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August 6, 2018

Colleen K. Reilly, PMP
President, Lake Park Friends
P.O. Box 71197
Milwaukee, WI 53211

Re: Lake Park Arch Bridge over Ravine Road – Phase 3: Structural Analysis Results

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WESLEY R.
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E-38563-006
EAU CLAIRE
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PROFESSIONAL ENGINEER
P.E.
08/06/18

TranSystems was contracted by Lake Park Friends to conduct structural engineering services for the Lake Park Concrete Arch Bridge, including a structural analysis of the bridge in order to determine its load carrying capacity. All members were analyzed in accordance with Allowable Stress Design (ASD) with the AASHTO *Standard Specifications for Highway Bridges, 17th Edition*. The results of the structural analysis are presented as capacity to demand ratios based on service loads and allowable stresses. All information used in the analysis is based on existing plans, field measurements, material testing, and pertinent historic documentation.

The structural analysis was performed in two (2) stages with different live loads considered:

1) **Original Design Loads** – This analysis serves as a proof of concept for the bridge's original design intent. The analysis was performed based on the original 80 psf design live loading shown in the plans, and capacities were calculated based on design-level allowable stresses (ASD methodology) as shown in AASHTO Articles 8.15.2.1 and 8.15.2.2.
2) **Modern Design Loads** – This analysis determines the ability of the bridge to resist modern code-prescribed design loadings. The analysis is based on a 90 psf pedestrian loading at the Inventory level, and a 90 psf pedestrian loading in conjunction with an H5 Truck (5-ton maintenance vehicle) at the Operating level. For this analysis, allowable stresses utilized for reinforcing steel and concrete are based on AASHTO MBE Tables 6B.5.2.3-1 and 6B.5.2.4.1-1.

The structural analysis for both stages considered the following three (3) alternatives:

1) **As-Built** – The analysis consists of the original structure in its as-constructed condition, utilizing original section properties, geometry, and material specifications. This alternative represents the original design criteria for the structure as shown on the original plan set with verification from field observation.
2) **As-Configured** – The analysis consists of the structure in its existing configuration, accounting for modifications to the structure such as new railings or wearing surfaces, with original as-built section properties and material specifications. This alternative demonstrates the impact of structural modifications to the bridge from its original design intent.
3) **As-Inspected** – The analysis consists of the as-configured bridge as it stands today, any observed section loss and deterioration, and any revisions to the material specifications based on testing. This alternative represents the ability of the structure to carry the original design loading in its current configuration and condition and provides a baseline for any future loading considerations.

ANALYSIS METHODOLOGY

Structural analysis calculations provide a basis for determining the safe load capacity of a bridge. These analyses require engineering evaluation in determining a capacity to demand ratio that is applicable to maintaining the safe use of the bridge and arriving at posting and permit decisions. A capacity to demand ratio of less than 1.0 indicates that the structure does not have sufficient capacity to carry the specified loading. As part of every inspection cycle, bridge analyses (or load ratings) should be reviewed and updated to reflect any relevant changes in condition or dead load noted during the inspection.

The Allowable Stress Design (ASD) method was used to rate all primary members of the bridge. The ASD method is based on analyzing the structure at service load levels (actual loads) and comparing those load effects to allowable stresses. Allowable stresses are used to calculate capacities that are lowering than the ultimate capacities of members, introducing a factor of safety into structural analysis calculations. The capacity to demand ratios for each bridge element are determined by dividing the allowable stress capacity of the member by the sum of the applied services loads.

- Inventory Level (INV) - Generally corresponds to the customary design level of stresses, but reflects the existing bridge and material conditions with regard to deterioration and loss of section. Structural analyses based on the Inventory level allow comparisons with the capacity for new structures and, therefore, result in a live load which can safely utilize an existing structure for an indefinite period of time.

- Operating Level (OPR) - Structural analyses based on this level generally describe the maximum permissible live load to which the structure may be subjected. While permitting live loads on the structure at this level of stress is acceptable, allowing unlimited numbers of loading conditions/vehicles to use the bridge at the Operating level may shorten the life of the bridge.

CAPACITY TO DEMAND RATIOS

Original Design Loading

The capacity to demand ratios from the structural analysis based on original design loading are shown in **Table 1**.

Bridge Element	Capacity to Demand Ratio (80 psf Original Design Load)		
	As-Built	As-Configured	As-Inspected
Deck	1.10	1.02	1.06
Longitudinal Spandrel Beam	1.25	1.16	1.04
Arch Rib	1.11	1.07	1.22

Table 1 – Summary of structural analysis results (presented as capacity to demand ratios) for each bridge element based on the original design loading of 80 psf.

The results of the As-Built analysis demonstrate capacity to demand ratios above 1.0 for all bridge elements. The results of the As-Configured analysis are slightly lower due to the added weight of the new railing and wearing surface since original construction, although the capacity to demand ratios are still above 1.0. The capacity to demand ratios are higher in the As-Inspected analysis than the As-Configured analysis due to the increased concrete strength from recent material testing despite minor section loss noted in the arch rib reinforcement.

Modern Design Loading

The capacity to demand ratios from the structural analysis based on modern design loads are shown in **Table 2**.

Bridge Element	Capacity-to-Demand Ratios (ASD)					
	As-Built		As-Configured		As-Inspected	
	Inventory (90 psf)	Operating (90 psf + H5)	Inventory (90 psf)	Operating (90 psf + H5)	Inventory (90 psf)	Operating (90 psf + H5)
Deck	1.13	1.02	1.05	0.98	1.10	1.02
Longitudinal Spandrel Beam	1.35	1.52	1.25	1.43	1.11	1.29
Arch Rib	1.14	1.55	1.07	1.50	1.23	1.69

Table 2 – Summary of structural analysis results (presented as capacity to demand ratios at both Inventory and Operating levels) for each bridge element based on modern pedestrian bridge loads.

The results of the structural analysis for modern design loadings demonstrate that the bridge has sufficient capacity to carry a 90 psf pedestrian loading at an Inventory level, as well as the same pedestrian loading with an additional H5 truck at the Operating level. The only capacity to demand ratio below 1.0 is the deck for the As-Configured analysis alternative at the Operating level, which represents the structure with existing modifications and original allowable stresses considered. Note that this capacity to demand ratio is 1.02 in the As-Inspected alternative due to the increased concrete strength utilized for analysis based on material testing.

ANALYSIS ASSUMPTIONS AND RESULTS

Deck

The deck slab on the structure consists of a 6" thick reinforced concrete deck with a 12'-0" width between bridge railings **(see Figure 1)**. Based on an allowable compressive stress of 400 psi shown in the original design plans and a factor of safety of 4 which is typical of the time period, an ultimate compressive strength (f'_c) of 1600 psi was assumed for the As-Built and As-Configured analysis. Based on the original design plans, transverse reinforcing steel consists of 1/2" by 1 1/2" historic Khan bars. A yield strength (f_y) of 33 ksi was assumed for the reinforcing steel based on an allowable stress of 16 ksi shown in the plans. A 1" thick concrete wearing surface has been added to the structure since its original construction.

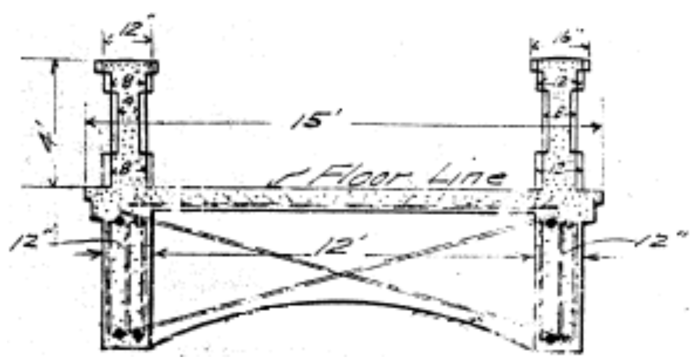


Figure 1 – Typical cross section of the bridge near the center of the arch span (from the original design plans).

Note that the original design plans indicate a transverse spacing of 18" on center; however, photographs from a field investigation show a spacing much closer than this **(see Figure 1 and Photo 2)**. Based on these photographs, a transverse spacing of 7" on center was conservatively assumed. It was assumed that the Khan reinforcing system was placed in such a way that the entire bar is effective in the primary moment region, while shear bars were bent at 45 degrees in the shear regions near the edge of the slab.

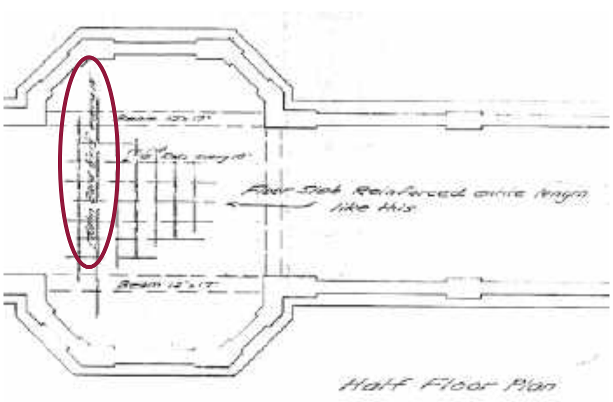


Figure 2 – Floor plan with deck slab reinforcement as shown in the original design plans. Note that the transverse and longitudinal rebar spacing is shown at 18".

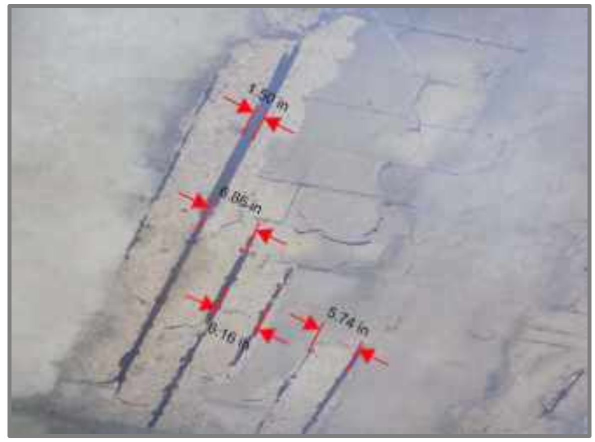


Photo 1 – Underside of deck spall from field investigation. Note that scaled dimensions from this photograph indicate a transverse rebar spacing much closer than 18".

Below is a list of assumptions made for the analysis of the deck:

1. Based on the original plans, the reinforced concrete deck is 6" thick and spans 12'-0" between the arch lines.

2. The original live load utilized for the analysis consists of a uniform distributed load of 80 psf from the original design plans. A 90 psf pedestrian load was utilized for the Inventory level analysis, while a 90 psf pedestrian load and H5 truck were utilized concurrently for the Operating level analysis.

3. For the As-Built and As-Configured analyses, an ultimate compressive strength (f'_c) of 1600 psi was utilized for the concrete. For the As-Inspected analysis, the ultimate compressive strength was increased to 2000 psi based on concrete testing that has been performed on the structure.

4. A yield strength (f_y) of 33 ksi was assumed for the reinforcing steel based on an allowable stress of 16 ksi shown in the plans. For the analysis of the bridge under original design loads, this 16 ksi allowable stress was used for reinforcing steel in accordance with the original plans and AASHTO Article 8.15.2.2. For the analysis of the bridge under modern design loads, allowable stresses were based on AASHTO MBE Table 6B.5.2.3-1 with an allowable stress of 18 ksi for Inventory level and 25 ksi for Operating level.

5. Transverse reinforcing steel consists of 1/2" by 1 1/2" Khan bars with an allowable tensile stress of 16 ksi. The original design plans indicate a transverse spacing of 18" on center; however, photographs from a field investigation show a spacing much closer than this. Based on these photographs, a transverse spacing of 7" on center was conservatively assumed. It was assumed that the Khan reinforcing system was placed in such a way that the entire bar is effective in the primary moment region, while shear bars were bent at 45 degrees in the shear regions near the edge of the slab.

6. A 1" thick concrete wearing surface was considered for the As-Configured and As-Inspected analyses, as shown in the deck core taken for petrographic analysis during the concrete testing **(see Figure 3)**. No concrete wearing surface is included in the As-Built analysis because it was not present following the original construction.

7. Longitudinal reinforcing steel consists of 1/4" diameter rods spaced at 18" on center.

8. Concrete clear cover was assumed to be 1".

9. No significant section loss has been documented on the reinforcing steel; therefore, the full reinforcing steel was considered in all analyses.

Figure 3 – Overall profile of Core 3 (north end of deck) from the petrographic analysis for concrete testing.

10. The deck was analyzed was a simply supported one-way slab spanning transversely. While potential two-way bending was investigated, the deck was determined to span transversely due to the much higher flexural capacity in the transverse direction than in the longitudinal direction based on reinforcing steel provided, which appears to match the design intent.

Based on the results of the structural analysis for the original design loading, the capacity to demand ratios for the deck are above 1.0 for all analysis alternatives. The As-Built capacity to demand ratio was calculated to be 1.10, indicating that the deck satisfies the original design criteria for the structure. The As-Configured capacity to demand ratio is 1.02 due to the added dead load from the concrete wearing surface. The As-Inspected capacity to demand ratio is 1.06 based on the increased concrete strength due to material testing.

The capacity to demand ratios for the deck for modern pedestrian bridges loadings are shown in **Table 3**.

Bridge Element	Capacity-to-Demand Ratios (ASD)					
	As-Built		As-Configured		As-Inspected	
	Inventory (90 psf)	Operating (90 psf + H5)	Inventory (90 psf)	Operating (90 psf + H5)	Inventory (90 psf)	Operating (90 psf + H5)
Deck	1.13	1.02	1.05	0.98	1.10	1.02

Table 3 – Summary of structural analysis results (presented as capacity to demand ratios at both Inventory and Operating levels) for the deck based on modern pedestrian bridge loads.

All capacity to demand ratios for the deck are above 1.0 under current pedestrian bridge design loads, except for the As-Configured analysis alternative at the Operating level (0.98), which represents the structure with existing modifications and original allowable stresses considered. Note that this capacity to demand ratio is increased to 1.02 in the As-Inspected alternative due to the increased concrete strength utilized for analysis based on material testing.

Longitudinal Spandrel Member

The longitudinal spandrel beam consists of the rectangular reinforced concrete beam section over the teardrop openings on each side of the structure **(see Figure 4)**. These members were analyzed as flexural members with a simply supported 20' clear span length. Based on the original design plans and field measurements, the beams are 12" wide by 3'-2" tall from the bottom face to the top of deck, with the entire height included because the deck was poured monolithic with the beams. According to the original design plans, the beam reinforcing steel consists of two (2) 1" by 3" Kahn bars in the bottom face.

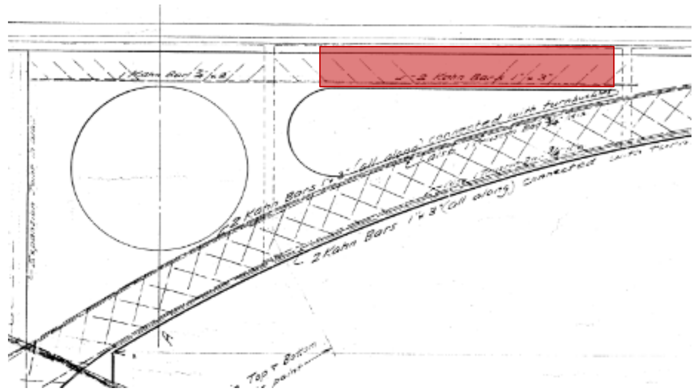


Figure 4 – Typical arch elevation showing the longitudinal spandrel members over the teardrop openings from the original design plans (highlighted red).

Flexural reinforcing steel consists of two (2) 1" by 3" Kahn bars in the bottom face of the beam. Based on the guidelines in the 1904 Khan Bar manual for reinforced concrete beams, the full cross-sectional area of the reinforcing bars as given in the manual should be assumed for strength calculations in flexural members **(see Figure 5)**. As such, this analysis assumes that the Khan reinforcing system is placed in such a way that the entire bar is effective in the primary moment region while shear bars were bent at 45 degrees in the shear regions beginning 5' from each end of the beam (assumed one-quarter points).

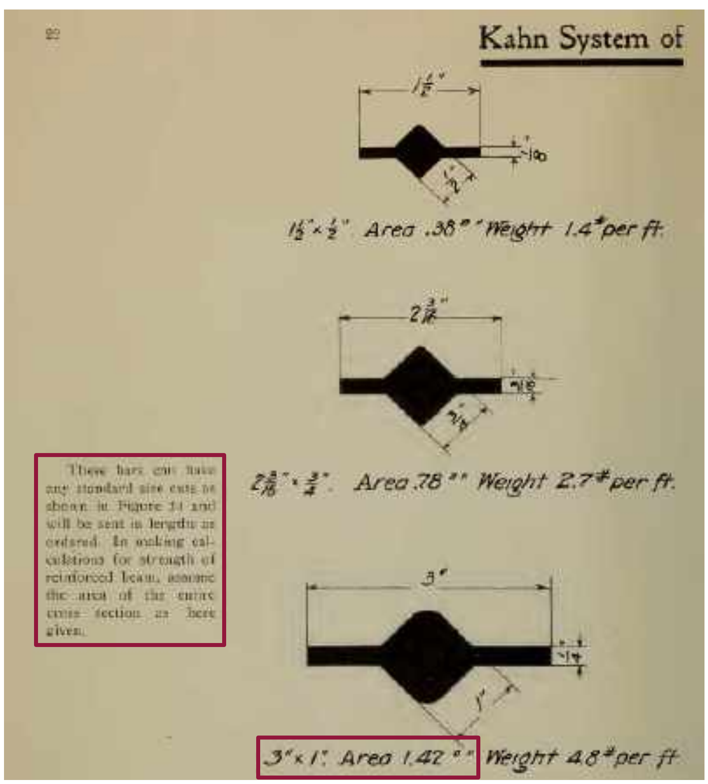


Figure 5 – Excerpt from "Kahn System of Reinforced Concrete" manual dated 1904 providing which provides guidance on the calculation of effective reinforcing steel in reinforced concrete flexural members.

