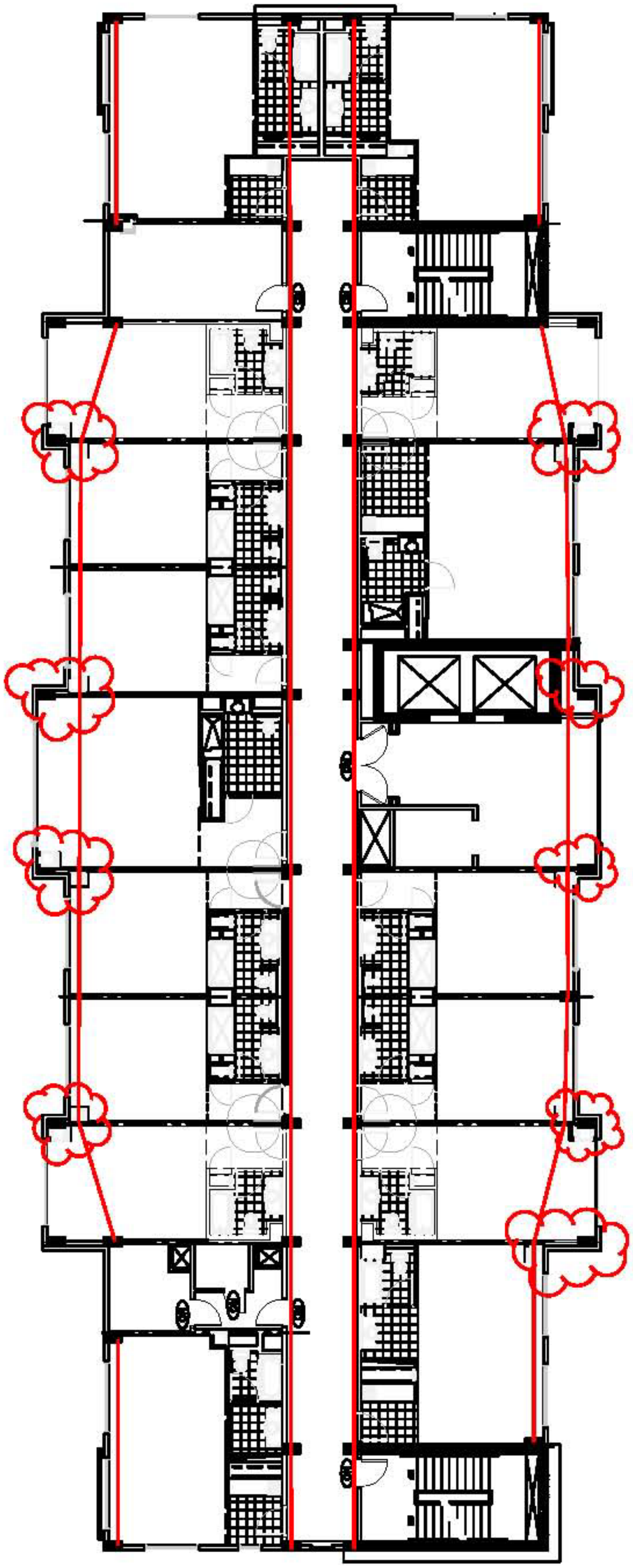

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Fig. 2

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.

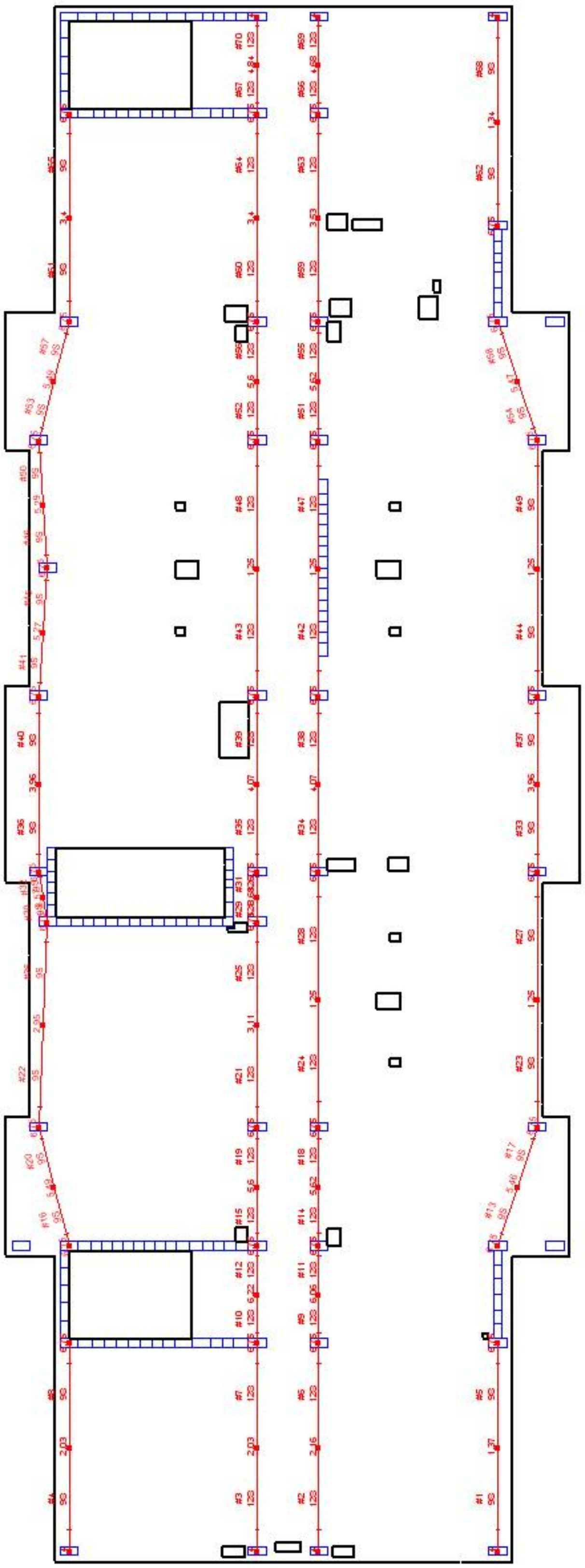

Fig. 3

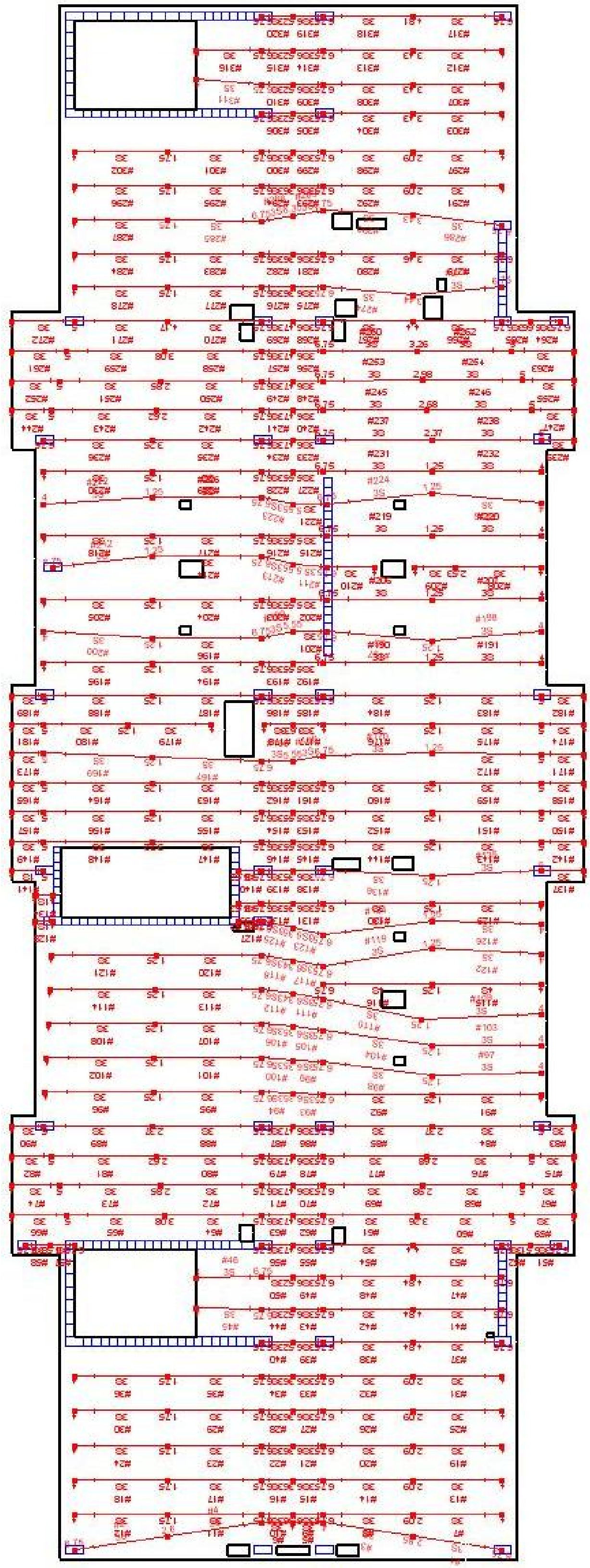

Fig. 4

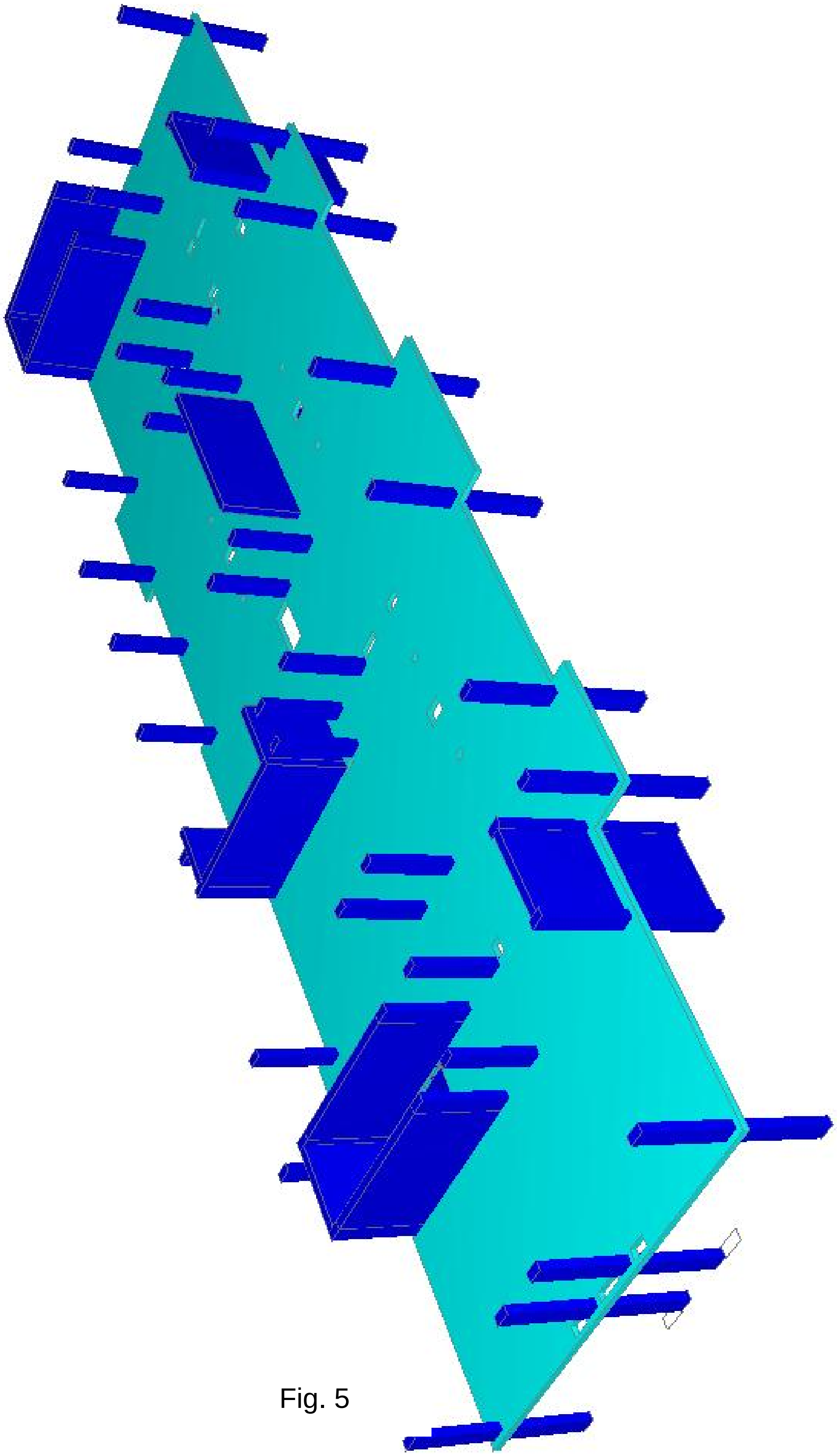

Fig. 5

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.

