April 7th, 2011

UNIT 5:
STRUCTURAL REPORT

KGB Maser
Management. Engineering. Consulting.

IPD/BIM TEAM #3

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STRUCTURAL EXECUTIVE SUMMARY

The purpose of this report is to detail the process of structural analysis that was used to redesign the floor system, façade, and cantilever of the Millennium Science Complex. This laboratory building is located at the corner of Pollock and Bigler in University Park, PA. The existing design will be evaluated, and redesigned based on the goals of KGB Maser in an effort to engineer a system that functions as an integral part of three systems while maintaining economy and constructability.

The current floor system of the Millennium Science complex uses a lightweight, composite floor system to meet a strict vibrational criterion. Lightweight concrete on top of 3-inch metal deck is used with 24-inch deep girders in order to retain a certain level of rigidity. The proposed design replaces these wide flanges with 30-inch deep cellular beams to increase stiffness while preserving low mass.

An analysis was performed in SAP in order to calculate the existing floor's vibrational velocity. The results of this analysis were used to size the cellular beams that would replace the current wide flanges. It was found that strength, rather than stiffness controlled the new design, although stronger concrete was used to largely increase performance for a relatively low cost.

The façade was identified by KGB Maser as a point of interest due to its exisiting weight of 36 thousand pounds. In order to decrease the weight of the panels, and subsequently the amount of materials, the team investigated decreasing the profile depth.

After an analysis was completed on the strength of the current panels, the face depth of each panel was decreased to 5 inches from 6. That analysis revealed it was also possible to decrease the flange depth, decreasing its profile depth a foot. Thin brick was used to further decrease the weight of materials at its face.

The existing cantilever stretches 154 feet unhindered by support over a landscaped plaza at the North West corner of the building. This cantilever is a large source of structural costs and was considered by KGB Maser as an opportunity to reallocate money for more practical purposes. The redesign proposed two columns that would sit between two intersections of the four main trusses in order to reduce stresses in their members and eliminate unnecessary diagonals.

The trusses were completely redesigned, eliminating all but one floor of web members in the overhang. The existing base columns were able to be reduced in weight and several bays of bracing, previously purposed to resist the cantilever's inherent overturning moment, beyond the overhang's base were removed.

A lateral system analysis was also completed. This analysis confirmed the strength of the current lateral system using ETABS to check shear, story drift, and maximum displacement. Due to a torsional irregularity, panel zone shear and cracked concrete sections had to be considered in the analysis of the analytical model.

For a complete IPD/BIM discussion, please refer to Unit 1. The following explains only the structural depth of KGB Maser's redesign.

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FLOOR SYSTEM

Of the revisions proposed, the floor system was the first system to be changed. Due to the projected time required to complete the analysis, three weeks were allotted to entirely redesign the floor system. It took four and a half weeks in total from initially researching vibrations to the point at which a final SAP model was completed and run to move on to the next structural system.

EXISTING CONDITIONS

The existing floor system utilizes steel beams and girders to support a composite deck in square, 22’ x 22’ bays. Wide Flange, 21 inch deep beams frame into 24 inch deep girders in typical fashion, as seen in the figure below, throughout the Life Sciences and Material Sciences wings (please note the orientation of the center row of bays in each wing as it is oriented 90 degrees from the direction of the adjacent bays). Strict vibrational criterion necessitates the use of heavier beams and lightweight concrete in areas where labs and offices are located. To minimize weight while maintaining stiffness, the engineers used 3000 psi lightweight concrete on top of 18 gauge 3” metal decking for a total floor height, including girders, of about 30 inches, as shown in Figure 5.3. Normal weight concrete is used elsewhere, in varying thicknesses, in locations not regularly populated by indoor traffic.

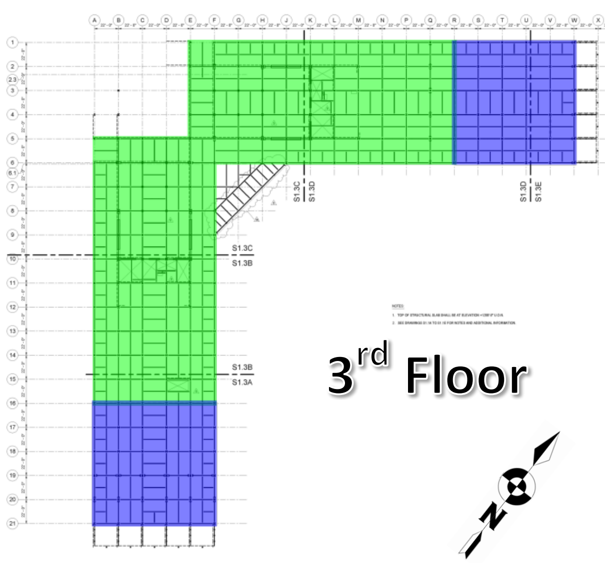


Figure 5.1: Plan View of the Third Floor

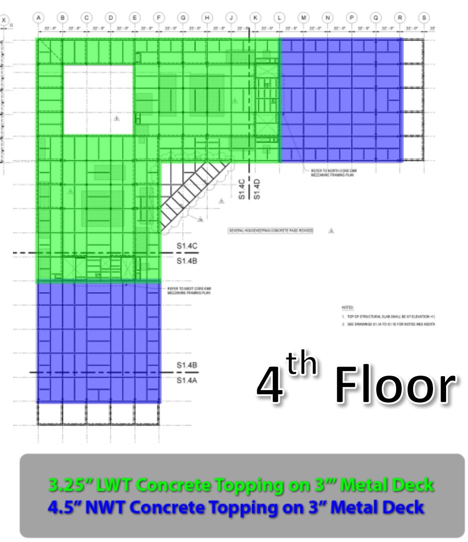


Figure 5.1: Plan View of the Fourth Floor

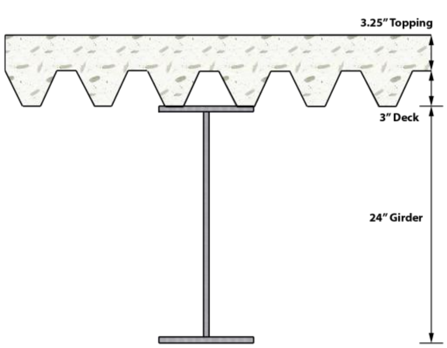


Figure 5.2: Typical Floor Profile

Figures 5.1 and 5.2 show a plan view of the 3rd and 4th floors, respectively. It must first be explained that KGB Maser chose the 3rd floor to focus on rather than the entire building, as that would have been a massive undertaking requiring time the teams were not afforded. Since the 3rd floor plenum is the area being studied by the Mechanical and Electrical/Lighting disciplines, the 4th Floor was studied in order to coordinate their systems through the 4th floor structure. The structure of the 3rd floor was also redesigned with an emphasis being placed on vibrational impedance to accommodate the vibrational criterion required of this floor. Also shown in figures above, are green and blue areas representing slab variants as well as defining the different areas of green roof, in blue, and office/lab space, in green. The area in blue on the 4th floor represents the most congested space in the plenum of the 3rd floor; this area was given special focus by the mechanical and structural disciplines during their research.

PROPOSED DESIGN

A preliminary gravity analysis was conducted on the existing floor system to confirm member sizes in each wing. It was found that the members were two to three times stronger than required by strength or deflection. Information garnered from an information session courtesy of Thornton Tomasetti revealed that the members were, in fact, oversized due to a vibrational criterion of 4000 and 2000 micro inches in the Life Sciences and Material Sciences wings, respectively. With this information, it was posited that a different solution could be used to meet vibrational requirements while relieving congestion in the third floor plenum.

Figure 5.3: Example castellated beams coordination with distribution systems from ArcelorMittal

Since frequency is dependent on mass and stiffness, the proposed alternative had to be either stiffer or lighter. Replacing the existing wide flanges with cellular beams was proposed thereby decreasing mass while maintaining stiffness. This solution also provided a convenient alley by way of the beam's inherent voids through which mechanical equipment could snake as demonstrated in Figure 5.4. The deck and concrete topping would remain unchanged, as would the W14 columns and lateral system.

Although this solution would have actually increased the cost of the floor system, it was anticipated that it would have allowed the plenum space to shrink, decreasing floor to floor heights. The amount of material saved by decreasing story heights would have, theoretically, offset the increased cost of the floor system.

