Structural Breadth

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The structural breadth for this thesis was chosen late in development as a response to the use of a seemingly exorbitant cantilever at the ground floor of the building's lobby. Fully aware of the questions this canopy would produce in its effectiveness and/or constructability, it was decided that a structural breadth analyzing the canopy would serve better purpose for the overall thesis project. Additionally, following the choice of a structural breadth altogether, the analysis of all architectural modifications was chosen. Therefore a small breadth relative to the skylight wells cut into the roof floor slab, and the stairwells cut into lower floor slabs should also be analyzed for implementation. Note: the analysis was not completed on the entirety of the structure, as the elements chosen do not affect the structure on the whole by relative comparison, but also that modeling of this building is, as peers have deemed it, very difficult.

Relationship with Thesis Project

The canopy being designed as the major focus of the structural breadth would be constructed between the two existing canopies at the north and east entrances to the main lobby at the northeast corner. This canopy's intention is to block out direct sunlight and glare resulting from early morning direct sun. Modeling the new canopy off of the current canopy design and following the architectural form of that section of building, a longer, broader, and curved canopy had to be developed to maintain the architectural form of the northeastern corner.

Façade Canopy

The façade canopy in plan is shown in Appendix B. The canopy has the elliptical shape as described previously and this makes it relatively hard to model through a single representation or at the expense of modeling the entire structure to detail. Based on the largest dimension found for the canopy, a single "bay" type of model was developed for a section of the canopy that can be applied equally too all sections of the model including an additional factor of safety built in by the "worst-case scenario" analysis.

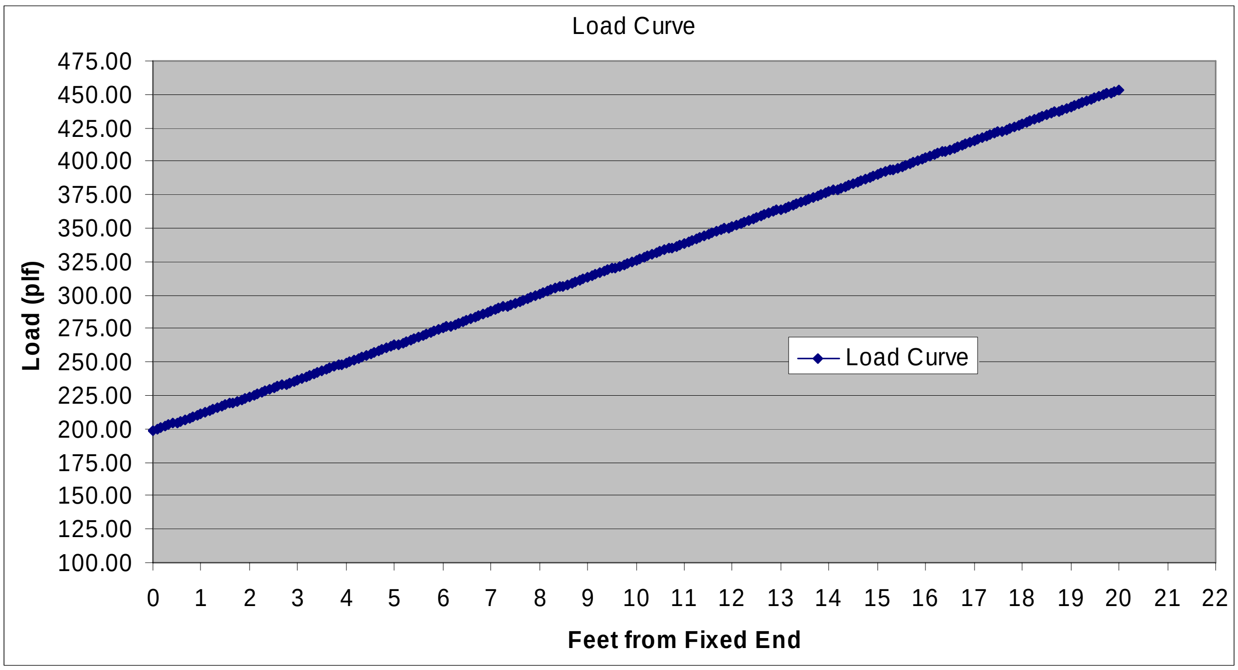
Based on the section of the existing canopy and its dimensions similar shapes were used for the cantilevered members and a similar shape was intended to be used for the cantilever's support members. The following design loads were applied to the structure to be analyzed:

Dead Load –	MEP – 10psf	
	Deck Wt – 3psf	
	Insulation – 2 psf	
	Collateral – 5 psf	TOTAL = 18psf
Live Load –	Snow – 30psf	TOTAL = 30psf
Self Wt –	Wt. – 8.7psf *	TOTAL = 8.7psf

TOTAL LOAD = 56.7psf

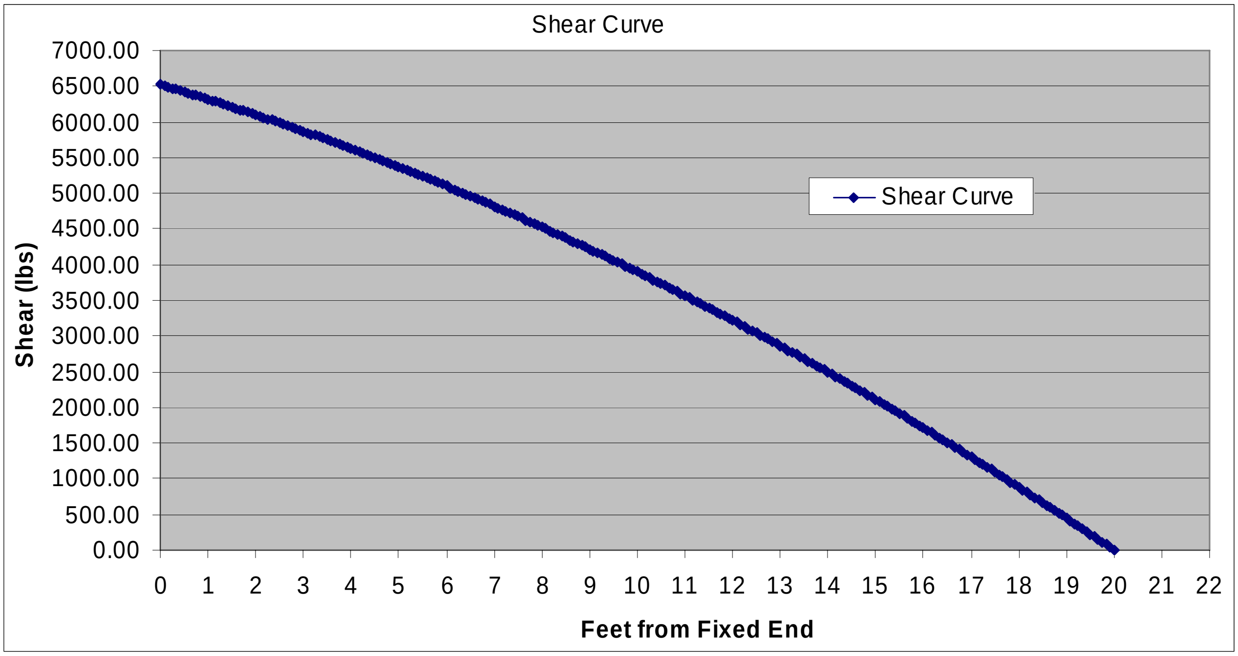
* Calculated from 50plf member (conservative) averaged over the wedge area

Because the shape is a curve, the tributary area at the "base" will be less than the tributary area at the "head". From the dimensions, the average end dimension was 3'6" and the average cantilevered dimension was 8'. This resulted in a load diagram as seen below.

