Structural Redesign

Theory

Innovative architecture demands innovative engineering solutions. The unusual design of the Athletic Center presented a substantial challenge for the engineering team to find a creative answer. Often when a building exterior is as unique as the Athletic Center the structural engineer must assume more responsibility for the architectural design. In this way they become more of an architect-engineer, attuned to the aesthetics of shape, form, and balance, while maintaining a firm grasp on the practical requirements of safety, economy, and constructability. The primary goal of the structural redesign was to take advantage of the opportunities which exist in the original design to develop a creative yet sensible alternative.

Therefore, there was no hesitation to alter the architectural look and feel of the building. Liberties were taken in changing the façade to meet demands of the new structural designs. However, complete disregard of the Athletic Center's contextual and programming requirements would be irrational and irresponsible. The general shape, height, and space layout was kept consistent with the original intent of the architect and owner. This restraint also helps reduce the scope of research and focuses the redesign on more comparable alternatives.

In order to further refine the above theory and to provide a base by which designs can be evaluated, the following specific goals are outlined below:

- Increase overall structural efficiency.
- Decrease the cost of the building as a whole, not just structure.
- Keep the design feasible from a construction standpoint.
- Reduce system complexity if possible.
- Limit redesign to the diagrid system only, however, check major effects that the changes will have on the rest of the building, such as foundation overturning and torsion

The Solution Area approach was developed to obtain a complete picture of the available alternatives to the diagrid system. It starts with a relatively non-disruptive replacement of the diagrid material, moves to a visibly changed exterior geometric change, and ends with the total discarding of the diagrid itself. Therefore, there are three progressive levels of architectural deviance.

For each Solution Area the general method was threefold:

1) Research – Available options were obtained through background research
2) Design – Those options were analyzed to find size, efficiency, feasibility, etc.
3) Select – The most reasonable option (if any) is chosen for comparison to the others

Specific selection criteria for comparison between alternative systems and the original system vary by Solution Area. They are identified and explained within each section.

The structural design of the Athletic Center utilized the 1998 Ohio Basic Building Code. Other major codes and standards used for the redesign are ACI 318-02, AISC LRFD Design Manual 2001, NDS 2001, and ASCE 7-98.

Solution Area I – Changing the material of the diagrid

The purpose of Solution Area I was to keep the perimeter lateral system in the current diagrid configuration while changing the material and/or detailing of its members. Before undertaking this task, examples of other diagrid systems were found. It was determined that 5 alternative materials have been or could be used in such a configuration. Those 5 alternatives are rectangular HSS, round HSS, glulam timber, precast concrete, and cast-in-place concrete.

Procedure

The alternatives were evaluated through a wide spectrum of categories. Each alternative in every category was rated on a scale of 0-100 by either an analytical procedure or simple educated assumptions. The categories which used an analytical procedure are:

- Weight – Force output from a computer model was used with a spreadsheet to find a typical axial to moment force relationship. This relationship was found to be about 2. A representative load of 200 kips (compression)/100 ft-kips was employed to size members using the respective design methods for each material. The sizes multiplied by material density gave approximate weights per foot of diagrid member.
- Cost – Once the members were sized, basic costs from Sweet's Unit Cost Data converted the weights or lengths to cost per foot. This cost was used with the consideration of fireproofing, insulation, etc.
- Size – The force output used to determine typical member weights was also used to find the most loaded member (475 kips/550 ft-kips). This member was sized in each material by the same design method as above.

The categories rated using educated assumptions were weighted for importance according to their contribution to the material's overall feasibility. They are:

- Availability
- Lead time
- Erection time
- Flexibility
- Durability
- Labor cost
- Fire resistance

Results

All of this information above was tabulated in Excel. The resulting spreadsheet indicates that no single system performs head and shoulders above the rest (Appendix A.1). Summaries of the ranking and scoring are shown in Table 1 and Chart 1.

Material	Rank
Wide Flanges	1
Rectangular HSS	3
Round HSS	2
Glulam Timber	6
Precast	5
Cast-in-place	4

Table 1: Alternative Diagrid Material Ranking

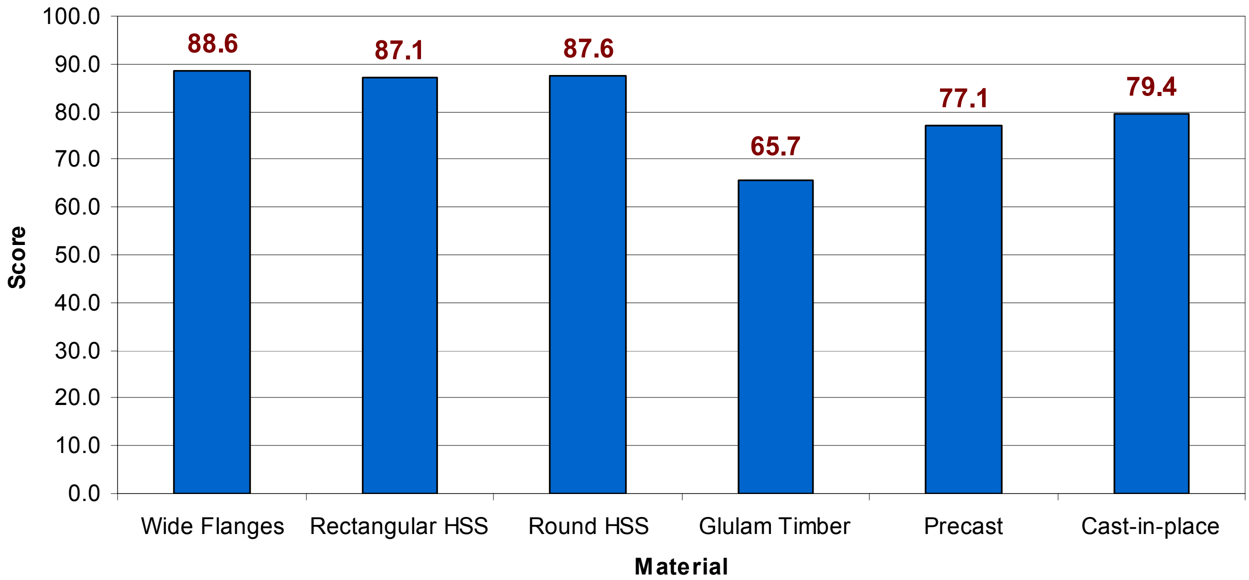
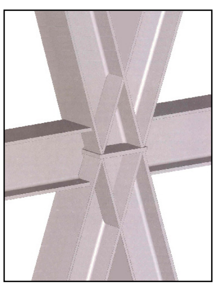


Chart 1: Alternative Diagrid Material Scoring

The original steel wide flange diagrid slightly edged out rectangular HSS and round HSS. In general, the steel options scored better than the concrete or wood options. An overview of each alternative's advantages and disadvantages are explained below.



Steel Wide Flange

Advantages – The weight and size of wide flanges are optimized to resist the high bending loads many of the members experience. This results in reduced structure weight and flexibility of size.

Disadvantages – Pre-fabrication of the diagrid sections requires a longer lead time. Careful planning can overcome the additional scheduling time.

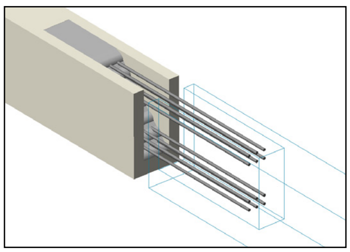
Rectangular and Round HSS

Advantages – As with wide flanges, HSS sections can be prefabricated in multi-panel sections, which would allow quick erection by crane. The quick erection also reduces labor costs in the field.

Disadvantages – Floor layouts will be changed because beams will need to frame into node points. This reduces floor flexibility and efficiency.

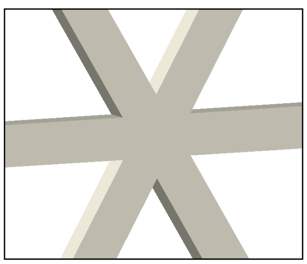
Glulam Timber

Advantages – Multi-panel sections can reduce erection time.
Disadvantages – Timber costs, both for material and connections, are much higher than the traditional structural materials of steel and concrete. The large sizes required by strength design (not even considering deflection and creep) prohibit its use in this application. Additionally, durability and weathering of the timber are issues.



Precast Concrete

Advantages – The flexibility of precast allows it to fit the curved form of the building. Concrete is also an extremely safe material against structural fire damage.
Disadvantages – Additional dead weight due to the large cross sections impact the foundations below grade, as well as increasing deflections of the long spans. Concrete creep is also an issue.



Cast-in-place Concrete

Advantages – Material cost is excellent, due to the low pound-for-pound concrete/steel cost ratio. Lead time is virtually nothing because cast-in-place is available on demand.
Disadvantages – Counteracting the nonexistent lead time is a lengthy erection time, complicated by the need for unusual formwork shapes and rebar splices. Labor costs will reflect the increased field time.

Discussion

Overall, wide flange steel is still the most reasonable choice. None of the other choices seem to offer substantial benefits over the original diagrid material. It must be concluded that this solution area did not produce a reasonable alternative to the current system, and therefore the original design will be kept. Consequentially, wide flange steel will continue to be used in the upcoming redesigns.

Solution Area II – Modifying the geometry of the diagrid system

Although the perimeter diagrid system functions as both the gravity and lateral load carrying system for the above grade levels, it is not an exceptionally efficient structure. The dense array of this virtual "wall of steel" uses relatively little of its potential strength. Many members are barely stressed under factored loads while a select few approach their practical load limits. The obvious solution would be to adjust the size of each individual diagrid segment to more fully utilize its strength capacity and/or deflection contribution. Unfortunately, this creates fabrication, erection, and cladding complications, increasing cost and scheduling of the building simultaneously.

The purpose of Solution Area II was to maintain the perimeter gravity and lateral system while taking liberties to modify its architectural (and hence structural) geometry. The goal was to develop a more efficient structure similar in concept to the original diagrid which would overcome weight, complexity, deflection, and drift issues.

There are two main ways to modify the geometry of the structure. They can be executed alone or in combination:

1) Open up the grid
   This can be accomplished by either removing members in a consistent fashion along the entire façade or by reducing the density of the grid in certain sections only, such as on the upper stories. One of the most well-known examples of this approach is the architecturally and structurally acclaimed John Hancock Center in Chicago. Its characteristic diagonal bracing was derived from a fine diagrid mesh on each face. The grid was made progressively coarser (Figure 10), increasing its structural efficiency while creating the clear X shapes the skyscraper is associated with (Iyengar, 47).

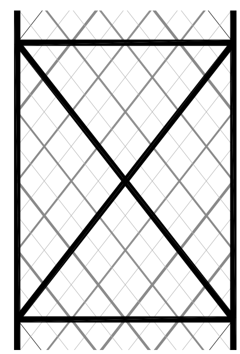


Figure 10: John Hancock Center grid

2) Adjust configuration
   The arrangement of the diagrid can change by either reorienting the members to different slopes or by letting the members "follow the load path." The latter option is well documented in engineering literature and sometimes produces unpredictable yet elegant results. An example of this theory is the Central China Television (CCTV) tower currently in development by the Office for Modern Architecture and Ove Arup engineers (Figure 11). The daring expressed design allowed the structural team to modify its diagrid configuration to accommodate areas of high load flow. (Reina, para. 7)

Figure 11: CCTV

To assess and compare the performance of the various options which were developed, the following categories were taken into consideration:

- Structural Efficiency – weight of superstructure
- Structural Stability – strength through redundancy, deflection
- Architectural Impact – geometry of the plan, V column layout
- Floor Framing Impact – orientation of framing members, connection ability
- Material Cost – steel, glazing, insulation, cladding
- Complexity – labor and connection material cost

In selecting the most desirable system, more emphasis was given to the structural categories of efficiency and stability.