Below is a list of assumptions made for the analysis of the longitudinal spandrel beams:

1. Based on the original plans and field measurements, beam dimensions of 12" wide by 3'-2" tall (measured from bottom of beam to top of deck) were utilized in the analysis.

2. The beam was analyzed as a simply supported flexural member with a clear span length of 20'-0", spanning the horizontal top face of the teardrop opening based on the original design plans.

3. The original live load utilized for the analysis consists of a uniform distributed load of 80 psf from the original design plans with a tributary area of half of the deck. A 90 psf pedestrian load was utilized for the Inventory level analysis, while a 90 psf pedestrian load and H5 truck were utilized concurrently for the Operating level analysis.

4. Transverse live load distribution factors for the H5 truck in the Operating level analysis were calculated by the lever rule.

5. For the As-Built and As-Configured analyses, an ultimate compressive strength (f'_c) of 1600 psi was utilized for the concrete. For the As-Inspected analysis, the ultimate compressive strength was increased to 2000 psi based on concrete testing that has been performed on the structure.

6. A yield strength (f_y) of 33 ksi was assumed for the reinforcing steel based on an allowable stress of 16 ksi shown in the plans. For the analysis of the bridge under original design loads, this 16 ksi allowable stress was used for reinforcing steel in accordance with the original plans and AASHTO Article 8.15.2.2. For the analysis of the bridge under modern design loads, allowable stresses were based on AASHTO MBE Table 6B.5.2.3-1 with an allowable stress of 18 ksi for Inventory level and 25 ksi for Operating level.

7. Flexural reinforcing steel consists of two (2) 1" by 3" Kahn bars in the bottom face with an allowable tensile stress of 16 ksi. Based on the guidelines in the 1904 Khan Bar manual, the Khan reinforcing system was placed in such a way that the entire bar is effective in the primary moment region, while shear bars were bent at 45 degrees in the shear regions beginning 5' from each end of the beam (assumed one-quarter points).

8. The dead load utilized for the analysis of each beam includes half of the deck, the weight of the railing, the self-weight of the beam, and the wearing surface (if applicable).

9. The original decorative concrete railing was considered for dead load in the As-Built analysis, while the heavier railing currently installed was considered in the As-Configured and As-Inspected analyses.

10. The 1" thick concrete wearing surface was considered for the As-Configured and As-Inspected analyses only.

11. Concrete clear cover was assumed to be 2".

12. Based on photographs from the field investigation, 1/16" deep section loss was assumed for the reinforcing steel in the spandrel beams in the As-Inspected analysis.

Based on the results of the structural analysis for the original design loading, the capacity to demand ratios for the longitudinal spandrel beams are above 1.0 for all analysis alternatives. The As-Built capacity to demand ratio was calculated to be 1.25, indicating that the spandrel beam satisfies the original design criteria for the structure. The As-Configured capacity to demand ratio is 1.16 due to the added dead load from the concrete wearing surface and newer bridge railing. The As-Inspected capacity to demand ratio is 1.04 based on the increased concrete strength due to material testing and section loss noted to the reinforcement.

The capacity to demand ratios for the longitudinal spandrel beams for modern pedestrian bridges loadings are shown in **Table 4**.

Bridge Element	Capacity-to-Demand Ratios (ASD)					
	As-Built		As-Configured		As-Inspected	
	Inventory (90 psf)	Operating (90 psf + H5)	Inventory (90 psf)	Operating (90 psf + H5)	Inventory (90 psf)	Operating (90 psf + H5)
Longitudinal Spandrel Beam	1.35	1.52	1.25	1.43	1.11	1.29

Table 4 – Summary of structural analysis results (presented as capacity to demand ratios at both Inventory and Operating levels) for the longitudinal spandrel beam based on modern pedestrian bridge loads.

All capacity to demand ratios for the longitudinal spandrel members are above 1.0 under current pedestrian bridge design loads. The As-Built capacity to demand ratio is 1.35 for Inventory level and 1.52 for Operating level, indicating that this member in the original structure was designed with sufficient strength to carry these modern loads. The As-Inspected capacity to demand ratio is 1.11 for Inventory level and 1.29 for Operating level. These values differ from the As-Built capacity to demand ratios due to increased load from additional wearing surface and railing loads, section loss noted to reinforcement, and added concrete compressive strength due to material testing. This indicates that the bridge is capable of carrying modern pedestrian design loads in its current configuration with all existing factors included.

Arch Ribs

The reinforced concrete arch ribs have a span length of 118"-0" from spring line to spring line with a rise of 18'-0" at the arch center. The arch ribs were analyzed with a combination of hand calculations, STAAD models, and Excel workbooks. The arch ribs were modeled as a two-dimensional frame model created in STAAD.Pro v8i with fixed supports at the ends **(see Figure 6)**. Dead loads were calculated by hand and applied using distributed or concentrated loads within the model. The position of live loads utilized for the analysis were varied to maximize load effects. These load effects were then exported from the STAAD output and charted on axial-moment interaction diagrams that were created based on allowable stress analysis methods.

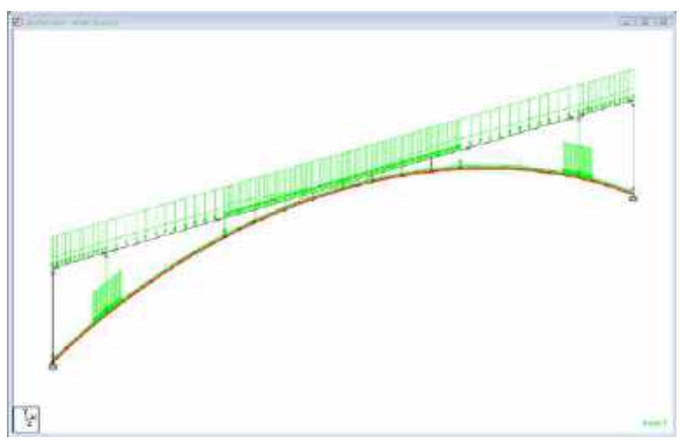


Figure 6 – Two-dimensional arch rib model in STAAD.Pro (dead load case shown).

Below is a list of assumptions made for the analysis of the arch ribs:

1. Based on the original plans and field measurements, arch dimensions of 4'-6" high by 12" wide were used in the analysis. The additional 9" wide by 11" high portions of concrete on the interior faces at the bottom of the arch ribs were not included in the calculation of arch rib capacity to account for spalls and to be conservative, although this concrete weight was included for dead load purposes.

2. The arch ribs were analyzed using a two-dimensional frame model created in STAAD.Pro v8i with fixed supports at the bases.

3. The reinforced concrete arch ribs have a span length of 118"-0" from springing line to springing line with a rise of 18'-0" at the arch center.

4. For the As-Built and As-Configured analyses, an ultimate compressive strength (f'_c) of 1600 psi was utilized for the concrete. For the As-Inspected analysis, the ultimate compressive strength was increased to 2000 psi based on concrete testing that has been performed on the structure.

5. Flexural reinforcing steel consists of both historic Khan and Truscon reinforcing systems. There are three unique reinforcing patterns across the length of the arch ribs, as follows:
    a. Top Arch Segments - Two (2) 1" by 3" Kahn bars (top and bottom)
    b. Middle Arch Segments - Two (2) 1" by 3" Kahn bars and one 3/4" diameter Truscon bar (top and bottom)
    c. Lower Arch Segments - Two (2) 1" by 3" Kahn bars and one 1" diameter Truscon bar (top and bottom)

6. A yield strength (f_y) of 33 ksi was assumed for the reinforcing steel based on an allowable stress of 16 ksi shown in the plans. For the analysis of the bridge under original design loads, this 16 ksi allowable stress was used for reinforcing steel in accordance with the original plans and AASHTO Article 8.15.2.2. For the analysis of the bridge under modern design loads, allowable stresses were based on AASHTO MBE Table 6B.5.2.3-1 with an allowable stress of 18 ksi for Inventory level and 25 ksi for Operating level.

7. The dead loads were calculated by hand and applied using distributed or concentrated loads within the model. Loads were applied directly to the arch ribs, to the spandrel columns and walls, or through the deck, as appropriate.

8. The original decorative concrete railing was considered for dead load in the As-Built analysis, while the heavier railing currently installed was considered in the As-Configured and As-Inspected analyses.

9. The 1" thick concrete wearing surface was considered for the As-Configured and As-Inspected analyses only.

10. The live load utilized for the analysis of each beam consists of a uniform distributed load of 80 psf from the original design plans with a tributary area of half of the deck. A 90 psf pedestrian load was utilized for the Inventory level analysis, while a 90 psf pedestrian load and H5 truck were utilized concurrently for the Operating level analysis. The extents and position of the live load was varied in order to create maximum load effects on the structure, based on recommendations of load positions for pedestrian loading on arches and frames, as well as applied along the full length of the structure.

11. Transverse live load distribution factors for the H5 truck in the Operating level analysis were calculated by the lever rule.

12. In order to be conservative and due to the uncertainty of placement of Truscon bars based on the original plans, concrete clear cover was assumed to be 2 1/2" for all reinforcement.

13. Axial-moment interaction diagrams were developed based on KDOT Column Expert v6.0 utilizing the arch rib section properties, provided reinforcement, and allowable stresses. The arch rib sections were input into the program as symmetric, and interaction diagrams were plotted assuming unconfined concrete.

14. Services loads from the STAAD output were charted on axial-moment interaction diagrams for each arch rib section for the appropriate arch rib members, and the ratio of the governing load effects to the capacity shown on the interaction diagram were used in order to determine the capacity to demand ratios for each member.

15. Based on photographs from the field investigation and field measurements from previous analysis, 1/8" deep section loss was assumed for the reinforcing steel in arch ribs in the As-Inspected analysis.

Based on the results of the structural analysis for the original design loading, the capacity to demand ratios for the arch ribs are above 1.0 for all analysis alternatives. The As-Built capacity to demand ratio was calculated to be 1.11, indicating that the arch ribs satisfy the original design criteria for the structure. The As-Configured capacity to demand ratio is 1.07 due to the added dead load from the concrete wearing surface and newer bridge railing. The As-Inspected capacity to demand ratio is 1.22. For this analysis alternative, despite the decrease in reinforcing steel area due to section loss, the capacity to demand ratio increases because the arch ribs function primarily in compression and ultimate compressive strength is increased in this alternative due to concrete testing.

The capacity to demand ratios for the longitudinal spandrel beams for modern pedestrian bridges loadings are shown in **Table 5**.

Bridge Element	Capacity-to-Demand Ratios (ASD)					
	As-Built		As-Configured		As-Inspected	
	Inventory (90 psf)	Operating (90 psf + H5)	Inventory (90 psf)	Operating (90 psf + H5)	Inventory (90 psf)	Operating (90 psf + H5)
Arch Rib	1.14	1.55	1.07	1.50	1.23	1.69

Table 5 – Summary of structural analysis results (presented as capacity to demand ratios at both Inventory and Operating levels) for the arch ribs based on modern pedestrian bridge loads.

All capacity to demand ratios for the arch ribs are above 1.0 under current pedestrian bridge design loads. The As-Built capacity to demand ratio is 1.14 for Inventory level and 1.55 for Operating level, indicating that the arch ribs in the original structure was designed with sufficient strength to carry these modern loads. The As-Inspected capacity to demand ratio is 1.23 for Inventory level and 1.69 for Operating level. Despite accounting for additional loads and section loss of reinforcing steel, the capacity to demand ratios for the arch ribs in the As-Inspected condition are higher than those in the As-Built because of the additional compressive strength considered due to concrete testing. Because arch ribs function primarily as compression members, this higher strength provides a significant increase in capacity. As such, the analysis results indicate that the arch ribs are capable of carrying modern pedestrian design loads in its current configuration with all existing factors included.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the structural analysis for original design loads with design-level ASD allowable stresses, the primary load carrying bridge elements exceed the 1.0 capacity to demand ratio utilizing ASD methodology where a factor of 0.4 was used for the allowable stresses of the concrete and a factor of approximately 0.5 was used for steel, per AASHTO Article 8.15.2. TranSystems believes the results of our analysis best corroborate the original design intent of the structure and demonstrates that the bridge has sufficient structural capacity to carry the loading specified at the time of construction. Furthermore, our analysis verifies that based on the current configuration and condition of the bridge, the bridge components maintain their safe loading capacity.

In addition, TranSystems performed a structural analysis of the bridge for modern code-prescribed design loadings. This analysis is based on a 90 psf pedestrian loading at the Inventory level, and a 90 psf pedestrian loading in conjunction with an H5 Truck (5-ton maintenance vehicle) at the Operating level. For this analysis, allowable stresses for Inventory and Operating levels utilized for reinforcing steel and concrete are based on AASHTO MBE Tables 6B.5.2.3-1 and 6B.5.2.4.1-1. The capacity to demand ratios for the longitudinal spandrel members and arch ribs are all above 1.0 for all three analysis alternatives. The capacity to demand ratios for the deck are above 1.0 for the As-Built and As-Inspected analysis alternatives, while the capacity to demand ratio for the As-Configured analysis alternative is 0.98. Overall, these results indicate the bridge is capable of carrying these modern design loads in its originally constructed state and in its current condition.

Based on the results of the analysis, the capacity of the primary structural members, in both Inventory and Operating cases, exceed the existing loading condition of the bridge and our recommended future loading cases (representing modern-day loads) and therefore bridge rehabilitation would not require structural strengthening or replacement due to their load-carrying capacity.

In conjunction with the concrete material testing results [see TranSystems' Letter: *Lake Park Arch Bridge over Ravine Road – Concrete Testing Results dated June 18, 2018*], epoxy injection of the cracks and structural patching of the bridge would be recommended where shallow depth concrete repairs with doweled-in rebar mesh could be implemented for long term aesthetic improvements. These types of concrete repairs, along with the application of a concrete sealant as a 5 year routine maintenance item, would prevent further concrete and steel deterioration and would eliminate potential falling hazards due to spalling concrete and extend the service life for another 50 years.

If you have any questions, comments, or require further information, please contact me at wrweir@transystems.com or 216-408-5394.

Very truly yours,

P.E.

Wesley Weir, P.E.
Senior Bridge Engineer / Vice President

Calculations

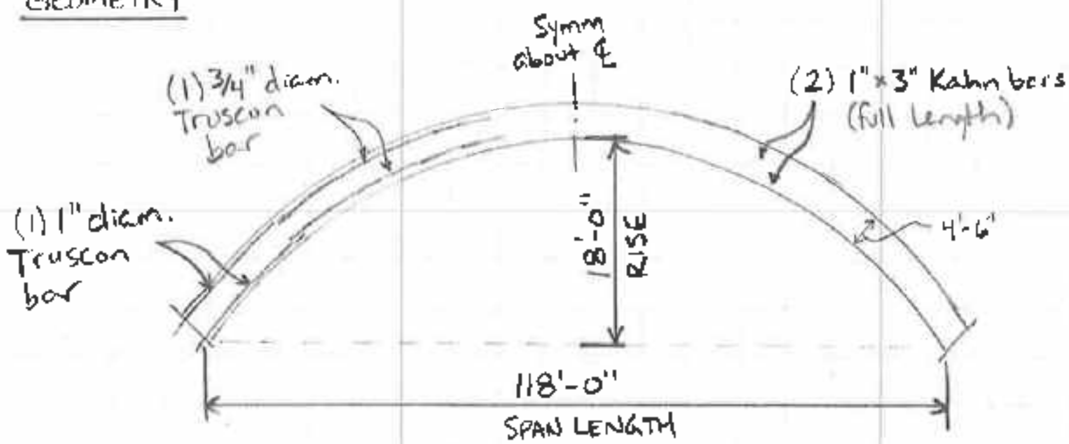
Original Design Loads

LAKE PARK ARCH BRIDGE - LOAD RATING

LOAD RATING ASSUMPTIONS

- Normal Weight Concrete
  - ↳ Weight = 150 pcf
  - ↳ Compressive Strength
    - a) As-built : f'_c = 1,600 psi (based on 400 psi allowable from plans w/ safety factor 4)
    - b) Concrete Testing : Say f'_c = 2,000 psi
- Reinforcing Steel
  - ↳ Khan and Truscon historical bar systems
  - ↳ Tensile Strength : f_y = 33 ksi (based on 16,000 psi allowable from plans)
- Live loading : 80 psf (from original plans)

GEOMETRY



ANALYSIS ALTERNATIVES

#1) AS-BUILT - Consists of the original structure in its as constructed condition with original section properties, geometry, material specifications, etc.