ANALYTICAL PROCESS

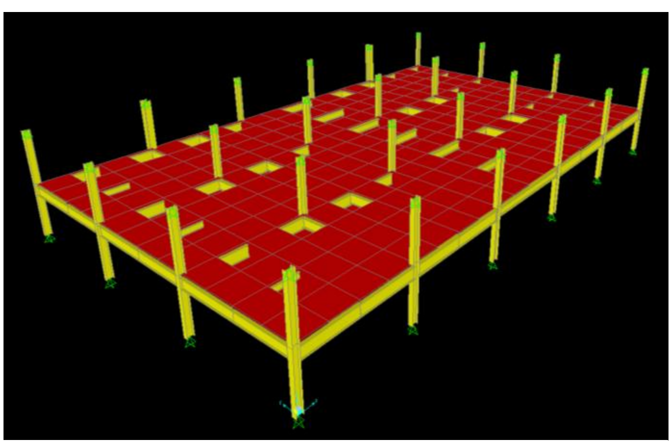


Figure 5.4: Final SAP Model (the seemingly protruding beams are a result of an error in rendering the extrusion in SAP)

Determining the stiffness of the existing floor system was the first step in the redesign process. The vibrational benchmark was given to us by Thornton Tomasetti, but numerical stiffness of the existing system was unknown. Initial research was conducted by reading AISC Design Guide 11 to both learn the evaluation process of stiffness in a floor system and to gather a general list of elements which would be needed to complete a vibrational analysis.

As per chapter 6 in Design Guide 11, the equation $[V = U_v \Delta_P / f_n]$ determines the velocity of a system based on a footfall impulse parameter $[U_v]$, its deflection $[\Delta_P]$, and its frequency $[f_n]$ acquired from another, separate equation. With this information, a model was begun in SAP2000 to determine the maximum deflection, when subjected to a concentrated load, of the existing floor system. The results of this procedure would set the bar for future redesign alternatives.

EXISITING CONDITIONS SAP MODEL

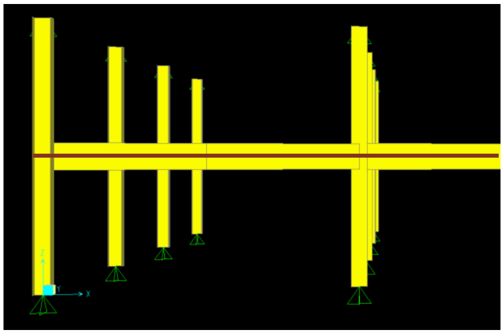


Figure 5.5: Initial SAP Model

The most costly procedure in this process, in terms of time, was building an accurate existing conditions analytical model. Rather than modeling the entire 3rd floor, it was chosen to use a representative section using 15 bays, 5 wide and 3 long as seen in Figure 5.5. All dimensions were taken directly from the structural drawings including the column and beam sizes. The columns were placed ten feet above and below the slab, fixed with pins on either end where moments are assumed to be zero due to the bending curvature of the element. The beams and girders were then modeled and released from moment at their connections, assuming shear connections only. Each bay was given its own slab, modeled as a shell thin, which was assigned modified material properties due to its behavior differing in one direction versus the other. To account for this behavior, due mainly to the ribs, the shell's modulus of elasticity was increased by a factor of 1.5 the direction in which the deck spanned.

An issue arose when trying to mimic composite action inside SAP. Since SAP, by default, places every element on a gridline by its centerline, as shown in Figure 5.6, it was necessary to offset the beam or slab to attain the correct depth, and therefore inertia, of the floor structure. In the figure, the yellow elements represent wide flanges and the red horizontal line penetrating the center of the beams represents the default placement of the shell element. A question was brought up regarding the accuracy of simply using insertion points to gain composite action of the slab and beam, leading to an investigation of composite beam action in SAP.

Four options were explored during this investigation. The first option simply offset the top of the beam, using insertion points, to 4.625" below the centerline of the slab. The second option did exactly the opposite, offsetting instead the slab above the top of the beam. The third option was a blend of the first two, exploring different combinations of offsetting both the beam and slab while maintaining a distance of 4.625" between them. The fourth option used rigid elements to connect the slab and the girder, which were placed on different gridlines at different elevations from one another. A series of trials were conducted using every method to determine the most accurate way of modeling a composite floor system.

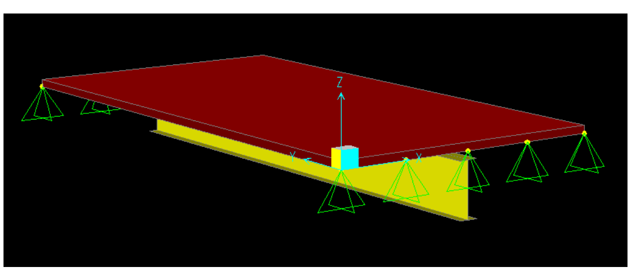


Figure 5.6: Composite Floor System Trial

To set up these different trials, a simple bay was used, 8 feet wide and 20 feet long, and a beam was drawn across it. A 3" slab was used and pinned at the edges to prevent bending in two directions. The beam was first offset, followed by the slab, and finally both were offset at intermediate values between 0 and 4.625 inches, while still maintaining a constant distance of separation. Through all these trials, inconsistent results were being returned as beam size and weight changed. One combination of offsets would return an accurate deflection (corroborated by hand calculations) for a particular beam, however when the beam was changed, a different combination of offsets was needed to return the same amount of accuracy. Not only did this prove to be inconvenient when attempting to replicate it on a larger scale, the results were inconsistent with hand calculations performed using a transformed moment inertia and a basic deflection equation. It was found, after searching through SAP's included manual, that in order to return results which match those of a simple deflection equation, the simply supported beam had to be determinate. Pinning both ends of a simply supported beam, and using insertion points to offset the beam below the grid line, created an invisible line of tension that resulted in a lower deflection than what was predicted. After using a roller on one end of the beam and discretizing the frame as well as the slab, deflections closely aligned with what was foreseen (within 10%). Insertion Points were used in the final existing conditions model, offsetting the beams and girder below the grid line, 3 inches below the bottom of the slab as seen in Figure 5.8.

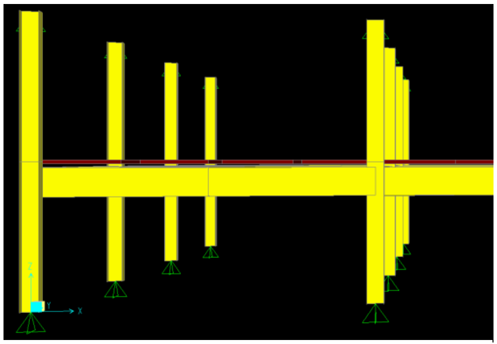


Figure 5.7: Final Existing Conditions Model with Insertion Points

Completing the model was fairly straight forward from that point onward. A point load was assigned to critical points in each bay of relevance. The slab was divided up and then discretized further in order to properly distribute mass as assumed by vibrational calculations. Figure 5.9 illustrates where loads were placed with green, blue and red circles. These points produced the most deflection when subjected to a point load and are representative to the behavior of the remaining bays.

VIBRATIONAL ANALYSIS OF EXISTING CONDITIONS

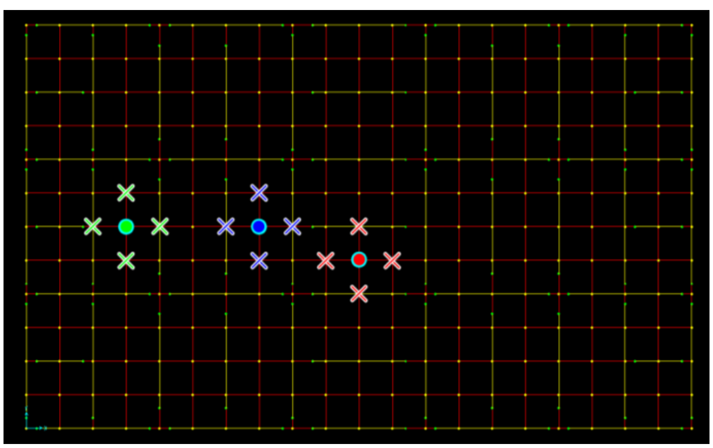


Figure 5.8: Plan View of SAP Model - Deflection Due to Point Loads

Once the model was complete with all necessary elements and loads, it was run for deflections. These deflections were used to calculate an approximate fundamental period of each bay which would then be used to calculate velocities. The equation and factors used for this analysis was taken directly from Design Guide 11 and is as follows:

$V = U_v \Delta_P / f_n$; where $U_v = 5500\ lb.Hz^2$ for moderate walking.