Procedure

Structural Efficiency and Structural Stability

Using the structure modification techniques above, alternative geometric configurations of the diagrid were developed. Initially, these were drawn in two dimensional views (see Appendix B.1), which seemed to be the easiest way to visualize the patterns and proportions. It was determined that analyzing these representative 2D elevations would be much more efficient than analyzing the actual 3D shells. Although this approach is a simplification of the actual curved façade, the inaccuracies were assumed to be negligible. Furthermore, the alternatives could be easily evaluated for major axis bending and axial forces, the two most prevalent member forces in the original diagrid structure.

In order to choose a section of the perimeter for analysis, moment and axial force diagrams of the original diagrid system were studied. Looking at Figures 12a and 12b below, the areas of highest force occur at the northern end of the building. A section centered at the Northeast corner which includes two of the greatest areas of stress and the longest span was chosen. It is highlighted in green.

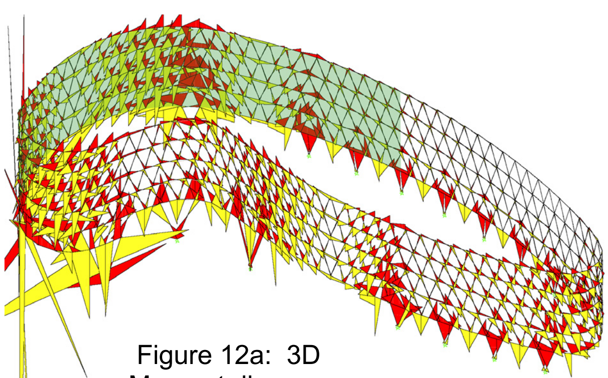


Figure 12a: 3D Moment diagram

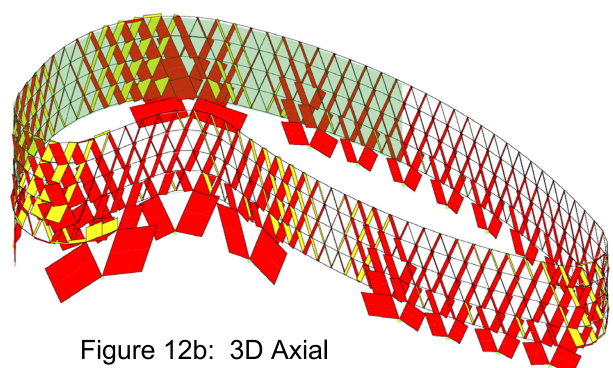


Figure 12b: 3D Axial Force diagram

The final elevations for all of the options, or cases, were set using this Northeast section. Support conditions were modeled to be as close to the original V columns as possible. Cases 1, 2, 3, 4, 5a, 5b, and 5c are shown in Figures 13a through 13h. For Case 6, the technique of following the flow of forces was used. To do this, the section was modeled in the STAAD structural analysis program as a uniformly loaded simple beam with relative support conditions similar to the elevation. The moment diagram of the beam (Figure 14) gave insight as to how the diagrid members could be oriented more efficiently. The new orientations allow members to carry primarily tensile load. Case 6 configuration is shown in Figure 15, with the "tendon" members highlighted red.

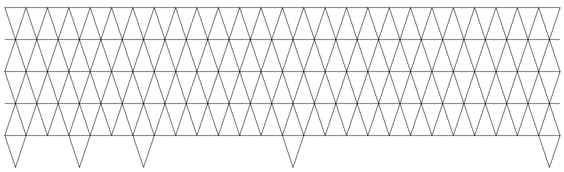


Figure 13a: Case 0 elevation

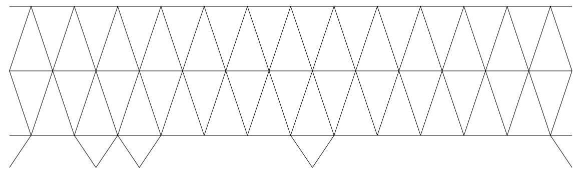


Figure13b: Case 1 elevation

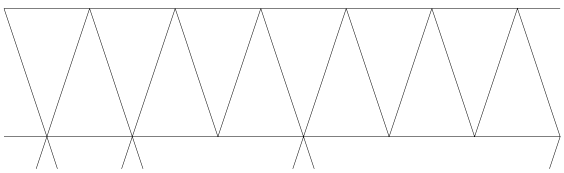


Figure 13c: Case 2 elevation

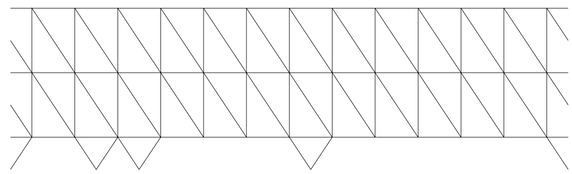


Figure 13d: Case 3 elevation

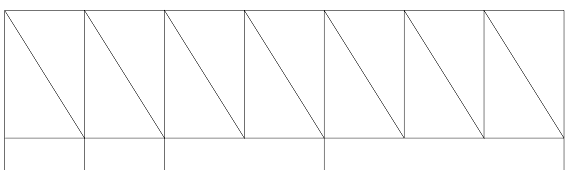


Figure 13e: Case 4 elevation

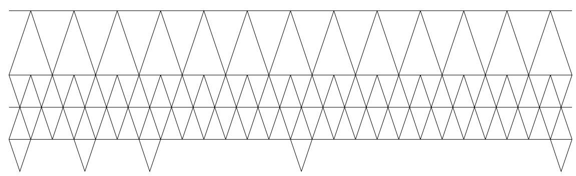


Figure 13f: Case 5a elevation

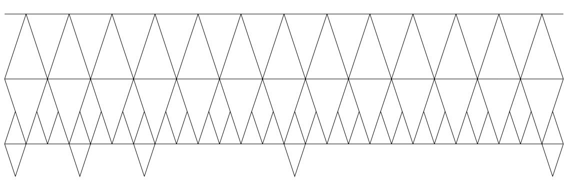


Figure 13g: Case 5b elevation

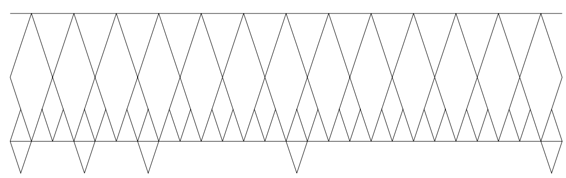


Figure 13h: Case 5c elevation

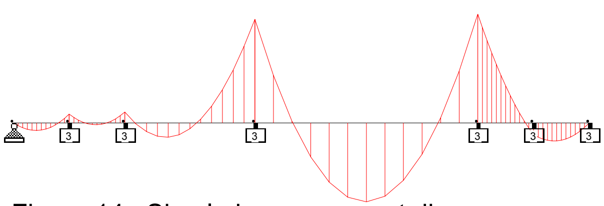


Figure 14: Simple beam moment diagram

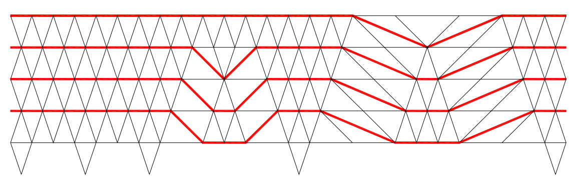


Figure 15: Case 6 elevation
with highlighted tendons

With the elevations set, a STAAD analysis of each case was undertaken. An AutoCAD drawing was imported as a 2D plane model. Pinned support was placed in the lower left corner, while roller supports were placed along the bottom and left edges. All members were assigned the same member section. A W14x53 was chosen as the approximate average member in the original diagrid. Each node was loaded for gravity according to its tributary floor width. Live load reduction was not taken into consideration. In the interest of developing feasible construction techniques, members were grouped by similar function in order to size that particular function only. The system was then analyzed, and output for midspan deflection and member stress was recorded.

It is important to note the inherent inaccuracies of such a model. Because of the simplified loading scheme, systems with larger tributary widths, especially Cases 2 and 4, will seem more inefficient than in actuality. Using the same section properties for all members creates a distribution of forces slightly different than one using final member sizes. The exclusion of lateral wind forces can also affect the results. Additionally, no load was put directly on the horizontal members. This may or may not be the case, depending on how the floors are constructed at levels with horizontals. Even with these shortcomings, this modeling method is justified by its ability to provide relative conclusions rather than absolute approximations.

In order to compare the performance of every case, resulting data from the STAAD models were analyzed with Excel spreadsheets. Each member group was tabulated to find the average and maximum stress for that group. From these stresses the relative weight of every case was obtained. An example of the spreadsheets used to calculate these values is found in Appendix B.2. Summaries of all cases are in Appendix B.3. Table 2 was prepared to directly compare the structural properties of the cases against each other.

	Str. Efficiency	**Redundancy**	**Deflection**
Case	**Weight**	**%**	**in.**
0	42170	71.6	0.029
1	36192	54.4	0.059
2	51648	42.5	0.079
3	33417	53.4	0.044
4	65833	46.0	0.095
5a	40845	64.3	0.037
5b	45110	58.8	0.057
5c	68016	66.3	0.074
6	33176	69.0	0.029

Table 2: Structural properties comparison

In the table above, the relative weight of the structure corresponds directly to its structural efficiency. The lower the weight, the more efficient the configuration. Redundancy is its reserve strength, calculated by dividing the average member stress by the maximum member stress and subtracting from 1. A higher redundancy percentage is desirable. Deflection values at the midpoint of the largest span were taken directly from STAAD. These are not actual deflections because unit loads were used in the model as opposed to real loads.

Architectural Impact

The architectural impact considers the effect of the changed geometry on the building footprint and on its V column placement. The original diagrid has a standard grid width of 9 feet. Although the horizontal members are slightly curved to match the smooth perimeter, the diagonal members cannot economically be modified in the same way. Therefore, any diagonal member of the alternative structures is out of plane with the perimeter. This is barely noticeable for a 9 foot grid width, more visible for an 18 foot grid width, and problematic with a 36 foot grid width. Examples of the effect of grid spacing are shown below in Figure 16. The structure profile is black while the actual smooth perimeter is red.

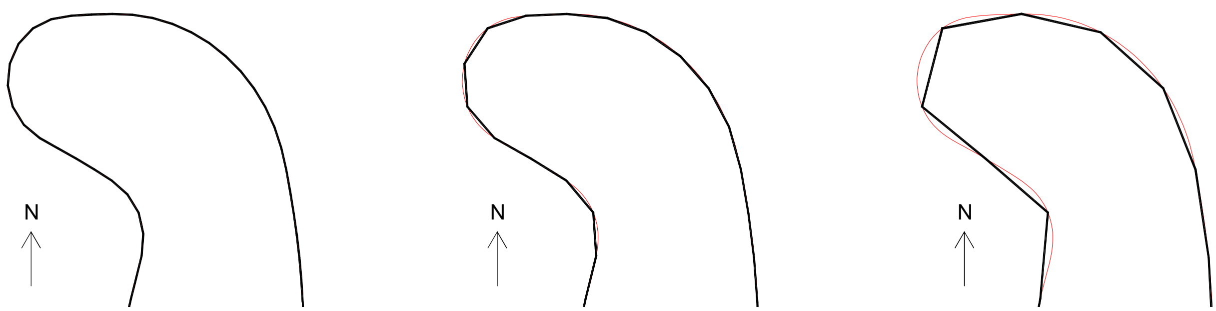


Figure 16: Grid spacing comparison

Cases where the spacing is 36 feet will significantly alter the aesthetic continuity of the Athletic Center design. Spacings of 18 feet will have a few negative effects on the cladding installation and interior layout, but they are not as severe as with 36 feet. Spacings of 9 feet will have little or no impact at all.