#2) AS-CONFIGURED - Consists of the structure in its existing configurations, accounting for modifications to the structure such as new railings or wearing surfaces, with original as-built section properties and material specifications

#3) AS-IS - Consists of existing structure, accounting for modifications to the structure, section loss, material testing, etc.

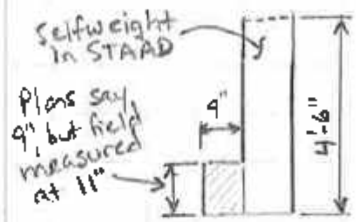
Calculations for 1/4 US Grid

LAKE PARK ARCH BRIDGE – LOAD RATING

DEAD LOADS

- Arch weight will be modeled with selfweight within STAAD w/ additional load applied for extra section not modeled.
- Deck and parapet weight will be modeled as uniform distributed load (each arch line supports one parapet + half of deck)
- Struts and walls will be modeled with point loads.
- Spandrel wall input as combination of varying distributed load and concentrated load based on hand calculations.

Arch Dead Load :



Arch section

Weight = $(0.15\,\text{kcf})(9''/12)(11''/12) = 0.103\ \text{k/ft}$
(Remaining arch dead load covered by selfweight in STAAD)

Deck and Parapet :

As-Built = 0.98 k/ft , As-configured = 1.13 k/ft (see additional calculations for analysis alternatives)
(Also As-is)

Walls :

Five 8" thick R/C walls extend from underside of deck to bottom of arch ribs. Height varies and was scaled off drawings imported into CAD. Walls are 12' wide, each arch carries half.

4'-6" wall :	$(0.15\,\text{kcf})(8''/12)(4.5')(12'/2)$	= 2.7 kips	(midspan)
7'-6" wall :	" " (7.5') "	= 4.5 kips	(at end of teardrop)
16'-6" wall :	" " (16.5') "	= 9.9 kips	(between openings)

Struts :

Based on photographs, says struts are 16" tall x 12" wide.

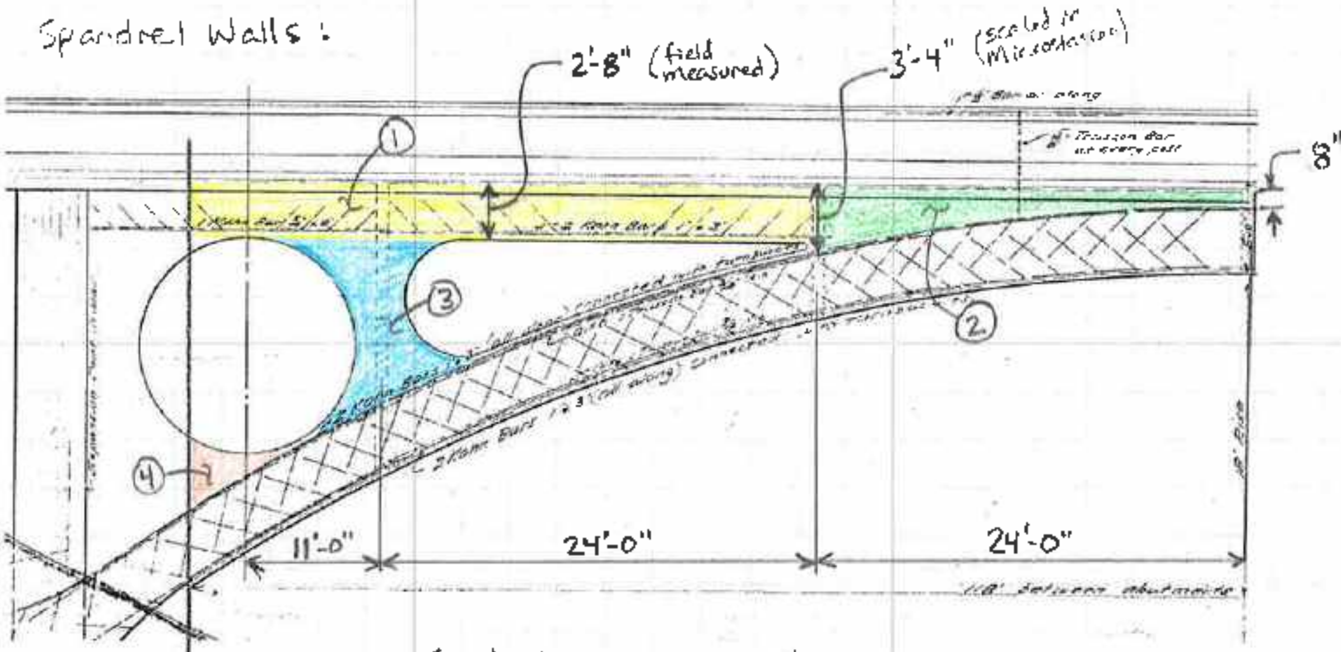
Weight = $(0.15\,\text{kcf})(16''/12)(12''/12)(12'/2) = 1.2\ \text{kips}$

Calculators for 1/4 US Grid

Made By: DWC	Date: 7/24/18	Job No: P402180060
Checked By: SFH	Date: 7/25/18	Sheet No.

LAKE PARK ARCH BRIDGE - LOAD RATING

Spandrel Walls:



SPRING LINE

Spandrel wall is 12" wide

① SPANDREL BEAMS: $g_1 = (0.15\text{ kcf})(2.667')(12''/12) = 0.40\text{ k/ft}$

② TAPERED PORTION: Use trapezoidal distributed load over 24' length

$$g_{start} = (0.15\text{ kcf})(3.333')(12''/12) = 0.50\text{ k/ft}$$
$$g_{end} = (0.15\text{ kcf})(8''/12)(12''/12) = 0.10\text{ k/ft}$$

③ BETWEEN OPENINGS: Calculate total weight and distribute load over two arch elements in STAAD (length measured 6.5' in STAAD)

Width = 5'-6" average

Height = 16.5' - 4.5' - 2.667' = 9.333'

$$\therefore g_3 = (0.15\text{ kcf})(9.333')(5.5')(12''/12)\left[1/6.5'\right] = 1.18\text{ k/ft}$$

④ BELOW CIRCULAR OPENING: Ignore this area due to proximity to spring line and this portion being monolithic with portion of wall beyond spring line.

LAKE PARK ARCH BRIDGE - LOAD RATING

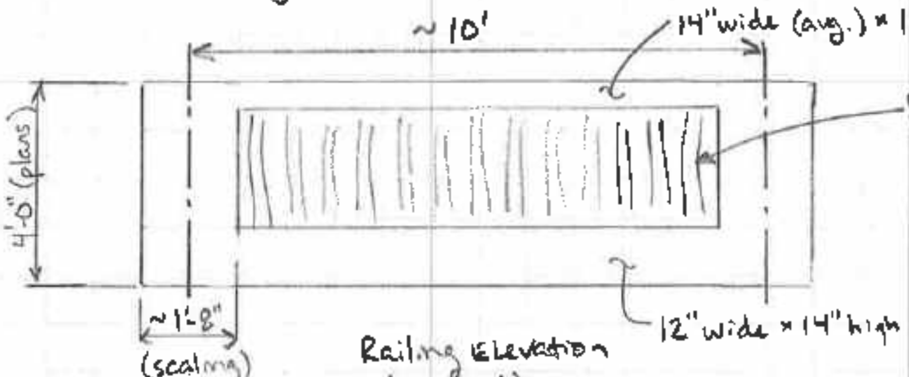
DEAD LOAD - AS-BUILT vs. AS-CONFIGURED / AS-IS

The as-built structure consists of a 6" R/C deck with no wearing surface and a R/C railing with posts, openings, and decorative balusters.

For the as-configured and as-is analyses, consider the updated railing detail consisting of a solid railing w/ decorative panels. Also, a concrete wearing surface appears to have been added to the structure.

As-Built: Weight $= 0.56^{k/ft} + 0.42^{k/ft} = \underline{0.98^{k/ft}}$

- Deck: $(0.15\text{ kcf})(6''/12)(15'/2) = 0.56\ ^{k/ft}$
- Railing: $\left[(0.15\text{ kcf})\left[(1.667')(4')(12''/12) + (8.333')\left(\frac{14\times10}{12^2} + \frac{12\times14}{12^2}\right)\right] + 13(0.041^k)\right] \times 1/10'$ $= 0.42\ ^{k/ft}$



Railing Elevation (original)

Decorative Baluster (typ.)
- 13 balusters per bay
- Scaling from plans, balusters appear to be 24" high, and diameter varies from 2½" to 6" (say 5" average)

$\therefore$ Volume $= (2')\frac{\pi}{4}(5/12)^2 = 0.273\ ft^3$

$\therefore$ Weight $= (0.15)(0.273) = 0.041\ k$

As-Configured / As-Is: Weight $= 0.66^{k/ft} + 0.47^{k/ft} = \underline{1.13^{k/ft}}$

- Deck: $(0.15\text{ kcf})(15'/2)\left(\frac{6''+1''}{12}\right) = 0.66\ ^{k/ft}$
- Railing: $(0.15\text{ kcf})(48''/12)(9.5''/12) = 0.47\ ^{k/ft}$

Deck thickness includes 6" original deck plus 1" thick concrete wearing surface based on photo of Core #3 from petrographic analysis. Railing dimensions based on field measurements and photographs.

Ravine Concrete Arch Footbridge at Milwaukee Lake Park

A Cultural Heritage Assessment Study and Report

Historic Preservation Office
City of Milwaukee

200 E. Wells Street, Milwaukee, WI 53202
Phone 414-286-5712, fax 414-286-3004
carlen.hatala@milwaukee.gov

Carlen Hatala
Emma Rudd
Leila Saboori
Nader Sayadi

July 2016

APS PROJECT NO: 10-09372 **DATE:** MAY 7, 2018
PROJECT: LAKE PARK FOOTBRIDGE
GILES PROJECT NO.: 1M-1803020, MILWAUKEE, WI

PHOTO: 1

Deck Core
(5" long
sample)

SAMPLE ID: 3 **DESCRIPTION:** The overall profile of the core as received, with the top surface oriented to the left.

PHOTO: 2

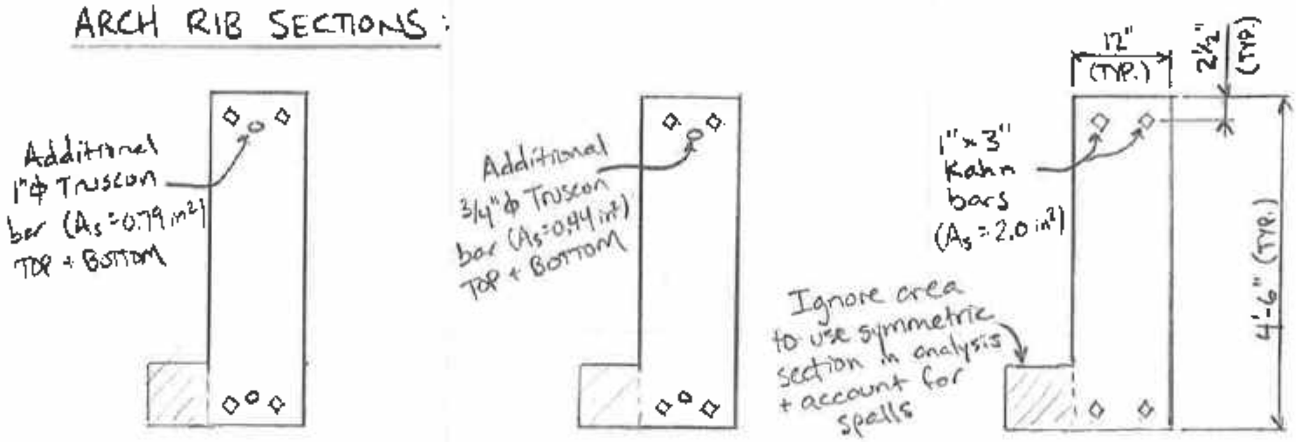
SAMPLE ID: 3 **DESCRIPTION:** The top surface of the core as received.

TranSystems

Made By: DWC	Date: 7/24/18	Job No. P402180060
Checked By: CFH	Date: 7/25/18	Sheet No.

LAKE PARK ARCH BRIDGE - LOAD RATING

ARCH RIB SECTIONS:



LOWER ARCH
- A_s = 2.79 in² TOP + BOTTOM
- FROM SPRINGING LINE TO WALL BETWEEN ARCHITECTURAL OPENINGS (11'-0" ±)

MIDDLE ARCH
- A_s = 2.44 in² TOP + BOTTOM
- FROM WALL BETWEEN OPENINGS TO 6'-0" ± BEYOND END OF TEARDROP (24'-0" ±)

TOP ARCH
- A_s = 2.00 in² TOP + BOTTOM
- TOP PORTION OF ARCH BELOW CONTINUOUS SPANDREL WALL

Treat arch ribs as rectangular beam:
$b = 12"$, $h = 54"$

Although clear cover can be assumed as 2", use clear cover of 2½" for all members due to uncertainty of placement for additional Truscon bars and to be conservative.

Reinforcement : Use cores of bars only due to bars being bent for shear throughout arch ribs.

AS-BUILT / AS-CONFIGURED

TOP ARCH: (2) 1"x3" Khan bars $A_s = 2 \times (1.0\text{ in}^2) = 2.00\text{ in}^2$
→ Use (2) #9 bars for analysis

MIDDLE ARCH: (2) 1"x3" Khan bars + (1) 3/4" Truscon bar
$A_s = 2.00\text{ in}^2 + 0.44\text{ in}^2 = 2.44\text{ in}^2$ $A_{bar} = \frac{2.44}{3} = 0.813\text{ in}^2$
→ Use (3) custom bars $d_{bar} = 1.02"$

LOWER ARCH: (2) 1"x3" Truscon bars + (1) 1"φ Truscon bar
$A_s = 2.00 + 0.79 = 2.79\text{ in}^2$ $A_{bar} = 2.79/3 = 0.93\text{ in}^2$
→ Use (3) custom bars $d_{bar} = 1.09\text{ in}^2$

TranSystems

Made By: DWC | Date: 7/25/18 | Job No: P402180060
Checked By: SFH | Date: 7/25/18 | Sheet No

LAKE PARK ARCH BRIDGE - LOAD RATING

AS-INSPECTED

- Account for additional compressive capacity due to concrete testing with f'_c = 2000 psi ($\therefore f_c = 0.4(2000) = 800$ psi)
- Include section loss on reinforcement due to corrosion based on field measurements. From field investigation, exposed 1"x3" Khan bar cores measured 7/8"x7/8" ∴ Assume 1/8" section loss.

TOP ARCH: $A_{bar} = (7/8" \times 7/8") = 0.766 \text{ in}^2$ each
→ Use (2) #8 bars

MIDDLE ARCH: $A_s = 2(0.766) + 0.44\left(\frac{.625}{.75}\right) = 1.84 \text{ in}^2$
$A_{bar} = 1.84/3 = 0.61 \text{ in}^2$
→ Use (3) #7 bars

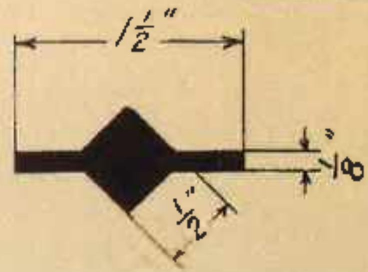
LOWER ARCH: $A_s = 2(0.766) + 0.79\left(\frac{.875}{1}\right)^2 = 2.14 \text{ in}^2$
$A_{bar} = 2.14/3 = 0.713 \text{ in}^2$
$d_{bar} = 0.95"$
→ Use (3) custom bars

Beam Definitions in STAAD:

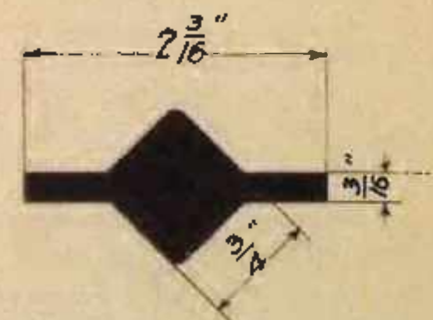
Beams 1-4: LOWER ARCH
Beams 5-14: MIDDLE ARCH
Beams 15-20: TOP ARCH

Calculations for 1/4 US Grid

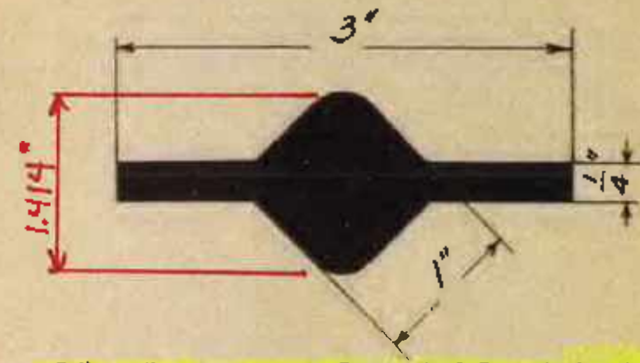
These bars can have any standard size cuts as shown in Figure 11 and will be sent in lengths as ordered. In making calculations for strength of reinforced beam, assume the area of the entire cross section as here given.



1½"×½". Area .38□" Weight 1.4# per ft.



2 3/16"×¾". Area .78□" Weight 2.7# per ft.



3"×1". Area 1.42□" Weight 4.8# per ft.