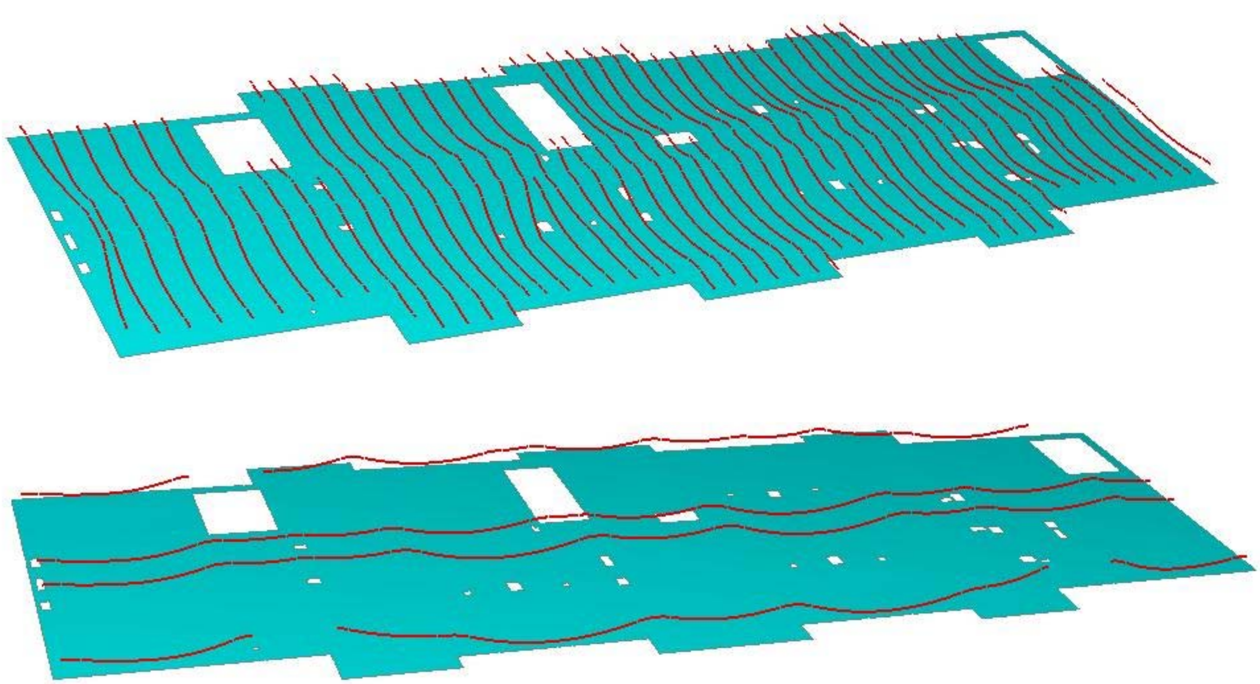

Fig. 6

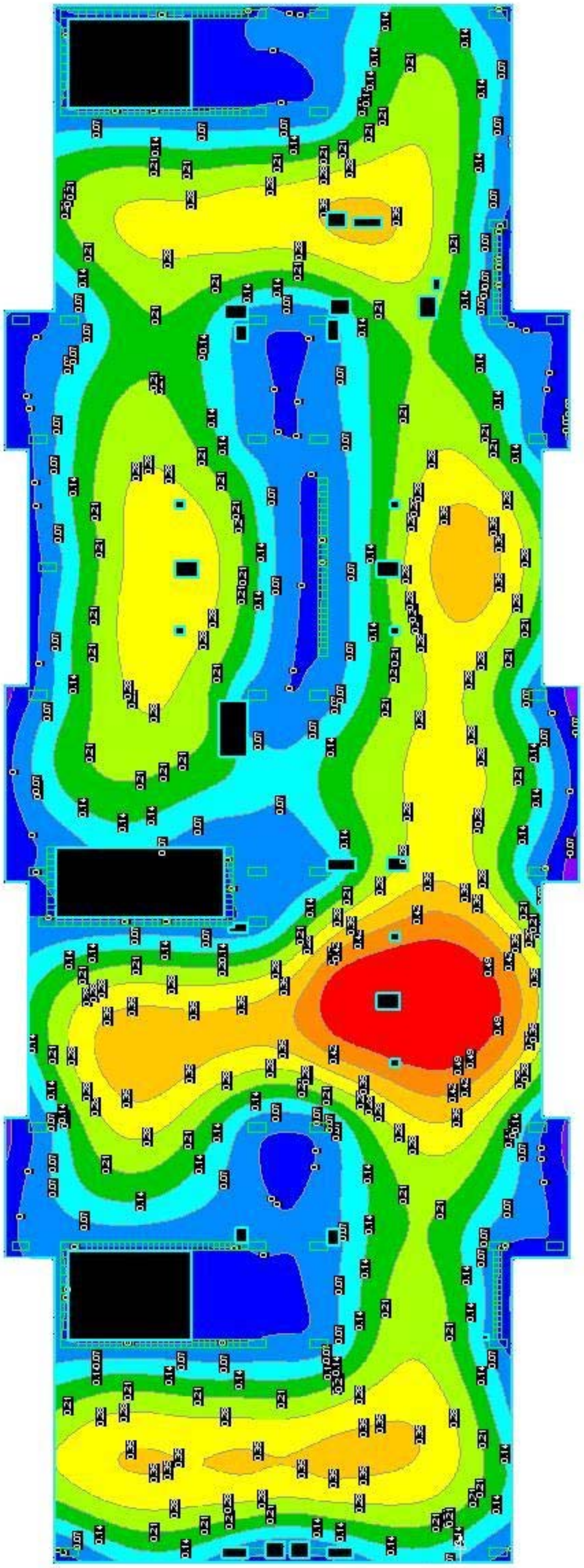

Fig. 7

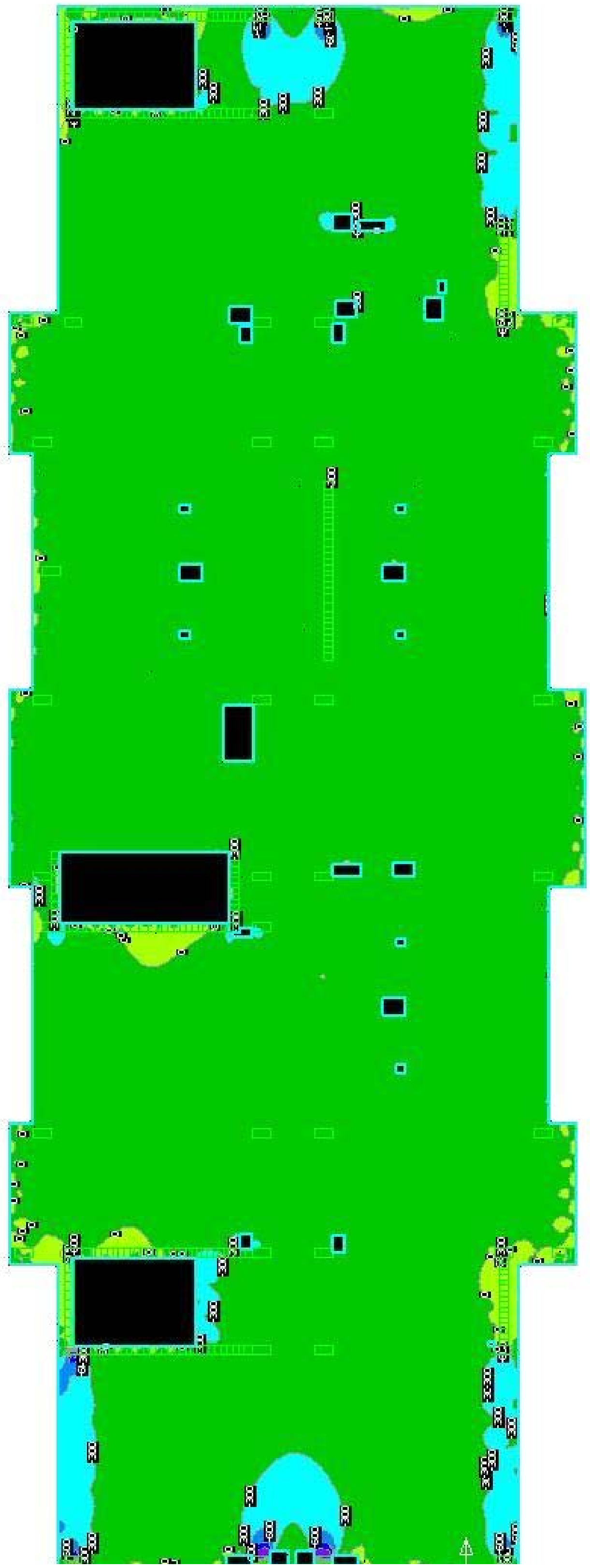

Fig. 8

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.

Lateral System Analysis

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.