As for V column placement, new supports were designed for each case. These supports were evaluated for how well they would perform structurally, how well they fit into the geometric pattern of the building, and how close their bases are to the original base placement. The overall combination of the footprint and support considerations determined the index value for architectural impact (Table 3). The higher the number the better the case fit with the original design intent.

Case	Architecture Index
0	100
1	90
2	75
3	90
4	75
5a	95
5b	95
5c	95
6	90

Table 3: Architecture impact

Floor Framing Impact

The structural reconfigurations of the alternatives present complications to the original floor layout. Because the perimeter diagrid is meant to handle gravity as well as lateral forces, dead and live loads from the interior spaces are transferred through floor beams to the horizontal members of the grid, which then pass these loads to the diagonal members and eventually to the V columns below. The problem which arises from the geometric change to the structure is that some or all of the horizontal members are now removed. Load-carrying floor beams now have no where to frame into, especially in the office bays on the west side of the building. There are two potential solutions to this situation. They are:

1) Change the direction of beam span, allowing the beams to frame into girders attached to the exterior structure.
2) Maintain the span direction, but provide a heavy cross beam which will support the original beams.

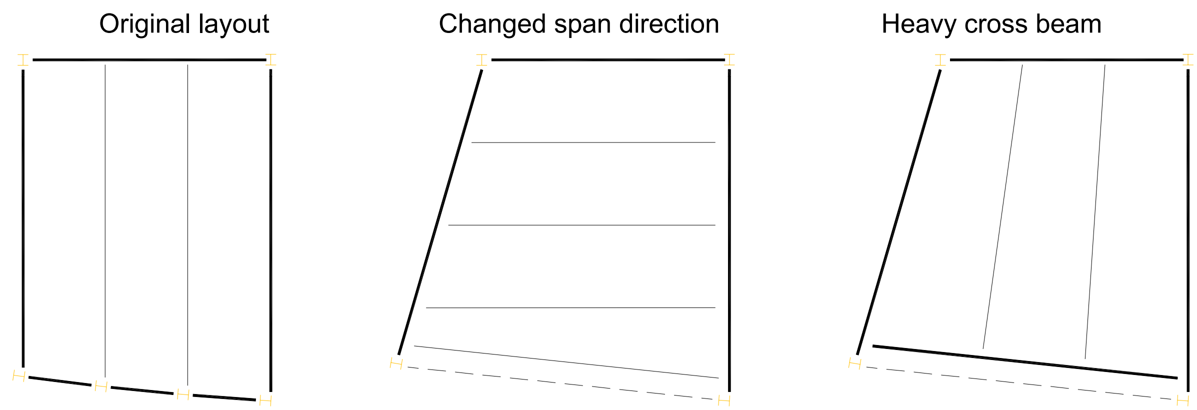


Figure 17: Floor framing schemes

It is apparent from Figure 17 above that the original floor scheme is the simplest, most consistent layout. The other two options create unnecessary steel or inconsistent tributary widths. Therefore any of the cases which require the use of these options do not perform as well as cases with horizontals at every level. The floor framing index (Table 4) takes this idea into consideration and subtracts 10 for every floor which lacks horizontal members. A higher index is desirable.

Case	Floor Framing Index
0	100
1	80
2	70
3	80
4	70
5a	90
5b	80
5c	70
6	100

Table 4: Floor framing impact

Material Cost

An estimate was made that the cost of steel and its related fireproofing, insulation, and cladding is more expensive per square foot than its glazing counterpart. Therefore, each case has been indexed based on its relative structure-to-window percentage (Table 5). The index was taken out of 100. The higher the index value the higher the material cost.

Case	Material Cost Index
0	100
1	80
2	70
3	80
4	70
5a	90
5b	85
5c	80
6	95

Table 5: Material cost

Complexity

Connection material and labor cost can be a significant portion of the overall structure cost of a building. The fewer pieces to join together and the less welds or bolts to secure the less expensive the system. This is especially important for a connection-intense structure such as a diagrid. Any reduction in the number of nodes or the members framing into them would be beneficial. Each case was indexed based on its number of nodes and members (Table 6). A lower index value is more desirable.

Case	Complexity Index
0	100
1	75
2	50
3	75
4	50
5a	85
5b	80
5c	75
6	100

Table 6: Complexity

Results

In order to compare the overall picture between cases, tables were used to collect the individual considerations into a logical scoring system. Table 7 scores the cases for each consideration, and then weights those considerations. Chart 2 calculates the final score, while Table 8 ranks each case from 1 through 9.

	Str. Eff.		Redundancy		Deflection		Architecture		Flr. Framing		Mat. Cost		Complexity	
Case	**Weight**	**Score**	**%**	**Score**	**in.**	**Score**	**Index**	**Score**	**Index**	**Score**	**Index**	**Score**	**Index**	**Score**
0	42170	0.79	71.6	1.00	0.029	1.00	100	1.00	100	1.00	100	0.70	100	0.50
1	36192	0.92	54.4	0.76	0.059	0.49	90	0.90	80	0.80	80	0.88	75	0.67
2	51648	0.64	42.5	0.59	0.079	0.37	75	0.75	70	0.70	70	1.00	50	1.00
3	33417	0.99	53.4	0.75	0.044	0.66	90	0.90	80	0.80	80	0.88	75	0.67
4	65833	0.50	46.0	0.64	0.095	0.31	75	0.75	70	0.70	70	1.00	50	1.00
5a	40845	0.81	64.3	0.90	0.037	0.78	95	0.95	90	0.90	90	0.78	85	0.59
5b	45110	0.74	58.8	0.82	0.057	0.51	95	0.95	80	0.80	85	0.82	80	0.63
5c	68016	0.49	66.3	0.93	0.074	0.39	95	0.95	70	0.70	80	0.88	75	0.67
6	33176	1.00	69.0	0.96	0.029	1.00	90	0.90	100	1.00	95	0.74	100	0.50
Weight		1.0		0.8		0.8		0.7		0.3		0.5		0.4

Table 7: Alternative diagrid scores

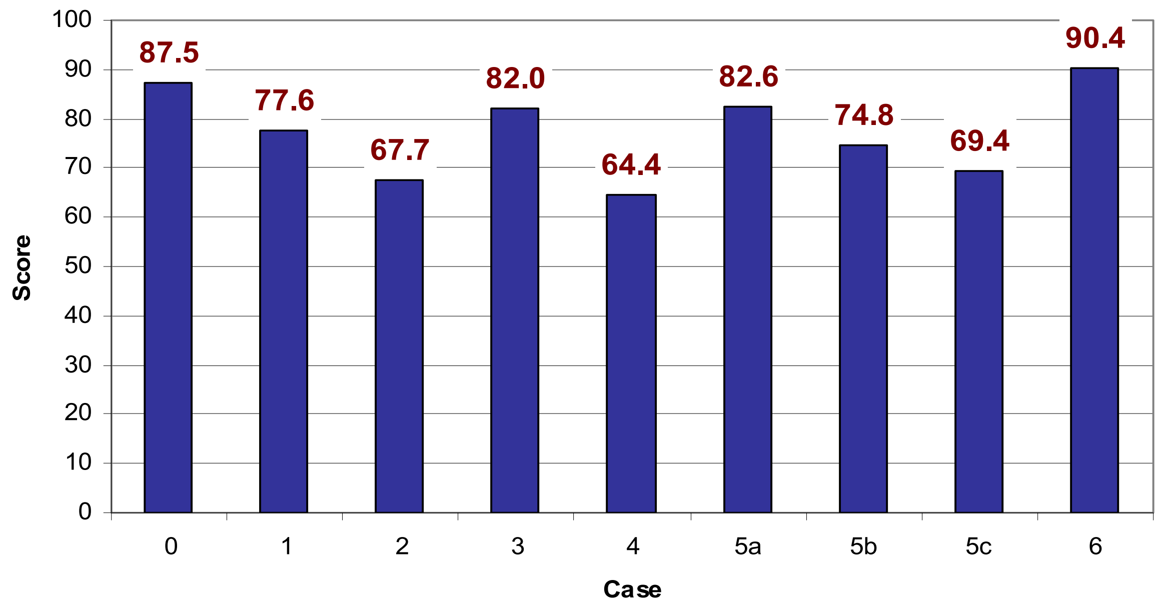


Chart 2: Alternative diagrid final scores

Case	Rank
0	2
1	5
2	8
3	4
4	9
5a	3
5b	6
5c	7
6	1

Table 8: Alternative diagrid rankings

Discussion

A few interesting observations and conclusions can be inferred from the results of the structural analysis and the final scores.

1) Varying member lengths and heights has substantial impact on structural efficiency.

Alternatives with medium height members (Cases 1 and 3) weigh considerably less than single story alternatives (Cases 0 and 5a). The longer lengths of their members are optimized to the loading conditions they must carry. Continuing this trend, it would be expected that alternatives with large height members (Cases 2 and 4) weigh even less. However, this is clearly not true. In fact, the system weights of the large member cases are nearly twice as much as the medium cases. This is because the long unbraced lengths of Cases 2 and 4 cause column buckling failure to occur well before material yielding. It must be noted that the floor slabs at each level could potentially provide lateral support for the members, especially in Cases 3 and 4 which have vertical truss elements. This bracing would certainly help reduce member sizes, making large member cases more efficient than what is represented above.

2) In general, there is a noticeable tradeoff between architectural impact and cost.

The configurations which minimize floor plan and floor framing impact tend to have higher material and complexity scores. Conversely, configurations which create large bays and column spacings (Cases 2 and 4) also reduce cladding, insulation, and labor costs. This tradeoff virtually negates any of the scores from the four leftmost columns in Table 7. Because of this, final scores, and ultimately system selection, are primarily dependent on structural performance considerations.

3) High system redundancy helps control deflection.

There is one exception to this general observation. Case 5c has relatively high redundancy, yet it performs poorly in deflection. This is due to its inefficient diamond-shaped configuration. Without the cross beams to tie opposite corners of the diamond together it becomes unstable. This is clearly shown in a deflection diagram from STAAD (Figure 18).

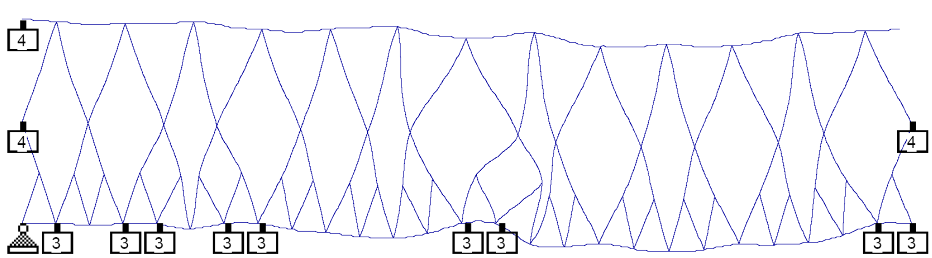


Figure 18: Case 5c deflection

Overall, Case 6 is the best geometric configuration out of the proposed alternatives. The combination of small bay spacing and floor beams at each level keep architectural impact to a minimum. Structurally, its "tendons" over the long spans are highly effective in limiting midspan deflection, all while providing adequate redundancy and weighing less than every system. It outperforms the other options in many categories.

Even with its advantages, Case 6 is only marginally better than the original diagrid system. Though the alternative may result in nearly a 25% reduction of steel weight, all the other criteria for evaluation do not justify a replacement of the diagrid system. Its score is simply too close to the original system score.

Therefore, an entirely new approach to the structural redesign of the Athletic Center is necessary. The concepts which have been exploited so far should be taken further. A more unique treatment of the diagrid must be addressed.