Using SAP to find the fundamental frequency would have required averaging different modes, whose shapes do not always reflect the period of one particular bay. A more straight forward method was used to calculate the fundamental frequency by way of Rayleigh's Method. His equation is as follows:

$\frac{1}{2}\omega^2\Delta^2 m = \frac{1}{2}P\Delta$; Solving for frequency yields $T = 2\pi\sqrt{\Delta^2 m / P\Delta}$; where $P = 100k$, the point load applied, and m is the mass of the bay.

The procedure of steps follows the table below from left to right. First each bay's mass was calculated by adding the total weight of the slab and beams in one bay and dividing by 484 square feet, the area of each bay, to obtain a distributed load. The nodes in three bays (those to which point loads were applied) were then renamed and used to gather the deflections at each of these 25 points. Weight was distributed to each node by way of tributary area and then multiplied by the square of its deflection as per Rayleigh's Method; this value was then divided by the product of the 100 kip point load and the deflection of the node where it was placed. After the period was calculated, the equation garnered from Design Guide Eleven was used. This equation uses the floor's maximum deflection from a unit point load to calculate velocity. In day to day use, deflections will not be caused by a single point load, rather it will be caused by a human foot walking on the weakest part of the slab over more area than a single point. A weighted average of the deflection at the point of the unit load and its neighboring nodes was therefore used, out of practicality, as depicted in Figure 5.9 by "X's". Sample calculations detailing the bias given to each point used in the calculation of Δ_P can be found in Appendix A.

SPAN	Lx	Ly	t	w	Wslab	Wbeams	NODE	Wi	Δ	Wi.Δ^2	P.Δ	Tcalc	T(SAP)	Vel
	ft	ft	in	ksf	kip	kip		kip	in		P=100 k	sec	sec	μ in/sec
SPAN-A	22.0	22.0	3.3	0.049	23.619	4.103	1	0.533	0.0012	0.0000	178.6212	0.0639		3916
							2	0.902	-0.0195	0.0003				
- due to load at A13							3	0.902	-0.0330	0.0010				
							4	0.902	-0.0195	0.0003				
							5	0.533	0.0012	0.0000				
							A1	0.902	0.0551	0.0027				
							A2	1.640	0.0596	0.0058				
							A3	1.640	0.0774	0.0098				
							A4	1.640	0.0596	0.0058				
							A5	0.902	0.0552	0.0027				
							A6	0.902	0.0913	0.0075				
							A7	1.640	0.2216	0.0805				
							A8	1.640	0.2886	0.1366				
							A9	1.640	0.2217	0.0806				
							A10	0.902	0.0914	0.0075				
							A11	0.902	0.0614	0.0034				
							A12	1.640	0.6814	0.7615				
							A13	1.640	1.7862	5.2335				
							A14	1.640	0.6818	0.7624				
							A15	0.902	0.0614	0.0034				
							A16	0.533	0.0052	0.0000				
							A17	0.902	0.0818	0.0060				
							A18	0.902	0.1219	0.0134				
							A19	0.902	0.0826	0.0062				
							A20	0.533	0.0051	0.0000				

Figure 5.9: Calculation of Vibrational Velocity

As illustrated by the table above, velocities were very close to what was required of the building. The values calculated from the existing conditions served as a baseline for the redesign.

REDESIGN ANALYSIS

The existing analytical model served as the base for redesign. Since design changes were minimal, the existing model was simply updated by replacing the unaltered wide flange sections with modified W21 sections. Initial sizing of members was done by matching the inertia values of the existing beams and girders with inertia values of particular cellular beams obtained from RAM SmartBeam. The cellular beams were then checked for strength by using an excel spreadsheet taken from a steel manufacturer's website and increased in weight as necessary (see Appendix A for spreadsheet and manufacturer). By using W21 members in the model, the components which comprise a 30" cellular beam, the weight of the cellular members were maintained. The shear areas of these wide flanges were reduced by roughly 10 percent and their inertias were increased twofold in order to mimic the behavior of an actual cellular beam. The updated model was then run and its results were used in the spreadsheet created for existing conditions; only the beam weight per bay had to be changed. These results were then compared to the ones gathered before and member sizes and decking were re-evaluated and changed as appropriate to exceed those conditions set by the existing conditions model.

COLUMN CHECK

After changing the floor system, a column check was conducted in order to confirm the sizes of the existing conditions. Loads were quantified based on the categories listed in the Figure below. Columns were sized for loads every two to three floors, splices lying between the 2nd and 3rd floors.

Column Check								
	Occupancy	Area	Dead Load (lbs.)				Live Load	Totals
			Slab	Beams	Panels & Column	SDL	(lbs.)	(lbs.)
Roof	Roof	484	24200	1804	0	12100	8580	59452.8
Floor 4	Mechanical	484	53240	3817	1755	12100	72600	201254.4
Floor 3	Green Roof	0	0	4559.5	1620	0	0	131319.4
	Office	0	0			0	0	
	M.S. Labs	0	0			0	0	
	L.S. Labs	484	24200			14520	48400	
	Corridors	0	0			0	0	
	Elevator Lobbies	0	0			0	0	
Floor 2	Green Roof	0	0	6303	1620	0	0	78851.6
	Office	484	24200			14520	14300	
	M.S. Labs	0	0			0	0	
	L.S. Labs	0	0			0	0	
	Corridors	0	0			0	0	
	Elevator Lobbies	0	0			0	0	
Floor 1	Plaza Landscape	242	26620	3476	1800	72600	24200	231875.2
	Office	0	0			0	0	
	M.S. Labs	0	0			0	0	
	L.S. Labs	0	0			0	0	
	Corridors	242	16940			7260	24200	
	Mech. Mezzanine	0	0			0	0	
	Elevator Lobbies	0	0			0	0	

Floors Three and Four			
W14X61	392.0	k	
A=	17.9	in.2	
k=	1.0		
l=	18.0	ft.	
r=	2.5	in.	
E=	29000.0	ksi.	
Fy=	50.0	ksi.	
k*l/r=	88.2		
Fe=	36.8	ksi.	
Fcr=	28.3	ksi.	
ϕPn=	456.3	k.	OK

Floors One Two and Three			
W14X90	702.8	k	
A=	26.5	in.2	
k=	1.0		
l=	20.0	ft.	
r=	3.7	in.	
E=	29000.0	ksi.	
Fy=	50.0	ksi.	
k*l/r=	64.9		
Fe=	68.0	ksi.	
Fcr=	36.8	ksi.	
ϕPn=	876.7	k.	OK

Figure 5.10: Column Check at the Intersection of Grids 9 & C

RESULTS

When a first analysis was run in SAP with preliminary cellular beam sizes, the results showed surprising deflections. When these SAP results were inserted into the excel sheet that translated their values, the resulting velocities were actually slightly higher than those of the existing conditions. Shear deflections were not considered a major factor with the design in SAP up until this point. The decreased area of the area resulted in larger shear deflections than the existing wide flanges, and even despite their doubling of inertia, the cellular simply deflected more. To solve this issue, the cellular had to become heavier in order to increase their stiffness.

As these changes were being made to account for shear deflections, a mistake was realized with the strength calculations. The loads on each beam had been miscalculated, and they had to be sized up due to strength. Given that development, the beams being sized up as a consequence of strength, the model was run again with the new beams in place. The results that were returned gave deflections aligning more along the lines of those of the existing conditions. They were still slightly higher than the existing conditions, but their weights, although only slightly less, brought down the floor's vibrational velocity to within a few percent of the existing conditions.

Velocity Comparison			
	Existing Conditions (µ.in./s.)	Redesign (µ.in./s.)	Percent Change
Span A	3916	3099	20.86%
Span B	3317	2737	17.49%
Span C	4063	3458	14.89%

From the onset of the analysis, it was desired that the floor system exceed the existing system's performance. Two options were then considered. The current vibrational velocity would suffice, so the system could be left as redesigned, or the floor could be made stiffer in order to gain a more desirable velocity.

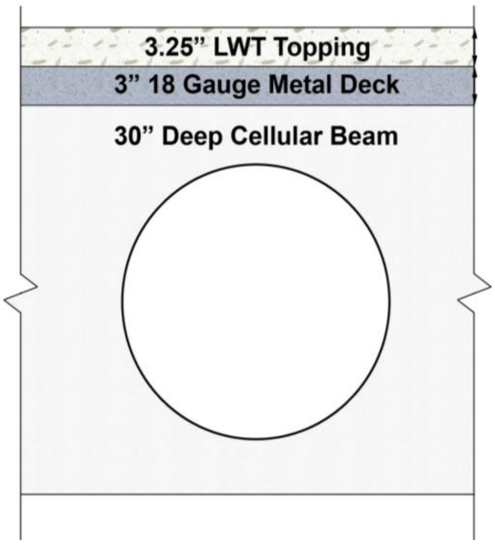


Figure 5.11: Cellular Beam Profile

It must be noted that a discovery was made in the midst of this process. Allegedly the material sciences wing suffers from a stricter vibrational criterion than the life sciences wing. However, when the material sciences wing was modeled in SAP, the members that were needed to be changed were few in number and lacked any significant strength advantage over the life sciences wing's members. When the deflections were run through excel to calculate velocities, the results did not represent a floor that was stiff enough for the criteria required of it.