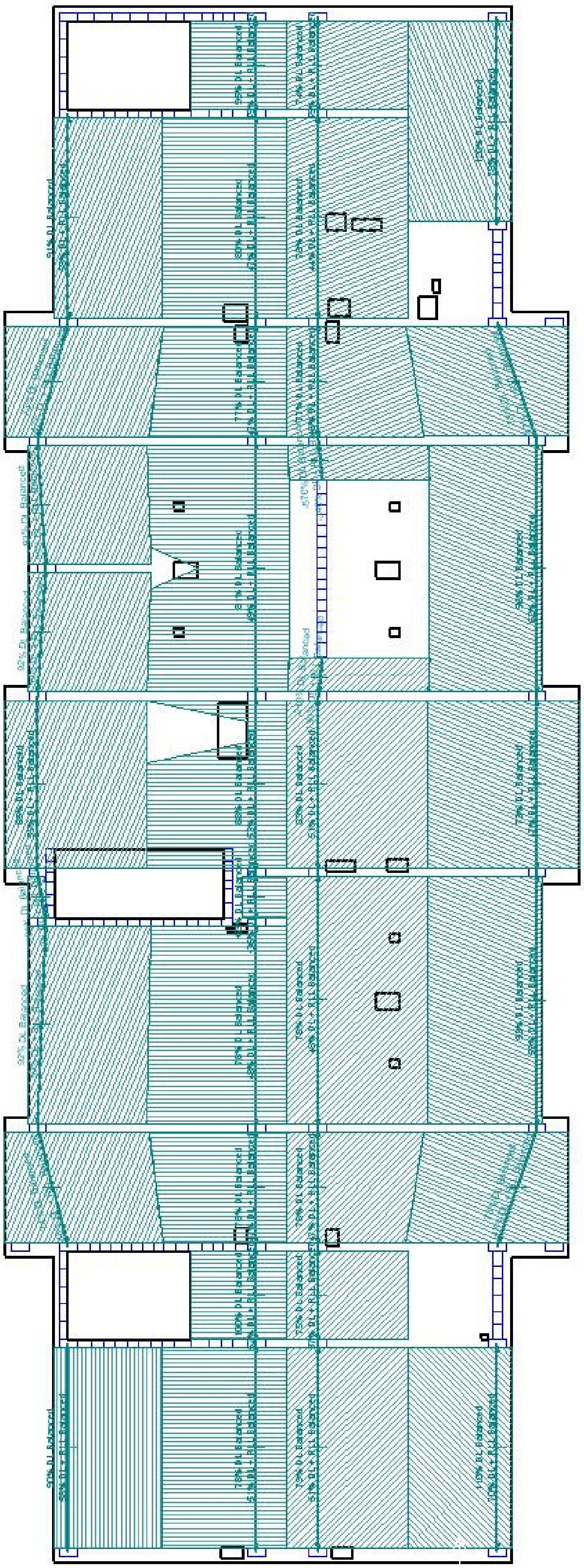

Fig. 9

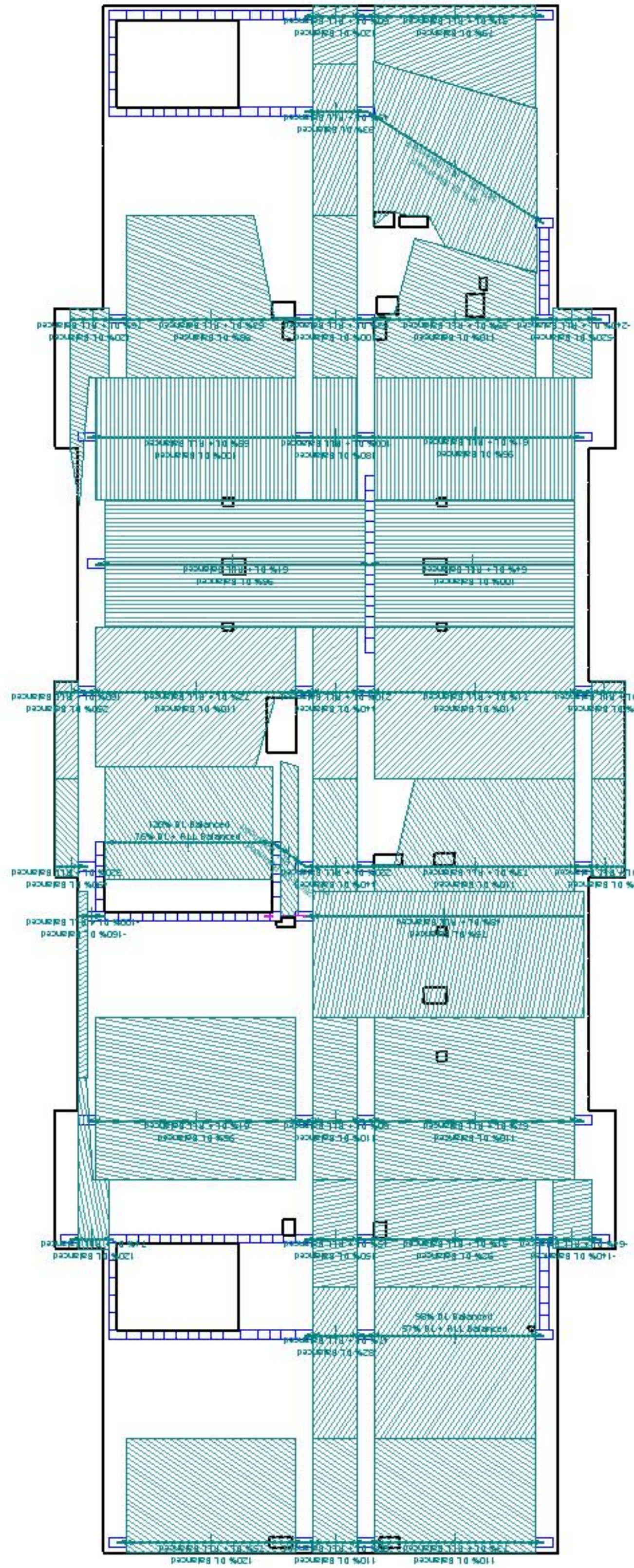

Fig. 10

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.

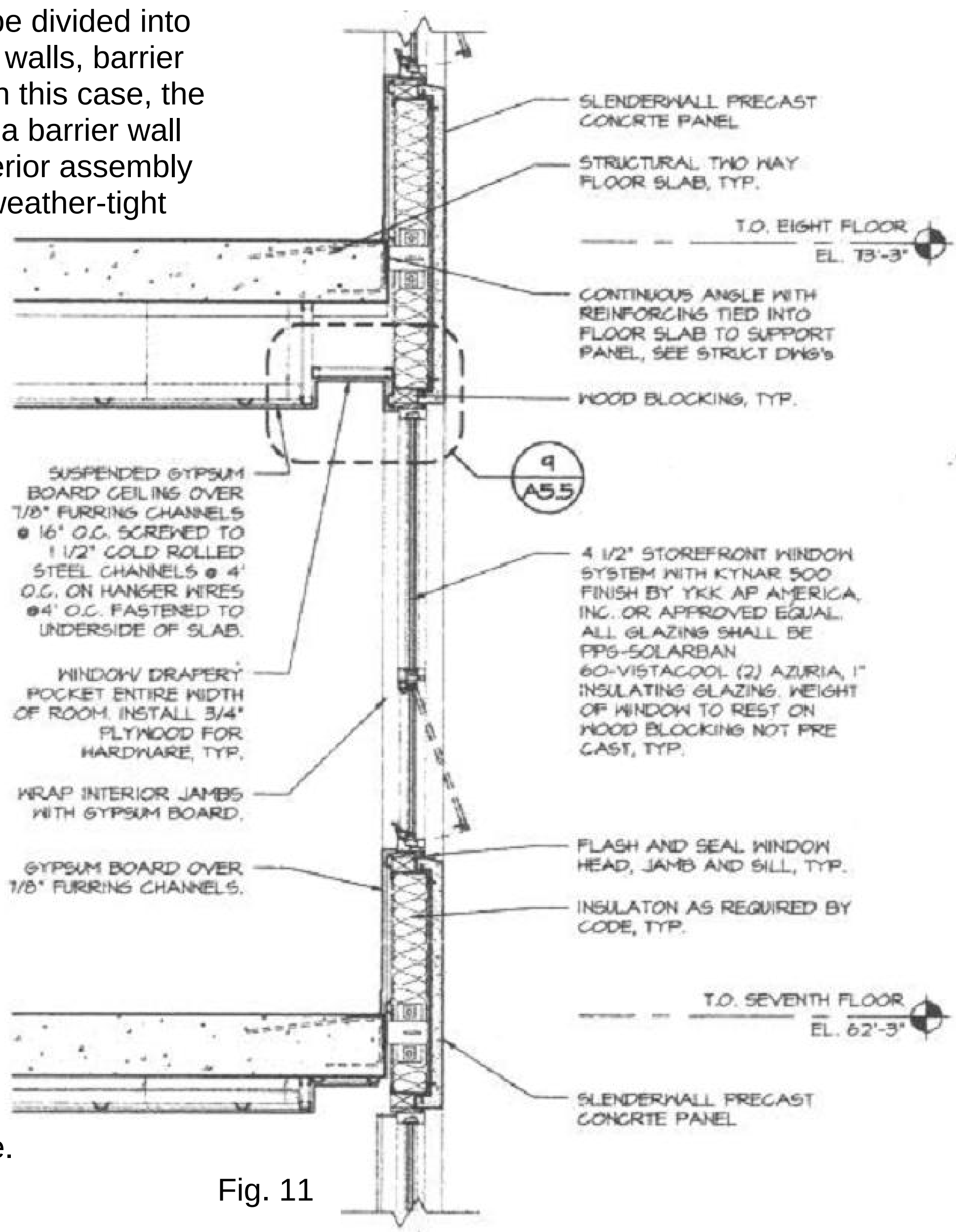

Fig. 11

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.

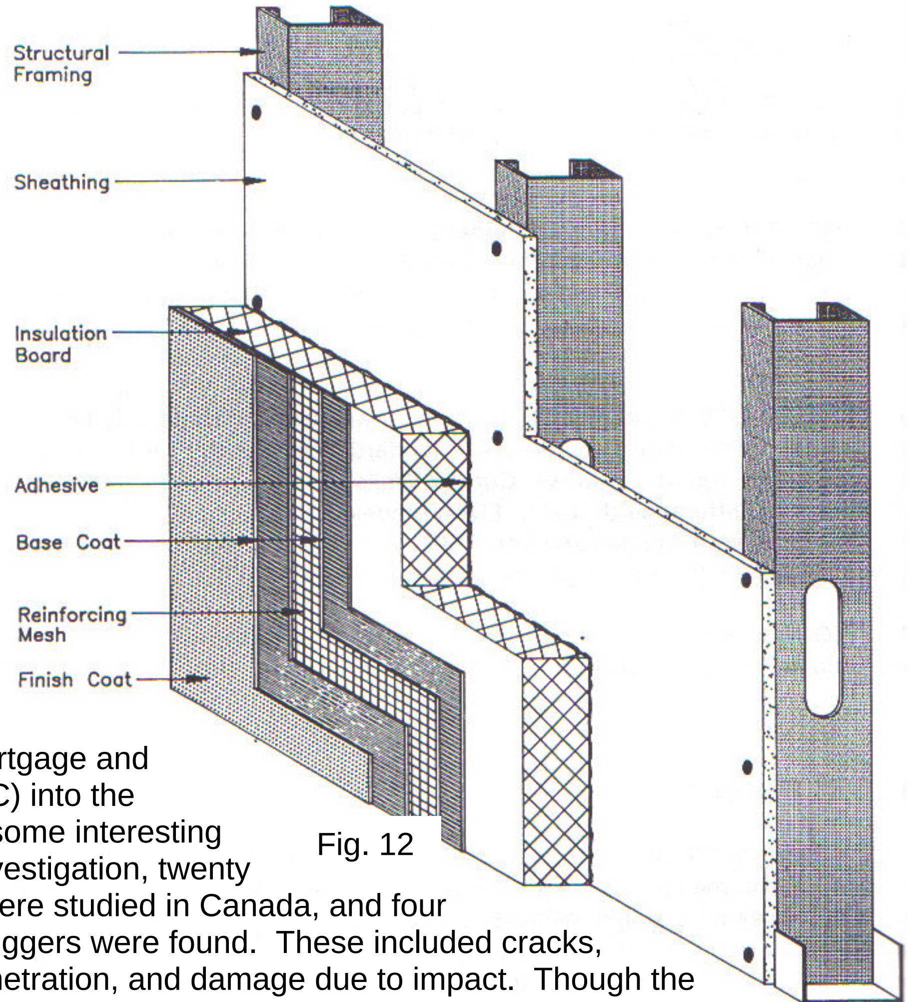

Fig. 12

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.

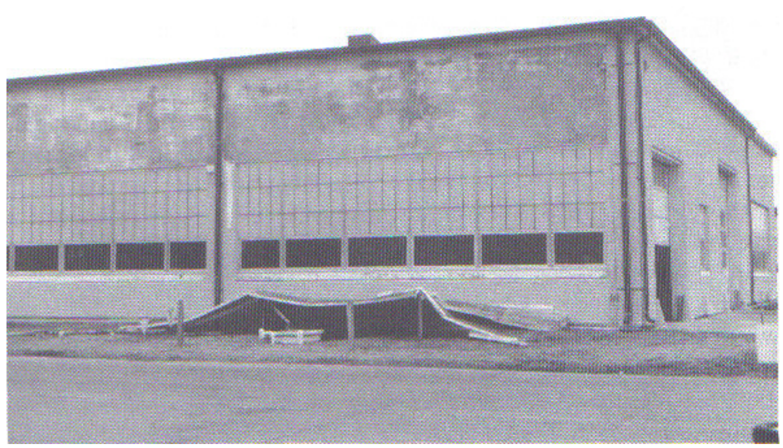

Fig. 13

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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And now, in no particular order, the informal thank you list. Consider yourselves special. Thanks to early morning homerun derby games with one of the worst pitchers to ever throw a duct tape ball, thanks to the couch in the thesis library, thanks to Lauren for keeping my head on straight, thanks to Easter dinner for holding me over until this report was finished, thanks to Microsoft Excel, thanks to The Diner for being open all night, thanks to portable mp3 players, thanks to AE for five good years, and thanks to Penn State for being such a fine institution.

Appendix A

Structural Depth Calculations

Shear Wall Calculations

Estimates on how much load a certain shear wall absorbs can be made from the principle of relative stiffness, which involves direct shear, torsion and bending. After calculating the center of rigidity and the torsional constant for this building, it became clear that the overall effect of eccentric loading on the center of stiffness had a negligible impact on the outcome of the shear calculation. In fact, calculating each shear wall using the direct shear method yielded a shear value to within 99.2% of the actual shear.