Solution Area III – Removing the diagrid to bring the lateral system inwards

The conclusions of the previous two sections made it quite clear that another approach was necessary to obtain a valid redesign of the Athletic Center. Changing only the material was not advantageous at all. Modifying the diagrid geometry had benefits, but not enough to substantiate a redesign. A whole new design approach, one which eliminates the concept of a diagrid, would have to be used. In this way the lateral system would be moved from the perimeter inward, and a new gravity system would need to be developed to replace the diagrid. A curtain wall would become the new vertical building envelope. Rethinking the entire above-grade load carrying structure would be considerably more difficult than either Solution Area I or Solution Area II.

The primary goals of the diagrid removal were as follows:

- Change the façade's architectural look and feel. In an interview with Charles Thornton, the Athletic Center's architect, Bernard Tschumi, leans toward a theoretical view of not expressing or exposing structure if it is not required (Thornton, 73). Though the original design expresses the structural system of the Athletic Center, it is out of necessity rather than intent. In keeping with Tschumi's perspective on expressed structure, the aesthetic look of the building was intentionally modified to hide the structure. This would be relatively easy due to the ability to add a curtain wall around the perimeter.
- Keep the shape of the building intact. Changes to the plan would mean changes to space layout and programming characteristics.
- Reduce structure weight and complexity. Structural efficiency remained a key indication of the feasibility of a redesigned system.
- Provide as much glazing opportunity as possible. As indicated in the problem statement, very little of the usable window viewing height is glazed. Opening the façade to views of the football stadium and surrounding landscape would be desirable.
- Minimize impact to the interior layout. The redesigned structure should not have major negative effects on the amount or quality of interior space.
- Maintain floor height. Added floor-to-floor height generally equates to added cost.

Several additional considerations were taken into account:

- Placement of columns. According to architect David Zelman, existing spaces from the adjacent facilities could not be compromised by the construction of the Athletic Center (para. 4). The diagrid was developed to span over these spaces. It successfully allowed only one column to be brought through the existing space. More than that would have forced closure of Shoemaker Arena. The redesigned system had to solve the same problem, maintaining the relative positions of Level 500 support columns.
- Penetration of open spaces. There are several vertical openings such as atriums, stairwells, and elevators through which beams cannot pass.
- Lateral system placement. Location of any braced or moment frames must be invisible to the occupants.
- Floor system impact. Because the floor system originally framed into the perimeter diagrid, it would be beneficial to require the new perimeter to allow similar framing opportunities.
- Foundation impact. Gravity and lateral load flow should follow a path as in the original system in order to retain foundation design applicability.

Procedure

The redesign of the structural system followed the same development phases as a typical new construction project: Conceptual Design, Schematic Design, Design Development, and Construction Documents.

Conceptual Design
Initially, conceptual ideas of potential gravity-carrying structural systems were drawn up. These drawings are shown below in Figures 19a through 19f. The building outline is blue, the proposed structure is yellow, and the main supports are represented by maroon pyramids. The light blue line in Figures 19b and 19c represents the edge of existing facilities below the Athletic Center. No columns except for the existing perimeter column may extend past this line.

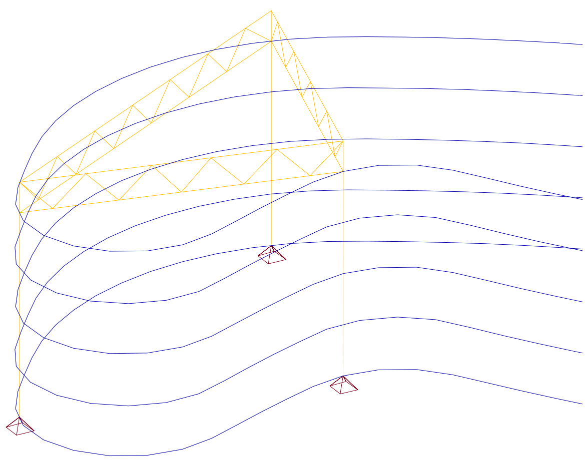


Figure 19a: Interior hat truss

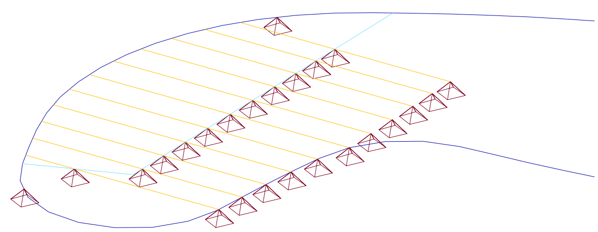


Figure 19b: Cantilevers over columns

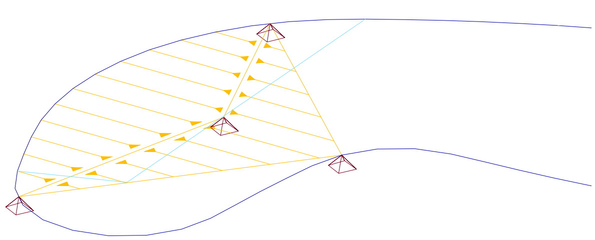


Figure 19c: Cantilevers over girders

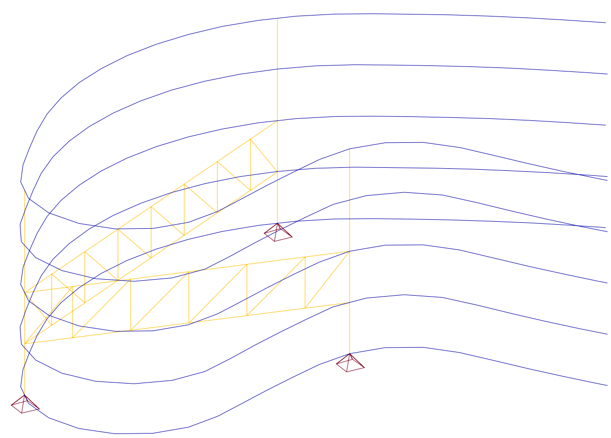


Figure 19d: Level 600 truss

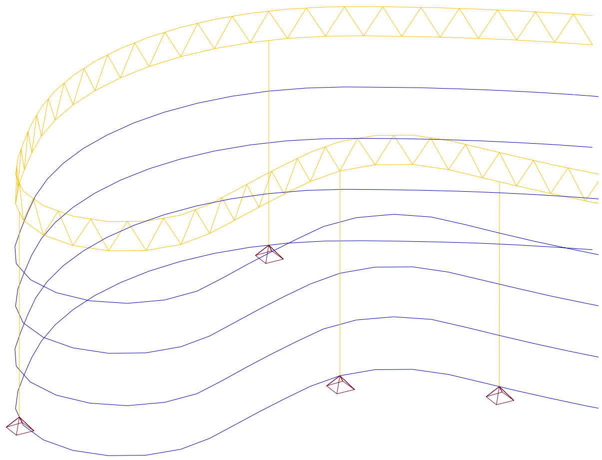


Figure 19e: Perimeter truss

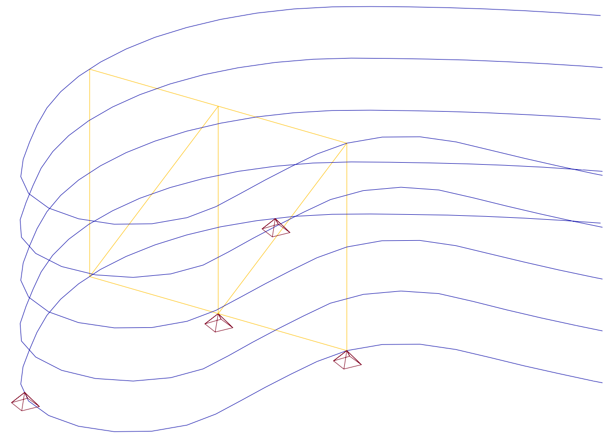


Figure 19f: Reverse truss

A pro-con comparison was made between all of the alternative gravity systems. Each option was evaluated for advantages and disadvantages based on the goals and considerations outlined previously. They were then subjectively ranked from 1 to 6. The results of the study are below in Table 9.

Option	Advantages	Disadvantages	Rating
Interior Hat Truss	Hidden, flexible, can be applied over whole building	Small cantilevers remain, construction sequence will be an issue, truss will add depth to total height, some openings may need to be adjusted, tensile columns	**2**
Cantilevers Over Columns	Invisible structure, no height increases	Backpinning will be a major issue, no columns can be put through auditorium	**6**
Cantilevers Over Girders	Hidden, flexible, no height increases	Floor layout will have to be changed drastically, downward slant through auditorium will be extremely hard to negotiate, open space prevents girder from reaching columns	**5**
Level 600 Truss	Truss can be deep and efficient through mechanical room	Truss will interfere with some mechanical equipment, layout of some public space will have to be replanned, combination of tensile and compression columns	**4**
Perimeter Truss	Out of the way of the rest of the building, very efficient, can be applied over whole building	Height increase, construction sequence will be an issue, tensile columns	**1**
Reverse Truss	Provides both gravity and lateral stiffness, fairly efficient, no height increase	Not flexible, diagonals will interfere with spaces and atrium layout will have to change	**3**

Table 9: Gravity system comparison

The perimeter truss option seemed to be the least disruptive to the floor layout and open spaces of the Athletic Center, unlike the majority of the other options. However, it still had several issues which needed to be investigated and resolved.

Next, ideas for the lateral system were evaluated. There were fewer options available, but the three main types of systems were evaluated, braced frames, moment frames, and shear walls. A pro-con comparison was also done for the lateral systems, summarized in Table 10 below. Braced frames were chosen as the most viable lateral option, due to their relatively easy incorporation into the existing floor/column scheme, and their ability to be placed at several locations throughout the building

Option	Advantages	Disadvantages	Rating
Braced Frames	Braced frames from Level 100-500 are already in place, no impact on floor-to-floor height, less labor than rigid frame	Reduces usable interior space, placement will be a slight issue	1
Moment Frames	Maintains interior spaces, potentially less steel weight	Predominant grid system is not available to develop sufficient frame action, potentially deeper beams	3
Shear Walls	No impact on floor-to-floor height	Heavier loads on foundation, reduces usable interior space, placement will be an issue, introduces concrete construction on site	2

Table 10: Lateral system comparison

Schematic Design

The systems chosen during the conceptual phase were developed further. Several issues were resolved to further refine the design. These issues were:

1) Floor beam sweep
   The original design called for horizontal members of the diagrid to be slightly curved. Perpendicular floor members framing into the curve cause torsion, which is transferred as moment at the supporting column connections (Figure 20). This moment was undesirable; however, the 9 foot spans of the original design were short enough to consider torsion negligible. In order to open up the façade and eliminate some of the columns, it was necessary to work with longer spans. These spans would cause an unacceptable amount of torsion on 18' or 27' beams. Therefore, the beams were designed to connect straight between the columns, eliminating the effects of torsion.

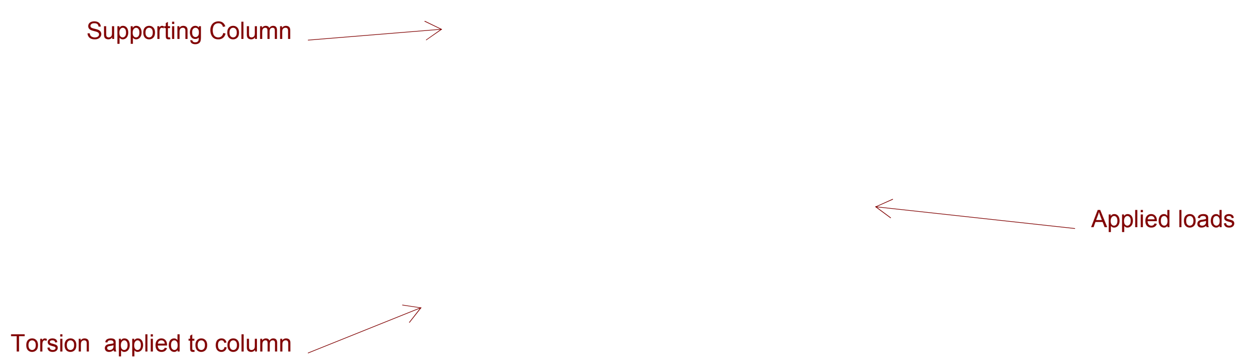


Figure 20: Curved beam torsion

2) Column spacing
The decision to design each floor beam as a straight member brings back consideration of how well the building follows the smooth perimeter outline. Unlike the diagrid geometry alternatives discussed in Solution Area II, the perimeter truss will actually allow column spacing to vary along the perimeter. Naturally, spacing will be 9' in sections of high curvature and 27' in section of low curvature. The column layout is shown below in Figure 21, with small pink dots representing columns in tension and large red dots representing columns in compression.