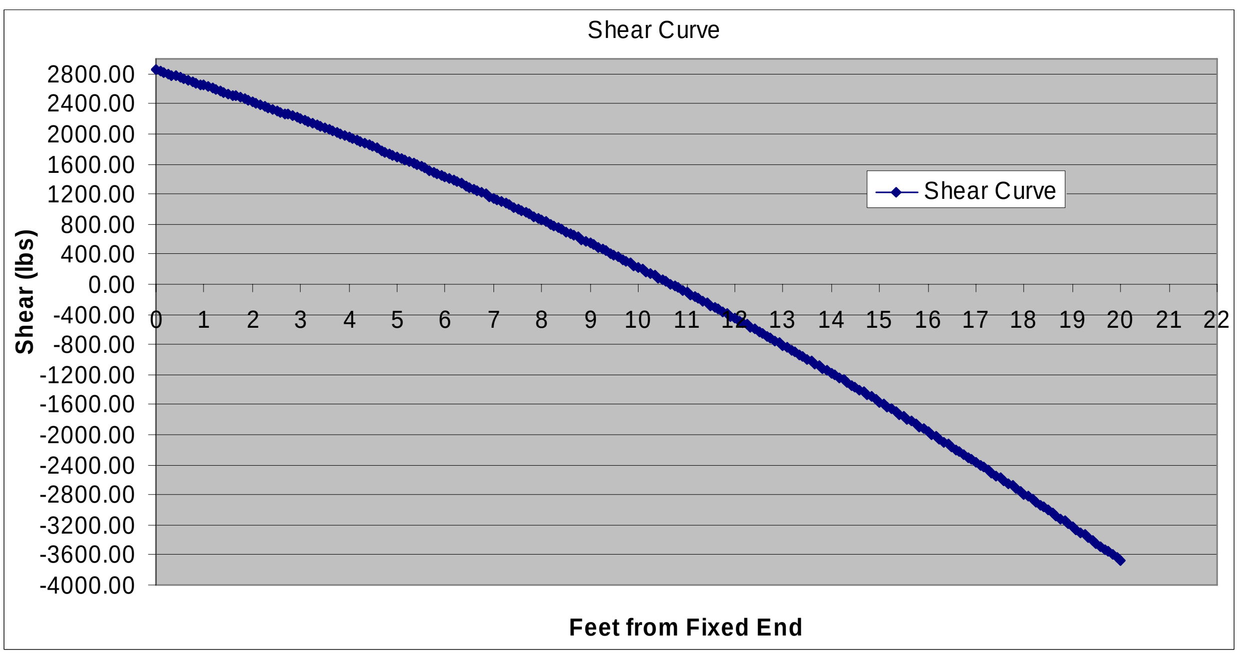


From this, the reaction forces were found for two conditions. First, the condition where the cantilever was purely cantilevered and only supported at the base member was considered. For a second analysis (to be explained), a cantilever with a tension cable was considered. Based on the reactions found, the tension in the cable for the second condition could be calculated. The resulting shear and moment diagrams were found for the two conditions and are shown below.

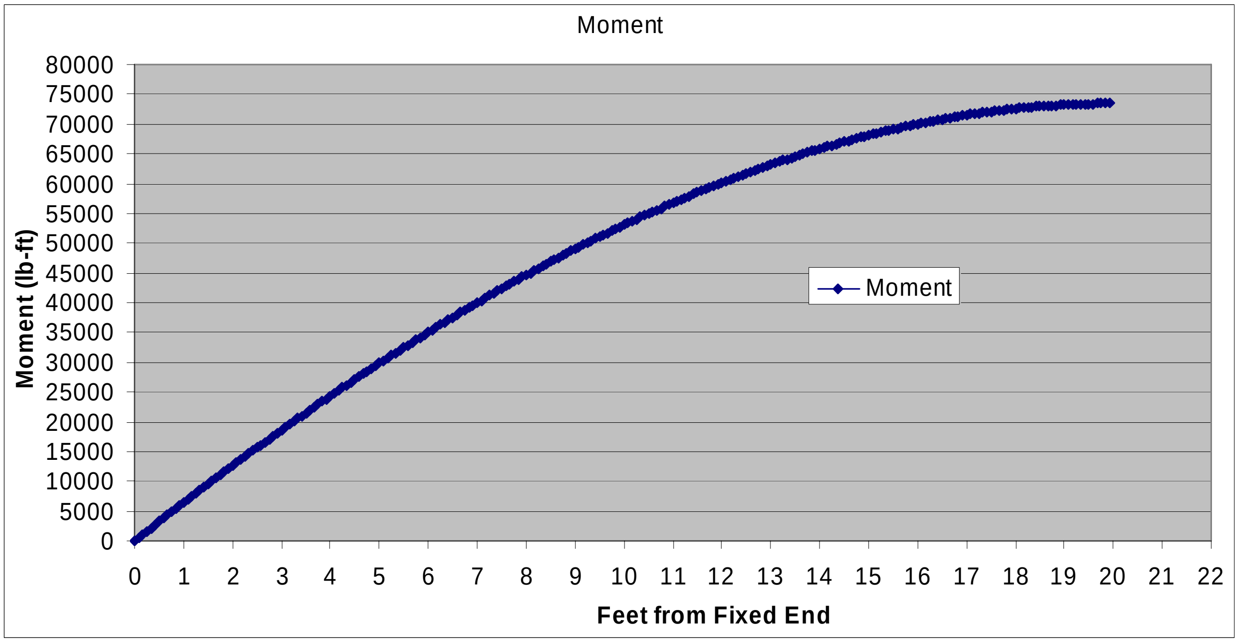
Cantilever Option – Shear Maximum = 6521 lbs At Base = 6521 lbs