Shear wall calculation (revised)

$\boxed{\text{SW 3}}$ $\quad l = 21'$

$V = 178k$

ACI 21.7.2.2

$2(12)(21)(12)\sqrt{6000}\,/1000 = 468.5 < V_u \quad \therefore$ 2 curtains

$\rho_l, \rho_t = \dfrac{A_{sl}}{A_{cv}} \geq 0.0025$

$A_{cv} = (144\,in^2/ft)(0.0025) = 0.36\,in^2/ft$ req'd

Assume #5

$A_{sl} = 0.62\,in^2 / \$$, $\$ = $ spacing

$\dfrac{0.36}{12} = \dfrac{0.62}{\$} \Rightarrow \$ = 20.67''$ MAX

Try #5 @ 18" o.c. Both Directions

$\dfrac{h_w}{l_w} = \dfrac{130}{21} = 6.19 > 2 \quad \therefore \alpha = 2.0$

$V_u = (12)(12)(21)(2\sqrt{6000} + 0.0043(60{,}000))\,/1000 = 1249k$

$\phi V_n = 0.6(1249) = 749.4k > V_u \therefore$ ok

Distribution of Lateral Loads

Force Distribution Calculations

Element	Height	Depth	h/d	(h/d)^3	3(h/d)	ΔF	R
SW1	10	11	0.909090909	0.751314801	2.727272727	3.478587528	0.287473002
SW2	10	11	0.909090909	0.751314801	2.727272727	3.478587528	0.287473002
SW3	10	21	0.476190476	0.1079797	1.428571429	1.536551128	0.650808152
SW4	10	9	1.111111111	1.371742112	3.333333333	4.705075446	0.212536443
SW5	10	11	0.909090909	0.751314801	2.727272727	3.478587528	0.287473002
SW6	10	11	0.909090909	0.751314801	2.727272727	3.478587528	0.287473002
SW7	10	9	1.111111111	1.371742112	3.333333333	4.705075446	0.212536443
SW8	10	21.33	0.468823254	0.103045121	1.406469761	1.509514882	0.662464486
SW9	10	15.33	0.652315721	0.277570646	1.956947162	2.234517808	0.447523844
SW10	10	20	0.5	0.125	1.5	1.625	0.615384615
SW11	10	21.33	0.468823254	0.103045121	1.406469761	1.509514882	0.662464486
SW12	10	21.33	0.468823254	0.103045121	1.406469761	1.509514882	0.662464486

Center of Mass

Center of Mass Calculations

					Distance from Reference			
Element	Area	Height	Unit Weight	W	x	y	Wx	Wy
Floor	9790	0.666	0.15	978.021	64.66	18.33	63238.83786	17927.12
SW1	11	10.25	0.15	16.9125	5.5	0	93.01875	0
SW2	11	10.25	0.15	16.9125	145.16	0	2455.0185	0
SW3	21	10.25	0.15	32.2875	111.16	20.33	3589.0785	656.4049
SW4	9	10.25	0.15	13.8375	76	31.16	1051.65	431.1765
SW5	11	10.25	0.15	16.9125	29.17	48.66	493.337625	822.9623
SW6	11	10.25	0.15	16.9125	168.83	48.66	2855.337375	822.9623
SW7	9	10.25	0.15	13.8375	76	51.16	1051.65	707.9265
SW8	21.33	10.25	0.15	32.794875	0	45.66	0	1497.414
SW9	15.33	10.25	0.15	23.569875	48.665	11	1147.027967	259.2686
SW10	20	10.25	0.15	30.75	48.83	47.84	1501.5225	1471.08
SW11	21.33	10.25	0.15	32.794875	45.66	139.66	1497.413993	4580.132
SW12	21.33	10.25	0.15	32.794875	45.66	150.66	1497.413993	4940.876

Xmass	Ymass
63.95047	27.11300782

Center of Rigidity

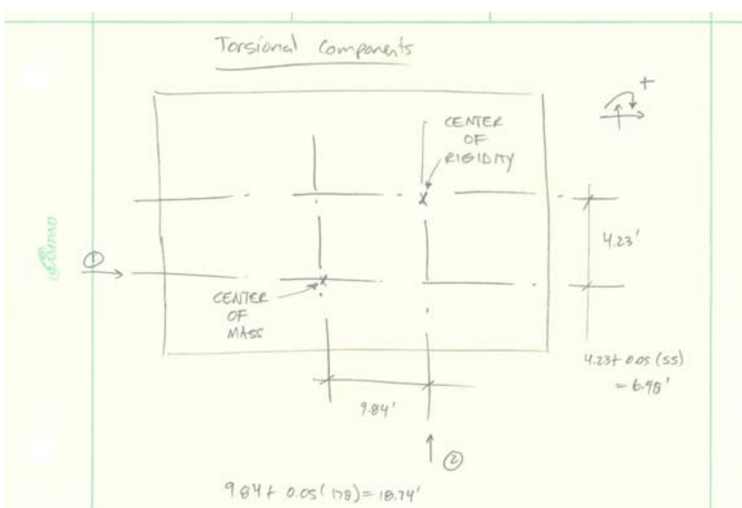
Center of Rigidity Calculations

	Distance from Reference					
Element	x	y	Rx	Ry	RxY	RyX
SW1		0	0.287473002		0	
SW2		0	0.287473002		0	
SW3		20.33	0.650808152		13.23092973	
SW4		31.16	0.212536443		6.622635569	
SW5		48.66	0.287473002		13.98843629	
SW6		48.66	0.287473002		13.98843629	
SW7		51.16	0.212536443		10.87336443	
SW8	0			0.662464486		0
SW9	11			0.447523844		4.922762289
SW10	47.83			0.615384615		29.43384615
SW11	139.66			0.662464486		92.51979008
SW12	150.66			0.662464486		99.80689942
			2.225773047	3.050301917	58.7038023	226.6832979

Xrigidity	Yrigidity
74.315036	26.37456788

Torsion Issues

Because the centers of mass and rigidity do not coincide, any wind or seismic loading will create inherent torsion on the building. The distance between the two centers is 9.84' East-West and 4.23' North-South. By taking a consistent sign convention, the results from the hand analysis closely match those found from the RAM output.



Lateral Load Distributions, Forces Parallel to Short Dimension

Controlling Shear (k): 541

Element	Ksn	Cn	Ksn	Cn	KsnCn^2	Direct Shear	Torsional Shear	Hn
SW1	0.277428033	28.83			230.5895527	0	2.313813323	-2.31381
SW2	0.277428033	28.83			230.5895527	0	2.313813323	-2.31381
SW3	0.63268393	8.51			45.8190335	0	1.557577614	-1.55758
SW4	0.204336725	2.32			1.099821987	0	0.137141137	0.137141
SW5	0.277428033	19.798			108.7409147	0	1.588930842	1.588931
SW6	0.277428033	19.798			108.7409147	0	1.588930842	1.588931
SW7	0.204336725	22.29			101.523496	0	1.31761894	1.317619
SW8			0.644080967	75.24	3646.179454	117.5532719	14.01917915	131.5725
SW9			0.433880125	64.24	1790.526782	79.1888457	8.063222091	87.25207
SW10			0.598046304	27.41	449.3170325	109.1513387	4.742170661	113.8935
SW11			0.644080967	64.44	2674.554976	117.5532719	12.00685678	105.5464
SW12			0.644080967	75.44	3665.589478	117.5532719	14.05644438	103.4968

Lateral Load Distributions, Forces Parallel to Long Direction

Controlling Shear (k): 541

Element	Ksn	Cn	Ksn	Cn	KsnCn^2	Direct Shear	Torsional Shear	Hn
SW1	0.277428033	28.83			230.5895527	69.77392639	6.212157833	75.98608
SW2	0.277428033	28.83			230.5895527	69.77392639	6.212157833	75.98608
SW3	0.63268393	8.51			45.8190335	159.1217786	4.181805801	163.3036
SW4	0.204336725	2.32			1.099821987	51.39125793	0.368198411	51.02306
SW5	0.277428033	19.798			108.7409147	69.77392639	4.265983377	65.50794
SW6	0.277428033	19.798			108.7409147	69.77392639	4.265983377	65.50794
SW7	0.204336725	22.29			101.523496	51.39125793	3.537561453	47.8537
SW8			0.644080967	75.24	3646.179454	0	37.63888499	-37.6389
SW9			0.433880125	64.24	1790.526782	0	21.64824957	-21.6482
SW10			0.598046304	27.41	449.3170325	0	12.73184501	-12.7318
SW11			0.644080967	64.44	2674.554976	0	32.23617423	32.23617
SW12			0.644080967	75.44	3665.589478	0	37.73893519	37.73894

Story Drift (┴ to long direction)

Level	Wind	Seismic
1	0.023	0.048
2	0.07	0.16
3	0.122	0.31
4	0.19	0.49
5	0.26	0.7
6	0.34	0.93
7	0.43	1.18
8	0.53	1.47
9	0.61	1.74
10	0.71	2.02
11	0.8	2.3
Roof	0.88	2.58
Penthouse	1.2	3.88

	Building Height to Low Roof
	130'-0"
	Equivalent Drift, Seismic
L/	402.0618557
	Equivalent Drift, Wind
L/	1300

Wind Calculations

Wind load calculations were performed according to ASCE 7-05 using method 2 – analytical procedure. K_{zt} was assumed to be equal to 1.0 and the building was considered enclosed when analyzing the main wind force resisting system (mwfrs) according to case 1. Through seismic calculations, the building was determined to be rigid. Linear interpolation was used where permitted.

Velocity Pressures by Floor

Level	Height	Kz	qz	
1	0	0.57	12.4032	
2	12	0.57	12.4032	
3	22.25	0.64	13.9264	
4	32.5	0.715	15.5584	
5	42.75	0.7725	16.8096	
6	53	0.8234	17.91718	
7	63.25	0.8634	18.78758	
8	74.25	0.908	19.75808	
9	84.5	0.943	20.51968	
10	94.75	0.975	21.216	
11	105	1.0025	21.8144	
Low Roof	115.25	1.0275	22.3584	
High Roof	130	1.065	23.1744	qh
Parapet	132	1.07	23.2832	

Design Pressure		⊥55'			⊥178'		
Level	Height	p w-w	p l-w	p roof	p w-w	p l-w	p roof
1	0	8.371749	-4.51661	-18.96587	8.147242	-9.51405	-19.7892
2	12	8.371749	-4.51661	-15.64195	8.147242	-9.51405	
3	22.25	9.399858	-4.51661		9.147781	-9.51405	
4	32.5	10.5014	-4.51661		10.21979	-9.51405	
5	42.75	11.34592	-4.51661		11.04166	-9.51405	
6	53	12.09351	-4.51661		11.76919	-9.51405	
7	63.25	12.681	-4.51661		12.34093	-9.51405	
8	74.25	13.33605	-4.51661		12.97841	-9.51405	
9	84.5	13.8501	-4.51661		13.47868	-9.51405	
10	94.75	14.3201	-4.51661		13.93607	-9.51405	
11	105	14.724	-4.51661		14.32914	-9.51405	
Low Roof	115.25	15.09118	-4.51661		14.68648	-9.51405	
High Roof	130	15.64195	-4.51661		15.22248	-9.51405	
Parapet	132	34.9248	-23.2832		34.9248	-23.2832	

Story Shear	⊥55'	⊥178'
Level		
1	8.506319	37.72452
2	7.265814	32.22303
3	7.845411	34.04851
4	8.466408	36.00438
5	8.942505	37.50389
6	9.363954	38.83127
7	10.40455	42.79204
8	10.06444	41.0375
9	10.35424	41.95024
10	10.6192	42.78475
11	10.84689	43.5019
Low Roof	15.90682	63.53848
Total	118.5866	491.9405

Overturning	⊥55'	⊥178'
Level		
1	51.03792	226.3471
2	124.4271	551.8193
3	214.7681	932.0779
4	318.5486	1354.665
5	428.1224	1795.499
6	544.2799	2257.068
7	715.3131	2941.953
8	798.8648	3257.351
9	927.9985	3759.79
10	1060.592	4273.127
11	1194.514	4790.647
Low Roof	1950.574	7791.406
Total	8329.041	33931.75

Seismic Calculations

As the vertical distribution of forces shows, seismic analysis was the controlling factor in both directions. That is, the seismic base shear, which is the same in both directions, was larger than either direction of wind base shear. This result is not surprising, as the seismic response is based on the building weight. Concrete buildings tend to carry more mass per story, and consequently are often controlled by seismic design criteria.