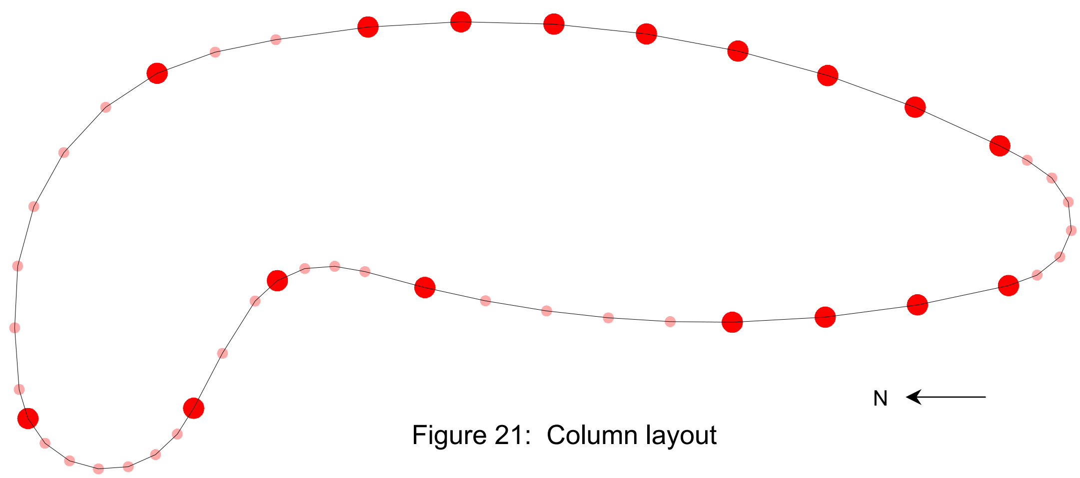


Figure 21: Column layout

3) Pinned vs. fixed connections
Because load is now being carried primarily by the perimeter truss and braced frames, minimizing additional moments on secondary members would reduce sizes while maintaining adequate strength. Therefore, pinned connections (shown in Figure 22 as red circles) were used at all floor beam connections and at the tops of columns, both in tension and compression. Rigid connections are maintained for column continuity and the perimeter truss.

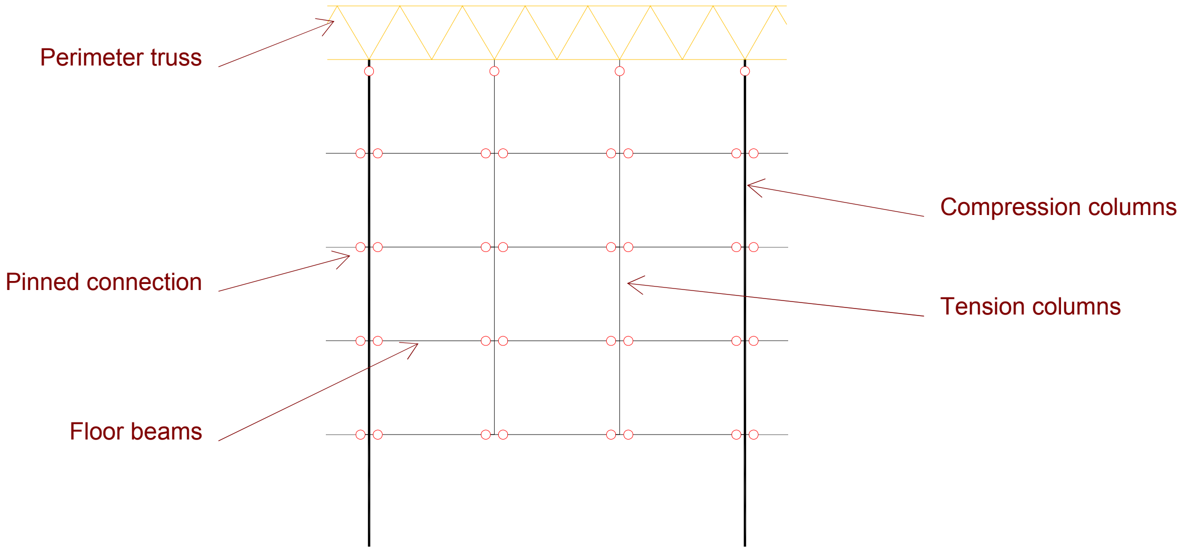


Figure 22: Pinned connection schematic

4) Column deformation compatibility
The differences in load deformation between columns in tension and compression (Figure 23) could have substantial drawbacks to the curtain wall structure. A solution to this situation would be to carefully sequence the construction of the curtain wall. A detailed evaluation of column deformation compatibility will be addressed later in the Construction Study section.

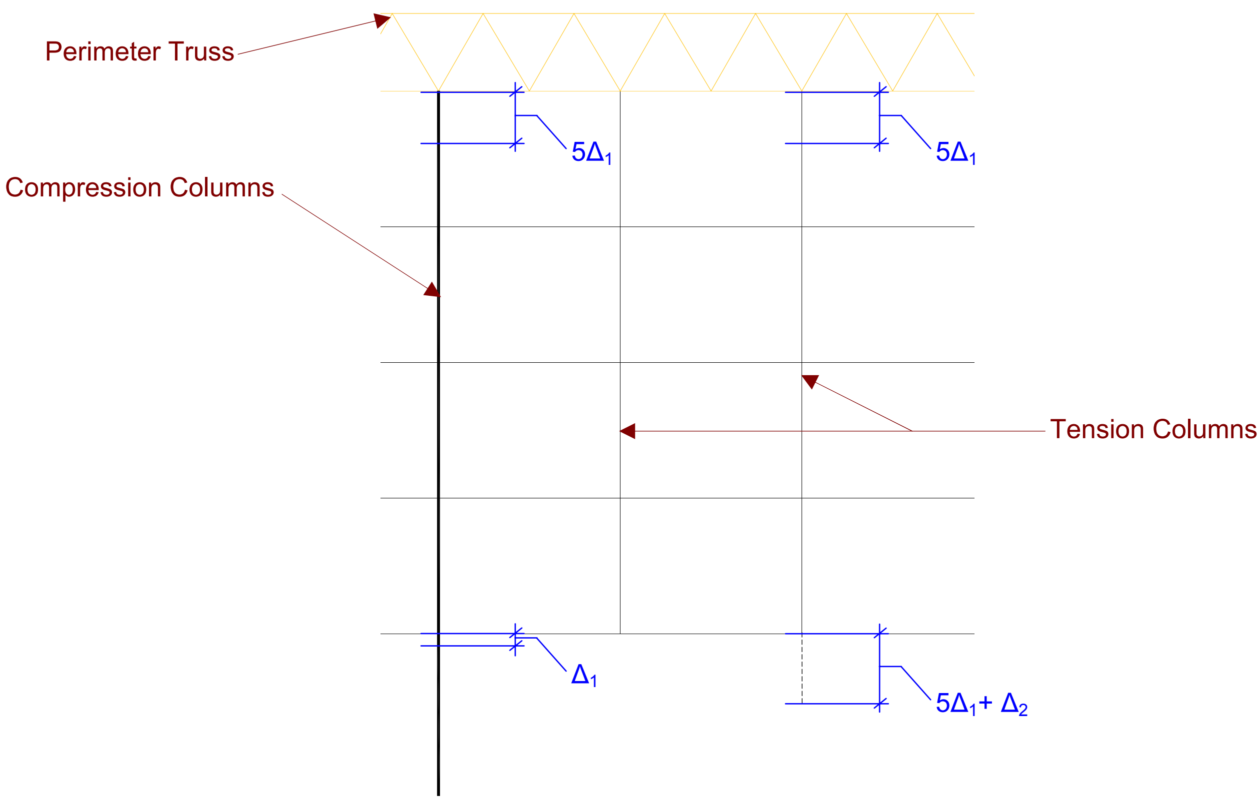


Figure 23: Column deformation

5) Fire Resistance of perimeter truss.
Because the Athletic Center contains A-3, B, and M occupancy types, it is categorized by Table 503 of the Ohio Basic Building Code as a Type 1B construction class. For structural frame elements the fire-resistance rating requirement is 2 hours. This can be obtained by using a spray-applied fire resistive material. The thickness of this fireproofing in the original diagrid system is 2", therefore the same minimum thickness is specified for the new perimeter truss.

6) Thermal movement and stresses
Though the original diagrid design was certainly expressed, the steel structure was insulated behind 3 inches of expanded foam insulation, protecting it from temperature extremes. If left unprotected, the perimeter truss would encounter considerable thermal variation due to sol-air effects and night sky radiation. In order to mitigate detrimental thermal movement, it was decided to insulate the perimeter truss using 3-4 inch thick rigid insulation. The architectural desire to hide the structure will conceal the insulated truss from public view. See Figure 24.

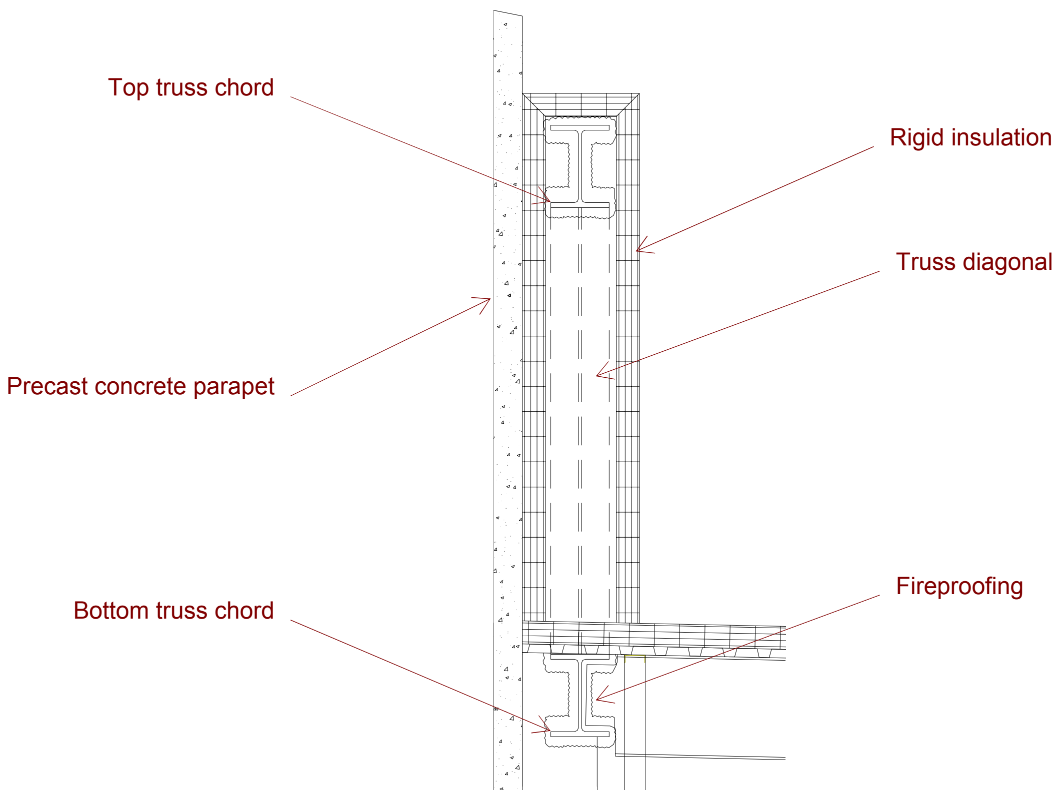


Figure 24: Perimeter truss insulation

7) Truss height
Bay width was set at 9 feet to be consistent with the column layout and spacing. Three options for height were conceived (Figure 25). The first, 4'-6" high, produced 45˚ diagonals. The second, 7'-8" high, created an equilateral triangle pattern. The third, 9'-0" high, matched the width. After consideration of all three, the second option was chosen for its sufficient depth and reasonable member lengths.