It was chosen to increase the concrete strength rather than upsize the beams. This increase from 3000 psi to 4000 psi added $5 per cubic yard to the price of lightweight concrete (or 4.2%). The model was run again with 4000 psi concrete in order to gauge the value of the change. Velocities decreased 20% in each bay with the added strength, so the change was made permanent.

A large consideration for this floor type was cost. Cellular beams are more expensive than traditional w-shapes for the same weight. Since they are made from the two halves of a wide flange, they retain the cost of the original w-shape on top of the added cost of manufacturing. It was chosen therefore to only use this system where congestion in the plenum is heaviest. This occurs mainly at the end of each wing where the laboratories require large quantities of ventilation. For this reason, only two beam sizes were needed for the entire 4th floor. And also, as a consequence of this lack of size diversity, the bays weigh relatively the same and are also mostly lighter than the existing conditions. Though this does not account for much in the way of velocity performance or lateral forces, it is nonetheless an improvement.

Figure 5.12: Cellular Beam Layout

Because the beams were placed every eleven feet, the system was very uniform. Nearly every beam uses the same amount of tributary area so the loads experienced by each are nearly identical, and the same can be said of the girders. Different layout schemes were considered, but ultimately disregarded due to cost and incompatibility with the mechanical system. If, for example, the beams were placed every 7 or so feet, deflection would most likely decrease enough so that shallower beams could be used while still meeting velocity requirements. The smaller beams, however, would use smaller voids and prevent most of mechanical equipment from using that space. If the beams were again increased in depth, 3000 psi concrete could be used, but the cost of the extra beam in every bay would far offset the savings in concrete cost. The current layout, it seems, uses the most efficient spans it can, given the size of the bays.

Choosing cellular beams over a concrete system was done in light of the collaborative effort between disciplines. An integrated design process calls for decisions to be based on the consequences of multiple systems, rather than basing them on a solely structural objective. Thus the analytical process was completed in order to realize a larger goal of mechanical integration with the structural system. The system, in this way, finds success in being a true alternative because it functions as a participant of a larger machine.

FAÇADE

The façade was designed as an ongoing study of panel alternatives throughout the entire semester. The façade was constantly referenced as keys points of study for the mechanical, construction management, and electrical/lighting disciplines. The design that resulted from the structural analysis was also influenced by the other discipline's systems analysis. For more information on this topic, please refer to Unit 1.

EXISTING CONDITIONS

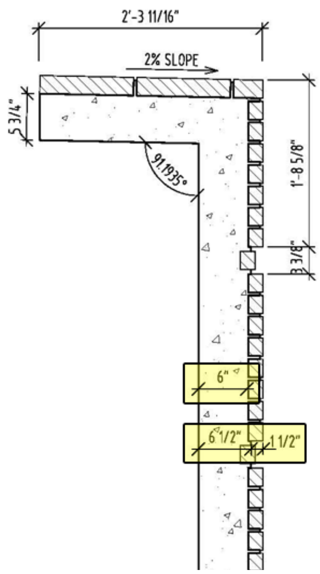


Figure 5.13: Existing Facade Panel

The existing panels weight in at 36 thousand pounds apiece. They impart all of their weight onto the exterior columns, which are then needed to be sized up in order to resist the additional force. It was proposed from the start of this project that the panels were needlessly heavy and could be thinned in some way to achieve better economy of materials and to reduce the forces on the superstructure.

This enormous weight is derived from sheer volume. The panels are 6 inches at their faces, which are embedded with 2-inch, masonry half-brick, as illustrated by the diagram on the left. The shape they take, a "C", is due to a cantilevered slab at the edge of the building. They also sport flanges that flank all four edges of each panel. These flanges shoot 2 feet from the rear of the panel towards the building and are used to resist bending under the panel's self-weight. Each panel stretches 22 feet across the exterior frames to connections at each exterior column. Two bearing connections are used along with two lateral connections, which brace the panel against wind and seismic loads.

In order to begin the redesign process, the existing panel dimensions were analyzed for strength. It was understood that each panel needed to remain uncracked in order to maintain the illusion of a real brick façade, so analysis was conducted for a stress of 477 psi, derived from the strength of an uncracked section of 5000 psi concrete.

	Precast Panel Dimensions		
	Panel Height	141.125	in.
	Panel Depth	4.25	in.
	Brick Depth at Face	2	in.
	Flange Height	5.75	in.
	Brick Height at Flange	2.25	in.
	Flange Depth	27.6875	in.
	Panel Width	263.25	in.
	Return Thickness	14	in.
	Return Depth	21.4375	in.
	Return Height	129.625	in.
	Volume Concrete	173.9579	ft.3
	Weight Concrete	26093.68	lb.
	Volume Brick	61.9801	ft.3
	Weight Brick	7437.612	lb.
(factored)	Total	46943.81	lb.

Figure 5.14: Existing Facade Dimensions

The dimensions were first taken from the construction documents, and inserted into a table made in excel, on the left. This spreadsheet related these various measurements to volumetric dimensions and, by multiplying these volumes with the density of a particular material, weight was found. The weight calculated did not match the values given by the precast manufacturer, who presumably evaluated each panel in more detail, and with more accuracy. However, as information was limited on the process by which they found those weights, the weight by way of the method as described above was used.

The largest panel was chosen to be evaluated for various loading cases. These loading cases included the panel sitting on its connections as part of the façade, the panel laying down

	Cracking Stress	
(factored)	477.2971	psi

Self Weight Check Prone			
Weight/in.	8.53125	lb./in.	(factored)
Inertia of Strip	76.765625	in.4	
Moment	16695.94	lb.in.	
Stress	462.17134	psi.	OK

Figure 5.15: Self-Weight Inducing Moment of Existing Panel

at the site before assembly, and it subjected to wind pressure. As shown in the above table, the controlling load case for the existing conditions was gravity in the case of the panel being laid prone before construction.

The biggest surprise during the analysis process was the panel's self-induced moment, as shown in the Figure on the left. Previously it had been assumed that the panel was 6 inches thick merely for architectural reasons, but as discovered from the strength calculations, the required thickness, based purely on a maximum uncracked stress, is 4.25 inches at the face. 6 inches, while conservative, was reasoned based on quality control which could fall short during the transportation and erection processes. Surprisingly the controlling factor in this case was gravity rather than wind.

REDESIGN

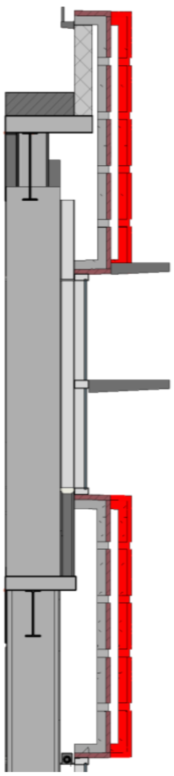
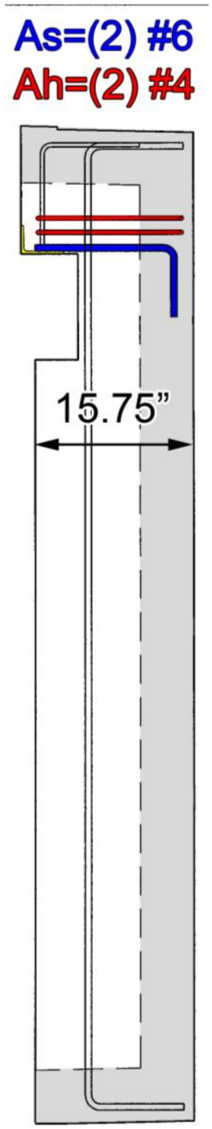


Figure 5.16: Redesigned Panel - Existing in Red

As it was desired to make the panel profile thinner, the most susceptible element of the panel to change, given its conservative construction, was the flange. 1 foot was immediately removed from the flange profile, shrinking the entire panel depth to 15.75 inches while also removing one inch from the panel face thickness. The new dimensions shifted strength control from gravity to wind.