The overturning moment also turned out to be larger for seismic than wind. This can be attributed to larger forces being present at higher elevations for the seismic design. The vertical distribution of forces equation attempts to take a whiplash effect into account. As the base of the building moves one way, the top wants to catch up to it. As it does this, the base of the building switches directions and moves back, thus pulling the top of the building back to its original position with much greater force.

Once the seismic and wind forces are determined, the analysis of the lateral elements of the building can begin. Because the seismic load controls, the shear walls will be analyzed according to their relative stiffness within the group using seismic load.

Seismic Inputs	
Variable	Value
S_s	0.152
S_1	0.5
F_a	1.6
F_v	2.4
I	1
SM_s	0.2432
SM_1	1.2
SD_s	0.16213333
SD_1	0.8
R	5
C_s	0.03243
C_t	0.02
h_n	130
x	0.75
T_a	0.7699943
T_o	0.98684211
T_s	4.93421053
V (k)	530.594332

Total Weight by Floor		
Floor	Total Weight	Elevation
1	0	0
2	1276136	12
3	1566041	22.25
4	1566041	32.5
5	1566041	42.75
6	1566041	53
7	1566041	63.25
8	1566041	74.25
9	1566041	84.5
10	1566041	94.75
11	1566041	105
Low Roof	924500	115.25
High Roof	66214	130

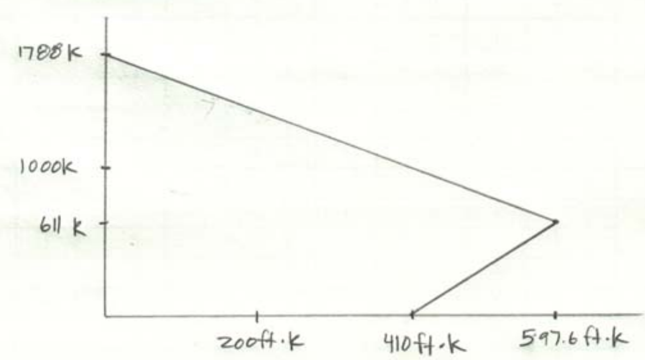
Weight Seen by Floor		
Floor	Weight	Story Shear
High Roof	66214	2.14732002
Low Roof	990714	32.128855
11	2556755	82.9155647
10	4122796	133.702274
9	5688837	184.488984
8	7254878	235.275694
7	8820919	286.062403
6	10386960	336.849113
5	11953001	387.635822
4	13519042	438.422532
3	15085083	489.209242
2	16361219	530.594332
1	16361219	530.594332

Vertical Distribution of Forces		
Floor	C_{vx}	F_x (k)
High Roof	0.00906249	4.80850739
Low Roof	0.11036758	58.5604134
11	0.16819967	89.2457906
10	0.14968997	79.4246482
9	0.13144911	69.7461519
8	0.1135052	60.225216
7	0.09461914	50.2043777
6	0.07741558	41.0762699
5	0.06065796	32.1847679
4	0.04443886	23.5790064
3	0.02890646	15.3376016
2	0.01168799	6.20158118
1	0	0
	1	530.594332

Base Moment	
First Floor	42546.1255

Column Calculations

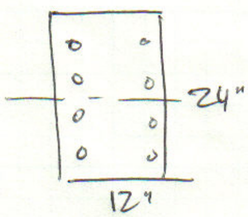
Column D-3 is a 12”x24” column with eight number 9 vertical reinforcing bars, 4 in each face. Assuming a cover of 1-1/2” all around, I found the pure axial capacity of the column to be 1788k. Similarly, the pure bending capacity of the column, about an axis perpendicular to the 24” side, was found to be 410 ft-k. The balanced strain condition is the last point needed to make a preliminary column interaction diagram. After calculating the balanced condition, which yielded 611k of compression and 597.6 ft-k of bending capacity, the diagram looked like this:



COLUMN INTERACTION DIAGRAM
FOR COLUMN D-3

If the actual point lies somewhere inside this conservative area, the column is deemed adequate.

Column Interaction



$f'_c = 6$ ksi
$f_y = 60$ ksi
8-#9 bars

Pure axial $P_o = (0.85)(6)(12 \times 24 - 8.0) + (8)(60)$
$= 1788$k

Balanced Condition

$\varepsilon_y = 60/29{,}000 = 0.00207$

$c = \frac{.003}{.003 + .00207}(22.5) = 11.84"$

$\varepsilon_{s1} = \frac{.003}{11.84}(11.84 - 1.5) = 0.00262 \quad f_{s1} = 60$ ksi

$\varepsilon_{s2} = \frac{.003}{11.84}(11.84 - 8.5) = 24.5$ ksi

$\varepsilon_{s3} = \frac{.003}{11.84}(11.84 - 15.5) = -26.9$ ksi

$\varepsilon_{s4} = -60$ ksi

$M_b = (.85)(6)(12)(.85)(11.84)(12 - \frac{(.85)(11.84)}{2})$
$+ 2(60)(12 - 1.5) + 2(24.3)(12 - 8.5)$
$+ 2(-26.9)(12 - 15.5) + 2(-60)(12 - 22.5) = 597.6$ ft·k

$P_b = (.85)(6)(12)(.85)(11.84) + 2(60) + 2(24.5) + 2(-26.9)$
$+ 2(-60) = 611$ k

pure bending   assume 2 dont yield, 2 do

$$f_{s1} = \frac{.003}{c}(c-1.5)(29k) =$$

$$f_{s2} = \frac{.003}{c}(c-8.5)(29k)$$

$$f_{s3} = f_{s4} = -60$$

$$\Sigma F = 0 \quad (.85)(6)(12)(.85)c + 2f_{s1} + 2f_{s2} + 2f_{s3} + 2f_{s4}$$

$$0 = 52c + \frac{174}{c}(c-1.5) + \frac{174}{c}(c-1.5) - 4(60)$$

$$0 = 52c^2 + \frac{348}{\cancel{c}}(c-1.5) - 240c$$

$$-52c^2 + 240c = .348c - 522$$

$$-52c^2 - 108c + 522 = 0$$

$$\frac{108 \pm \sqrt{108^2 - 4(-52)(522)}}{2(-52)}$$

$$52c - 240 + \frac{.003}{c}(c-1.5)(29000)(4) = 0$$

$$52c^2 - 240c + \frac{348}{\cancel{c}}(c-1.5) = 0$$

$$52c^2 - 240c + 348c - 522 = 0$$

$$52c^2 + 108c - 522 = 0$$

$$\frac{-108 \pm \sqrt{108^2 - 4(52)(-522)}}{2(52)} = \frac{-108 \pm 347}{104}$$

$c = 2.3''$

$fs_1 = \frac{.003}{2.3}(2.3-1.5)(29000) = 30.3k$

$fs_2 = \frac{.003}{2.3}(2.3-8.5) \quad > -60\ ksi \quad \therefore$ yielded

$.85(6)(12)(.85)c + \frac{.003}{c}(c-1.5)(29000)(2) \; 360$

$52c - 360 + \frac{.003}{c}(c-1.5)(29000)(2)$

$52c - 360 + \frac{174}{c}(c-1.5) = 0$

$52c^2 - 360c + 174c - 261 = 0$

$$\frac{186 \pm \sqrt{186^2 - 4(52)(-261)}}{104}$$

$c = 4.655''$

$fs_1 = 58.9k$

$fs_2 = \frac{.003}{4.655}(4.655-8.5) = -60$

$fs_3 = fs_4 = -60$

$M_o = .85(6)(12)(.85)(4.655)\left(12 - \frac{.85(4.655)}{2}\right) \quad 2427$

$+ 2(58.9)(12-1.5) + 2(-60)(12-8.5)$

$+ 2(-60)(12-15.5)$

$+ 2(-60)(12-22.5) = 4924$

$= 410\ ft{\cdot}k$

Post Tensioning Analysis

SPAN LENGTH = 29'-0

TRIB WIDTH = 17.5'

$\frac{29}{45} = 0.64' \Rightarrow$ 8" PT SLAB

LOADING

(3.5')(100) = 350

(14')(40) = 560

(29)(20) = 580

1490 lb/ft²

$\frac{1490}{17.5} = 85.14$ lb/ft²

LOADS

100 DEAD

85.14 LIVE

8 MEP

194 TOTAL

(194)(17.5) = 3395 plf

BALANCE 90% DEAD ⇒ 90 psf

USE ½" ϕ 270k TENDONS, ASSUMING 30k LOSSES

$f_e = (0.153)(0.7)(270-30) = 25.7$ k PER TENDON

$F_e = (0.09)(8)(12) = 8.64$ k/ft

FOR A 29'-0 SPAN, $\frac{(29)(8.64)}{25.7} = 9.75$, SAY 10 TENDONS

$\rightarrow F_e = \frac{10(25.7)}{17.5} = 14.7$ k

$\sigma = \frac{F}{A} = \frac{14.7}{(12)(8)} = 0.153$ ksi

NET LOAD

194 - 90 = 104 psf

$$\frac{P}{A} = \frac{18(25.7)(1000)}{8(284)} = 204 \text{ psi}$$

$$\frac{P}{A} = \frac{9(25.7)(1000)}{8(11)(12)} = 219 \text{ psi}$$

$$\frac{P}{A} = \frac{(\overset{18}{\cancel{24}})(25.78)(1000)}{8(12)(23.34)} = 206 \text{ psi}$$

$$A = (122)(8) = 976 \text{ in}^2$$

$$S = \frac{I}{c} = \frac{I}{4} = \frac{\frac{bh^3}{12}}{4} = \frac{\frac{(122)(8)^3}{12}}{4} = 1301 \text{ in}^3$$

$$w_b = -1.366 \text{ k/ft}$$

@ JACKING

$f'_{ci} = 3000$ psi

$0.6 f'_{ci} = 1800$ psi

$3\sqrt{f'_{ci}} = 164$ psi

@ SERVICE

$f'_c = 5000$ psi

$0.45 f'_c = 2250$ psi

$6\sqrt{f'_c} = 424$ psi

$w_b = 0.95$ k/ft

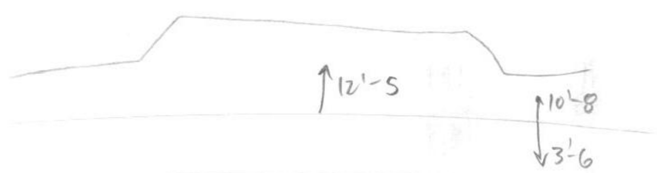
$\frac{P}{A} = \frac{15(25.7)}{13'\text{-}8\ (8)} = 293.8$ psi $\qquad \frac{15(25.7)}{15'\text{-}11(8)} = 252$ psi

12

$\frac{P}{A} = \frac{12(25.7)(1000)}{1312} = 235$ psi $\qquad \frac{12(25.7)(1000)}{1528} = 202$ psi

$\frac{9(25.7)(1000)}{976} = 237$ psi $\qquad \frac{9(25.7)(1000)}{12'\text{-}5\ (8)} = 194$ psi