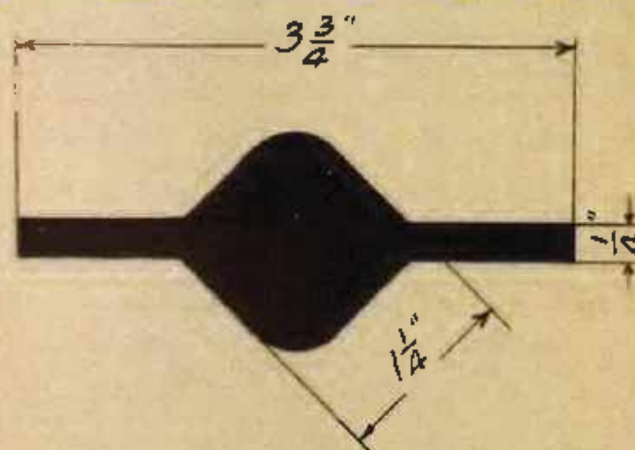


FIG. 13. 3¾"×1¼". Area 2.0□" Weight 6.9# per ft.

Kahn System of Reinforced Concrete

So much actual work is being done at the present time with reinforced concrete, and in general, the subject is receiving such intense interest by those taking part in buildings, bridges, or other constructions, that the new method of steel reinforcement herein described, it is believed, will be of interest.

The advantages of reinforced concrete above steel, masonry, or wood, are so well known, that it is hardly necessary to enter into comparison here. Reinforced concrete is absolutely free of any of the serious objections which exist in the use of these other materials. It is fire proof, and rust proof, but what is most advantageous about this type of construction, is the fact that its strength continually increases with age.

Reinforced concrete lends itself admirably to the construction of walls, columns, floors, roofs, and all parts of buildings; to bridges, arches, culverts, abutments, retaining walls, tunnels, foundations, railway ties, and in general, it replaces, to advantage, all masonry or steel construction.

The Kahn trussed bar consists of a half truss, struck up directly from a single rolled section, and provides the tensional members only. Concrete within itself is an excellent material to take up compressive strains, but is comparatively weak for resisting tensile strains. The Kahn bar when imbedded in a mass of concrete, therefore, supplies strength to the latter where this is

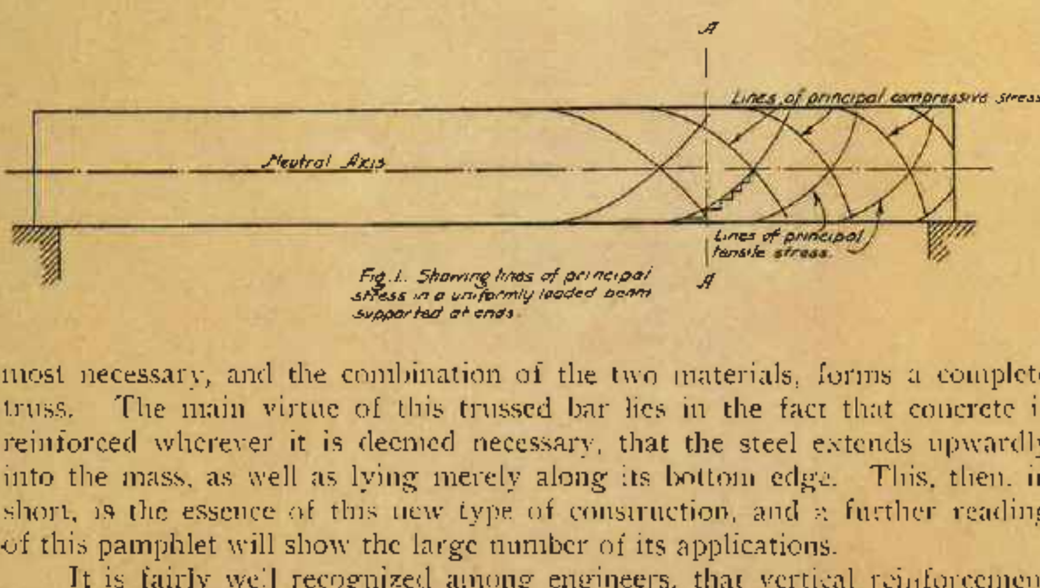


Fig. 1. Showing lines of principal stress in a uniformly loaded beam supported at ends.

most necessary, and the combination of the two materials, forms a complete truss. The main virtue of this trussed bar lies in the fact that concrete is reinforced wherever it is deemed necessary, that the steel extends upwardly into the mass, as well as lying merely along its bottom edge. This, then, in short, is the essence of this new type of construction, and a further reading of this pamphlet will show the large number of its applications.

It is fairly well recognized among engineers, that vertical reinforcement for concrete beams is just as essential as the horizontal reinforcement, and in many cases to accomplish this purpose, the horizontal rods are surrounded by U shaped stirrups of band or twisted iron. It was noticed at first by European engineers that a concrete beam, when tested to destruction under uniform loading, invariably failed by shear at the ends, the lines of rupture corresponding closely to the lines of principal compressive stress for such a beam, as is shown in Figure 1. In this country engineers were apparently very slow to

Tables

General Description In these tables it is assumed that floors have been constructed in accordance with the Kahn System of Reinforcement, as illustrated in our catalogue, and that bars have been inverted in their position over supports to procure the effects of continuous beam action.

2. Concrete to be composed of the best grade of Portland Cement, sharp, clean sand and broken stone or gravel, in the proportions of 1:2½:5 for floor slabs, and 1:2:4 for beams. Broken stone or gravel a 1" ring.

3. Bars to be placed at least ¾" from the bottom of the beam, and the concrete thoroughly rammed in place.

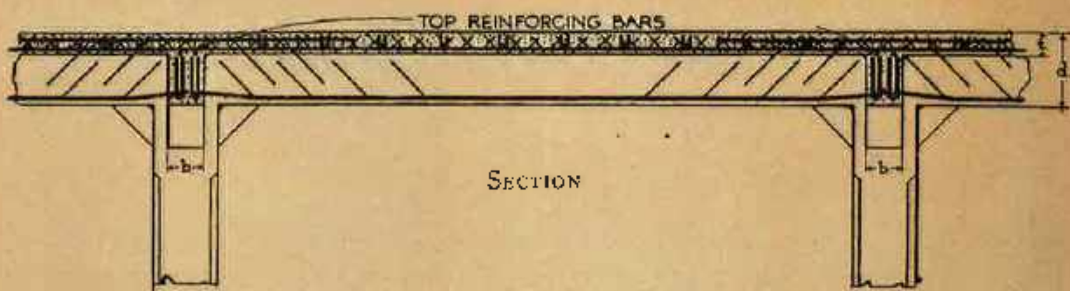
4. Centering not to be removed in less than two and one-half weeks, if the concrete has not been subjected to frost. If freezing has occurred, centering must not be removed until every indication of frost is removed, and the concrete thoroughly set.

Tables were calculated for a factor of safety of 4. However, when this system is incorporated into a combination of continuous beams, the resultant factor of safety rises to 6 or 7. This is due to arch action, tension in concrete, continuity, and slab action, as well as numerous other facts which, on account of the difficulty attending their exact calculation, it is deemed advisable to neglect in these tables.

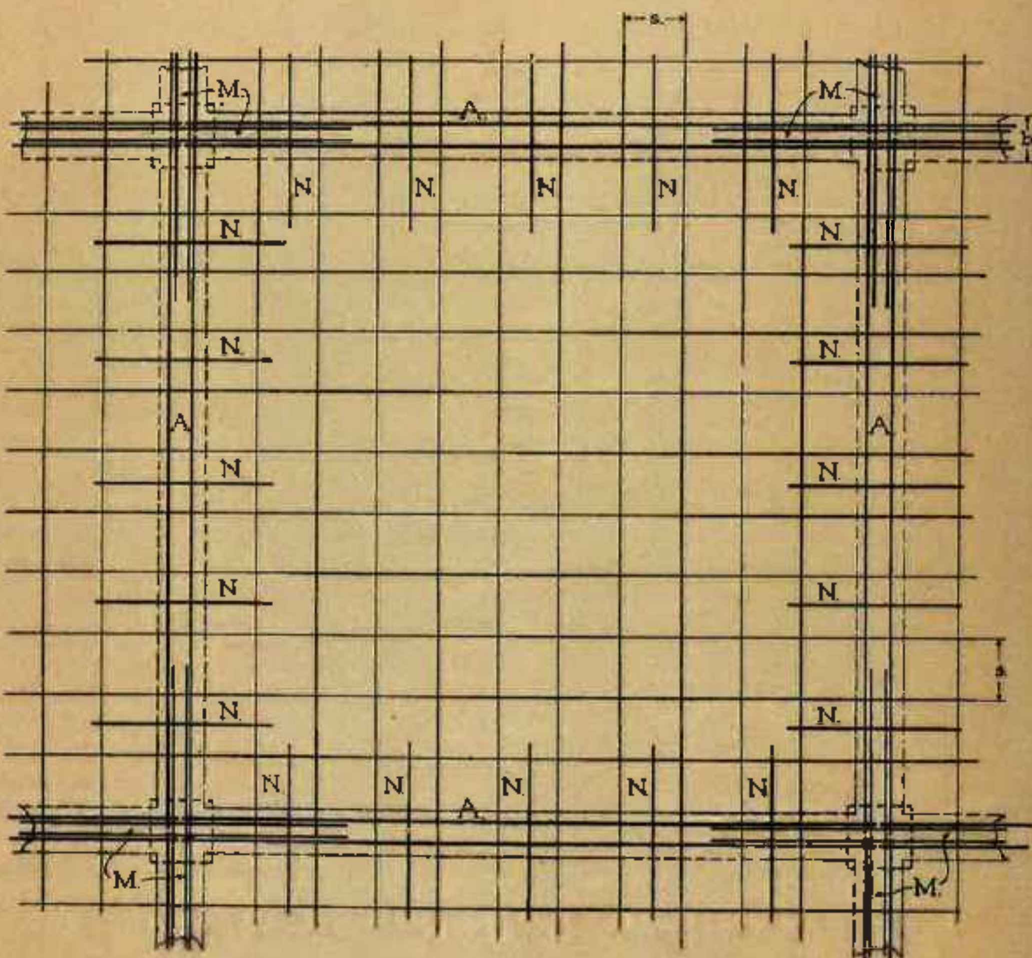
The following are the usual assumptions made in practice for superimposed loads:

Floors of dwellings and offices....................	70 lbs. per sq. ft.
Floors of churches, theaters, and ball rooms......	250 " " " "
Floors of warehouses.........................200 to	250 " " " "
Floors for heavy machinery......................250 to	400 " " " "

III



SECTION



Plan.

FIG. 17.

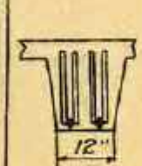
See note on bottom of page 28.

Safe loads in hundreds of pounds uniformly distributed for concrete beams reinforced with Kahn Trussed Bars

Safe Loads below are figured for fibre stress in steel of 16000 lbs. per square inch.

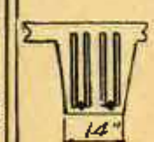
If beams are made continuous across supports by inverting reinforcement bars, safe loads may be increased by ⅓

D	Distance between center of supports in feet																					
	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
10	244	217	195	177	163	150	140	130	122	115	108	103	98	93	89	85	81	78	75	72	70	68	65
12	328	290	261	237	218	200	186	174	163	153	145	137	130	124	118	113	107	104	100	97	93	90	87
14	394	350	315	286	263	242	225	210	197	185	175	160	157	150	143	137	131	126	121	117	112	108	105
16	448	400	358	326	298	276	256	239	224	211	199	188	179	170	163	155	149	143	138	132	128	123	119
18	508	452	406	369	338	312	290	270	254	239	226	214	203	193	184	176	169	162	156	150	145	140	135
20	568	504	453	412	378	348	324	302	283	266	252	238	226	216	206	197	189	181	174	168	162	156	151
22	634	564	507	461	423	390	362	338	317	298	281	266	254	242	230	220	211	203	195	186	182	175	169
24	680	605	544	395	455	418	388	362	340	320	302	286	272	259	247	236	226	217	209	201	194	188	181



1"x3" Bars
A=2.84 sq. in
W=9.6 lbs.
L=12" & 18"

12	452	402	361	328	300	277	258	240	226	212	200	190	182	172	164	157	150	144	139	134	129	125	121
14	540	480	432	392	360	332	308	288	270	254	240	227	216	206	196	188	180	173	166	160	154	149	144
16	625	555	499	455	416	382	357	333	312	294	277	263	250	238	227	217	208	200	192	185	178	172	167
18	707	630	566	515	472	435	405	377	352	333	314	298	283	270	257	246	236	226	218	210	202	195	188
20	758	700	630	572	525	485	450	420	394	370	350	331	315	300	286	273	262	252	242	233	225	217	210
22	867	770	694	630	577	535	495	462	434	407	386	365	346	330	315	302	289	278	267	257	248	239	231
24	960	854	768	700	640	590	550	512	480	452	427	404	382	366	349	334	320	307	295	284	276	265	256
26	1040	925	832	757	695	640	595	555	520	490	462	438	415	396	378	361	346	333	320	308	297	287	277
28	1125	1000	900	820	750	692	642	600	563	530	500	474	450	428	410	392	375	360	346	334	321	310	300
30	1210	1075	968	878	805	744	680	645	605	568	536	508	483	460	440	420	404	387	372	358	345	333	321



1¼"x3¾" Bars
A—4.0 sq. in.
W=13.8 lbs.
L—18"

A Area of steel in sq. in. W—Weight of steel per linear foot L—Length of diagonals
Batter of sides of beams should not be less than 1 : 6 for beams up to 16 inches in depth.
Beams more than 16 inches deep may have vertical sides
D—Depth of beam, in inches, from top of slab to center of steel reinforcement.

Spacing of bars in Kahn Reinforced Floors for various uniform loads

¾"x2" Bars   Area=.78 sq. in.

Safe live loads per square foot		Distance between supports in feet																					
		10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
6" Floor Slab	100									19	17	15½	14										
	125								16½	15	13½	12½	11½										
	150						18½	16	14	12½	11¼												
	175					18	15½	13½	12	11	9¼												
	200				18	15½	13½	12	10½	9½	8½												
	250			17	14½	12½	11	9½	8	7½													
	300		17	14	12	10½	9	8	7	6													
	350	17½	14½	12½	10½	9	7½	6½	6														
	400	15½	12½	10½	9	8	7	6															
5" Floor Slab	100								18½														
	125						18½	17	15														
	150					18	15	14	12½														
	175				17½	15½	13½	12	10½														
	200			18½	15½	13½	12	10½															
	250		17½	15	12½	11	9½	8½															
	300	17½	14½	12	10	9	8																
	350	15	12½	11½	8½	7½	6½																
	400	13	11	9	8	7	6																

Spacing above is given in inches.