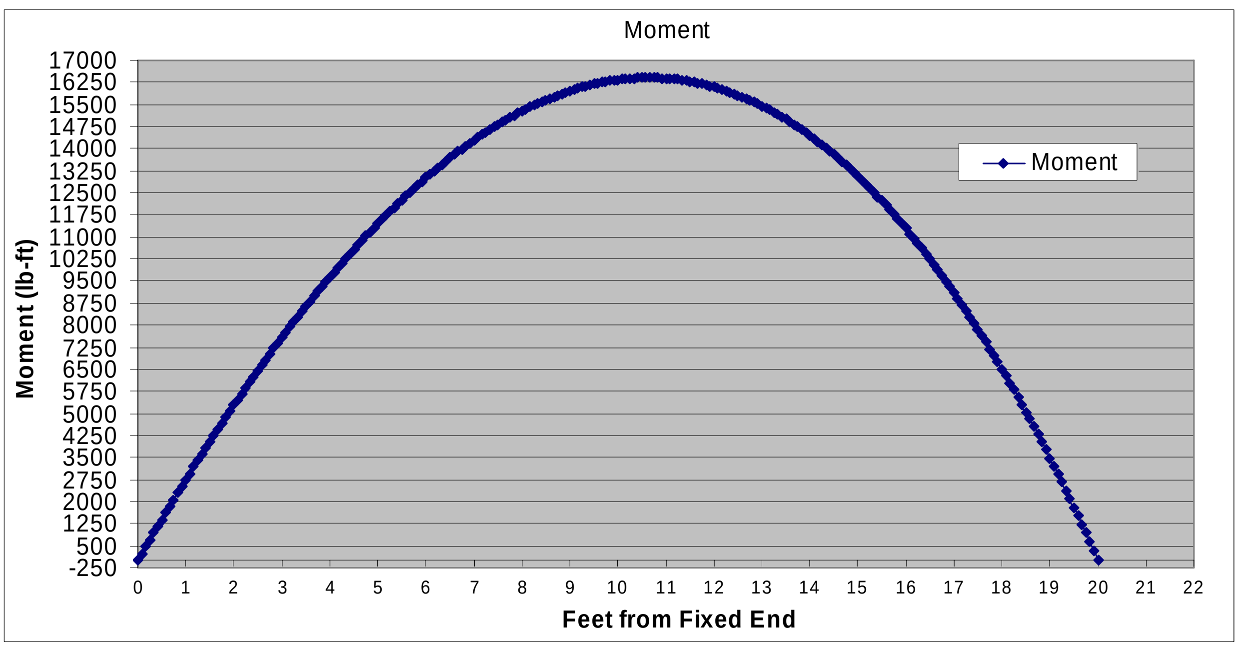
Tension Cable Option – Shear Maximum = -3619 lbs At Base = 2850 lbs



Cantilever Option – Moment Max = 73.4 ft-k At Base = 73.4 ft-k



Tension Cable – Moment Max = 16.4 ft-k At Base = 0 ft-k



Based on these diagrams and the maximum moment found for each condition, a member was chosen using the 2005 AISC Steel Manual. The maximum moment at an unbraced length of 20' was used for both the cantilever and tension cable options. These calculations and the resulting member chosen assumes that the base end of the cantilevered beam is purely fixed, with no base member rotation – a point found to be of great importance for a full cantilever option. The chosen members are as follows:

Full Cantilever Option	-	MC12x50
Tension Cable Option	-	MC8x21.4

These diagrams and the associated members were checked using STAAD Pro. Based on the moments and shear values matching, the maximum deflection was found using the program to save time.

STAAD output Cantilever Base	-	Moment = 72.4 ft-k	Shear = 6518 lbs
STAAD output Tension Cable Base	-	Moment = 5.7 ft-k	Shear = 2327 lbs
Deflection Cantilever	-	-1.675"	
Deflection Tension Cable	-	2.610"	

Because of the extreme positive deflection in the smaller, tension cable member (due to the assumed self weight of 50plf and actual self weight of 21.4plf), the load was adjusted in STAAD to determine the resultant vertical cable tension. The goal was to produce 0 ft-k moment at the base of the cantilever such that the supporting member would only see column-axial load.

Full Cantilever Option

With the full cantilever option, the initial cantilever member must be a very large channel, with a significant self-weight. Additionally, the member deflects at the end -1.675" even when considering the supporting member to be a perfectly fixed end. In reality, the supporting member will rotate and a considerable torsion of 72.4 ft-k will be placed on it at every connection point.

The results of attempting to size the supporting member to match the current canopy detail for the cantilever resulted in the following:

Max Moment (Torsion)	= 144.75 ft-k
Max Moment (Bending)	= 47.11 ft-k
Max Deflection (Cantilever end)	= 7.5"
Max Shear (Column End)	= 13.5 k

As one can see, the torsion placed on the member at the connection point with the columns is considerable compared to the bending moment. Due to the lack of knowledge in torsional failure (especially for HSS Round or HSS Rectangular shapes currently employed in the canopy section), further analysis of the Cantilever Option was deemed unnecessary. The deflection seen by the Cantilever is also considerable when considering a simple cantilever, and this results in a rotation of the supporting member of 1.79˚. Additionally, comparison to the tension cable member shows that one could purchase about 3 of the tension cable members for the equivalent weight and size of the cantilever member. At this point, it was decided that the Cantilever might be a better choice.

Tension Cable Option

With the tension cable, the resulting necessary cable force needed for a 0 ft-k moment at the supporting member end was 3333.3 lbs upward. This results in a 1.871in upward deflection. The goal was to add these elements to the proposed cantilever design for a single supporting member. Using STAAD Pro, it was decided that two analyses could be completed. One where the maximum deflection of the entire structure was 0" total, and one where the moment at the column was 0 ft-k.

Using the upward force of 3333.3 lbs and resolving the axial force in the cantilever produced by the angle of the tension cable (3897.4 lbs) the following results were obtained for the cantilever:

Max Moment x-axis (torsion)	= 0 ft-k
Max Moment y-axis (bending)	= 19.8 ft-k
Max Moment z-axis (lateral bending)	= 27.3 ft-k
Max Deflection	= 1.83" (upward)
Max Shear at Column (lateral)	= 7.8 k

All of these numbers fall well within the limits of the available bending moment of any HSS10x10 thickness. Therefore, from the steel framing standpoint, the canopy works beautifully. The supporting member could even be downsized to meet the new loading requirements; however, given the current canopies size, it would be more difficult to introduce a new size and possibly cost prohibitive.

The second analysis takes the cable system and is only concerned with the maximum deflection the cantilever can take by code – L/360 or 0.67". By adjusting the forces in STAAD to obtain a deflection that does not exceed 0.67" the following results are obtained.

Max Moment x-axis (torsion)	= 23.7 ft-k
Max Moment y-axis (bending)	= 23.9 ft-k
Max Moment z-axis (lateral bending)	= 22.5 ft-k
Max Deflection	= 0.656" (upward)
Max Shear at Column (lateral)	= 6.9 k

The concern that arises with this steel design is the implementation of the cable system into the current concrete column structure. Each of these cables would be tied to one of the four exterior columns either inside, or immediately outside of the façade. The four columns have been shown with cabling in Appendix B. Each of these columns requires a maximum lateral force of 3.9 k per cable for the 0-moment design and 3.3 k for the 0.66"-deflection design. The largest number of cable applied to a single column would be 5 under this design resulting in either 19.5 k shear or a 16 k shear. Whether or not the column would be able to withstand a shear force of this magnitude is uncertain for two reasons. First, analysis of the column would require more information about the column, and given the lack of structural drawings and information, one cannot determine the properties of the column to make any valid design. Second, this analysis begins to fall outside the scope of the breadth work which was to analyze whether or not such a canopy could exist. Since it can be constructed, can take the appropriate loading, and requires only a shear connection to the columns, it is reasonable to assume the canopy can be easily constructed.

Cable System

The cable system is relatively easy to design. Based on the maximum required vertical component (for the 0-moment design) of 3897 lbs, the tension in the cable is 5127.6 lbs. Based on the maximum required vertical component (for the 0.66" deflection design) of 2825 lbs, the tension in the cable is 4342.1lbs.