2366
2,366 k/ft

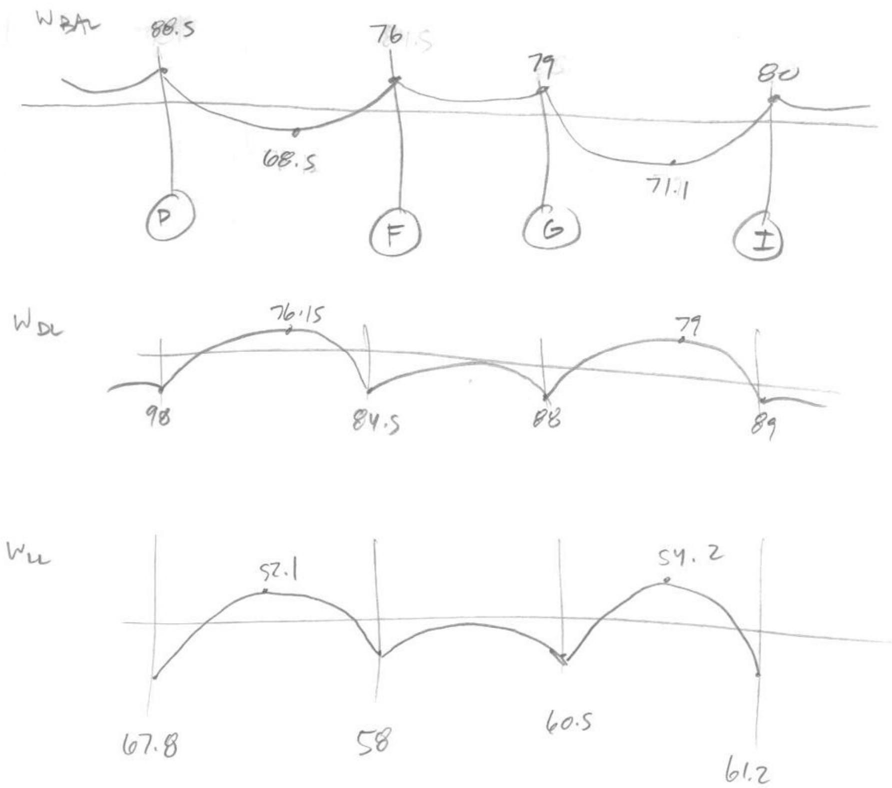


23'-8

24 TENDONS

$\frac{P}{A} = \frac{24(25.7)}{8(284)} = 271$ psi

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S
1	w DL-->	1.366		1.366		1.479		1.592		1.592		1.592		1.479		1.366		1.366	
2		AB	BA	BC	CB	CD	DC	DF	FD	FG	GF	GI	IG	IJ	JI	JL	LJ	LM	ML
3	K	0.127	0.127	0.273	0.273	0.222	0.222	0.103	0.103	0.15	0.15	0.103	0.103	0.222	0.222	0.127	0.127	0.273	0.273
4	DF	1	0.3175	0.6825	0.551515	0.448485	0.683077	0.316923	0.407115	0.592885	0.592885	0.407115	0.316923	0.683077	0.636103	0.363897	0.3175	0.6825	1
5	COF	1	0	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0	1
6	FEM	-63.7557	63.75572	-13.7738	13.77383	-22.4623	22.46231	-111.573	111.5727	-53.0667	53.06667	-111.573	111.5727	-22.4623	22.46231	-63.7557	63.75572	-13.7738	13.77383
7																			
8	DIST	63.75572																	-13.7738
9	CO	0	31.87786															-6.88692	0
10	SUM	0	95.63358	-13.7738	13.77383	-22.4623	22.46231	-111.573	111.5727	-53.0667	53.06667	-111.573	111.5727	-22.4623	22.46231	-63.7557	63.75572	-20.6608	0
11																			
12	DIST		-25.9905	-55.8693	4.791828	3.896651	60.86923	28.24113	-23.8186	-34.6874	34.68735	23.81865	-28.2411	-60.8692	26.26687	15.02654	-13.6827	-29.4123	
13	CO			2.395914	-27.9346	30.43461	1.948326	-11.9093	14.12056	17.34368	-17.3437	-14.1206	11.90932	13.13343	-30.4346	-6.84133	7.513271		
14	SUM		69.64311	-67.2472	-9.36898	11.86895	85.27986	-95.2409	101.8746	-70.4103	70.41034	-101.875	95.24086	-70.1981	18.29457	-55.5705	57.58634	-50.0731	
15																			
16	DIST		-0.7607	-1.63521	-1.37877	-1.1212	6.804128	3.15687	-12.8096	-18.6547	18.65469	12.80955	-7.93663	-17.1061	23.71134	13.5646	-2.38546	-5.12781	
17	CO			-0.68939	-0.81761	3.402064	-0.5606	-6.40478	1.578435	9.327344	-9.32734	-3.96831	6.404776	11.85567	-8.55307	-1.19273	6.782299		
18	SUM		68.88241	-69.5718	-11.5654	14.14982	91.52339	-98.4888	90.64347	-79.7377	79.73769	-93.0333	93.70901	-75.4486	33.45285	-43.1986	61.98317	-55.2009	
19																			
20	DIST		0.21888	0.470506	-1.42537	-1.15909	4.757888	2.207488	-4.4399	-6.46588	7.882801	5.412857	-5.78716	-12.4733	6.199332	3.546465	-2.15338	-4.62892	
21	CO			-0.71268	0.235253	2.378944	-0.57955	-2.21995	1.103744	3.941401	-3.23294	-2.89358	2.706428	3.099666	-6.23665	-1.07669	1.773232		
22	SUM		69.10129	-69.814	-12.7555	15.36967	95.70174	-98.5012	87.30731	-82.2622	84.38755	-90.5141	90.62828	-84.8222	33.41553	-40.7289	61.60303	-59.8298	
23																			
24	DIST		0.226277	0.486407	-1.44177	-1.17243	1.912271	0.887225	-2.05395	-2.99119	3.632322	2.494195	-1.84009	-3.96601	4.652035	2.6613	-0.563	-1.21023	
25	CO			-0.72088	0.243203	0.956136	-0.58621	-1.02698	0.443612	1.816161	-1.4956	-0.92004	1.247097	2.326018	-1.983	-0.2815	1.33065		
26	SUM		69.32757	-70.0485	-13.954	15.15338	97.02779	-98.641	85.69697	-83.4372	86.52428	-88.9399	90.03529	-86.4622	36.08456	-38.3491	62.37068	-61.04	
27																			
28	DIST		0.228881	0.492004	-0.66145	-0.53789	1.101933	0.511257	-0.91999	-1.33979	1.432197	0.983442	-1.1324	-2.44071	1.440459	0.824046	-0.42248	-0.90817	
29	CO			-0.33073	0.246002	0.550966	-0.26894	-0.45999	0.255629	0.716098	-0.66989	-0.5662	0.491721	0.720229	-1.22036	-0.21124	0.412023		
30	SUM		69.55645	-69.8872	-14.3695	15.16646	97.86078	-98.5897	85.03261	-84.0609	87.28658	-88.5227	89.39461	-88.1827	36.30467	-37.7363	62.36022	-61.9482	
31																			
32	DIST		0.105006	0.225721	-0.43954	-0.35743	0.497919	0.231017	-0.3956	-0.57612	0.732862	0.503232	-0.3841	-0.82786	0.910643	0.520954	-0.13082	-0.28121	
33	CO			-0.21977	0.112861	0.24896	-0.17871	-0.1978	0.115508	0.366431	-0.28806	-0.19205	0.251616	0.455322	-0.41393	-0.06541	0.260477		
34	SUM		69.66145	-69.8812	-14.6962	15.05799	98.17999	-98.5565	84.75251	-84.2706	87.73138	-88.2115	89.26213	-88.5552	36.80138	-37.2807	62.48988	-62.2294	
35																			
36	DIST		0.069777	0.149993	-0.19955	-0.16227	0.25719	0.119327	-0.1962	-0.28573	0.28465	0.195459	-0.22404	-0.48289	0.304907	0.174429	-0.0827	-0.17778	
37	CO			-0.09977	0.074997	0.128595	-0.08114	-0.0981	0.059663	0.142325	-0.14287	-0.11202	0.09773	0.152454	-0.24145	-0.04135	0.087214		
38	SUM		69.73123	-69.831	-14.8207	15.02431	98.35604	-98.5353	84.61597	-84.414	87.87316	-88.1281	89.13582	-88.8856	36.86484	-37.1476	62.49439	-62.4072	
39																			
40	DIST		0.031678	0.068096	-0.11228	-0.09131	0.122433	0.056805	-0.08223	-0.11976	0.15112	0.103769	-0.07929	-0.17089	0.179888	0.102909	-0.02769	-0.05952	
41	CO			-0.05614	0.034048	0.061217	-0.04565	-0.04112	0.028402	0.07556	-0.05988	-0.03964	0.051885	0.089944	-0.08545	-0.01385	0.051454		
42	SUM		69.76291	-69.8191	-14.899	14.99422	98.43282	-98.5196	84.56214	-84.4582	87.9644	-88.0639	89.10841	-88.9666	36.95928	-37.0586	62.51815	-62.4667	
43		0.0	69.76	-69.82	-14.90	14.99	98.43	-98.52	84.56	-84.46	87.96	-88.06	89.11	-88.97	36.96	-37.06	62.52	-62.47	0.0



AFTER JACKING

INTERIOR SPAN

MIDSPAN STRESS

$$f_t = \frac{(79 - 71.1)(12)(1000)}{1307} - 219 = -146 \text{ psi}$$

$$f_b = \frac{(-79 + 71.1)(12)(1000)}{1301} - 219 = -292 \text{ psi}$$

SUPPORT STRESS

$$\frac{(88.5 - 98)(12)(1000)}{1307} - 219 = -307 \text{ psi}$$

$$\frac{(-88.5 + 84.5)(12)(1000)}{1301} - 219 = -256 \text{ psi}$$

ULTIMATE STRENGTH

$M = P \cdot e$
$= (231k)(8-4)/12 = 77 \text{ ft}\cdot\text{k}$

$M_{sec} = M_{BAL} - M_1 = 88.5 - 77 = 11.5 \text{ ft}\cdot\text{k}$

@ MIDSPAN

$M = 1.2(79) + 1.6(54.2) + \frac{11.5}{2} = 187.3 \text{ ft}\cdot\text{k}$

@ SUPPORT

$M = 1.2(-98) + 1.6(-68) + \frac{11.5}{2} = -220.6 \text{ ft}\cdot\text{k}$

Punching Shear Calculations

$S_S = 0.1769$

$S_1 = 0.0629$

$S_{DS} = 0.1887$

$S_{D1} = 0.1006$

SDC B

$C_S = 0.0377$?

3'-6"
6'-9" 14'-6"
12'-5"

$A = 338.23 ft^2$

PUNCHING SHEAR CHECK

$V_c = 4\sqrt{5000}\,(6.5)(98) = 180k$

$V_c = \left(\frac{30(6.5)}{98} + 2\right)\sqrt{5000}\,(98)(6.5) = 179.7k$

$0.75(179.7) = 134.78k$

$V_u = 1.2D + 1.6L$

$1.2(100)(339) + 1.6(100)(75) + (1.6)(60)(264)$

$= 78\,k < 134.78k$ ∴ ok FOR PUNCHING SHEAR

EXTERIOR ~~SLAB~~ COLUMN

12'-5"
6'-9" 14'-6

$A = 264 ft^2$

$V_c = 4\sqrt{5000}\,(79.5)(6.5) = 146k$

$\phi V_c = 109.6k$

$V_u = 1.2(100)(264) + (1.6)(60)(264) = 57\,k$

$$V_c = \left(\frac{(30)(6.5)}{49} + 2\right)\sqrt{5000}\,(6.5)(49) = 163k\,(0.75) = 122k$$

$$\phi V_c = 0.75(2)\sqrt{5000}\,(49)(6.5) = 31.2k$$

$$\frac{57k - 31.2k}{0.75k} = 34.4k < 122k$$

$$A_v = \frac{34{,}400}{60{,}000(0.707)} = 0.81\,in^2$$

$$\frac{0.81}{4} = 0.2\,in^2 \Rightarrow (4)\text{-}\#5$$

Appendix B

Mechanical Breadth Calculations

EIFS References

resistant mesh in one instance, the spacing of both warp and weft was 4.25 mm in all samples. The mesh from two samples appeared to have no polymer coating. The remaining samples were coated with yellow, blue, white or clear polymer coatings.

In general the samples were reported as being of good quality, conforming to the EIMA Guideline Specifications [3], with a few exceptions.

Table 2--Individual test results

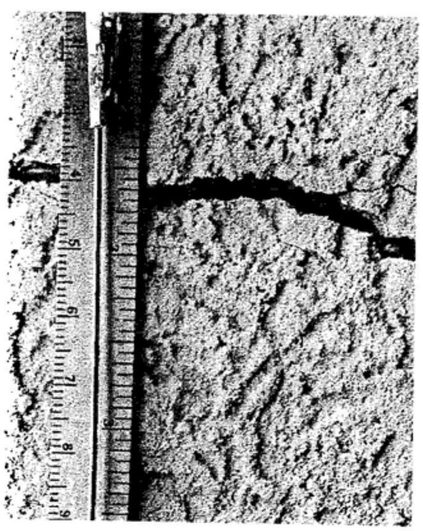
Bldg. No.	Lamina weight, kg/m²	Polymer content, %	Water absorption, %	Base coat(s), mm	Finish coat(s), mm	Mesh weight, kg/m²
1	3.1	10.2	7.2	0.5 - 0.75	0.1 - 2.0	0.12
4A	7.5	10.4	6.0	1.0 - 1.9	1.5 - 4.0	0.13
4C	4.5	10.6	8.2	1.0 - 1.2	0.2 - 2.5	0.14
5	4.0	10.9	8.9	0.5 - 2.0	0.5 - 2.0	0.13
10A	4.0	9.5	10.1	0.1 - 0.2 plus 1.2 - 1.5	0.2 - 1.0	0.13
10B	5.2	12.3	6.1	0.75 - 1.0 plus 0.75 - 1.0	0.5 - 0.75	0.13
11	6.9	12.4	2.7	2.0 - 2.25	1.0 - 1.5	0.11 x 2
12	5.7	14.0	5.7	1.1 - 1.3 plus 0.5 - 0.6 plus 0.1 - 0.3	0.1 - 1.5	0.11
15	6.0	12.6	7.9	2.5 - 3.2	0.1 - 1.5	0.14 plus 0.59
22	2.7	14.6	8.8	0.5 - 0.75	0.1 - 0.75	0.12

FIELD OBSERVATIONS

The finish was in excellent condition in many cases, including the oldest installation (13 years), on a high-rise cast-in-place concrete wall. More than half of the installations were in good to excellent overall condition, although none were entirely free of defect. Approximately 30% had visible problems serious enough to threaten serviceability. Ingress of moisture into the system and impact damage were the most common causes of damage serious enough to demand repair or replacement.