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 12-Jul-18
CHECKER DATE 25-Jul-18
JOB NAME LAKE PARK ARCH BRIDGE LOAD RATING
JOB COMMENT ARCH RIBS
ENGINEER NAME DWC
CHECKER NAME SFH
END JOB INFORMATION
INPUT WIDTH 79
UNIT FEET KIP
JOINT COORDINATES
*Node X Y Z
1 0 0 0; 2 2.75 1.789  ; 3 5.5 3.46  ; 4 8.25 5.019  ; 5 11 6.472 0;
6 14 7.942 0; 7 17 9.297 0; 8 20 10.542 0; 9 23 11.68 0; 10 26 12.716 0;
11 29 13.653 0; 12 32 14.493 0; 13 35 15.239 0; 14 38 15.893 0;
15 41 16.456 0; 16 44 16.93 0; 17 47 17.317 0; 18 50 17.616 0;
19 53 17.83 0; 20 56 17.957 0; 21 59 18 0; 22 62 17.957 0; 23 65 17.83 0;
24 68 17.616 0; 25 71 17.317 0; 26 74 16.93 0; 27 77 16.456 0;
28 80 15.893 0; 29 83 15.239 0; 30 86 14.493 0; 31 89 13.653 0;
32 92 12.716 0; 33 95 11.68 0; 34 98 10.542 0; 35 101 9.297 0;
36 104 7.942 0; 37 107 6.472 0; 38 109.75 5.019 0; 39 112.5 3.46 0;
40 115.25 1.789 0; 41 118 0 0;
50  0   19  0   ; 51    11  19  0   ;
52  35  19  0   ; 53    41  19  0   ;
54  47  19  0   ; 55    53  19  0   ;
56  59  19  0   ; 57    65  19  0   ;
58  71  19  0   ; 59    77  19  0   ;
60  83  19  0   ; 61    107 19  0   ;
62  118 19  0   ;

MEMBER INCIDENCES
1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 5 6; 6 6 7; 7 7 8; 8 8 9; 9 9 10; 10 10 11;
11 11 12; 12 12 13; 13 13 14; 14 14 15; 15 15 16; 16 16 17; 17 17 18; 18 18 19;
19 19 20; 20 20 21; 21 21 22; 22 22 23; 23 23 24; 24 24 25; 25 25 26; 26 26 27;
27 27 28; 28 28 29; 29 29 30; 30 30 31; 31 31 32; 32 32 33; 33 33 34; 34 34 35;
35 35 36; 36 36 37; 37 37 38; 38 38 39; 39 39 40; 40 40 41;
50  1   50  ; 51    5   51  ; 52    13  52  ; 53    15  53  ;
54  17  54  ; 55    19  55  ; 56    21  56  ; 57    23  57  ;
58  25  58  ; 59    27  59  ; 60    29  60  ; 61    37  61  ;
62  41  62  ; 70    50  51  ; 71    51  52  ; 72    52  53  ;
73  53  54  ; 74    54  55  ; 75    55  56  ; 76    56  57  ;
77  57  58  ; 78    58  59  ; 79    59  60  ; 80    60  61  ;
81  61  62  ;

DEFINE MATERIAL START
ISOTROPIC CONCRETE
E 453600
POISSON 0.17
DENSITY 0.150336
```

```
ALPHA 5e-006
DAMP 0.05
TYPE CONCRETE
STRENGTH FCU 576
ISOTROPIC STEEL
E 4.176e+006
POISSON 0.3
DENSITY 0.489024
ALPHA 6e-006
DAMP 0.03
TYPE STEEL
STRENGTH FY 5184 FU 8352 RY 1.5 RT 1.2
END DEFINE MATERIAL

MEMBER PROPERTY AMERICAN
1 TO 40  PRIS YD 4.5 ZD 1
50 TO 62 PRIS YD 0.6667 ZD 3
70 TO 81 PRIS YD 3.167 ZD 1

CONSTANTS
MATERIAL CONCRETE ALL

MEMBER RELEASE
50 51 61 62 BOTH MY MZ
70 71 START MY MZ
70 71 END MY MZ FX
80 81 END MY MZ
80 81 START MY MZ FX
52 TO 55 57 TO 60 START MY MZ

SUPPORTS
1 41 FIXED

LOAD 1 LOADTYPE Dead TITLE DEAD LOADS
*ARCH LOAD
SELFWEIGHT Y -1.0 LIST 1 TO 40
MEMBER LOAD
1 TO 40 UNI GY -0.103

*DECK AND PARAPET (AS-BUILT)
MEMBER LOAD
70 TO 81 UNI GY -0.98

****DECK AND PARAPET (AS-CONFIGURED/AS-INSPECTED)
***MEMBER LOAD
***70 TO 81 UNI GY -1.13

*TRANSVERSE WALLS
JOINT LOAD
5 37 FY -9.9
```

```
13 29 FY -4.5
21 FY -2.7

*STRUTS
JOINT LOAD
9 17 25 33 FY -1.2

*SPANDREL WALLS
MEMBER LOAD
70 71 80 81 UNI GY -0.40
MEMBER LOAD
72 TRAP GY -0.50 -0.40
73 TRAP GY -0.40 -0.30
74 TRAP GY -0.30 -0.20
75 TRAP GY -0.20 -0.10
76 TRAP GY -0.10 -0.20
77 TRAP GY -0.20 -0.30
78 TRAP GY -0.30 -0.40
79 TRAP GY -0.40 -0.50
MEMBER LOAD
4 5 36 37 UNI GY -1.18

LOAD 2 LOADTYPE Live TITLE LIVE LOAD 1
MEMBER LOAD
73 UNI GY -0.48 3.25 6
74 TO 77 UNI GY -0.48
78 UNI GY -0.48 0 2.75

LOAD 3 LOADTYPE Live TITLE LIVE LOAD 2
MEMBER LOAD
70 TO 72 79 TO 81 UNI GY -0.48
73 UNI GY -0.48 0 3.25
78 UNI GY -0.48 2.75 6

LOAD 4 LOADTYPE Live TITLE LIVE LOAD 3
MEMBER LOAD
73 UNI GY -0.48 3.25 6
74 TO 81 UNI GY -0.48

LOAD 5 LOADTYPE Live TITLE LIVE LOAD 4
MEMBER LOAD
70 TO 72 UNI GY -0.48
73 UNI GY -0.48 0 3.25

LOAD 6 LOADTYPE Live TITLE LIVE LOAD 5
MEMBER LOAD
70 TO 77 UNI GY -0.48
78 UNI GY -0.48 0 2.75

LOAD 7 LOADTYPE Live TITLE LIVE LOAD 6
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MEMBER LOAD
78 UNI GY -0.48 2.75 6
79 TO 81 UNI GY -0.48