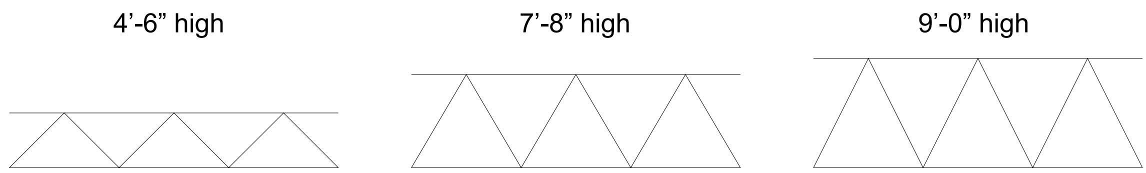


Figure 25: Truss height options

8) Truss lateral bracing
Because the truss acts as a deep beam, its chords undergo considerable compressive forces in some sections. They must be braced to prevent buckling failure. The bottom chord is automatically braced by the roof structure at Level 900. The top chord, however, was designed with angled wide flange braces oriented perpendicular to the perimeter to resist lateral truss movement. Figure 26 illustrates this bracing scheme.

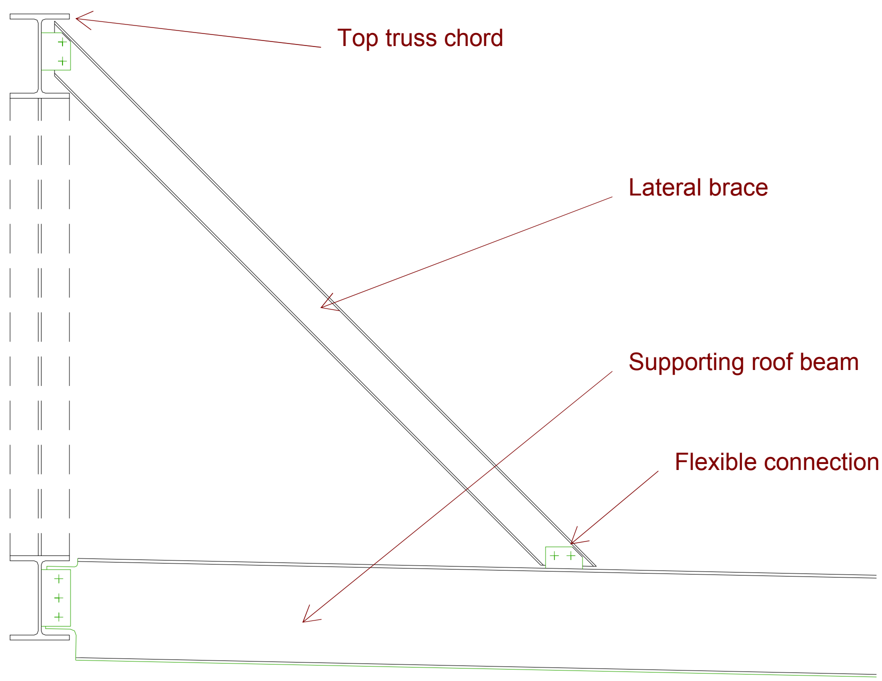


Figure 26: Truss lateral bracing

9) Corrosion
The addition of lateral truss bracing brought up the issue of weather protection for exposed steel. Though the truss will be covered with insulation and a water resistant membrane, the brace pieces are left exposed. Encasing them in a protective would likely be expensive and prone to damage, therefore the braces and their connections will have a corrosion-resistant paint applied after installation.

10) Braced frame placement

The original braced frames, labeled BF1 and BF4, extended from the foundation slab up to Level 500 and were oriented in the East-West direction only (Figure 27a). Their purpose was to transfer East-West lateral loads from the bottom of the diagrid to the foundation walls, while the V columns picked up the North-South load. The new braced frames, designed to carry all lateral load from the roof (Level 900) to the foundation, had to penetrate into previously unobstructed interior space. To minimize the impact of the new braced frames, upper level framing in the East-West direction was continued on top of existing framing. Additionally, BF4 was relocated from grid line D to grid line C, in order to take advantage of existing mechanical chases on Levels 500-800. Finally, new North-South braced frames were added above the elevator shaft on grid line 1 and along the central West stairwell on grid line 2 (Figure 27b).

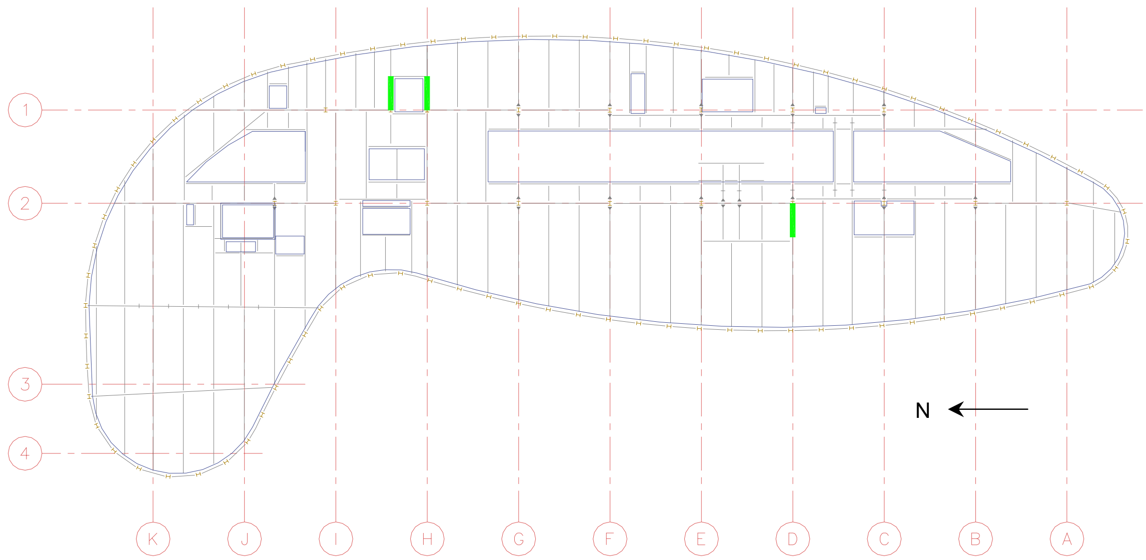


Figure 27a: Original braced frame positions

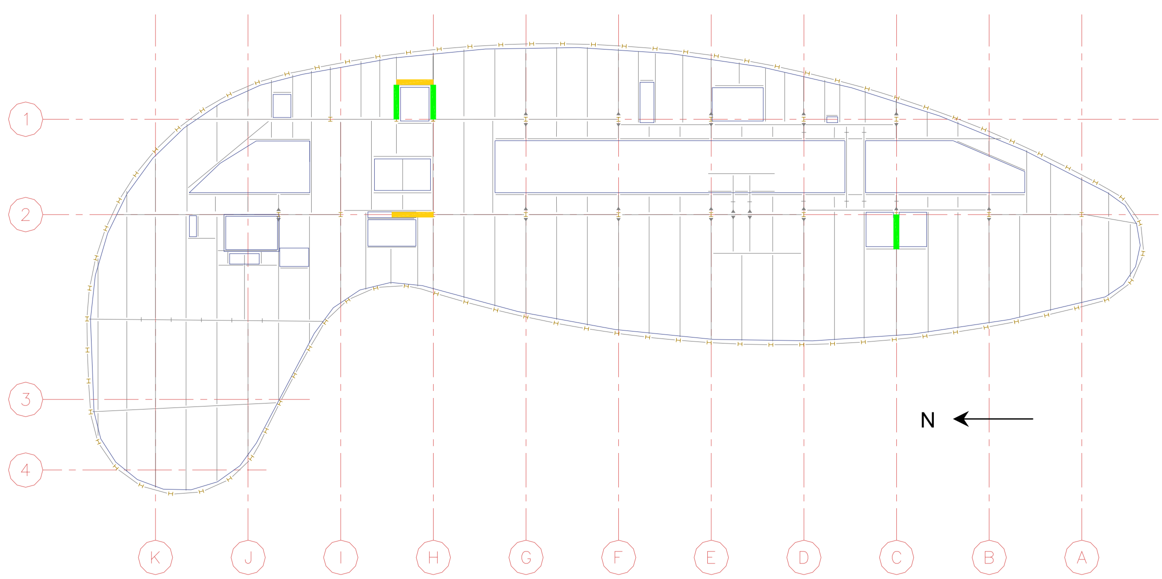


Figure 27b: New braced frame positions

Though these new frames were positioned to be nearly invisible to the interior space layout, they still impacted lower levels of the Athletic Center. It was necessary to assess the layout of the affected spaces. Of primary concern was the new N-S frame above the central West stairwell. The southern column was already there in the original design. The northern column was offset 8'-0" from this grid (Figure 28a). Diagonal members between these columns upset the mechanical ducts adjacent to the stairwell on Levels 400-800. This was resolved by moving the ducts just past the northern column (Figure 28b). At Level 300, the new column and bracing do not interfere with any other structure or equipment; however the frame will need to be covered up for aesthetic reasons. A false wall between the doors accomplishes this (Figures 29a and 29b). At Level 200, the brace cuts right into circulation space. A wall was designed around the brace, which protects the structure while providing storage space for the adjacent Football Meeting Room (Figures 30a and 30b). Level 100 contains a recycling area, which has been moved to another location. In the process more closet space was created for the nearby room (Figures 31a and 31b).