Although the concrete was dimensioned appropriately for strength under bending, connections still had to be considered. Two types of bearing connections were investigated. The first connection analyzed was a dap steel type, which places the connection in the middle or towards the bottom of the panel. The connection was first evaluated for required steel in order to properly identify rebar sizes. These rebar sizes were then used to calculate development length into the façade panel. It was found that ## inches of development were needed for strength, ## inches more than what was available. KGB Maser was unwilling to enlarge the profile beyond what was decided, so another connection type was considered.

A corbel was chosen instead of a bottom bearing connection. Reinforcement was calculated for the specific Vu, and rebar was sized based on the required steel. Development length was once again checked. By moving the connection to the top of the panel, the development length criteria is changed, requiring less development into the concrete, 9 inches versus the aforementioned ##. This option fit the desired panel depth so the connection was chosen.



PCI was constantly referenced throughout the entire process providing the appropriate equations for each bearing type connection. The precast manufacturer has also provided their drawing and sample calculations for reference. A quick check of their numbers confirmed the veracity of the above calculations.

CANTILEVER

Redesigning the cantilever fell at the middle of the overall analytical process. It was projected that the entire analysis, including an investigation of the existing conditions, would take two weeks. The entire redesign was completed in a week and a half, including changes made on a Revit model to reflect those member sizes which changed due to the analytical process.

EXISTING CONDITIONS

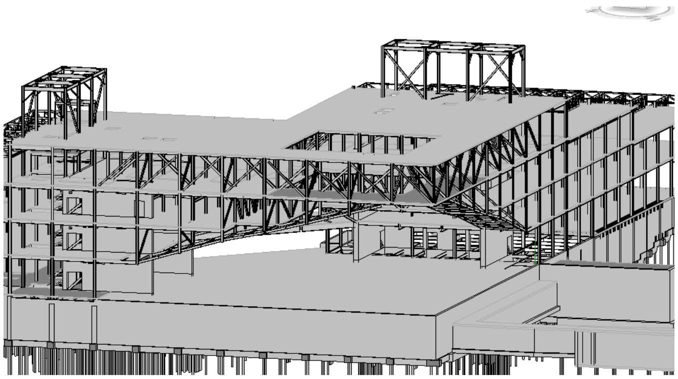


Figure 5.17: Existing Cantilever 3D Model

Situated at the corner of the Millennium Science Complex, a 154 ft. cantilever stretches out over a landscaped plaza. This architectural feature, conceived out of a purely aesthetic goal, adds an enormous amount of money to the overall cost of the superstructure. And on top of the expensive construction, the space inside the cantilever is mostly unoccupiable, including the last 88 ft. of the overhang. Its trusses crowd the mechanical penthouse with web members making placement of mechanical equipment inside the fourth floor even more difficult than it already is.

The cantilever is entirely supported by four main load bearing trusses which occur at grid lines 2, 5, B and E. Forces are collected by diagonal web members which then transfer loads into large wide flange columns and into the foundation by way of enormous pile caps, as seen in Figure 5.14 (Blue members represent compression whilst red ones represent tension). An overturning moment develops out of this cantilevered action, which is resisted by two more bays of trusses extending beyond a 30 inch thick shear wall, shown in yellow below. This shear wall was not used in the initial design of the cantilever. Its inclusion occurred later in the design process, when a vibrational consulting firm suggested it as a necessary factor in damping vibrations from the cantilever. Although it adds stiffness to the entire truss, it did not participate in the analytical model drawn up by Thornton Tomasetti, the design firm on this project. A deflection limit was given by the design firm of 2 inches at the tip of the overhang which greatly increased the cost of the cantilever, as truss members had to be both moment connected and sized based on stiffness rather than strength, as it may have been if deflection had meet code requirements of L/180 or 10 inches.

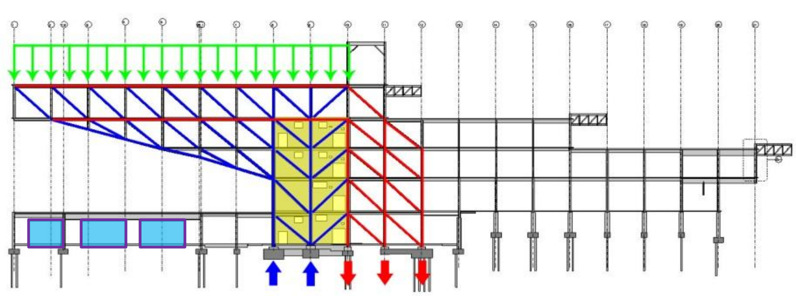


Figure 5.18: Existing Cantilever - Distribution of Forces

One of the main advantages to using a giant cantilever over the plaza, is its minimal interference with the basement level below. This level houses three laboratories, show in blue boxes on the left, each of which are subjected to strict vibrational criterion. Each lab sits on a 2 ft. thick slab completely separate of the surrounding foundation, poured independently of any other slab, as shown in Figure 5.14. The laboratories were designed for a vibrational velocity of 130 micro inches per second, achieved by its seclusion from any potential vibration inducing source.

PROPOSED DESIGN

It was a goal of KGB Maser's to reduce the structural cost of the building in order to afford the Mechanical and Lighting/Electrical disciplines more freedom with their energy efficient designs. The most obvious way of reducing the cost was to reduce the amount of materials used in the superstructure. Since most of the cost is concentrated in the cantilever, it was suggested that a column be placed at the end of it, thereby reducing the large stresses experienced by the existing truss members and allowing their weights to therefore be decreased.

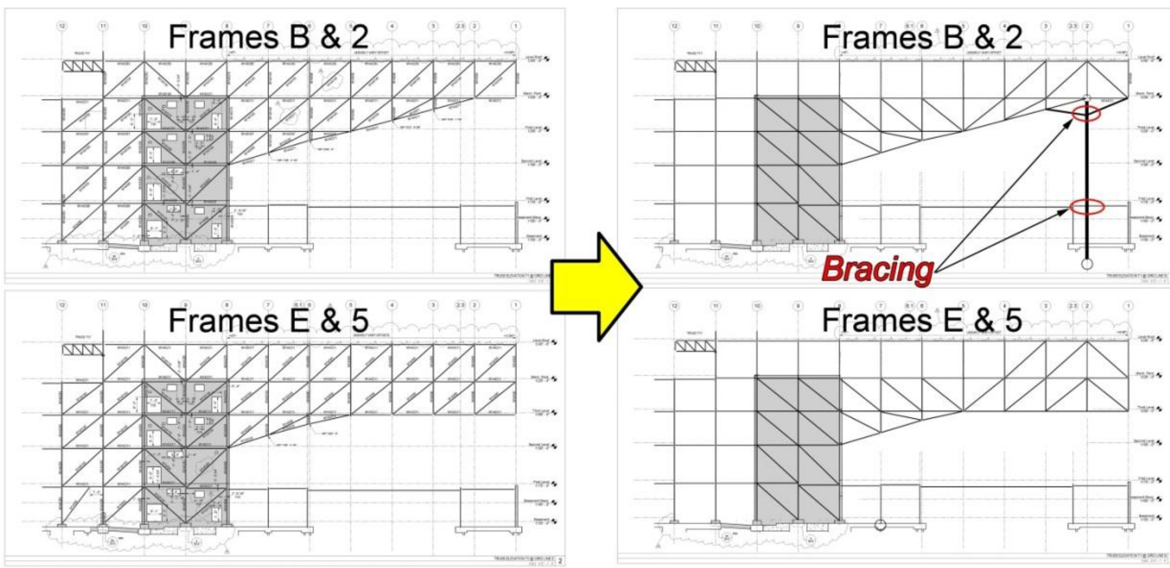


Figure 5.19: Initial Truss Layout

Initially one column was proposed, situated between the intersection of trusses 2 and B. A new web design had to be created, and since the information up until that point had led KGB Maser to believe that the existing truss's members were pinned, it was assumed that the redesign would be as well. By eliminating the cantilever, the trusses needn't be as stiff and therefore need be less encumbered by braces. The resulting design was anticipated to rid the mechanical penthouse, as well as the two bays beyond the shear wall, of diagonal bracing. However the new design also required a restructuring of the basement level due to interference with the isolation laboratories.

A column that would support the end of the cantilever would also need to penetrate through the plaza level and travel directly through the laboratory floor based on the existing layout. To reduce direct vibrational propogation through the slab, it was posited that the column could be, itself, isolated from the laboratories. By creating a premeditated hole in the floor of the laboratory and allowing the column to travel, unobstructed, through the slab, the isolation laboratory could retain some of its vibrationally resistive integrity. This column would also cause a disturbance, not only in the labs, but also in the visual experience created by the architect, Rafael Vinoly. These two factors ultimately shaped the resulting plan visually and schematically of the cantilever redesign.