Problems observed included: failed joints, cracking, impact damage, excessively thin applications, softening, erosion of the finish, delamination of the finish coat, delamination from the insulation, poor attachment of insulation to the building, fading, freezing prior to cure during construction, color variation dating from installation, color variation due to fading, cracking at locations of movement in underlying supports, unsatisfactory repairs, algae and moss growth on the surface, water saturated insulation, damage from interior water sources, and complete detachment of the system from the building.

The common problems were cracks, particularly at reentrant corners, failed joints where sealant had been used, deterioration due to moisture, and damage due to impact. A subjective view of the frequency and severity of these problems is tabulated (Table 3). "..." indicates negligible occurrence. More severe problems are rated as "■", "■■", or "■■■", in ascending order of severity. The "■■■" designation is reserved for instances which threatened the overall appearance of the building, the serviceability of the cladding (unless promptly repaired), or where failure necessitating replacement had already occurred. Figures 1 to 7 give some indication of the damage observed. On Building 4C one third of the windows had one or more cracks 0.1 to 1.0 metres long extending across the building face from window opening corners. Similar, but more extensive, cracking was observed on Building 10, where cracks extended from one window to the next in several locations, from grade to parapet. See Figure 1. On Building 5, impact damage was evident on every elevation, with each typical 10 metres of building perimeter bearing 50 to 100 impacts ranging from fist-sized dents to patches of bare concrete like that shown in Figure 2. Figure 2 also shows more typical damage, adjacent to a garbage container on Building 1, from a falling object on an upper floor of Building 15, and by snow sliding from a roof on Building 4A. On Building 11 the lamina was cracked at many of the joints in the insulation. In addition, in some areas, it was cracked at every strand in the glass mesh. See Figure 3. The insulation was saturated with water in several locations. On Building 7, sealant in building expansion joints had failed for the full building height (as in Figure 3) and one failed sealant joint had caused interior water damage. Figure 3 also shows moisture damage to finish coat on Building 4A. On Building 21, moisture damage to gypsum sheathing had resulted in loss of a section of EIFS cladding 3 stories high and approximately 15 metres wide.



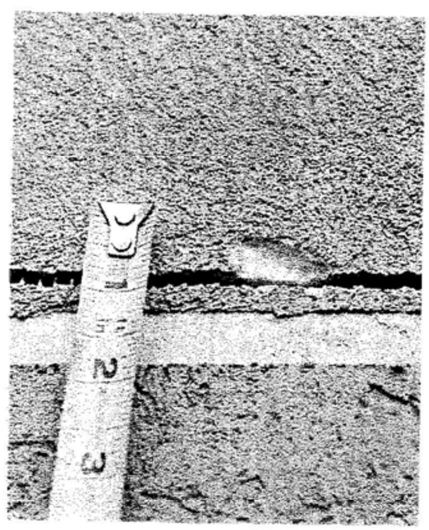


Fig. 1--*Window corner cracks: Building 4C at left, Building 10 at right. Joint-like feature on Building 10 is deliberately exposed white finish under contrasting top layer.*

Table 3--Observed problems

Building	Cracks	Joints	Moisture	Impact
1--High Rise Residential	■■	...	■	■
2--Recreation Centre	■	...	...	■■
3--Shopping Centre	■■	■■	■	■
4A--Hotel	■■	■	■■■	■
4B--Hotel	■	...	...	■
4C--Hotel	■■	■■■	...	■
5--Recreation Centre	■	■	■	■■■
6--High Rise Residential	■	■	...	■
7--Hotel	■	■■■	■	■
8--High Rise Residential	■■	■	■■■	■
9--Hotel	■■	■	■	■
10--Office	■■■	■■	■■	■
11--High Rise Residential	■■■	■■	■■■	■
12--High Rise Residential	■	■	■■	...
13--High Rise Residential	...	■	■	...
14--Hotel	■	...	...	■
15--Hotel	■	■■	■	■
16--Low Rise Residential	■■	■■	■	■
17--Restaurant	...	...	...	■
18--Low Rise Residential	■■	...	■	■
19--Retail/Residential	■■	■	■	■
20--Retail/Office	■	■	...	■
21--Shopping Centre	■■■	■	■■■	■
22--Retail/Office	■	■	■	■
23--Office	...	■	...	...

RECOMMENDATIONS

The following points summarize recommendations for application of EIFS to future projects, based on the failures observed:

- Mandate architectural technical input. Provide shop drawings indicating joint details, joint spacing and junctures. Have inspections completed by qualified and experienced third parties on all significant projects.
- Don't use EIFS in high impact areas or on stud framing with gypsum sheathing, if a physically secure wall is required. Use more impact resistant material such as concrete, or masonry, particularly at loading docks, parking spaces, and roadways.
- Use windows which can be cleaned from the interior, except where they are accessible from the ground or balconies without ladders or staging.
- Design the cladding, and all attachments, to withstand the full wind load.
- Use gypsum-based boards as substrata only when air sealed and waterproofed with a membrane on the outside. Otherwise, consider a more moisture resistant sheathing containing no gypsum; exterior and water resistant gypsum boards are not sufficient.
- Insulation in the stud spaces of frame walls supporting EIFS should be used only after thermal profiles have been considered for the winter design condition to ensure that the dew point will always fall in the EIFS insulation layer.
- Dark colors fade and deteriorate more rapidly, and subject the surface to more extreme temperatures, both in sunlight and on clear cold nights.
- Locate soft joints in line with corners of openings, to eliminate opportunities for cracking.
- Use two stage (rain screen) joints, like the typical drained joint described by Schaefer & McKechnie.[5] Use something other than sealant for the outer seal, where possible. Don't install finish coat on sides of joints where sealant adhesion is required.
- If sealant is used, use low modulus sealant with controlled minimum cross sections. Use a brand of sealant known to be compatible with the particular brand of EIFS. Take greater care than for sealant installations in other materials.
- Use metal expansion joints attached to the underlying structures to avoid bridging gaps between structurally independent EIFS-clad building segments with sealant, and terminate the EIFS at junctures where no movement will occur.
- Provide mounting for signs and other fixtures, independent of the EIFS.
- Secure EIFS to rigid substrata such as concrete or masonry where possible. If flexible supports are used, evaluate potential movement at connections and junctures between different parts.
- Don't use EIFS cladding as a window sill or roof parapet flashing.
- Ensure that drainage from other surfaces does not flow over the EIFS finish, and that icicles will not form on it.
- Do not use EIFS where snow will be in contact with the finish for extended periods of time.
- On edges of soffits and undersides of EIFS projections, provide drips (grooves or other breaks), to cause surface water to fall free, rather than wetting and staining under surfaces.
- Cure EIFS materials a minimum of 24 hours at temperatures above 5 °C.
- Promptly repair damage to EIFS to prevent ingress of moisture.

were the prime reasons for these failures. (The cracking and delamination problems associated with these failures are further described later in this paper. Additional details are presented in a USACERL Technical Report [6].)

INVESTIGATION OF INSTALLED SYSTEMS

As a result of the previously mentioned failures at the two different installations, concerns were raised regarding the quality of EIFS as a wall cladding system. Funding was obtained to investigate the performance of EIFS that had been applied to facilities at other Army installations. Close to a hundred different buildings at several different Army and Air Force installations were examined. The observed successes and problems are described below.

EIFS Successes

Given the number of EIFS applications versus the few major problems reported to date, the use of EIFS on Army and Air Force facilities currently can be considered successful. The field assessments showed that EIFS had been effectively used to upgrade both the appearance and energy efficiencies of most of the buildings on which the system was applied. Only one location was observed where catastrophic system failures had occurred (Figure 3). These system delaminations were determined to be a result of poor workmanship and not an inherent problem of the system.

Observed Problem Areas

Even though only one catastrophic system failure was noted, system problems or deficiencies were observed at every location. The most common problems observed were cracking of the lamina and impact damage. Examples of poor workmanship were common. Problems were also seen as a result of poor design choices; such as, use of EIFS on a loading dock.

Cracking

Cracking of the lamina was observed at virtually every location. System cracking ranged from minor hairline cracks to cracks up to 9.5 mm (3/8 in.) wide. Most of the cracking occurred at corners and window and door penetrations. The majority of the cracking observed was a result of installers not properly abutting the insulation boards as can be seen in Figure 4. Gaps of up to 12.5 mm (½ in.) wide were found. In a few cases, cracks originated from the corners of windows and doors were a result of missing diagonal strips of mesh at the corners. Either the mesh strips were omitted during installation or the system was installed prior to when this procedure became a manufacturer recommended practice.

Impact Damage

Damage to the system due to mechanical impact must be considered a major problem. Evidence of impact damage was common. Impact damage was seen on both Class PB and Class PM systems. Although some of the damage was due to unintentional impact, intentional acts of vandalism (e.g., throwing rocks or kicking with boots) were the most common (Figure 5). Having an EIFS that is resistant to expected in-service impact exposures is considered to be a very important system quality. A lamina damaged due to mechanical impact provides a potential path for water to enter the system. At minimum, the infiltrating water will degrade the insulation properties of the system. At worst, catastrophic damage to the substrate could result. A study conducted by USACERL [2], showed a tremendous range of values for impact resistance between various EIFS. Several different test methods were used to study these differences. A falling weight test method (different from the EIMA impact test method) and a falling ball test method were submitted to ASTM for consideration. Which of the various procedures to use and how each method relates to actual performance is not yet resolved within the industry.

Workmanship

Poor workmanship accounted for most system deficiencies. Improperly prepared substrate surfaces and insufficient adhesion contact area lead to the system delaminations shown in Figure 3. The use of incorrect backer rod (i.e., an open-cell rather than the specified closed-cell backer rod) caused sealant failures. Failure to properly abut the insulation boards lead to the system cracking shown in Figure 4. All of these were considered preventable if the contractor had followed the specifications and industry accepted installation practices.

<u>Solutions</u>

Most all of the system problems experienced were considered to be preventable. Greater than 90% of the system deficiencies could be attributed to poor workmanship. With proper inspection, most of the deficiencies could have been discovered in time to be corrected during initial construction. However, site inspectors need to be appropriately educated on what to look for that might lead to EIFS problems. Design issues including specifying the system in areas where it is not well suited (e.g., at a loading dock) or failure to specify high-impact systems in high-traffic areas, accounted for most of the other failures. A series of manuals was proposed to help provide this information to the Corps engineers specifying EIFS as well as the inspectors overseeing EIFS applications. A special report was developed jointly by Leo A. Daly, Kenney, Williams and Williams, Inc., the Omaha District Corps of Engineers, and USACERL to further provide information to field engineers responsible for EIFS installations [7].

structural steel beams/columns, etc.) are in place and ready to receive the cladding and framing assembly. For comparison purposes, it is assumed costs of the main structural components are identical, regardless of the cladding/framing assembly selected. Costs for structural/framing assemblies include labor and materials (i.e. gypsum wallboard and supporting framing if required) necessary to receive the interior finish. The cost of the framing assembly can vary widely depending upon the cladding and therefore is included in the analysis, although it is generally not considered to be a component of the cladding. Design of the framing assembly is based upon cladding requirements and local wind loads calculated as per The Guide to the Wind Load Provisions of ASCE 7-88 (formerly ANSI A58.1) [4]. The design evaluation considered wind load coefficient, building importance factor, deflection, spacing of the framing, and pullover values for sheathed substrates. For purposes of design, floor to floor heights were assumed to be ten feet (3 m) which is typical for a commercial building. The results of the design evaluation yields design wind loads on the components and claddings, along with the required moment capacity and required moment of inertia. All of the cladding assemblies were designed to meet the minimum thermal resistance requirements (R value of 12.5 F•ft^2•h/Btu) of the local building codes [5]. However, some assemblies may slightly exceed the R value of 12.5 if commercially available materials and types could not achieve an exact 12.5 R value. These costs were considered since they can vary widely depending on the type of insulation utilized as well as the method of installation. The analysis also takes into consideration the reduction in R value of the wall assembly due to thermal bridging (for framed assemblies) as per the American Society of Heating, Refrigerating and Air-Conditioning Engineers, Inc. ASHRAE 90.1-1989: Energy Efficient Design of New Buildings Except Low-Rise Residential Buildings (ASHRAE 90.1-1989) [6]. Inside and outside air flows, and interior finishes were not included in the ASHRAE energy design due to their relatively low values with respect to the overall wall assembly and would have an insignificant overall cost impact.