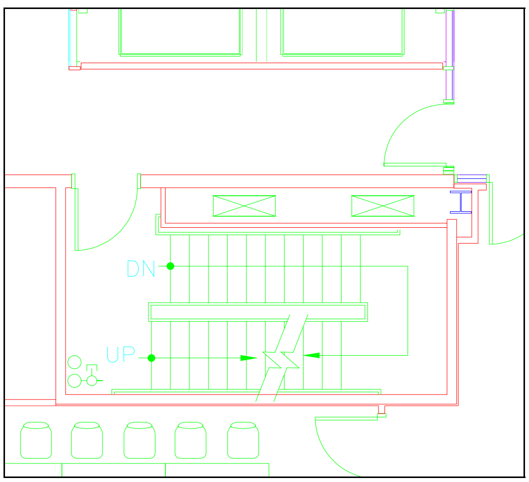


Figure 28a: Level 500 original layout

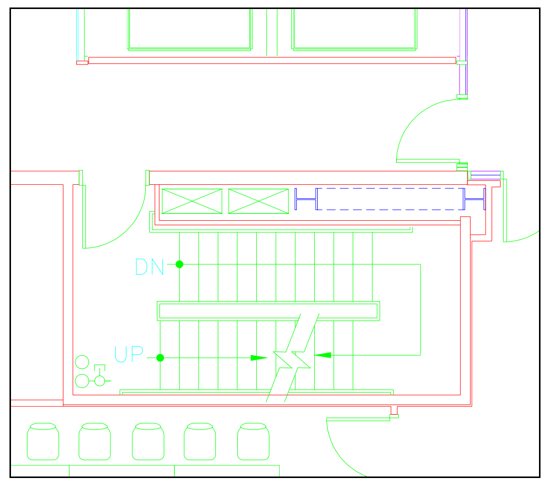


Figure 28b: Level 500 modified layout

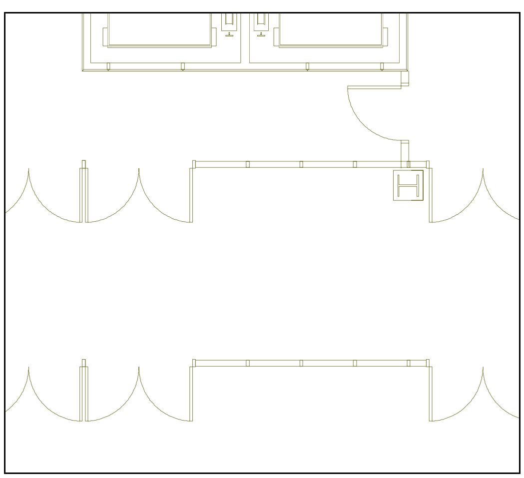


Figure 29a: Level 300 original layout

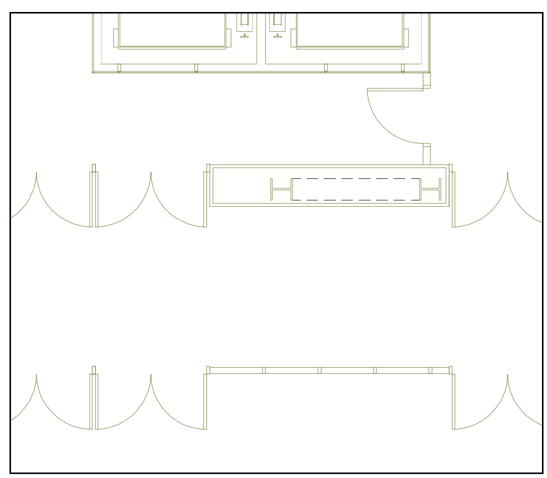


Figure 29b: Level 300 modified layout

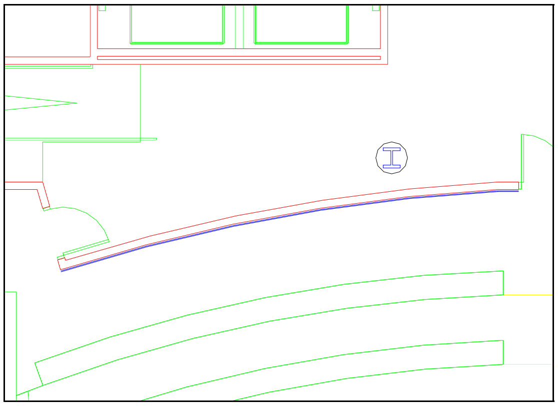


Figure 30a: Level 200 original layout

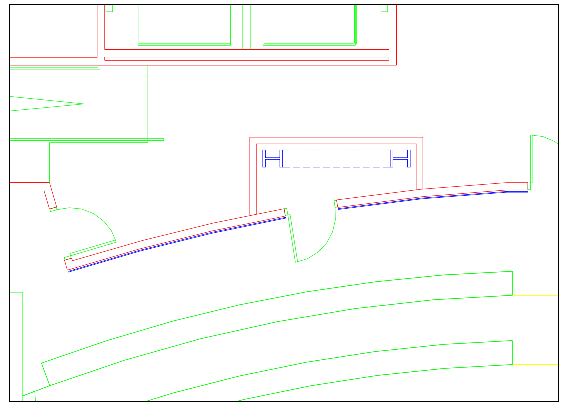


Figure 30b: Level 200 modified layout

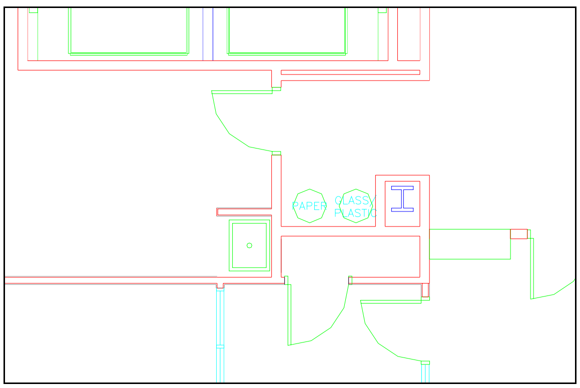


Figure 31a: Level 100 original layout

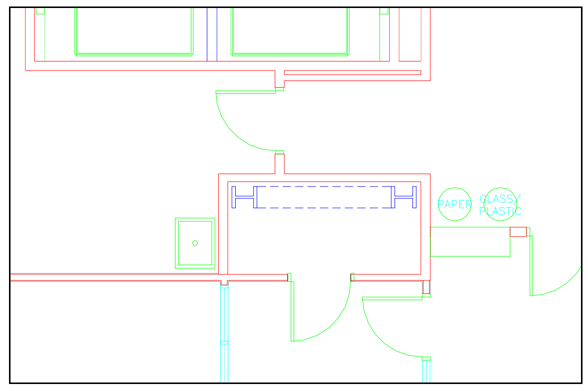


Figure 31b: Level 100 modified layout

Design Development

Once the main Schematic Design issues had been worked out, an actual structural analysis of the perimeter truss system could be performed to find member sizes, long span deflections, and story drifts. Though the building could have been simplified to undertake hand calculations, it was concluded that a more accurate 3-dimensional computer model was the best method of analysis. The Athletic Center was modeled in ETABS, a non-linear, finite element pre and post-processing software package written specifically for structural analysis of buildings.

The model skeleton including the perimeter truss, its columns, cross beams, and the braced frames was first constructed in AutoCAD. It was imported into ETABS as a fully rigid structure. Pinned members were released according to schematic design consideration #3 and base supports were added. Rigid diaphragms were assigned to Levels 500-800 to simulate the composite beam/composite deck action. Lateral bracing for the truss, as specified in schematic consideration #7, was modeled as supports released tangentially and vertically. Loads were added to model dead, live, and wind cases. Load calculations are found in Appendix C.1. Initial member sizes were created based on educated assumptions. Figure 32 is a three-dimensional view of the model.

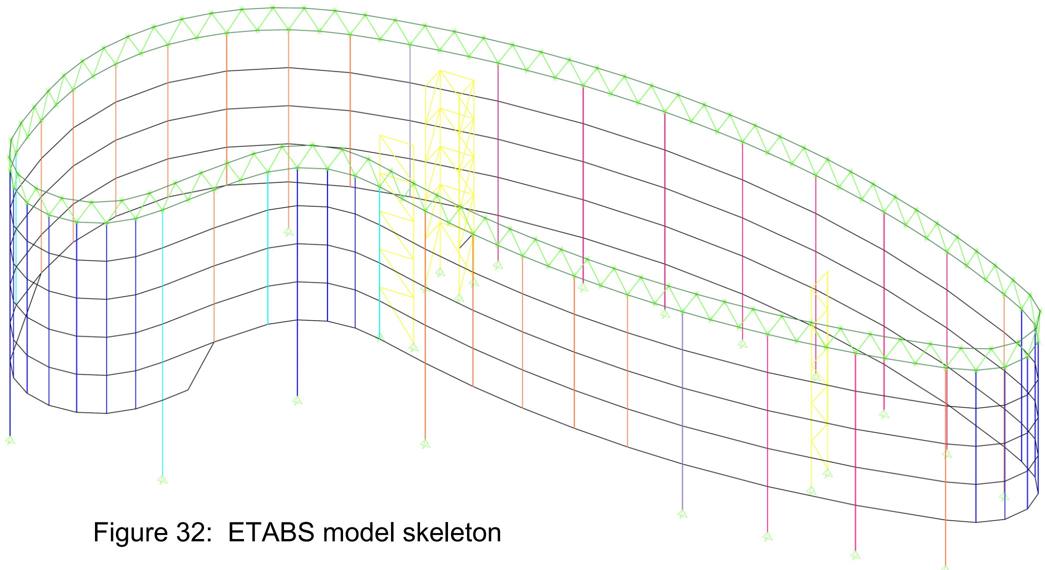


Figure 32: ETABS model skeleton

Once an analysis was performed, force output from ETABS was separated into five member categories: truss horizontals, truss diagonals, truss columns, braced frame diagonals, and braced frame columns. This output was used to find acceptable member sizes for both the perimeter truss system and braced frames with the help of a spreadsheet which applies the proper interaction equation. An example of this spreadsheet can be found in Appendix C.2. The process then became iterative. The new member sizes were inputted into the model, an analysis was performed, the results were compiled, members were resized based on strength or serviceability, and the cycle began again until all criteria were met. Rather than detailing each step of the process, a summary of the iterations, or trials, is provided in Tables 11a and 11b.

Trial #	Perimeter Truss System
1	Initial conditions. Truss sizes all W18x86. Column and BF sizes all W14x53.
2	Resized members for strength based on Trial 1
3	Strength criteria not met for some members. Resized based on Trial 2
4	Strength criteria met. Deflections are horrible. Main column deflections are as much as 2.24" vertically at top of column. Displacement at midpoint of 107' span is nearly 6.62" when allowable is 3.57". Unacceptable. Resized truss and column members.
5	Deflections still bad. Resized columns and increased long span truss sections based on virtual work diagrams.
6	Deflections still unacceptable. Increased truss horizontal "flanges" just over the main compression columns in areas of high negative moment.
7	Realized that factored loads were being used for deflection results. Scaled back gravity loads to represent service load levels.
8	Acceptable deflections from Trial #7. Not necessary.

Table 11a: Perimeter truss system trials

Trial #	Braced Frames
1	Initial conditions. Member sizes estimated from existing braced frames.
2	Strength criteria met. Story and overall drifts are unacceptable. Resized diagonals.

3	Story and overall drifts still bad. Increased Level 400 and 500 members based on virtual work diagrams.
4	Not much better. Realized that factored loads were used rather than service loads. Scaled back load cases.
5	Acceptable drifts. Decreased some overdesigned members because of the service load mistake.
6	Acceptable drifts from Trial #5. Not necessary.

Table 11b: Braced frame trials

The above tables make reference to "virtual work diagrams." These diagrams were displayed in ETABS to show the relative virtual work that each member contributed. The diagrams, an example of which is shown in Figures 33a and 33b, helped identify general areas where increasing member size would be most beneficial to limiting deflections or drifts.

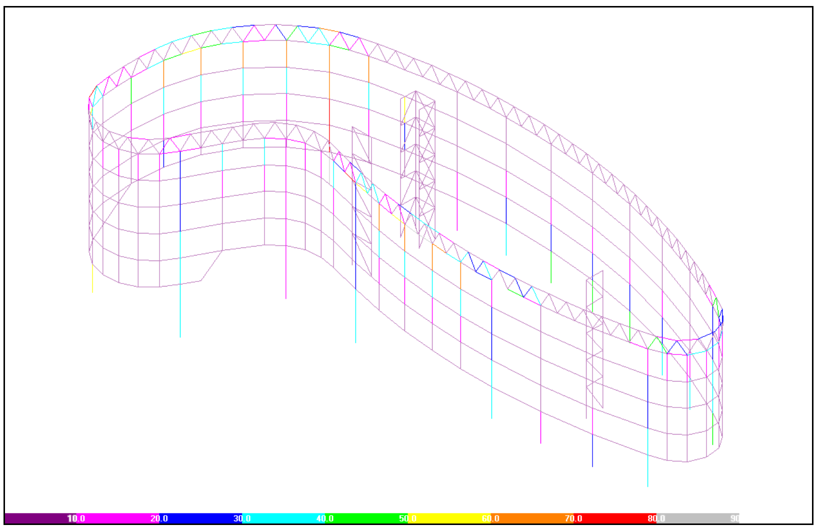


Figure 33a: Perimeter truss virtual work

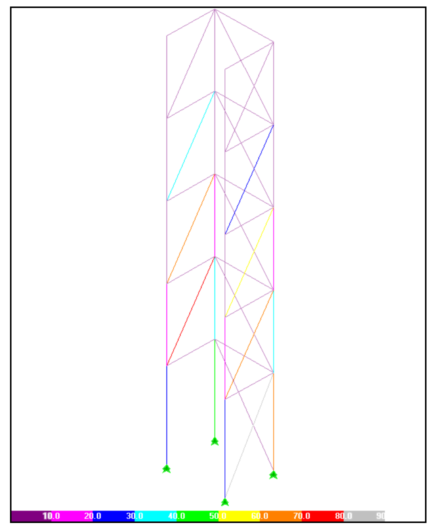


Figure 33b: Braced frame virtual work

To check deflection and drift outputs against allowable values stipulated by the building code, a series of spreadsheet calculations were employed. One set of spreadsheets analyzed gravity deflections of the long spans around the perimeter (Appendix C.3), while one analyzed diaphragm story drift at each level (Appendix C.4). Summaries of these spreadsheets are shown in Tables 12 through 14.