Using the yield strength of A36 steel as a relative minimum for typical steel elements, and the required tension force, the thickness of the steel cable can be computed. The calculation also includes a 0.6 safety factor since the structure would collapse in the case of a cable failure. This also accounts for excessive stresses built up over the course of many varying load conditions (read "winter seasons"). Based on these two facts, the 0-moment design requires a cable of 0.237 sq in and the 0.66" deflection design requires a cable of 0.201 sq in. This equates to a 0.55" diameter cable and a 0.506" diameter cable respectively. Given the difference and the probable standardization of cables, the 5/8" diameter cable would suffice for both designs.

Skylight stair-Well

Stair-Well

The skylight stair-well was the other structural entity that required analysis in the redesign of the architectural spaces relative to the Lighting Depth. Using the single bay of the steel frame at the 17th and 16th floors for the skylight and stairs respectively, the analysis had to account for any change in the weight distribution over the floor area.

At the time of design, completely unaware of the impact, the stairwell was cut into the entire bay. (See the Lighting renderings under "Reception" for a better view). This bay would allow excess light from the 17th floor to spill into the 16th and 15th floors – as was the design intention and goal. Because the stairwell "hole" was cut along the steel frame members, no actual design of supporting beams was needed. This part of the skylight stair-well design was, suffice to say, quite easy.

Skylight

The actual skylight was a little different, and did require the design of 4 separate members placed off-center. Each of these members had differing tributary areas and, therefore, had differing calculated uniform loads. The loads applied to the floor area of interest to each member are as follows:

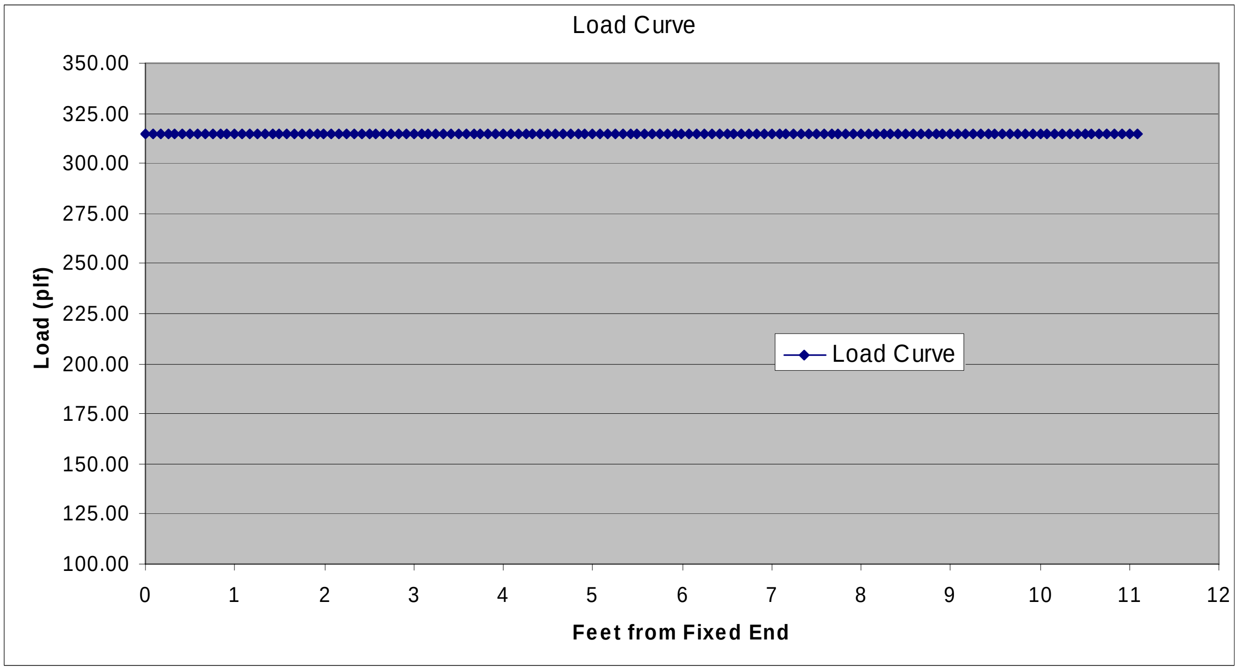
Dead Load –	SDL – 25psf	
	Deck + Concrete – 48psf	
	Finishes and MEP – 5 psf	
	Collateral – 5 psf	TOTAL = 83psf
Live Load –	Live – 80psf	TOTAL = 80psf
Self Wt –	Wt. – 20psf	TOTAL = 20psf

TOTAL LOAD = 103psf

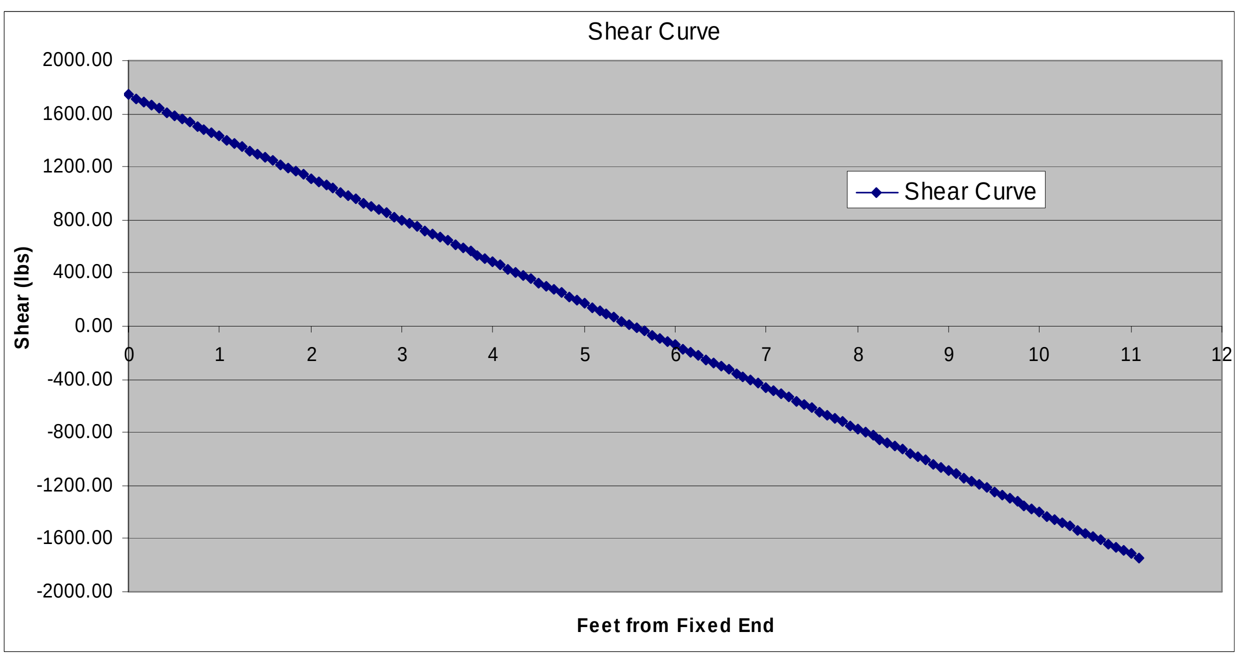
The loading, shear, and moment diagrams for the 4 members are shown below.