ANALYTICAL PROCESS

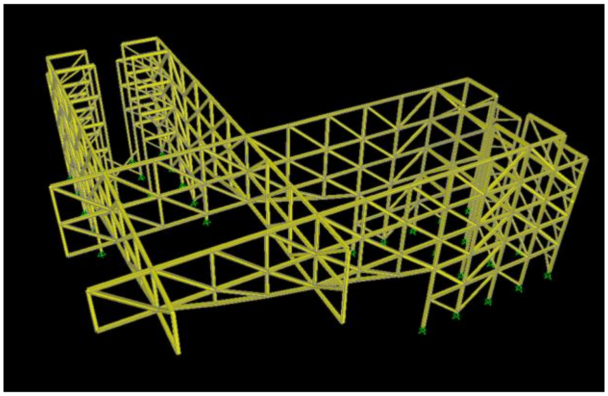


Figure 5.211: Initial SAP Model

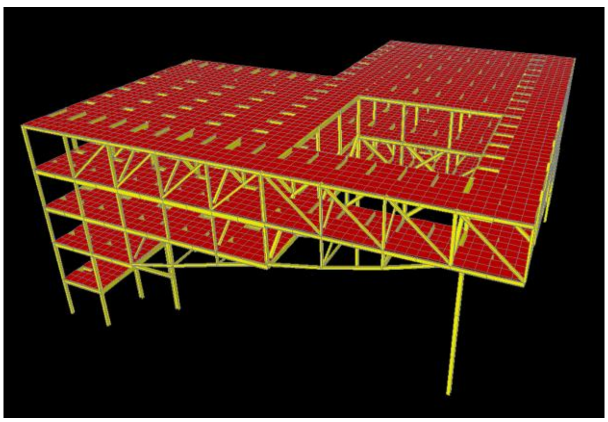


Figure 5.202: SAP Model Iteration

It was first decided that an analysis of the existing cantilever would lend some insight into the method of force distribution throughout the truss, so a SAP gravity model of the existing truss design was begun, seen in the Figure to the left. The model was based on the structural drawings and from an existing Revit model complete with member sizes. Only the four main trusses were modeled, as it was assumed that a simplified distributed load derived from a scheme of tributary areas would suffice over a complete modeling of the floor and its loads. After a rough plan of tributary area was created, it was realized that a more accurate way of approximating the actual loads experienced in the building would be to model everything, including the floor system through the end of the truss, as seen in the Figure to the left. After modeling the majority of components inside the truss, it was realized that the proposed redesign would be completely changed, from a cantilever to a simple truss spanning from one support to another; thus the existing conditions would have proved to be of little use to do its limited relevance to a simply supported truss. Although fundamentally, the analytical process had changed, the existing conditions continued to be modeled as it was decided that only the design of the four trusses would be altered. The frames which depend on these four mains for support were designed for loads which will remain unchanged. It was assumed that the transfer of forces from these dependent frames into the four main trusses will remain as is, where only the behavior of the forces through the independent trusses will be changed. The model was completed with the addition of two columns at the far and near corners of the window box, as shown in Figure 5.16; the theory behind using two columns being the more, the better.

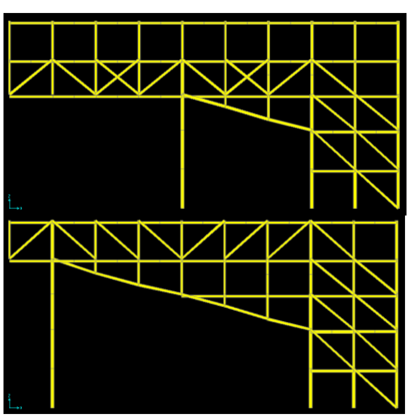


Figure 5.22: Frames B & E First Iteration

Preliminary sizing was based on a truss layout inspired by a basic Pratt truss where all members are in tension, as illustrated on the left, and would therefore need the very least amount of steel area. These members were also pinned rather than moment connected, as they are in the existing truss, a discovery made late in the analytical process which had little bearing on the redesign or its results although relevant to the modeling of existing conditions. W14X90's were chosen for web members and diagonal bracing inside the base of the truss. The chords and columns were left alone, to be sized after a first analysis.

The results of the first run revealed a stable model which behaved relatively identical in both the North/South and East/West directions. This was expected due to entire model being symmetrical, but it did lend credence to the accuracy of the model and its results. Members were resized based on this first run, changing the diagonal, horizontal and vertical elements. Changes in one truss reflected changes in its counterpart, revealing the redesign was successful in balancing forces. Deflection did not control at any point during the process of redesign, although multiple iterations were required due to strength.

A last check of was conducted based on beam-column interaction. Results gained from SAP were plugged into an Excel spreadsheet, seen above, which calculated each member's bx, by, and p or ty/tr based on its unbraced length. Being that the spreadsheet took into account bending around both major and minor axis, some member sizes were increased over the changes made via hand calculations. The columns were initially sized as W14X550's on the basis of an assumption of unbraced length and later checked based on the actual unbraced length acquired from the finished architectural feature used to mask them. Around seven iterations were completed in order to arrive at a completed model whose members met all strength requirements.

RESULTS

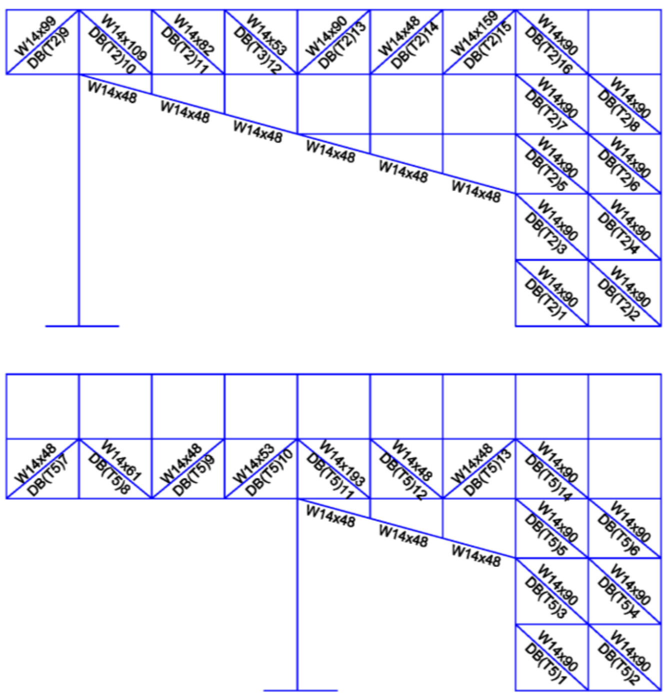


Figure 5.234: Final Truss Design

Ultimately, the truss redesigned truss was a success. Bracing was removed in two entire bays, previously necessitated in order to resist the overturning moment which has now been eliminated. The braces that remain were switched to tension, since stiffness was not a controlling factor in the redesign, and greatly reduced in size. Nearly all the members, were, in fact, reduced in weight. The columns at the base of each truss still need to carry half the load of the cantilever, so they were the biggest members besides the two columns added in the redesign. These members were all able to be downsized by the removal of the deflection limit. Since a large cantilever no longer exists, the required stiffness to limit deflection is greatly reduced to the point of strength controlling every member. The limit on deflection was 2 inches over a 154 foot cantilever set by the design firm; the allowed deflection of the new design, over a span of 66 feet in the interior truss is 2.2 inches in accordance with L/360. A maximum deflection of 0.83 inches was reached in the interior truss, well below its limit.

As described above, each member in the truss was put into an excel spreadsheet which checked the results returned from SAP by way of a unity equation, as seen on the following page. Shear was assumed not to have controlled at any point in the design process; a quick check of the largest shear of any truss versus the capacity of the smallest member in shear, reveals that it exceeds the maximum shear verifying that assumption. The only point at which forces required the addition of structure outside what was structurally proposed is in the two supporting columns at the end of the cantilever.