LOAD 8 LOADTYPE Live TITLE LIVE LOAD 7
MEMBER LOAD
70 TO 81 UNI GY -0.48

LOAD COMB 11 DL + LL1
1 1.0 2 1.0

LOAD COMB 12 DL + LL2
1 1.0 3 1.0

LOAD COMB 13 DL + LL3
1 1.0 4 1.0

LOAD COMB 14 DL + LL4
1 1.0 5 1.0

LOAD COMB 15 DL + LL5
1 1.0 6 1.0

LOAD COMB 16 DL + LL6
1 1.0 7 1.0

LOAD COMB 17 DL + LL7
1 1.0 8 1.0

PERFORM ANALYSIS
FINISH
```

Job Information

	Engineer	Checked	Approved
Name:	DWC	SFH	
Date:	12-Jul-18	25-Jul-18	

Project ID	
Project Name	

Comments

ARCH RIBS

Structure Type	SPACE FRAME

Number of Nodes	54	Highest Node	62
Number of Elements	65	Highest Beam	81

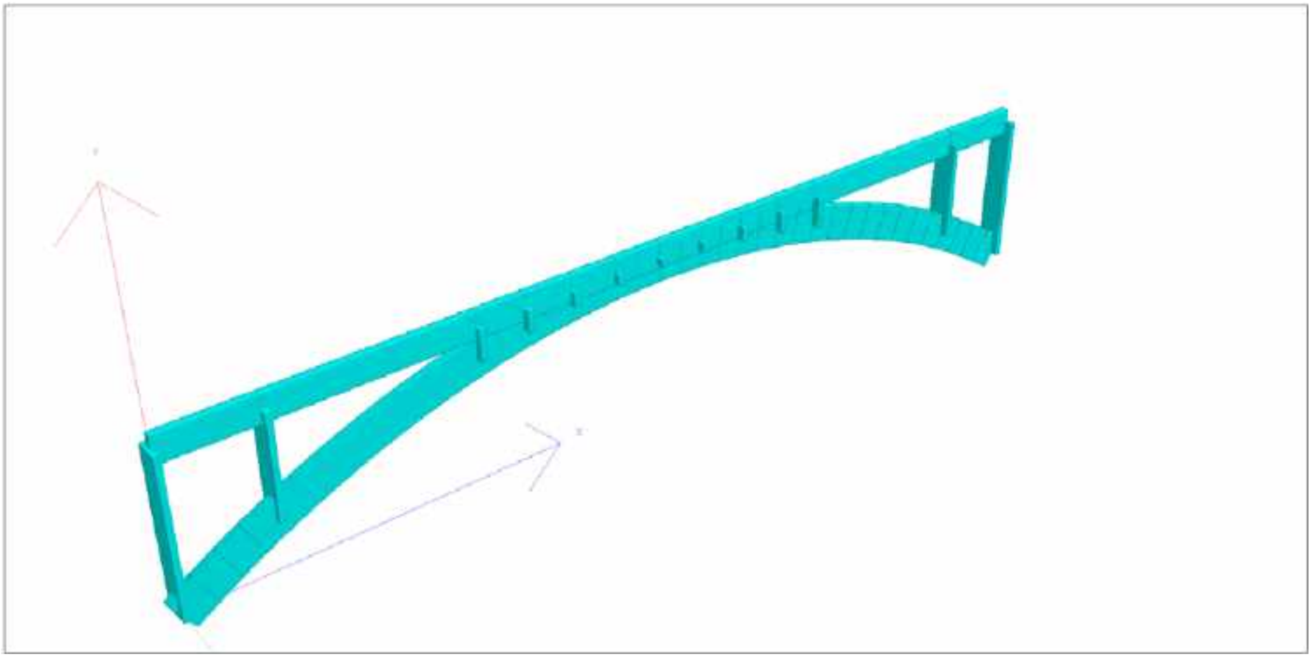
Number of Basic Load Cases	-2
Number of Combination Load Cases	7

Included in this printout are data for:

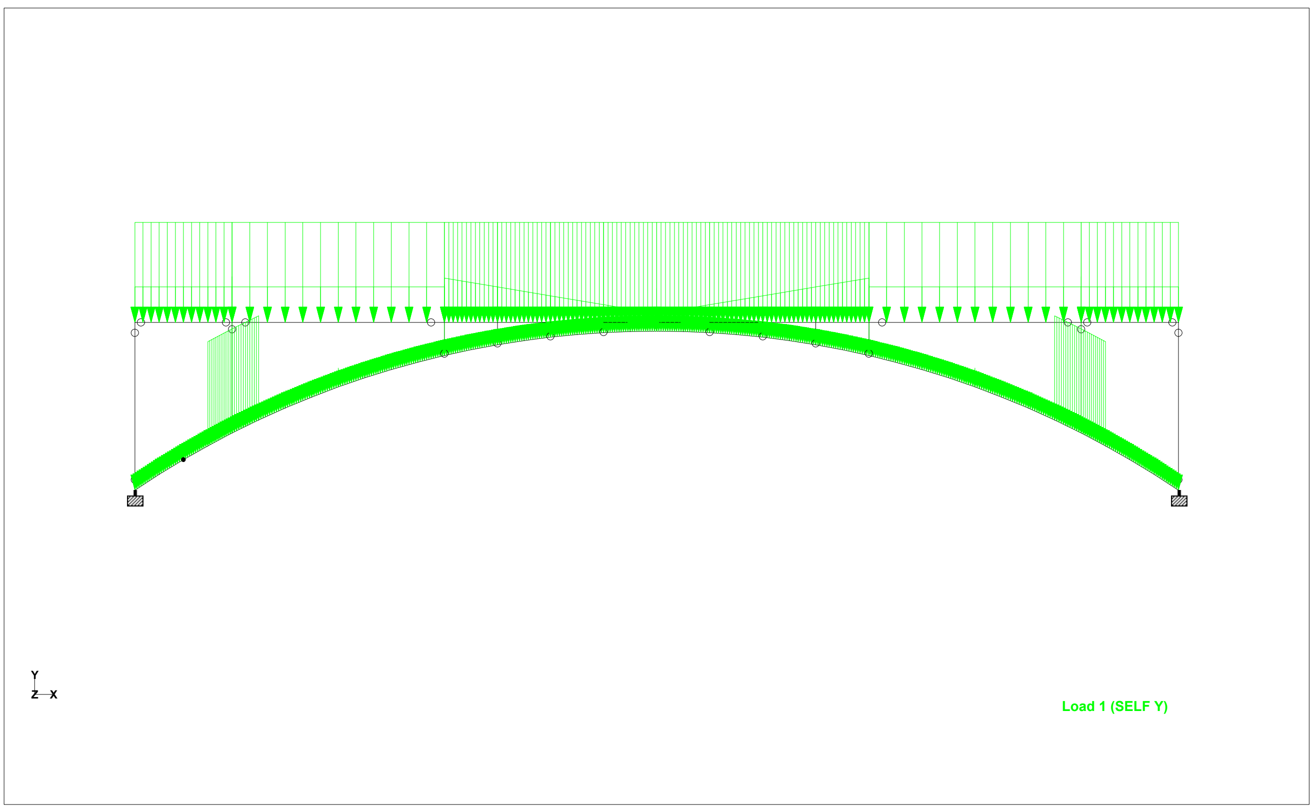
All	The Whole Structure

Included in this printout are results for load cases:

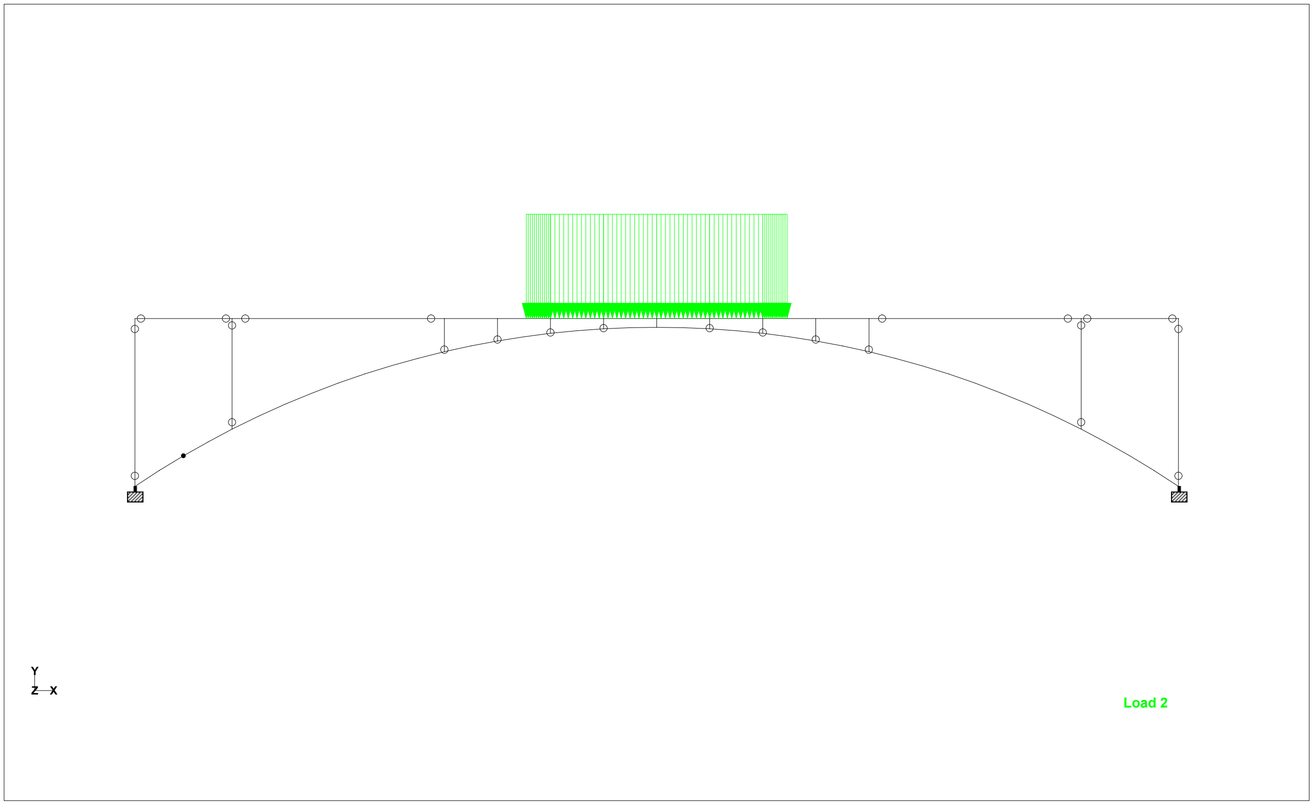
Type	L/C	Name
Primary	1	DEAD LOADS
Primary	2	LIVE LOAD 1
Primary	3	LIVE LOAD 2
Primary	4	LIVE LOAD 3
Primary	5	LIVE LOAD 4
Primary	6	LIVE LOAD 5
Primary	7	LIVE LOAD 6
Primary	8	LIVE LOAD 7
Combination	11	DL + LL1
Combination	12	DL + LL2
Combination	13	DL + LL3
Combination	14	DL + LL4
Combination	15	DL + LL5
Combination	16	DL + LL6
Combination	17	DL + LL7



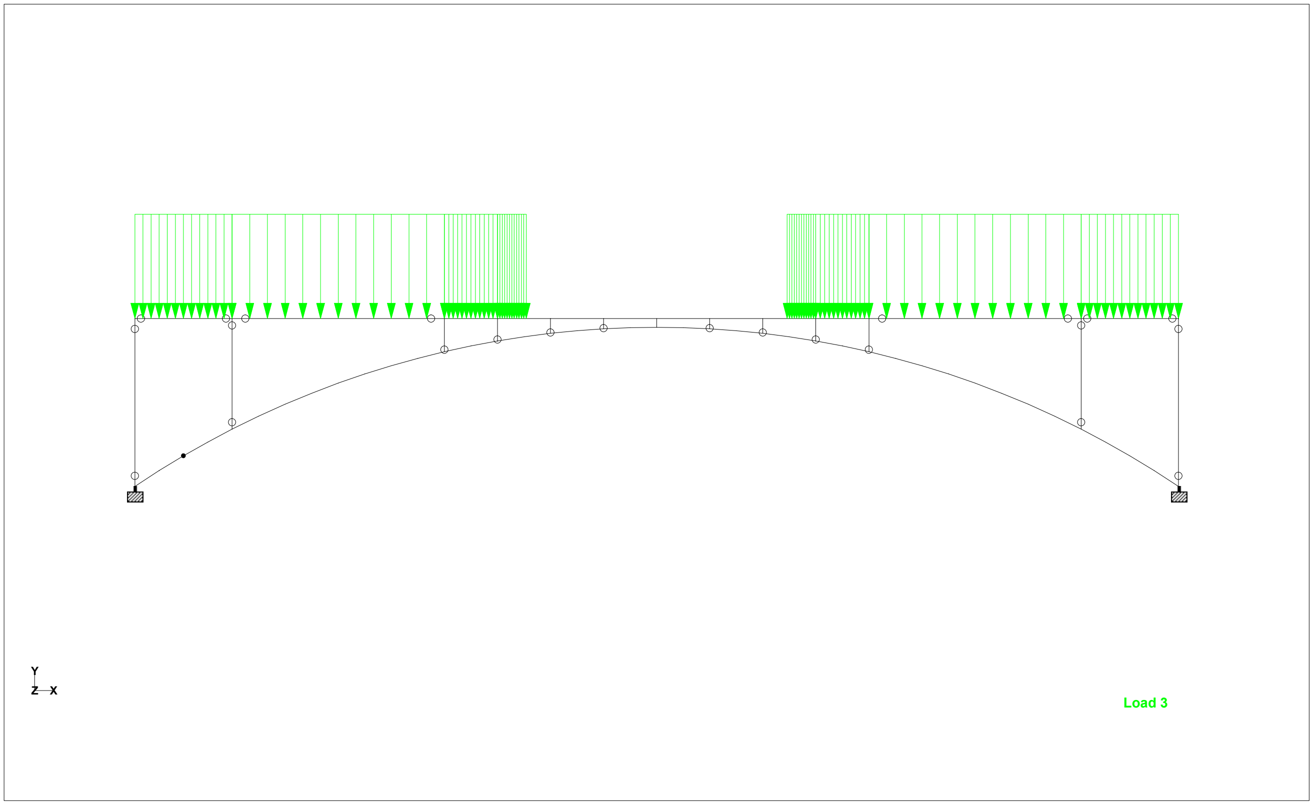
3D Rendered View



Dead Load



Live Load 1



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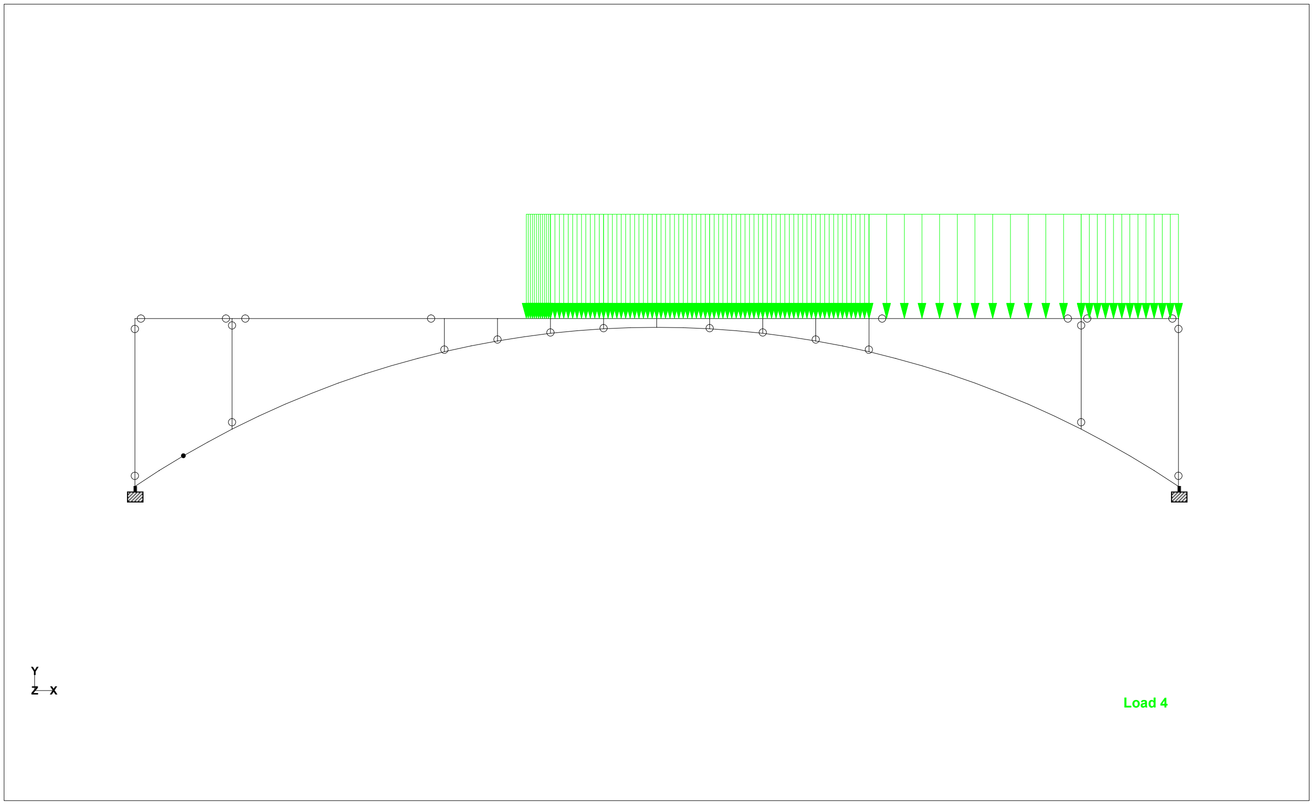

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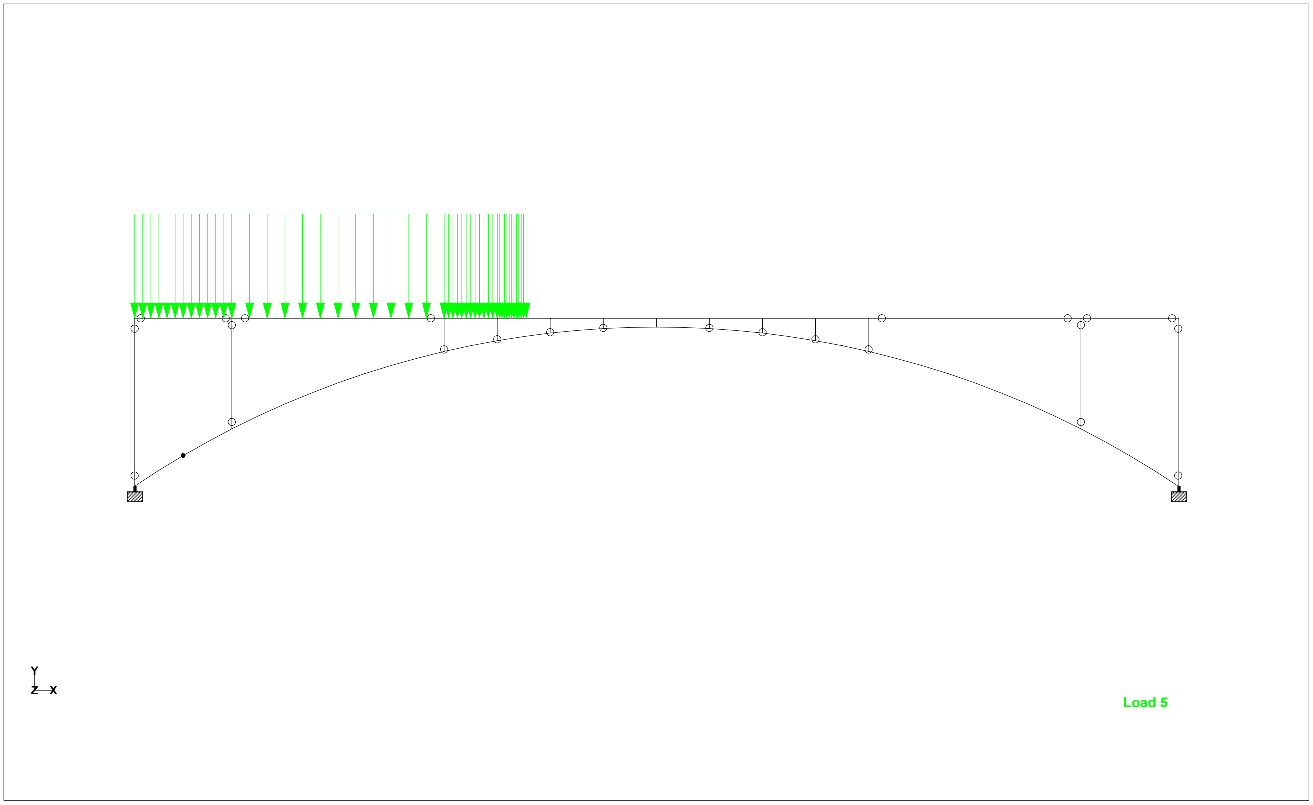

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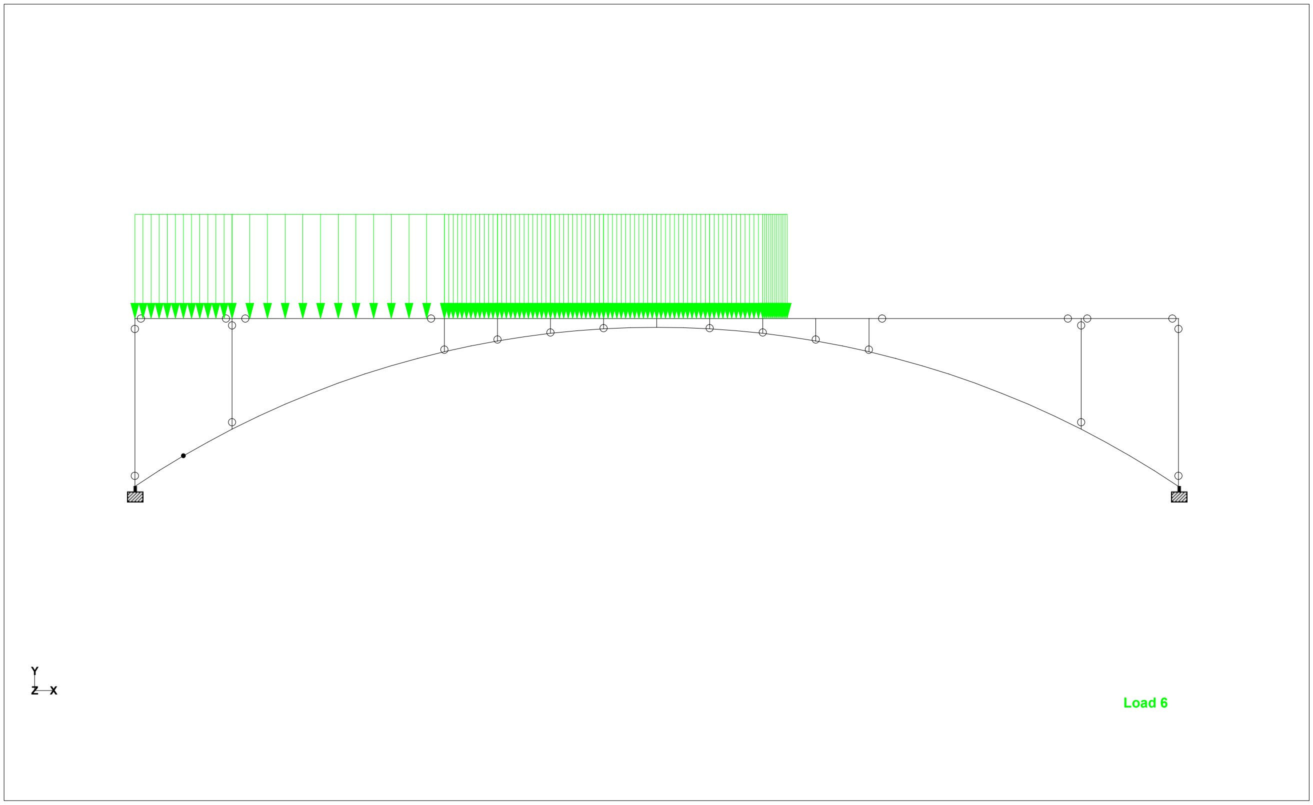
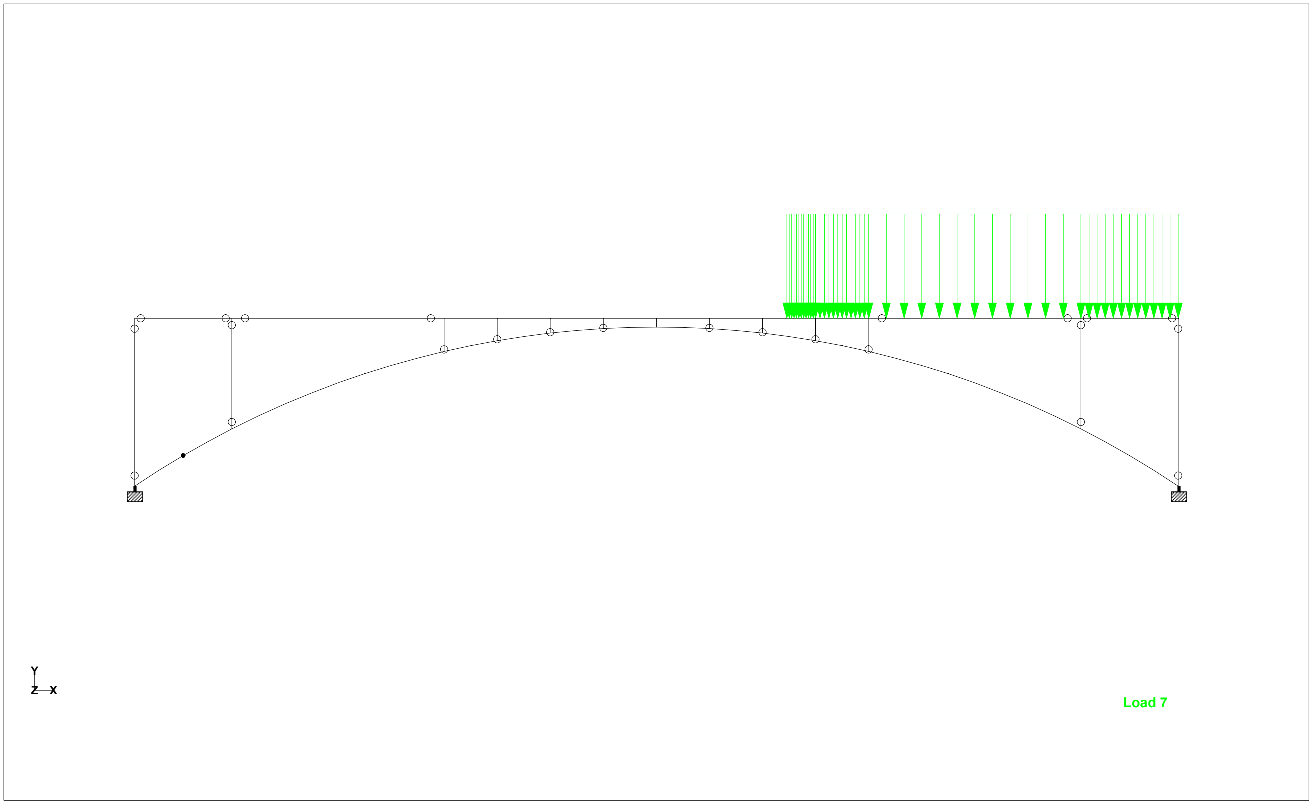
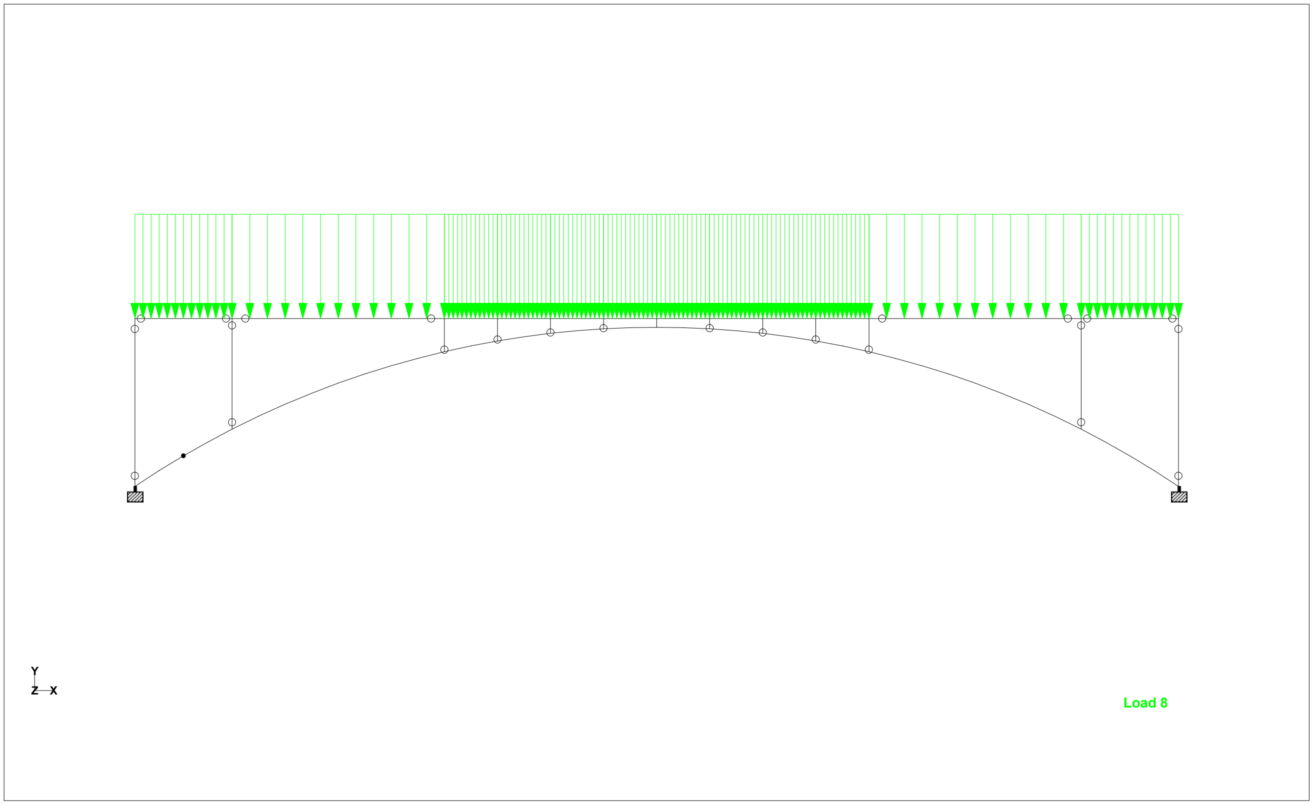


Live Load 5

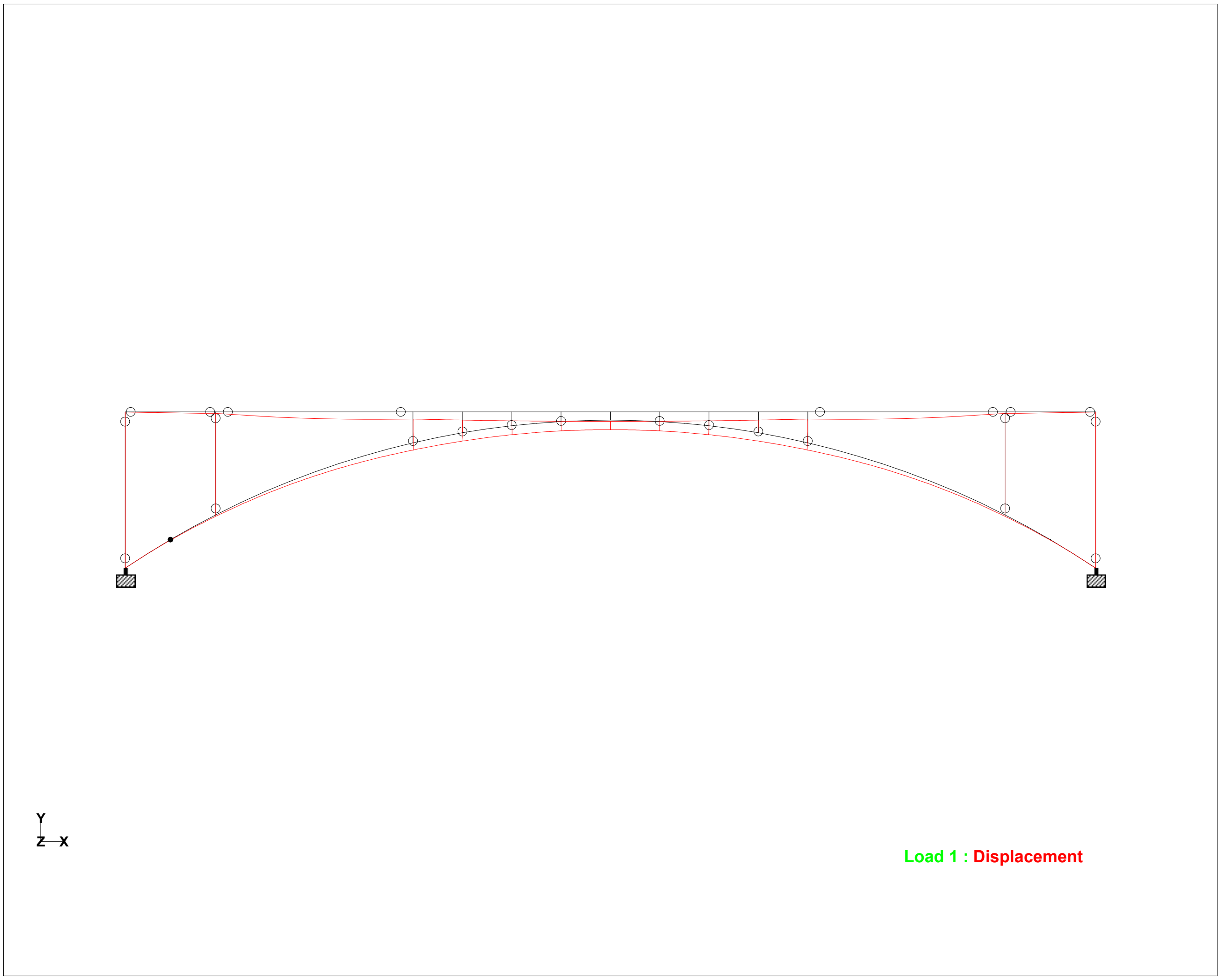


Live Load 6

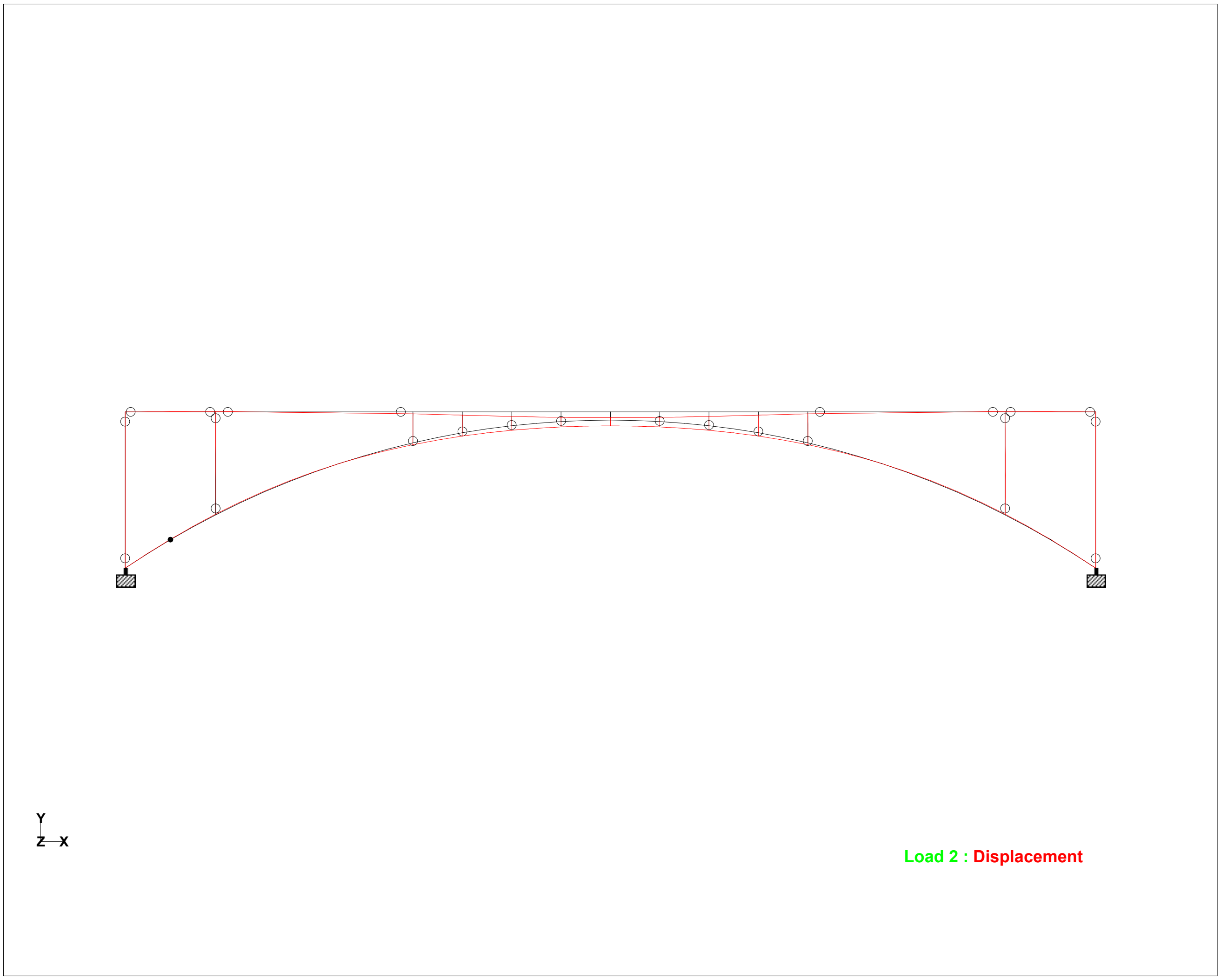
Software licensed to TranSystems	Job No	Sheet No 9	Rev
	Part		
Job Title LAKE PARK ARCH BRIDGE LOAD RATING	Ref		
	By DWC	Date 12-Jul-18	Chd SFH
Client	File Lake Park Arch.std	Date/Time 26-Jul-2018 15:18	



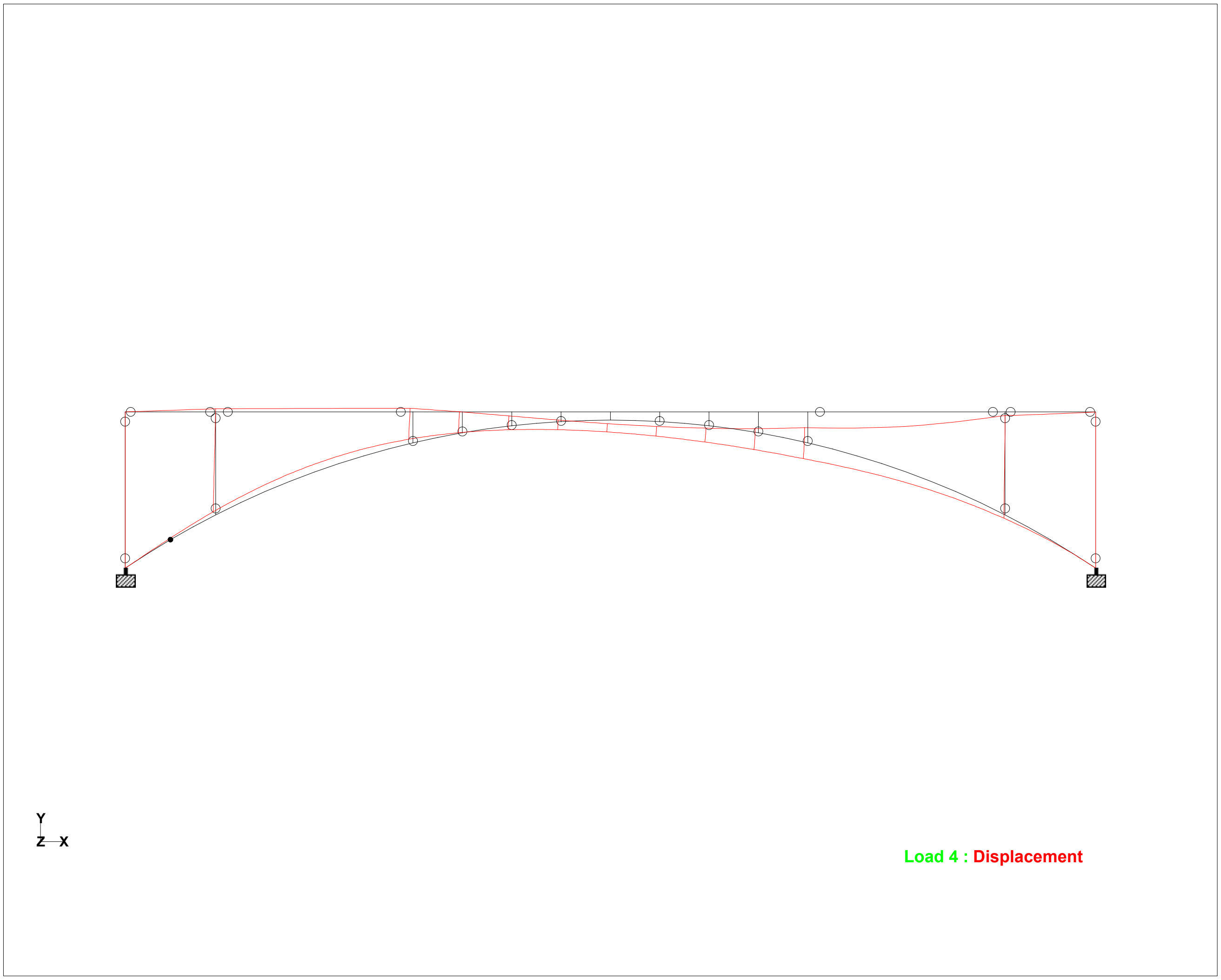
Live Load 7



Dead Load - Displacement



Live Load 1 - Displacement



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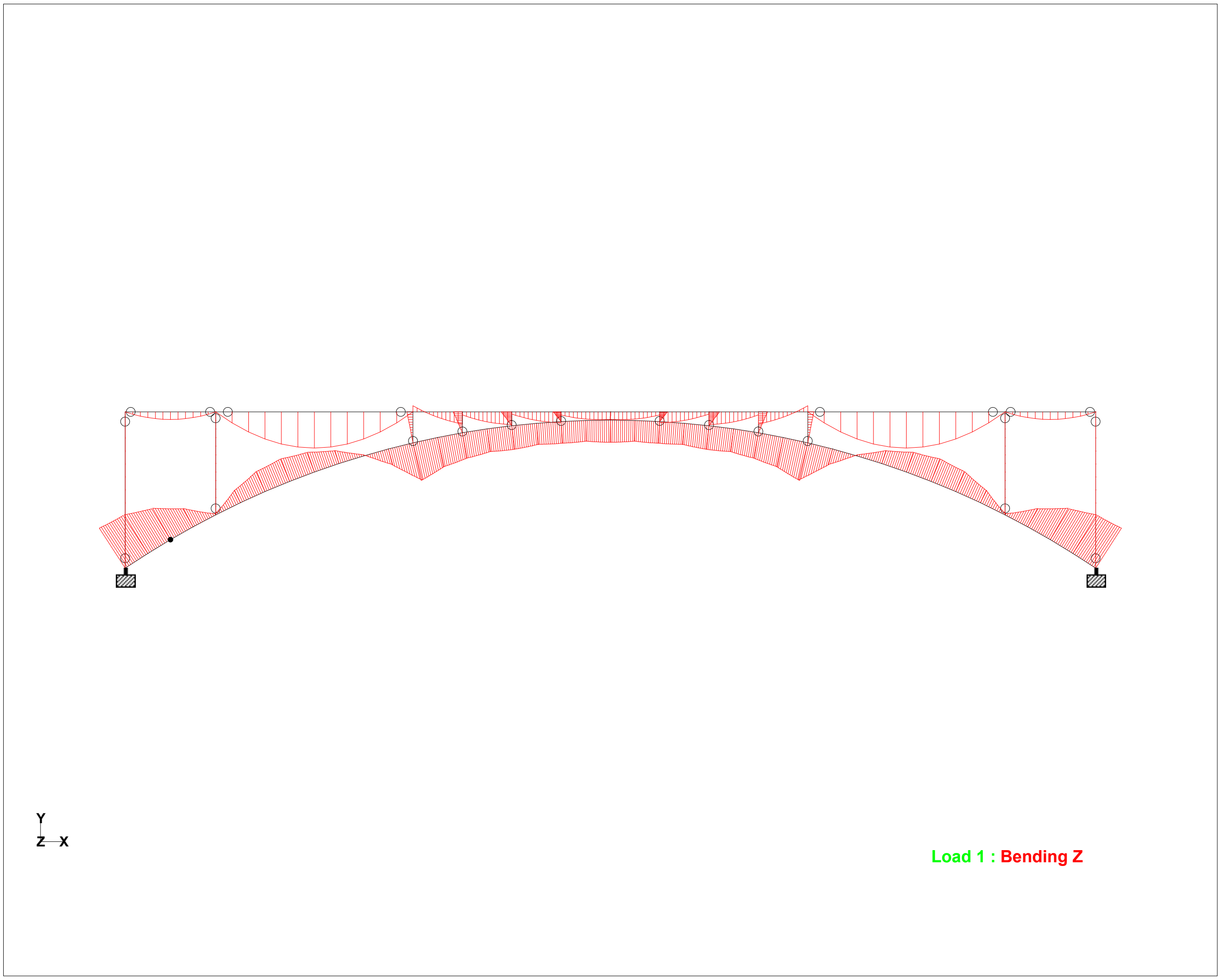
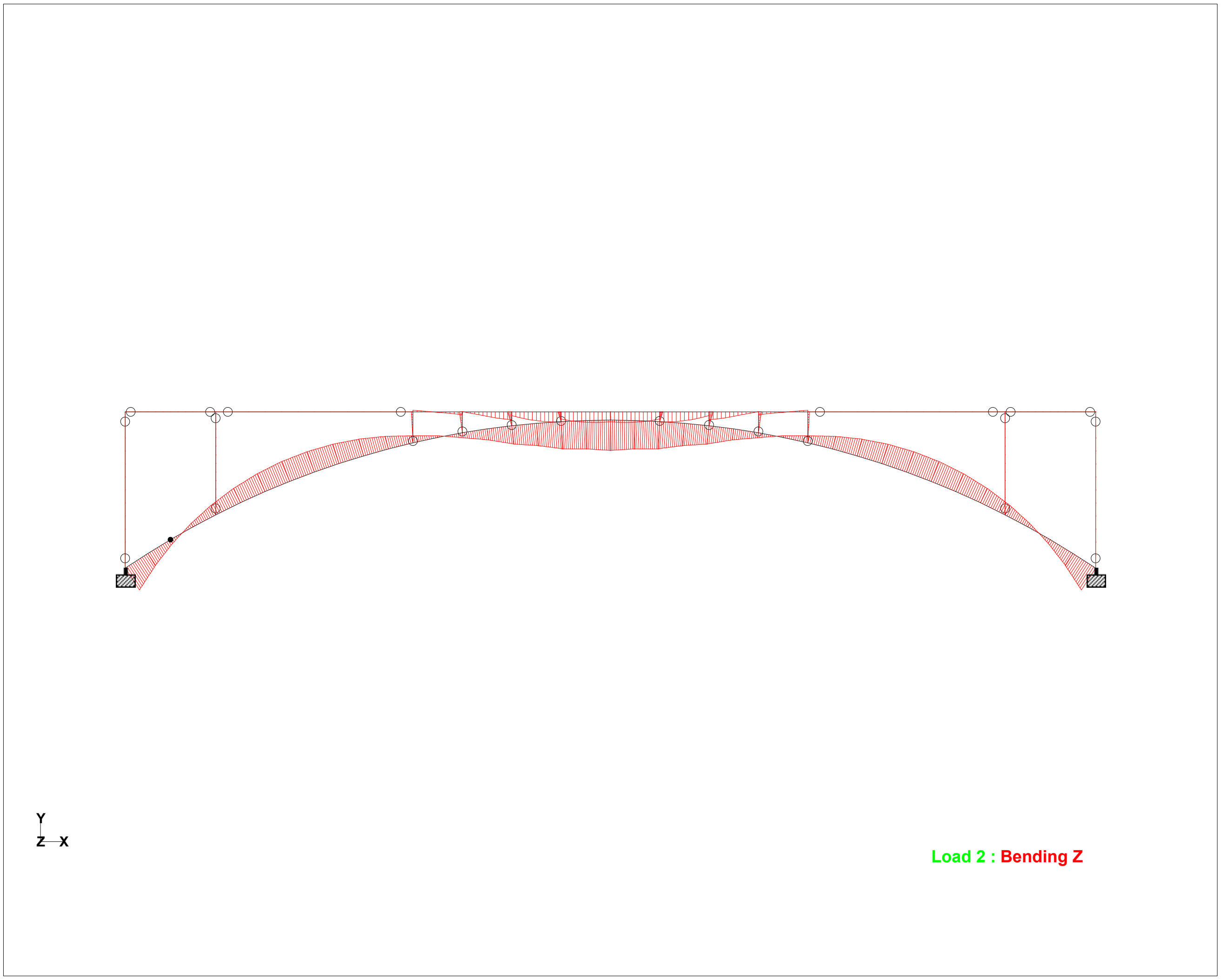
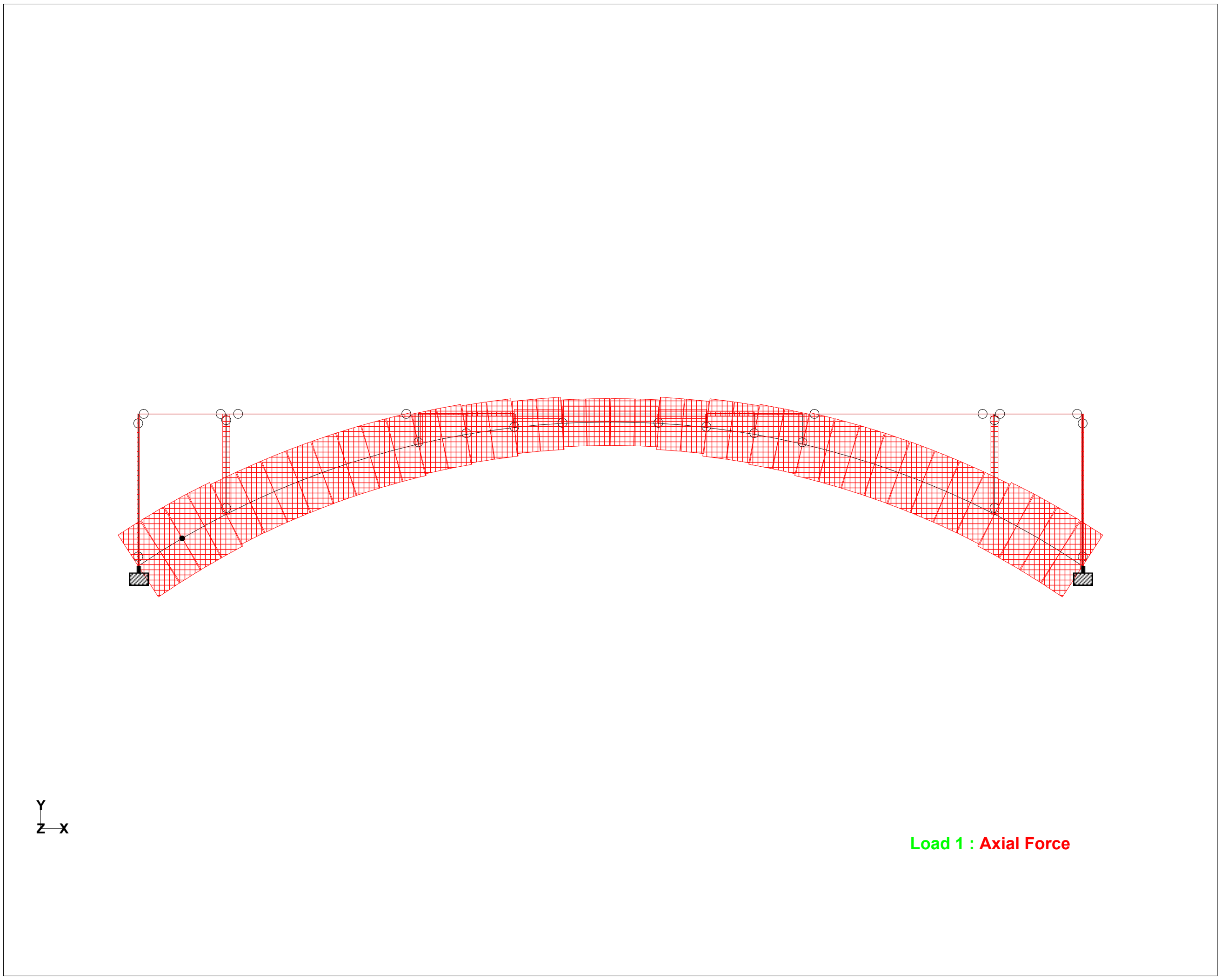