Member A

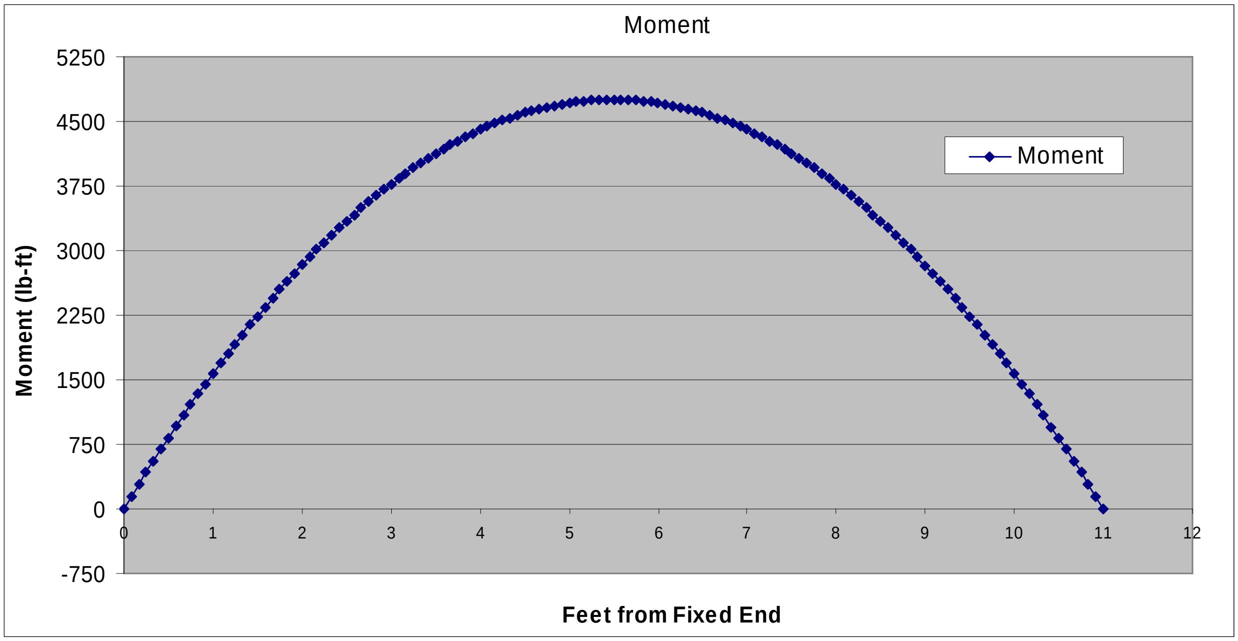
Loading



Shear Max = 1742.8 lbs

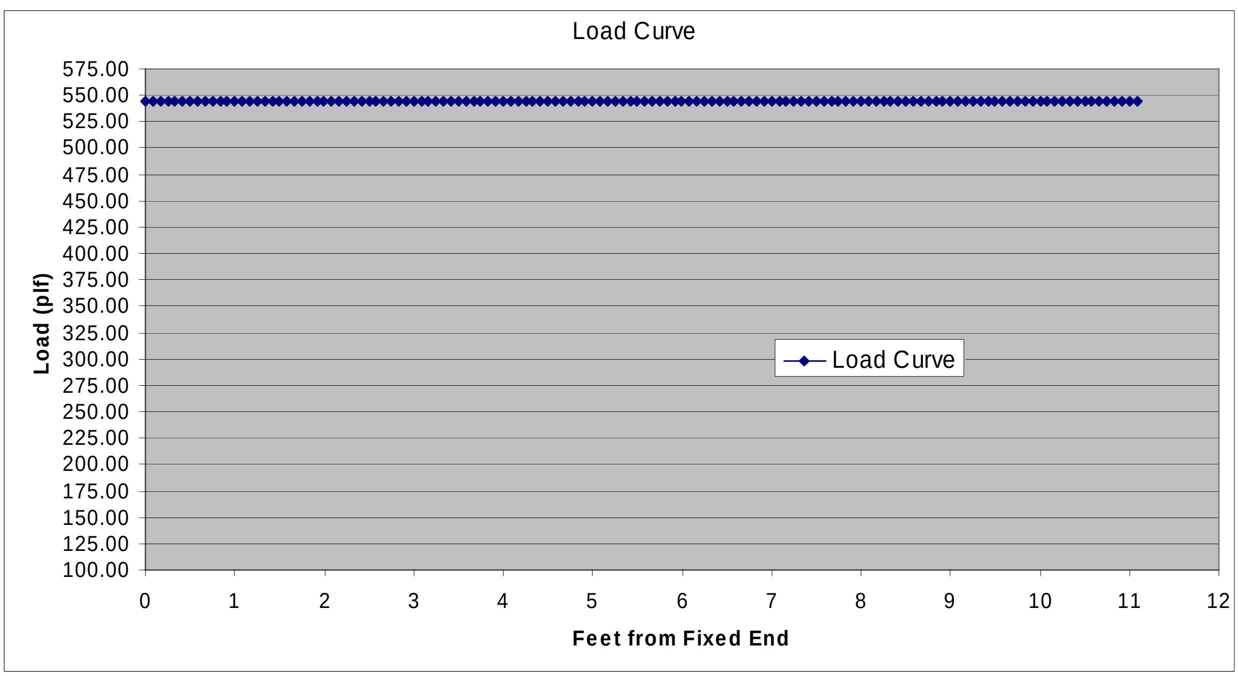


Moment Max = 4.76 ft-k

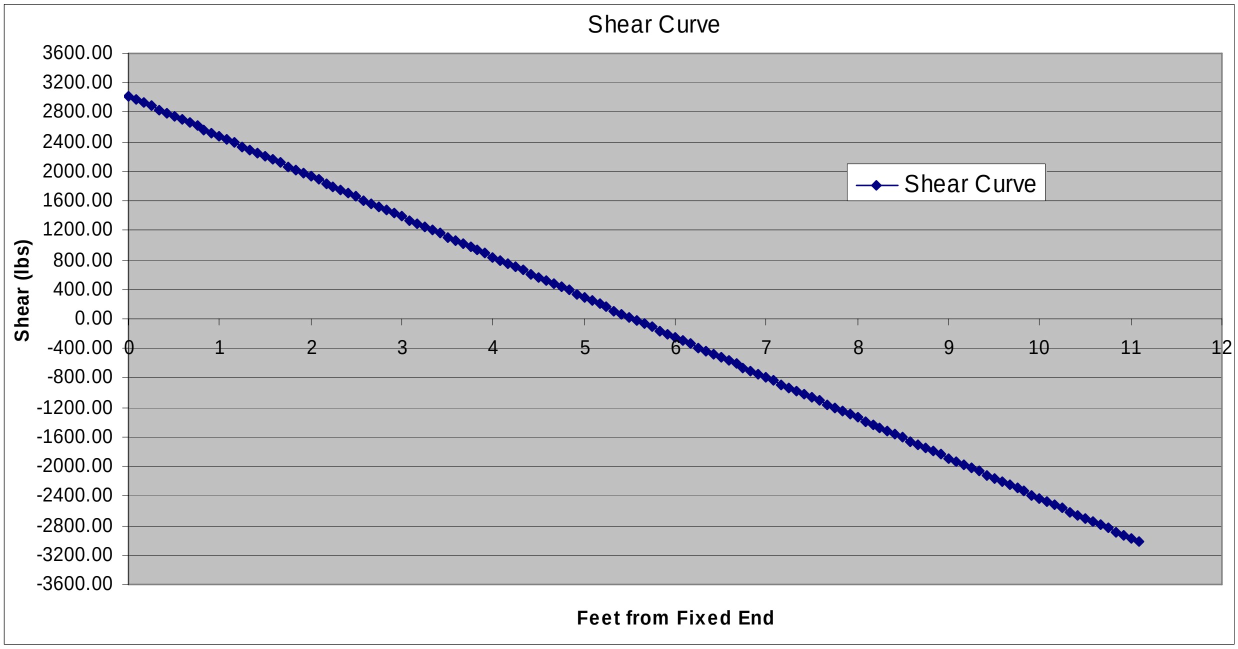


Member B

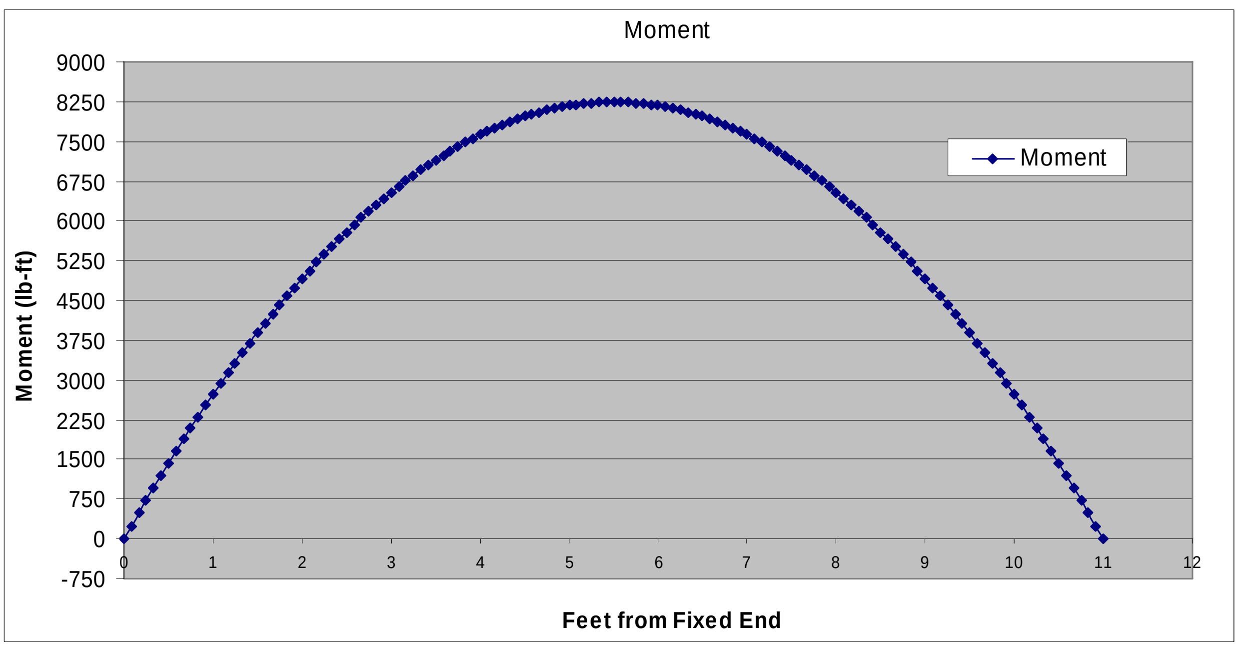
Loading



Shear Max = 3020.7 lbs

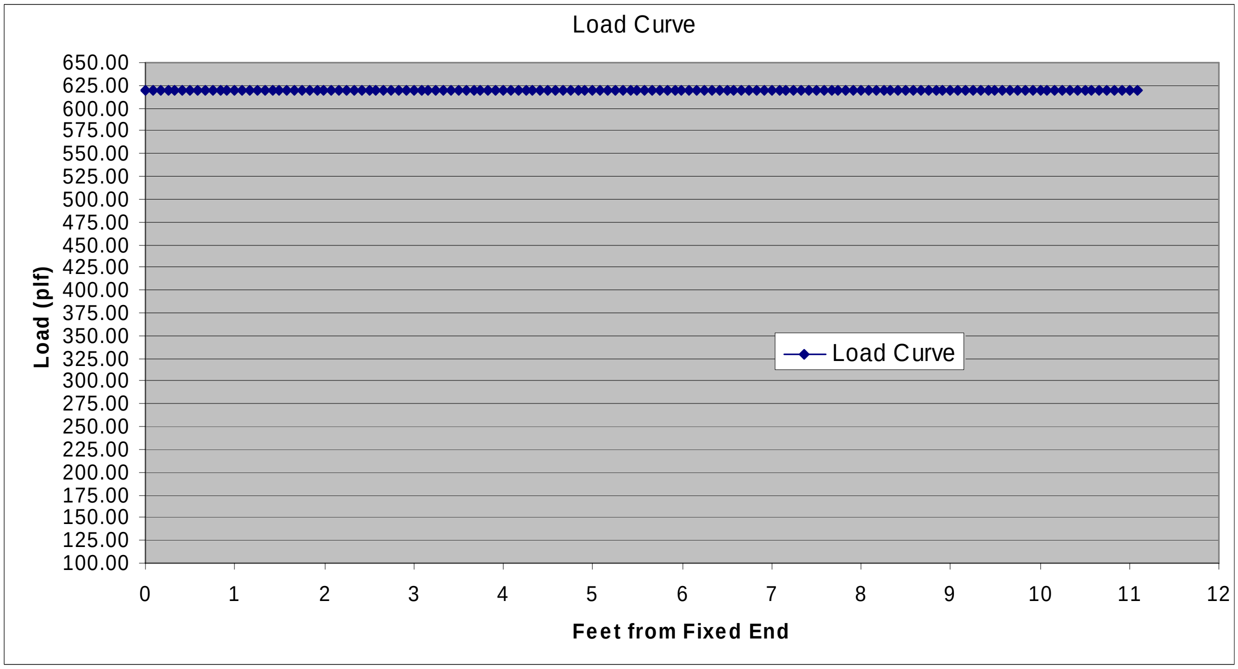


Moment Max = 8.24 ft-k

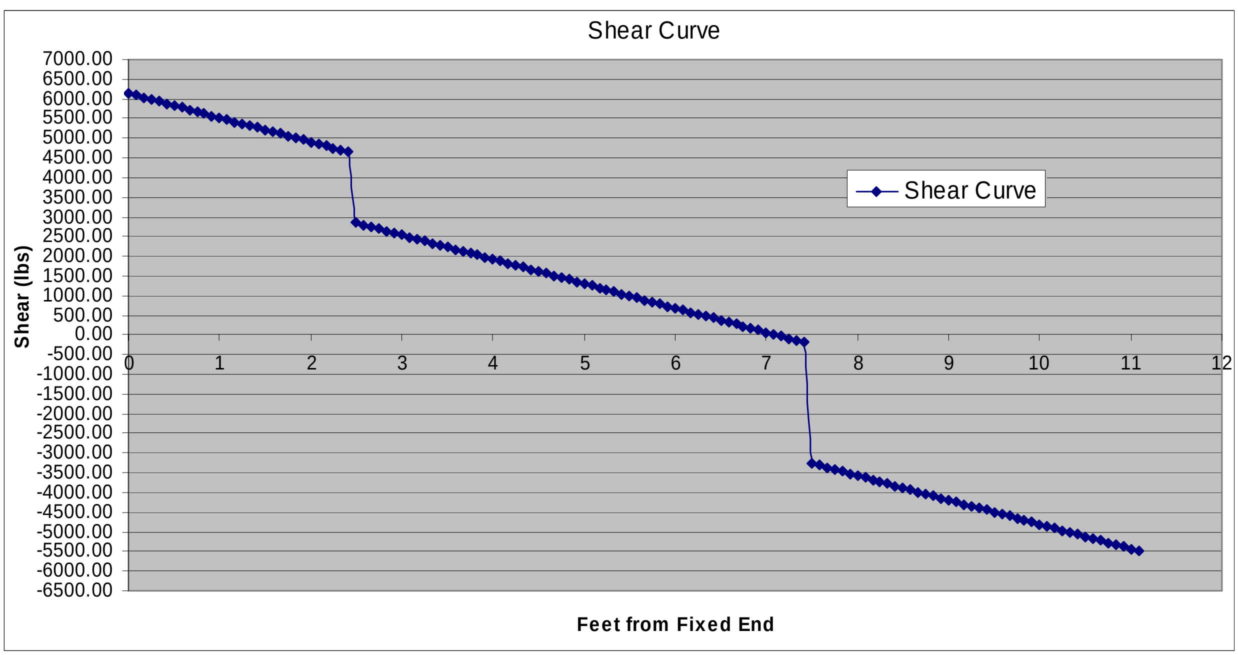


Member C

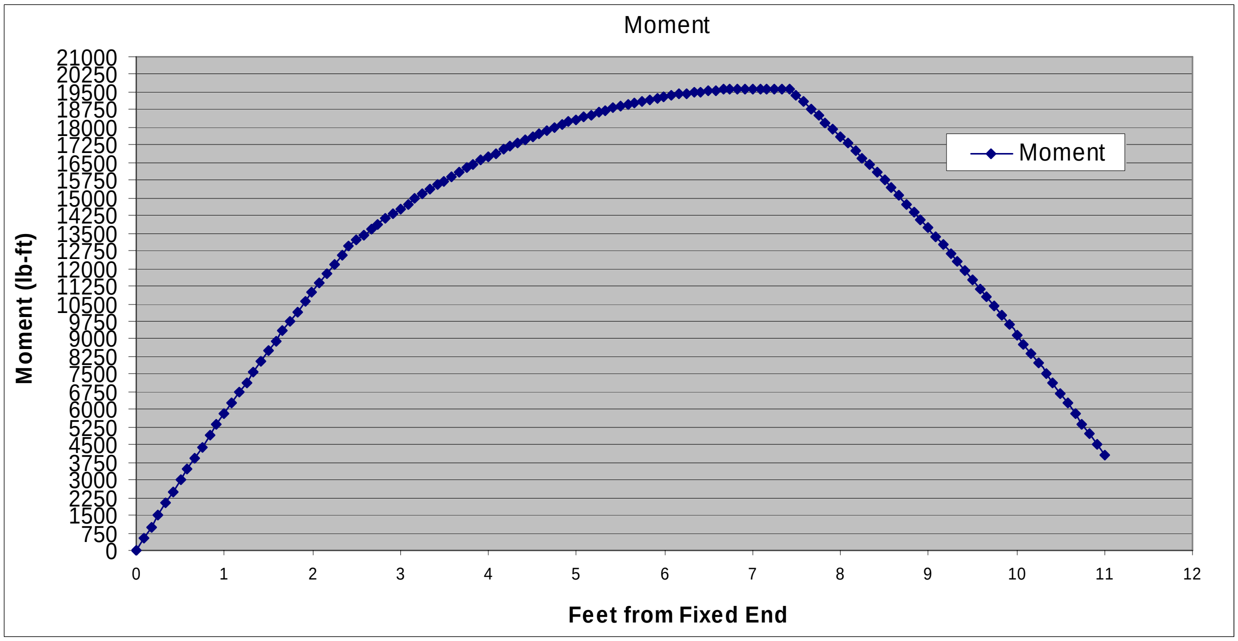
Loading



Shear Max = 6141.2 lbs

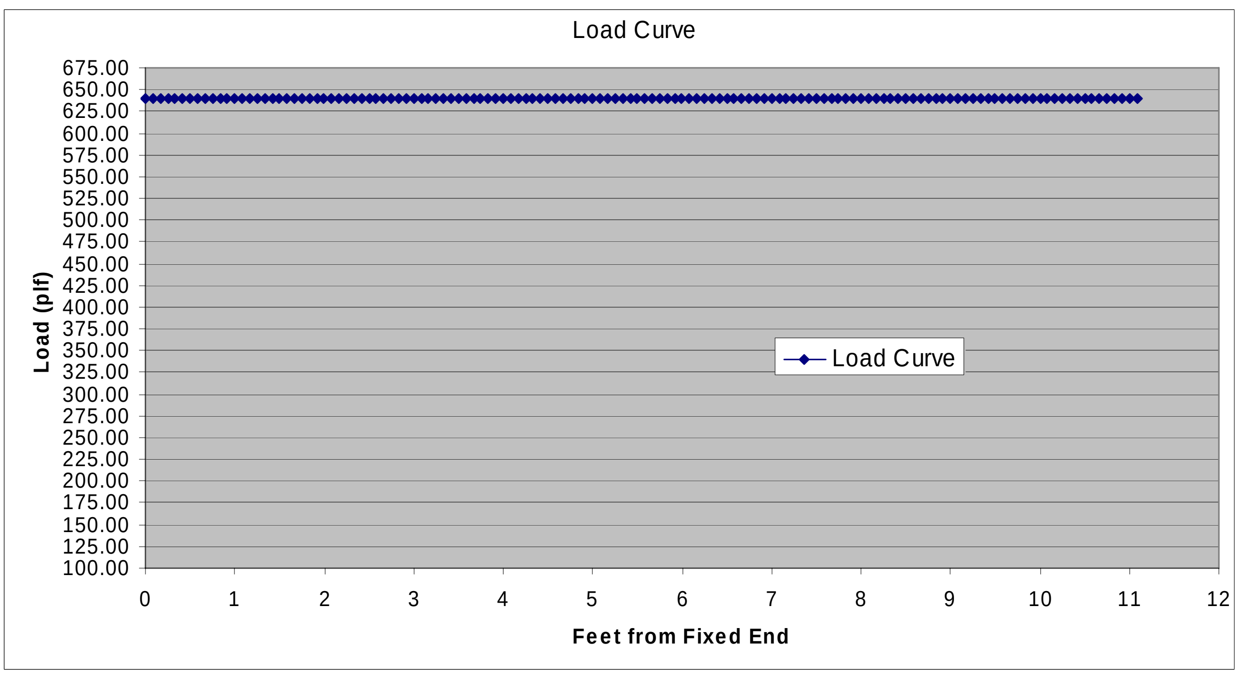