TABLE: Element Forces & Unity Equation												
Frame	Station	OutputCase	P	V2	V3	M2	M3	FrameElem	ElemStation	Section	Length	Length	Interaction
Text	in	Text	Kip	Kip	Kip	Kip-in	Kip-in	Text	in		in	ft	
CL(T2)1	0	All Factor	-2566.34	2.013	-0.485	-1.1E-14	-2.3E-13	CL(T2)1-1	0	W14X283	240	20	0.87
CL(T2)1	120	All Factor	-2562.6	2.013	-0.485	58.154	-241.525	CL(T2)1-1	120	W14X283	240	20	0.88
CL(T2)1	240	All Factor	-2558.87	2.013	-0.485	116.307	-483.049	CL(T2)1-1	240	W14X283	240	20	0.90

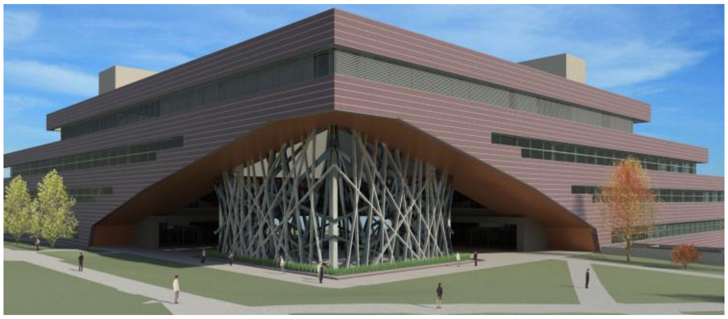


Figure 5.245: Bird Cage Rendering

The exterior column, or the column located at the intersection of the two outermost trusses, experiences 3400 kips over an unbraced length of 56 feet whilst the other, interior column experiences a larger load of 3900 kips. Extra bracing needed to be given to the columns lest the size and weight of them go up dramatically. It was originally proposed to mask the presence of a column with an architectural feature. This feature, or bird cage as it appeared to be, behaved dually both drawing attention away from the presence of the columns, and to bracing them intermediately. The unbraced length shrunk to 32' by using HSS tubes designed to resist 2% of the axial load of the column. These tubes, which appear as part of a mass of intertwined cage members, feed back into the truss for support and are braced, themselves by other members participating in the architectural feature.

The redesigned truss succeeded in alleviating congestion in the mechanical penthouse, it eliminated web members and turned the ones remaining into tension members reducing material and therefore cost. By virtue of two supports on each truss, the overturning moment present on the existing design becomes irrelevant to the new design and removes the need for bracing beyond the shear wall. However, with the presence of a column comes the need to resolve axial force via pile caps in the foundation. The location of the columns coincides with the location of the isolation laboratories, as shown on the left, requiring these foundation pile caps to be placed directly under the isolation slabs. This is an issue as the labs are under a 130 micro inch per second limit on vibrational velocity. Although no calculations were performed to verify the concept, a rational solution to this problem was proposed. The column pile cap would be placed several feet below the bottom of the isolation slabs. This depth of earth would provide impedance to any vibrational propagation initialized in the column. The column would be constructed first, and the isolation slab would be poured around it, allowing for an inch or so gap. This gap would then be filled with a compressive material to further mitigate vibrations. The frames of the plaza at the first floor would simply attach to the column. This method would be used for both columns.

Figure 5.256: Isolation Lab Interference

LATERAL SYSTEM

A check of the existing lateral system was the last process of the structural depth. This analysis was begun with a cooperative model between 2 other structural engineers, concluding with an individual check of the lateral elements. The entire analysis was completed in a week and a half.

EXISTING LATERAL SYSTEM REVIEW

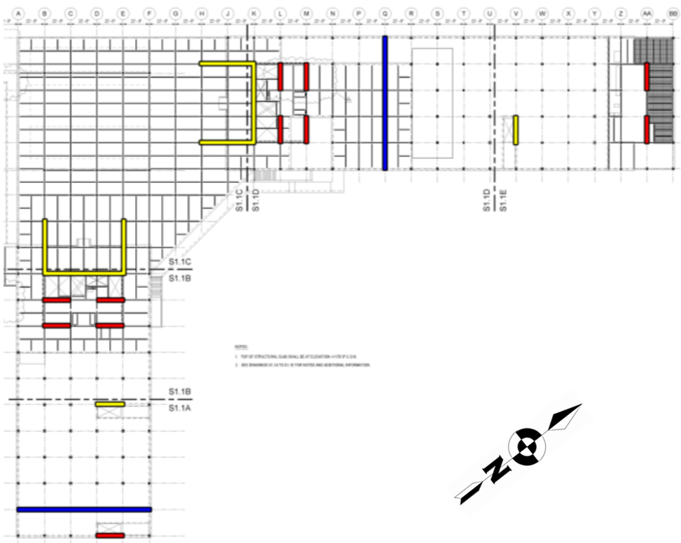


Figure 5.267: Plan of Lateral Elements

The existing lateral system is made up of various frame types throughout each wing. Shear walls, moment frames, braced frames, and gravity trusses, purposed for the cantilever, all partake in resisting the lateral loads. Most of the forces are taken by 3 shear walls toward one end of each wing, whose original designs were not meant to participate in the lateral system; rather they were included to dampen vibrations from the cantilever.

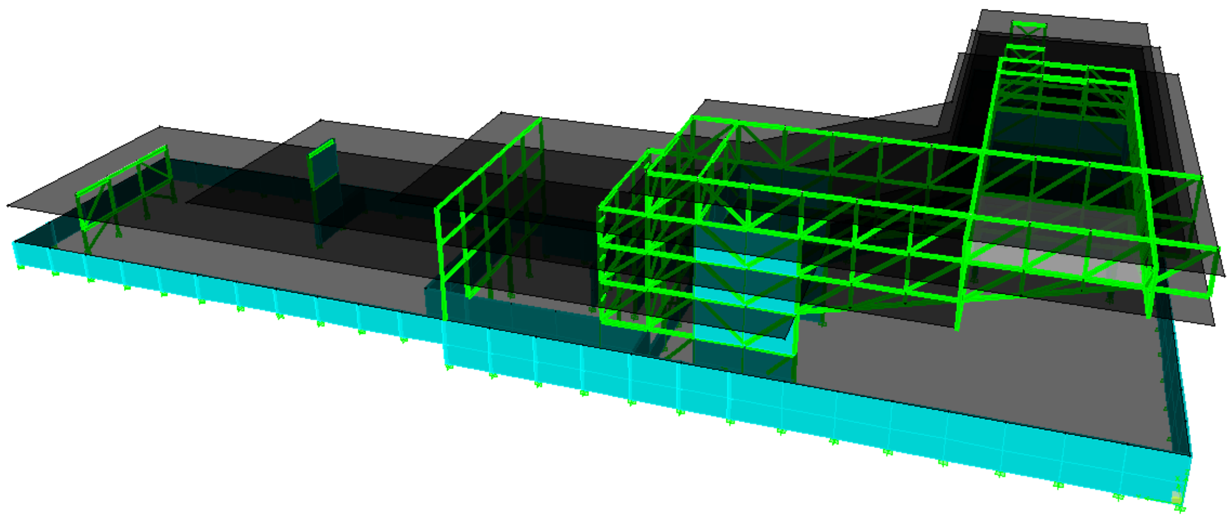
The plan on the left shows the placement of the various types of lateral resisting elements. They are staggered in such a way that force should be distributed evenly throughout each wing. Moment frames are shown in blue, shear walls in yellow and braced frames in red. The shear walls, and adjacent braced frames located closest to the Northwest corner of the building are also part of four large trusses that support the 154’ cantilever at that end of the building.

As the floor system redesign was developed, the lateral system was continuously changed to fit the appropriate floor system type. It ended up that the floor system was changed relatively minimally. The existing lateral system had been presumably designed correctly for the current layout, so it was thought that with only minor changes being made on the floor system, that the lateral elements needn’t be changed. An analysis was performed to confirm that the existing system did indeed meet code strength and drift requirements. The entire lateral system was replicated in ETABS and run through a series of checks.

When the redesigned floor system was being modeled, the lateral elements were avoided in being changed. It was guessed that due to the relatively low area for its depth that the cellular shapes would perform poorly in shear. The areas where beams were moment connected to their columns, the floor remains precisely as it was before the redesign. It was aimed to limit the amount of interference with the current lateral system as much as possible to retain the same integrity.

Where the lateral system did definitively change is in the cantilever trusses. Since the trusses were completely revamped, their diagonal members were downsized drastically. This of course led to reduced stiffness in these frames. And although these frames are mainly purposed to support the cantilever, they play a major role in the lateral system, so any changes could have been significant. If the shear walls were not present in these trusses, the lateral system would have needed to be completely redesigned, but these shear walls lend a tremendous amount of stiffness to each truss frame. It was proposed that the lateral system be left as is, changing only the diagonal members inside the base of each truss. This change, of course, would be trifle due to the 30 inch thick shear that encases the truss frames at their base as proved later by an analysis of the existing conditions.