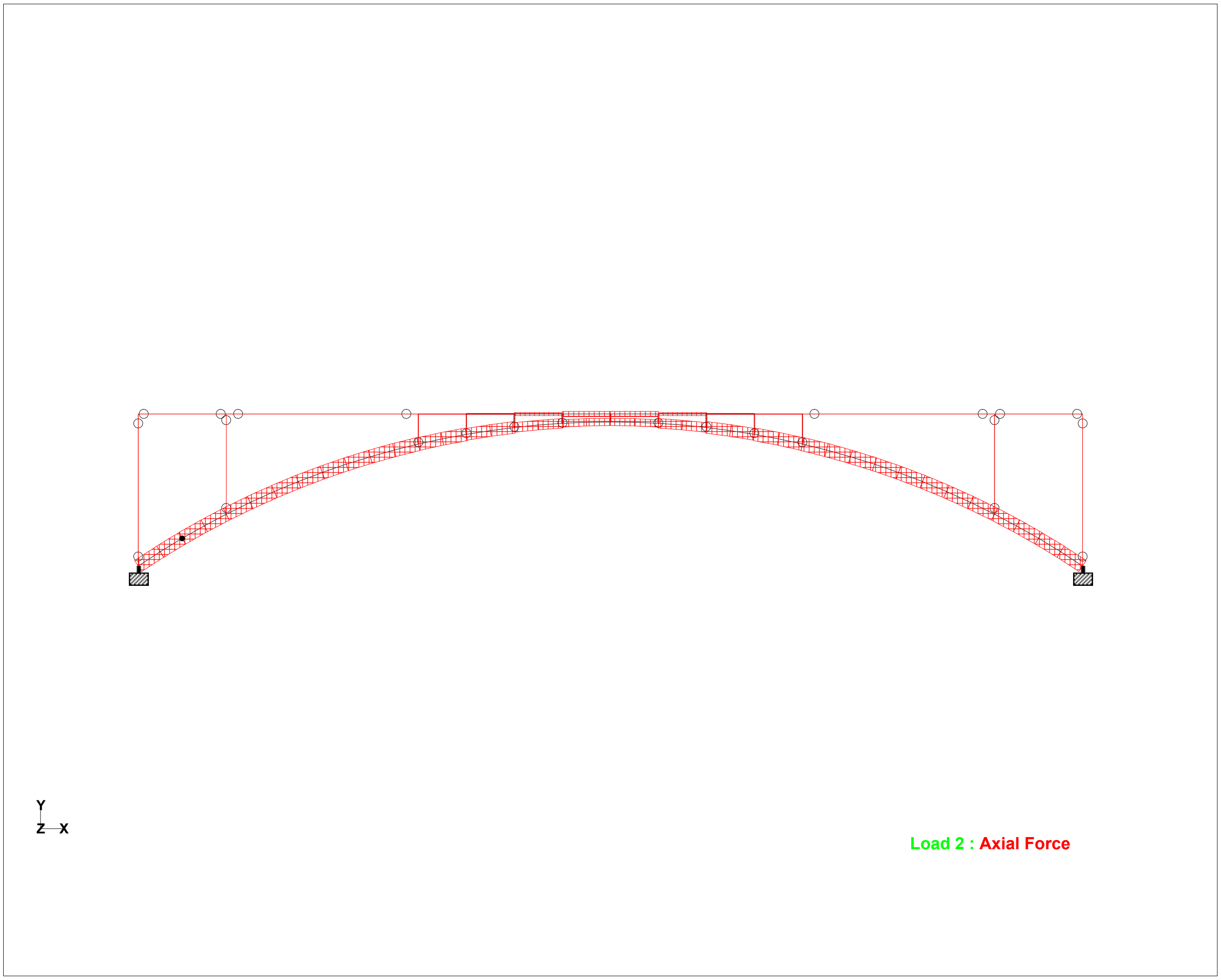
Dead Load - Moment



Live Load 1 - Moment



Dead Load - Axial Force



Live Load 1 - Axial Force

Beam End Forces

Sign convention is as the action of the joint on the beam.

Beam	Node	L/C	Axial Fx (kip)	Shear Fy (kip)	Shear Fz (kip)	Torsion Mx (kip-ft)	Bending My (kip-ft)	Bending Mz (kip-ft)
1	1	1:DEAD LOAD	277.325	3.318	0.000	0.000	0.000	144.802
		11:DL + LL1	297.037	-1.059	0.000	0.000	0.000	113.143
		12:DL + LL2	308.484	5.237	0.000	0.000	0.000	179.439
		13:DL + LL3	309.006	-5.648	0.000	0.000	0.000	53.642
		14:DL + LL4	296.514	9.826	0.000	0.000	0.000	238.940
		15:DL + LL5	316.226	5.450	0.000	0.000	0.000	207.282
		16:DL + LL6	289.295	-1.271	0.000	0.000	0.000	85.300
		17:DL + LL7	328.195	0.860	0.000	0.000	0.000	147.780
	2	1:DEAD LOAD	-275.931	-1.174	0.000	0.000	0.000	-137.433
		11:DL + LL1	-295.642	3.203	0.000	0.000	0.000	-120.133
		12:DL + LL2	-307.089	-3.094	0.000	0.000	0.000	-165.773
		13:DL + LL3	-307.612	7.792	0.000	0.000	0.000	-75.687
		14:DL + LL4	-295.120	-7.683	0.000	0.000	0.000	-210.219
		15:DL + LL5	-314.831	-3.306	0.000	0.000	0.000	-192.920
		16:DL + LL6	-287.900	3.415	0.000	0.000	0.000	-92.987
		17:DL + LL7	-326.801	1.283	0.000	0.000	0.000	-148.474
2	2	1:DEAD LOAD	275.764	9.655	0.000	0.000	0.000	137.433
		11:DL + LL1	295.601	5.887	0.000	0.000	0.000	120.133
		12:DL + LL2	306.849	12.532	0.000	0.000	0.000	165.773
		13:DL + LL3	307.706	1.668	0.000	0.000	0.000	75.687
		14:DL + LL4	294.744	16.751	0.000	0.000	0.000	210.219
		15:DL + LL5	314.581	12.982	0.000	0.000	0.000	192.920
		16:DL + LL6	287.869	5.436	0.000	0.000	0.000	92.987
		17:DL + LL7	326.686	8.763	0.000	0.000	0.000	148.474
	3	1:DEAD LOAD	-274.462	-7.512	0.000	0.000	0.000	-109.812
		11:DL + LL1	-294.298	-3.743	0.000	0.000	0.000	-104.640
		12:DL + LL2	-305.547	-10.388	0.000	0.000	0.000	-128.897
		13:DL + LL3	-306.403	0.476	0.000	0.000	0.000	-73.770
		14:DL + LL4	-293.442	-14.607	0.000	0.000	0.000	-159.767
		15:DL + LL5	-313.278	-10.838	0.000	0.000	0.000	-154.596
		16:DL + LL6	-286.567	-3.293	0.000	0.000	0.000	-78.942
		17:DL + LL7	-325.383	-6.619	0.000	0.000	0.000	-123.725
3	3	1:DEAD LOAD	274.108	15.819	0.000	0.000	0.000	109.812
		11:DL + LL1	294.050	12.652	0.000	0.000	0.000	104.640
		12:DL + LL2	305.092	19.635	0.000	0.000	0.000	128.897
		13:DL + LL3	306.277	8.802	0.000	0.000	0.000	73.770