	Midspan Deflection (inches)						
Trial #	**41' span**	**90' span**	**43' span**	**45' span**	**48' span**	**107' span**	**63' span**
4	-1.74	-3.39	-0.64	-0.89	-1.11	-4.52	-1.26
5	-1.58	-2.83	-0.70	-0.89	-1.11	-3.73	-1.21
6	-1.24	-2.37	-0.72	-0.89	-1.08	-3.41	-1.18
7	-1.00	-1.91	-0.58	-0.72	-0.87	-2.75	-0.95
Allowable	-1.37	-3.00	-1.43	-1.50	-1.60	-3.57	-2.10

Key -	Failed	Questionable	Acceptable

Allowable Deflection

L/350 = Span*12/350

Allowable Drift

Total: H/400 =

Table 12: Allowable long span gravity deflection comparison

East-West Direction

	Story Drift (inches)					
	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5	Acceptable
Level 900	0.65	0.61	0.54	0.42	0.42	0.46
Level 800	0.60	0.57	0.52	0.38	0.38	0.46
Level 700	0.60	0.56	0.50	0.34	0.34	0.46
Level 600	0.54	0.49	0.42	0.26	0.26	0.46
Level 500	0.38	0.35	0.29	0.18	0.17	0.55
Total Drift	2.78	2.58	2.27	1.57	1.58	2.09

Table 13: Allowable East-West drift comparison

North-South Direction

	Story Drift (inches)					
	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5	Acceptable
Level 900	0.47	0.44	0.40	0.29	0.32	0.46
Level 800	0.39	0.35	0.32	0.25	0.28	0.46
Level 700	0.41	0.37	0.34	0.23	0.26	0.46
Level 600	0.41	0.35	0.30	0.18	0.22	0.46
Level 500	0.22	0.19	0.15	0.09	0.12	0.55
Total Drift	1.90	1.70	1.50	1.04	1.20	2.09

Table 14: Allowable North-South drift comparison

Table 12 shows that midspan gravity deflections for Trial 7 satisfy all allowable deflection limits and are therefore acceptable. Tables 13 and 14 indicate that Trial 5 satisfies the allowable total and story drifts in both the North-South and East-West directions, though story drift for Levels 800 and 900 are close to the allowable limit.

Construction Documents

Final checks were made for foundation overturning and unbalanced wind load torsion.

- Foundation overturning

  Using unfactored wind loads from Appendix C.1 and total dead weights from Appendix C.5, overturning and resisting moments were calculated. Overturning was checked in the East-West direction only under the assumption it is the critical case. Summaries of the overturning and resisting moment calculations are given in Tables 15 and 16, respectively.

Level	Windward Pres. plf	Leeward Pres. plf	Trib width ft	Height ft	Moment ft-kips
900	217	132	300	70	7330
800	191	123	300	56.5	5316

700	178	123	300	43	3881
600	162	123	300	29.5	2521
500	158	143	300	16	1444
Sums					20493

Table 15: Overturning moment calculations

Level	Superimposed kips	Superstructure kips	Total kips	2/3 Total kips	Base Dist ft	Moment ft-kips
900	1973	542	2515	1677	40	67075
800	2084	291	2375	1583	40	63336
700	2100	291	2390	1594	40	63742
600	2361	291	2652	1768	40	70725
500	2209	286	2495	1663	40	66525
Sums			12428	8285		331404

Table 16: Resisting moment calculations

Even with the conservative assumption to use two-thirds dead load, the total resistive moment was much higher than the overturning wind moment. Therefore overturning is not an issue. This makes sense because the relatively low building height does not provide enough overturning moment to overcome the wide base of the perimeter frame.

- Torsion

As required by the Ohio Basic Building Code, a wind loading eccentricity of 5% was set up in the ETABS model to account for unbalanced loading conditions. The resulting displacement outputs were analyzed to find the maximum points of drift for each level under East-West and North-South wind loads. As shown in Tables 17 and 18 below, both the East-West and North-South conditions are satisfactory, though Levels 800 and 900 are very close to the allowable story drift.

East-West Wind

Level	Point	UX	Delta X	Allow.
900	70	1.822	0.458	0.46
800	70	1.3638	0.457	0.46
700	70	0.9067	0.402	0.46
600	70	0.5051	0.299	0.46
500	70	0.2057	0.206	0.55
		Total =	1.822	2.09

Table 17: Maximum East-West story drifts with torsion

North-South Wind

Level	Point	UY	Delta Y	Allow.
900	64	1.7259	0.396	0.46
800	64	1.3303	0.406	0.46
700	64	0.924	0.366	0.46
600	64	0.558	0.297	0.46
500	64	0.2607	0.261	0.55
		Total =	1.726	2.09

Table 18: Maximum North-South story drifts with torsion

With the checks completed, perimeter truss and braced frame member sizes were finalized. These sizes are found in Appendix C.6.

Results

Once the models were completed and checked, calculations were performed to size members, estimate weights, and compare systems. A steel take-off of the perimeter truss and braced frame system was carried out, followed by a steel take-off for the original diagrid system.

Before every trial, a spreadsheet recorded the sizes and lengths of each member in the perimeter truss and braced frame system. An example of this spreadsheet is found in Appendix C.7. In all trials, member sizes were repeated as much as possible to promote economy in fabrication. This data was then used to estimate the weight of steel being used per iteration. Tables 19 and 20 summarize the weights for the perimeter truss and the braced frames.

	Weight (tons)						
Member Group	**Trial #1**	**Trial #2**	**Trial #3**	**Trial #4**	**Trial #5**	**Trial #6**	**Trial #7**
Truss Horizontals	39.1	47.6	57.2	59.9	79.9	85.2	85.2
Truss Diagonals	28.8	33.4	38.2	49.8	49.8	54.5	54.5
Truss Columns	75.2	69.5	69.5	80.3	80.3	83.9	83.9
Sum =	143.0	150.6	164.9	189.9	209.9	223.7	223.7

Table 19: Perimeter truss weight summary

	Weight (tons)				
Member Group	**Trial #1**	**Trial #2**	**Trial #3**	**Trial #4**	**Trial #5**
Above Grade Braces	8.1	10.6	11.6	13.0	12.1
Above Grade Columns	87.1	87.1	92.6	63.5	59.4
Below Grade Braces*	4.1	5.3	5.8	6.5	6.1
Below Grade Columns*	43.5	43.5	46.3	31.8	29.7
Sum =	142.8	146.4	156.3	114.7	107.3

*Assumed at 50% of above grade sum

Table 20: Braced frame weight summary

In the sizing and weight tables above, perimeter floor beams were not included. This is because they were modeled in ETABS in a rigid diaphragm and without any loadings. The sizes for each typical span length were determined using basic bending analysis. The analysis itself was considerably conservative due to assumptions regarding perimeter load distribution, so the total weight is larger than what is required. Calculations are found in Appendix C.8. A summary of the weight calculations is found below in Table 21.

Length ft	Pieces per floor	Total Length ft	Weight lb/ft	Total weight tons
9	19	171	26	2.2
18	16	288	55	7.9
27	11	297	106	15.7
Per floor		756		25.9
			x4 Floors	103.5

Table 21: Perimeter floor beam weight summary

The total weight of the perimeter truss system was then found from the sum of the truss, column, brace, and beam components (Table 22).

Perimeter Truss	Tons
Truss Horizontals	85.2
Truss Diagonals	54.5
Columns	83.9
Filler Beams	103.5
Bracing	107.3
Total Weight =	434.4

Table 22: Perimeter truss system total weight

Steel take-offs for the original system were also carried out. Full calculations can be found in Appendix C.9. Total weights are given in Table 23.

Original System	Tons
Diagrid	407.0
V columns	46.9
Bracing	62.3
Total Weight =	516.2

Table 23: Original system total weight

Discussion

Unlike Solution Areas I and II, which utilized numerical criteria to evaluate and compare between alternatives, Solution Area III is most effectively evaluated through qualitative assessments of the perimeter truss and braced frame system. These assessments are based upon the primary goals and considerations outlined in the beginning of this section.

- Change the façade's architectural look and feel.
  The removal of the diagrid structure drastically opened up the perimeter to allow unlimited possibilities for treatment of the façade. The ability to have complete

control of curtain wall window quantity and placement presents the architect with more design freedom.

- Keep the shape of the building intact.
  Though bay spacings now vary from 9' to 27', layout of these spacings was carefully chosen to minimize visual impact to the perimeter. Interruptions to the smooth perimeter will be imperceptible to Athletic Center occupants and passersby.

- Reduce structure weight and complexity.
  The total structural steel weight of the perimeter truss system is over 15% less that the original system. This reduction of steel tonnage saves material costs, as well as having an impact on the foundation system. In addition, the complexity of the structure is greatly reduced. Simple shear connections are now used for every connection except the perimeter truss. Typical fabrication and erection costs account for much of the total structure cost in a steel building, and therefore reduction in the connection complexity can save a considerable amount of shop and labor costs.

- Provide as much glazing opportunity as possible.
  Whereas the original diagrid system utilized a mere 20% of the usable viewing level for windows, the elimination of tightly spaced diagonal columns now permits nearly 100% usage. This provides more impressive views of the football stadium and surrounding landscape.

- Minimize impact to the interior layout.
  Though efforts were made to prevent structural members from taking up interior space, this was not possible. The addition of braced frames to replace the diagrid lateral system requires placement of extra structure in spaces such as corridors, locker rooms, and mechanical chases. The frames must be hidden in existing walls or covered up. This is a definite drawback to the perimeter truss system; however, with careful space planning from the initial design stages it is possible to reasonably minimize structural impact.

- Maintain floor height.
  Floor-to-floor height was not affected at all by the perimeter truss design. The height of the parapet was increased by 2'-6" due to the position of the truss chords, but this increase is relatively insignificant.

- Placement of columns.
  The continuity of the perimeter truss allowed flexible positioning of supporting columns. Therefore, compression columns were able to be placed in the exact same locations as for the original diagrid system. This eliminated any need for lower level room redesign and did not compromise existing spaces from adjacent facilities.

- Penetration of open spaces.
  By placing the main gravity force resisting structure around the perimeter, interior column and floor beam layouts were maintained, and no additional members were

necessary to tie structural elements together. Penetration of open spaces was not an issue, with the exception of the vertical mechanical shaft affected by the new North-South braced frame.

- Floor system impact.
  Because it was possible to frame floor beams between perimeter columns, interior beams were not affected by the new system. Floor framing remains as originally designed.

- Foundation impact.
  The reduction of perimeter loads around the building due to lower structural system weight allows perimeter column footings to be slightly smaller than originally designed. However, the new addition of braced frames will require new piers as well, which could increase foundation costs.