Moment Max = 19.66 ft-k

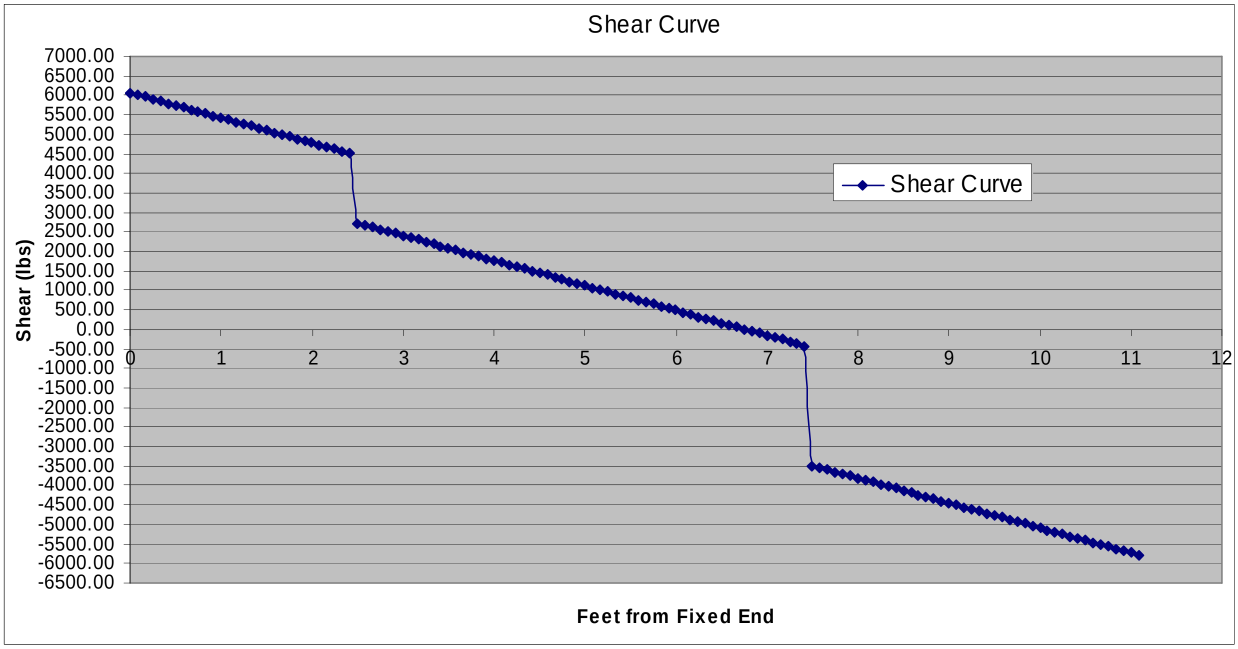


Member D

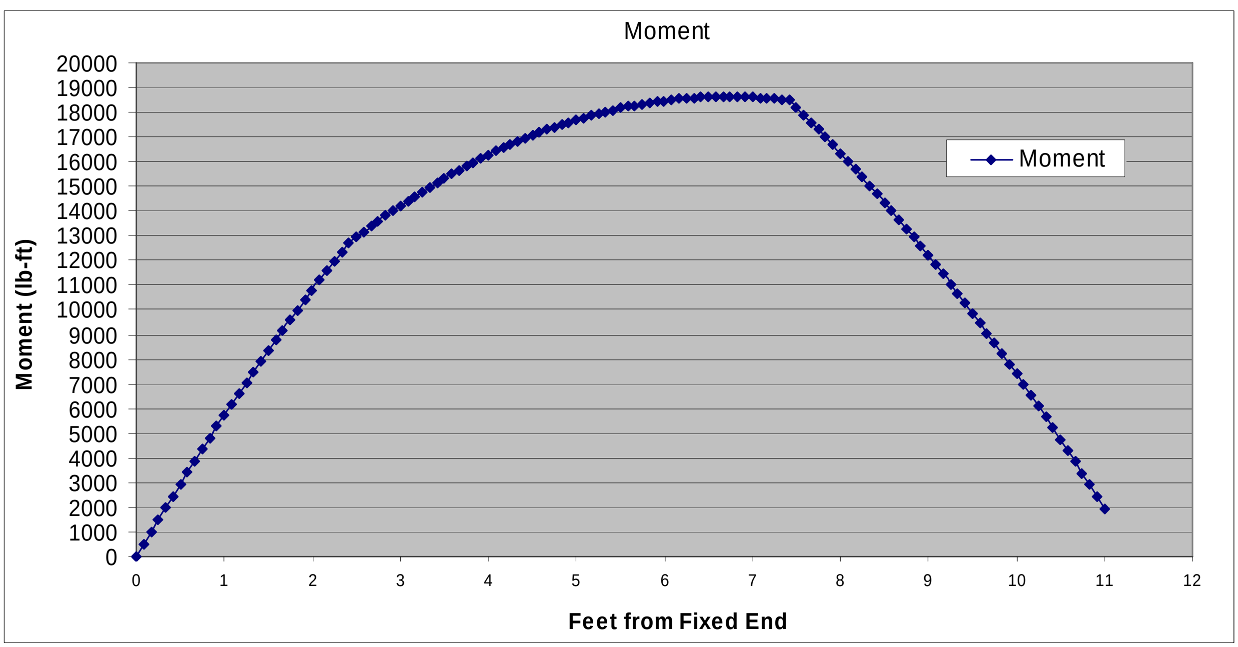
Loading



Shear		Max = 6063.6 lbs



Moment		Max = 18.64 ft-k



Using the maximum bending stress equation Fy / 1.67 = M_{max} / S with F_y = 36 ksi and the maximum moments found from the graphs, an appropriate angle was sized based on the section modulus. The following section modulus values and corresponding angle sizes are used for the appropriate members (based on building orientation):

Member A (East) :	M_{max} = 4.76 ft-k	S = 2.65 in^3	Size = ∠5x3x1/2
Member B (West):	M_{max} = 8.42 ft-k	S = 4.69 in^3	Size = ∠7x4x7/16
Member C (South):	M_{max} = 19.66 ft-k	S = 10.95 in^3	Size = ∠8x4x7/8
Member D (North):	M_{max} = 18.64 ft-k	S = 10.4 in^3	Size = ∠8x4x3/4

For the sake of cost efficiency and installation ease, one would simply choose the largest angle and use it to frame the entire skylight into the slab.

Conclusions

The structural breadth has proven that both systems required for the completion of the Lighting Depth and envisioned in the Architectural Breadth of the fall semester are both feasible from a structural point of view. Without considering the results obtained from the Lighting Depth the skylight will allow a vast amount of light to penetrate deep into the north core of the 15th through 17th floors. The canopy, acting as both a direct sunlight shade and prominent architectural element, can be implemented into the structural design of the northeast corner without causing too many headaches with the already complex design.