COST/DATA

Estimated installation costs of the cladding and structural framing assembly shown in Table 2 were derived from nationally recognized construction cost data publications [7,8]. Nationally recognized publications were used to derive the cost since it was most desirable to use an unbiased source and costs representing national averages. The national average costs were adjusted with city cost indices to accommodate local conditions as indicated in the publication.

Estimated maintenance costs and frequencies were derived from sources such as consultation with various industry professionals, publications, as well as a nationally recognized facilities maintenance and repair cost data publication [9,10]. The latter item was not used as the sole source of the cost data since the reported accuracy is plus or minus 20% because of the wide range of maintenance tasks and diverse environments addressed by the publication. Consequently, multiple sources were utilized in an effort to enhance the reliability of the data.

All installation and maintenance cost data shown in Table 2 represent current dollar costs. Table 3 provides projected life cycle costs for each of the cladding and structural/framing assemblies. Although opinions may differ slightly with the cost data shown, it provides an overall relative comparison of the various wall system alternatives.

LIFE CYCLE COST CALCULATIONS

Net life cycle costs tabulated in Table 3 were calculated in present value terms. The analysis assumes a thirty year study period as it is a reasonable time frame for an investor-owned property. Projected annual discount and inflation rates were obtained from a national financial institution and assumed to be a constant 4-3/4% and 3% respectively over the entire study period. Tax benefits are based on a combined 40% federal and state tax rate. A thirty nine year straight-line depreciation period was used based on current tax law.

CONSTRAINTS

All of the claddings and framing assemblies considered are representative of typical construction methods and materials and are viable assemblies for the model considered. It should be understood that additional cladding and structural/framing options exist, however, they are outside the scope of this paper. Additionally, all of the claddings and structural/framing assemblies satisfy the same basic functional requirements (i.e. protect building occupants, contents, provide pleasing aesthetics, etc.), which is essential for comparison of the assemblies. The structural/framing assemblies are assumed to be located outboard of the slab resulting in an equivalent net rentable floor area for all wall system assemblies. Costs related to windows, doors, sealants, flashings, etc. were assumed to be similar regardless of the cladding alternative selected so they have not been considered in this analysis. The only exception is precast concrete which must be installed in prefabricated panels and therefore included the additional cost of sealant and related maintenance at the panel perimeters since it is not required by any of the other assemblies under consideration.

TABLE 2--Cost Data (Current Dollar Costs)

Cladding and Structural Framing Assembly	Estimated Initial Installation Costs[5] ($/ft²)	Description of Maintenance[6]	Maintenance Costs and Frequencies[7]
Exterior Insulation and Finish Systems (Class PB) with metal stud framing	$234 086 ($11.43/ft²)	Clean 100% of the EIFS	$9 421 at year 15
		Clean and recoat 100% of the EIFS	$18 432 at year 30
Stucco with metal stud framing	$216 269 ($10.56/ft²)	Repair cracks in stucco, 2% of wall surface	$1 273 per 10 years
		Clean 100% of the stucco	$9 421 at year 15
		Clean and paint 100% of the stucco	$18 432 at year 30
Brick Veneer with metal stud framing	$416 973 ($20.36/ft²)	Clean 100% of the brick Repoint 30% of the brick	$14 398 at year 25
Brick Face Cavity Wall with concrete block	$429 261 ($20.96/ft²)	Clean 100% of the brick Repoint 30% of the brick	$14 398 at year 25
Stone Veneer with metal stud framing	$572 006 ($27.93/ft²)	Clean 100% of the stone Repoint 30% of the stone	$14 398 at year 25
Precast Concrete panels with metal stud framing (non-loadbearing)	$357 886 ($17.47/ft²)	Recaulk 100% of the panels	$30 925 at year 20
		General cleaning on 100% of panels	$9 421 at year 25
Reinforced Split Face Block Wall with metal stud framing (non-loadbearing)	$300 032 ($14.65/ft²)	Clean 100% of the brick Repoint 30% of the brick	$14 398 at year 25

[5]See Appendix for maintenance and installation cost calculations.
[6]The cost of maintenance on a three story building will vary from floor to floor due to scaffolding costs. The costs noted for each cladding alternative is average of the costs for each floor.
[7]All maintenance costs assume a thirty year study period.

TABLE 3--Projected Life Cycle Cost (Present Value Dollars)

Cladding and Structural/ Framing Assembly	Estimated Installation & Maintenance Costs (Year)							Cash Outflow[8] (years 0-30)	Cash Inflow[9] (years 0-30)	Net Life Cycle Cost (Net cash outflow)
	0	5	10	15	20	25	30			
EIFS (Class PB)										
a) cash outflow (initial costs and maintenance)	234 086	0	0	7 318	0	0	11 119	252 523	0	
b) cash inflow (tax benefits)								0	41 813	210 710
Stucco										
a) cash outflow (initial costs and maintenance)	216 269	0	1 076	7 318	908	0	11 887	237 458	0	
b) cash inflow (tax benefits)								0	59 927	177 531
Brick Veneer										
a) cash outflow (initial costs and maintenance)	416 973	0	0	0	0	9 449	0	426 422	0	
b) cash inflow (tax benefits)								0	73 055	353 367
Brick Face Cavity Wall										
a) cash outflow (initial costs and maintenance)	429 261	0	0	0	0	9 449	0	438 710	0	
b) cash inflow (tax benefits)								0	74 574	364 136

[8] See Appendix for present value and tax benefit equations, sample calculations as well as methods to determine tax benefits.
[9] Total cash inflows (tax benefits) were provided for the thirty year study period.

TABLE 3--Projected Life Cycle Cost (Present Value Dollars) (cont.)

Cladding and Structural/ Framing Assembly	Estimated Installation & Maintenance Costs (Year) 0	5	10	15	20	25	30	Outflow[10] (years 0-30)	Inflow[11] (years 0-30)	Net Life Cycle Cost (Net cash outflow)
Stone Veneer										
a) cash outflow (initial costs and maintenance)	572 006	0	0	0	0	9 449	0	581 455	0	
b) cash inflow (tax benefits)								0	99 836	481 619
Precast Concrete										
a) cash outflow (initial costs and maintenance)	357 886	0	0	0	22 079	6 183	0	386 148	0	
b) cash inflow (tax benefits)								0	69 446	316 702
Reinforced Split Face Block										
a) cash outflow (initial costs and maintenance)	300 032	0	0	0	0	9 449	0	309 481	0	
b) cash inflow (tax benefits)								0	52 881	256 600

[10]See Appendix for present value and tax benefit equations, sample calculations as well as methods to determine tax benefits.
[11]Total cash inflows (tax benefits) were provided for the thirty year study period.

Appendix C

Construction Management Breadth Calculations

STRESSING TENDONS

$12(25.7) = 300k$

UNGROUTED STRAND, 50' SPAN \$3.63 / lb

$(0.153\ in^2)(9\ \text{STRANDS})(178')(12''/ft) \times 2 = 5883\ in^3$

$+\ (0.153\ in^2)(12\ \text{STRANDS})(178')(12''/ft) \times 2 = 7843\ in^3$

$(0.153)(3\ \text{STRANDS})(55')(12''/ft)\left(\frac{178}{3}\right) = 17{,}975\ in^3$

$31{,}700\ in^3$

$\frac{31{,}700\ in^3}{12^3} = 18.34\ ft^3$ TENDONS @ $490\ lb/ft^3$

$(18.34)(490) = 9030$ lbs. TENDONS

FROM RS MEANS 2002, UNGROUTED STRAND, 300k, \$3.63 / lb.

\$32,779 TO LAY OUT & JACK TENDONS PER FLOOR

\$360,570 TOTAL FOR BUILDING

\$2.90 NOT INCLUDING O&P

$(2.90)(9030) = 26{,}187\ (11) = \$288{,}060$

OUTPUT: 1475 lb / DAY

$\frac{9030\ lbs}{1475} = 6.12$ DAYS PER FLOOR

STRUCTURAL CONCRETE

COLUMNS 16"x16" = 256 in² \$ 842 /C.Y.

COLUMNS 24"x24" = 576 in² \$ 636/C.Y.

ACTUAL COLUMNS 12"x24" = 288 in²

LINEAR INTERPOLATION BETWEEN TWO ⟹ \$ 822 /C.Y.

2½" SAVED PER FLOOR x 11 FLOORS = 27.5" COLUMN HEIGHT REDUCTION

27.5" (36 COLUMNS) (12"x24") = 285,120 in³

$\frac{285,120}{12^3} = 165\ ft^3$ $\frac{165}{3^3} = 6.11$ C.Y.

OUTPUT: 14 C.Y. /DAY $\frac{6.11}{14} = 0.4$ DAYS

ELEVATED SLABS

FLAT PLATE

AVERAGE SPAN = 19.37' → INTERPOLATE BETWEEN \$275.45 & \$394.50

$(10,000\ ft^2)(2\frac{1}{2}'/ft)(11) = 22,917\ ft^3$

$\frac{22,917}{3^3} = 849$ C.Y. @ \$335 /C.Y.

OUTPUT: 40 C.Y. /DAY $\frac{849}{40} = 21.2$ DAYS

PLACING CONCRETE

SLABS, 6-10" THICK, PUMPED
\$14.85 /C.Y.

OUTPUT: 160 C.Y. /DAY
$\frac{849}{160} = 5.3$ DAYS

COLUMNS \$26 /C.Y.

OUTPUT: 90 C.Y. /DAY

WALLS 12" THICK, PUMPED
\$21.60 /C.Y.

OUTPUT: 110 C.Y. /DAY

$(200\ ft)(27.5/12) = 456\ ft^3$

$\frac{456}{3^3} = 17$ C.Y.

PT MODEL

REBAR TOTAL ~ 38 TONS

38(1.2) = 46 TONS INCLUDING LAPPING & SPLICING

PER DRAWINGS

REBAR PER FLOOR ~ 10 TONS

10(11)(1.2) = 132 TONS TOTAL

DIFFERENCE = 86 TONS @ $975 / TON

OUTPUT: 29 TONS /DAY

ADDITIONAL COSTS	BARE	INCL. O&P
TENDONS & JACKING	$288,060	$360,570

SAVINGS		
COLUMNS	$ 5,022	$ 7000
SLABS	$284,415	$ 380,000
PLACING		
SLABS	$ 12,608	$ 17,800
COLUMNS	$ 160	$ 220
WALLS	$ 370	$ 510
REBAR	$ 83,850	$ 111,800
SAVINGS	$ 98,365	$ 156,760
LOCATION FACTOR = 91.5	$ 90,000	$ 143,435
INFLATION ≈ 1.2	$ 108,000	$ 172,100

SCHEDULE MODIFICATION

PER FLOOR

ADDITIONAL LABOR	DAYS
TENDON LAYOUT & STRESSING	6.12
REINFORCING PLACEMENT	1.5
LABOR SAVINGS	
PLACEMENT OF SLAB	0.5
REINFORCING PLACEMENT	4.13
WORKING OF SLAB	2
	1 DAY

SCHEDULE WOULD TAKE APPROX. 1 DAY LONGER PER FLOOR ABOVE GRADE FOR A TOTAL OF 11 DAYS EXTRA.