LATERAL SYSTEM ANALYSIS



First the entire lateral system of the millennium science complex was replicated in ETABS. The entire Northwest corner of the building had already been modeled in SAP for a gravity analysis and it was anticipated to simply export this model to ETABS to serve as a base for assembling the lateral system. However, the amount of gravity members that were modeled in SAP would have simply burdened ETABS with superfluous information, slowing its analysis and lengthening load times. It was also feared that errors would inevitably occur in the process leading to a lengthy period of correcting mistranslated information. Area properties as well as member properties would have needed to be altered to fit a lateral analysis and it was believed that creating a new model from scratch in ETABS would have been longer, but it would have caused less frustration and ultimately produced a model with less oversights.

Therefore each lateral element was recreated in ETABS. The floors were modeled as rigid elements and constrained to move with the lateral elements. Some of the frames required special joints to be placed off grid, especially those in the truss. The shear walls were modeled as shell elements, which were discretized for accuracy. There is also a diagonal foundation wall that was modeled at the interior corner of the meeting of the two wings. This shear wall required the creation of new elevations so it could be placed at the right location. The entire model's elements were placed even before the lateral loads and floor weights were corrected from the previous semester's calculations. In fact, the model was nearly complete from the start of the semester, but analysis was left to the end because of other priorities, including the cantilever, façade and floor system redesigns.

Floor weights had to be slightly corrected due to errors made when inserting floor areas into excel. These weights had a cascading effect on the rest of the seismic load calculations, whose story forces depend not only on ground acceleration, but of the floor masses as well. These new forces were corrected and applied to the model in two seismic load cases. Wind forces were inserted into the model based on story forces as well, with 8 load cases being necessary to cover all combinations of wind direction and moment due to eccentricity. The façade panel weights had to be applied to the model as point loads on the exterior columns. These weights were taken from construction documents created by the precast manufacturer.

RESULTS

After all necessary steps were taken to complete the model, an analysis was run. As was expected, seismic values controlled as they produced the largest story forces. In fact, seismic base shear was 1.5 times greater than the base shear produced by wind alone.

The analysis also revealed peculiar behavior in the distribution of forces. Forces were concentrated in the shear walls, taking over 90% of the load between four walls, three of which are located towards one end of each wing. This result could be explained by describing the size of each shear wall. The wall experiencing the most force is 16" thick and 66 feet long, an incredible amount of area over which shear can be distributed. It is no wonder that these walls take an inordinate percentage of lateral forces.

Another result that was, at first, perplexing was the amount of force in one of these walls. Looking at the layout of the lateral elements leads one to believe that the shear walls are favored towards one end of the building. In reality, the floor footprint is shaped like an "L" so the visual center when considering only the lateral elements in one direction appears farther from the actual center of mass. And since the story forces are applied at the center of mass, the shear wall that takes the most force serves as a fulcrum for the other three walls in that direction.

The period of the analytical model was then checked to corroborate the one calculated for seismic loads. Unfortunately, this period was much higher than what was calculated by seismic analysis, on the level of 3 seconds. Clearly a mistake had been made with the model, so loads were once again checked. The floor weights had been overestimated, and were far higher than should've been, so they were re-calculated and inserted back into the model.

One issue that could not be readily explained was with total amount of shear collected in all the elements. This total did not match the total base shear, being 6% lower than what was applied to the building. This was discovered when section cuts were used on all the modeled elements, in both direction, for seismic loads in the East-West direction. The first floor was chosen as the plane across which these section cuts would be used. Forces in each frame were separated into loads received by the columns, braces, and shear walls individually. Once their values were tallied, it was found that this total did not quite equal the amount of force that was applied to the building. One explanation that was proposed, involved the participation of an out-of-plane element. There exists a 45 degree foundation wall at the corner of the building that was thought to have been interfering with the results. Taking a section cut of this wall revealed that 20 kips were being taken in along its major axis. It was believed this force was the missing component of the total base shear, however even when added to the total, the forces still did not amount to a number equal to the base shear. After further consideration it was decided that this 20kips was due to more to eccentricity than from direct shear and was hence discarded as the problem.

Story Drift						
		Disp. (in.)	Disp. (in.)	Average	Max./Avg.	
Quake: East-West	Roof	0.041235	0.067931	0.054583	1.244545	
	Mech.	0.027331	0.027255	0.027293	1.001392	
Quake: North-South	Roof	0.067717	0.042757	0.055237	1.225936	
	Mech.	0.026746	0.028852	0.027799	1.037879	

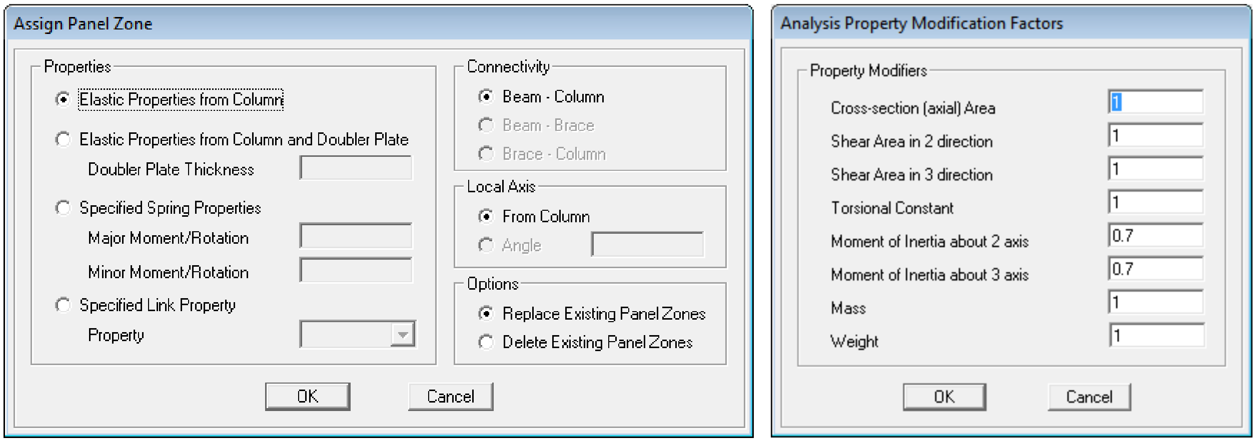


Figure 5.28: ETABS Torsional Irregularity Property Modifiers

Drift was also considered. The building was found to have torsional irregularity when subjected to seismic loading in both directions. According to ASCE7-05, the building falls under seismic design category B, and therefore, when classified as having a torsional irregularity, is required to be modeled mathematically. This had already been done with ETABS and its members were therefore checked for strength as per code. The mathematical model had to conform to a certain criteria; it had to be modeled in 3D, considering cracked section properties for concrete, and panel zone deformations for steel moment frames. All these requirements were easily met, and used to check for lateral element strength.

As was mentioned previously, lateral forces in the non-shear wall frames were small and ultimately piffling in the grand scheme of things. These forces totaled less than 10% of the base shear. Time constraints did not allow for detailed checks of the braced or moment frames, but checking them by hand against the beam-column interaction equation showed they exceeded strength requirements. Story drift was last checked, revealing a maximum story drift, including the Deflection Amplification Factor, of 0.00056, far below the allowable drift.

MAE COURSEWORK INTEGRATION

AE597B

A complete redesign of the existing cantilever was performed on the Millennium Science Complex. Methods of solving for chord and web forces, learned in Historical Methods of Structural Analysius, were used in creating a preliminary redesign for the four trusses of the cantilever. Using a design inspired by a Pratt Truss, web members were oriented so that they performed in tension. Due to different loading conditions, the live loads may cause these web members to experience a reversal in axial force, switching them from tension to compression in certain bays of the truss where dead load cannot supersede the influence of the live load. This was taken into consideration in the preliminary design with counters in bays near the midspan of the truss, between the column and the truss base. After an analysis was completed, it was decided that these counters were not needed as the live loads were too small to reverse the shear in the center bays.

AE 597A

Extensive use was made of computer modeling software including SAP and ETABS. SAP was used for redesigning the cantilever and floor system. The composite floor system was modeled by using normal wide flanges offset from the slab, whose material properties were edited to behave differently in different directions depending on the orientation of the deck span. The floor system also needed to be checked for vibrations, so it was analyzed for specific periods of vibrations. The cantilever was modeled using every beam, column and brace in the Northwest corner of the building to accurately depict the distribution of forces through its truss members. The lateral system was also modeled in detail with the relevant lateral force resisting elements. Loads calculated from an evaluation of seismic and wind forces were applied to the model and it was run to check member strength in those relevant structural elements. For a more in depth review of the model building process, please refer to the appropriate chapters above on the floor system and cantilever.

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