		14:DL + LL4	292.865	23.485	0.000	0.000	0.000	159.767
		15:DL + LL5	312.806	20.319	0.000	0.000	0.000	154.596
		16:DL + LL6	286.336	11.968	0.000	0.000	0.000	78.942
		17:DL + LL7	325.034	16.468	0.000	0.000	0.000	123.725
	4	1:DEAD LOAD	-272.893	-13.675	0.000	0.000	0.000	-63.195
		11:DL + LL1	-292.835	-10.508	0.000	0.000	0.000	-68.033
		12:DL + LL2	-303.877	-17.491	0.000	0.000	0.000	-70.217

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
		13:DL + LL3	-305.062	-6.658	0.000	0.000	0.000	-49.335
		14:DL + LL4	-291.649	-21.341	0.000	0.000	0.000	-88.915
		15:DL + LL5	-311.591	-18.175	0.000	0.000	0.000	-93.753
		16:DL + LL6	-285.120	-9.824	0.000	0.000	0.000	-44.497
		17:DL + LL7	-323.818	-14.324	0.000	0.000	0.000	-75.055
4	4	1:DEAD LOAD	272.368	21.760	0.000	0.000	0.000	63.195
		11:DL + LL1	292.394	19.186	0.000	0.000	0.000	68.033
		12:DL + LL2	303.225	26.493	0.000	0.000	0.000	70.217
		13:DL + LL3	304.730	15.699	0.000	0.000	0.000	49.335
		14:DL + LL4	290.889	29.979	0.000	0.000	0.000	88.915
		15:DL + LL5	310.915	27.405	0.000	0.000	0.000	93.753
		16:DL + LL6	284.704	18.273	0.000	0.000	0.000	44.497
		17:DL + LL7	323.251	23.919	0.000	0.000	0.000	75.055
	5	1:DEAD LOAD	-269.520	-16.371	0.000	0.000	0.000	-3.897
		11:DL + LL1	-289.547	-13.797	0.000	0.000	0.000	-16.741
		12:DL + LL2	-300.377	-21.104	0.000	0.000	0.000	3.802
		13:DL + LL3	-301.883	-10.311	0.000	0.000	0.000	-8.886
		14:DL + LL4	-288.041	-24.590	0.000	0.000	0.000	-4.053
		15:DL + LL5	-308.068	-22.016	0.000	0.000	0.000	-16.897
		16:DL + LL6	-281.857	-12.885	0.000	0.000	0.000	3.957
		17:DL + LL7	-320.404	-18.530	0.000	0.000	0.000	-9.042
5	5	1:DEAD LOAD	252.759	-8.361	0.000	0.000	0.000	3.897
		11:DL + LL1	272.855	-10.324	0.000	0.000	0.000	16.741
		12:DL + LL2	279.762	-10.233	0.000	0.000	0.000	-3.802
		13:DL + LL3	285.292	-13.433	0.000	0.000	0.000	8.886
		14:DL + LL4	267.325	-7.124	0.000	0.000	0.000	4.053
		15:DL + LL5	287.421	-9.087	0.000	0.000	0.000	16.897
		16:DL + LL6	265.196	-11.470	0.000	0.000	0.000	-3.957
		17:DL + LL7	299.857	-12.196	0.000	0.000	0.000	9.042
	6	1:DEAD LOAD	-249.879	14.239	0.000	0.000	0.000	-41.649
		11:DL + LL1	-269.975	16.202	0.000	0.000	0.000	-61.049
		12:DL + LL2	-276.881	16.112	0.000	0.000	0.000	-40.205
		13:DL + LL3	-282.411	19.311	0.000	0.000	0.000	-63.581
		14:DL + LL4	-264.445	13.003	0.000	0.000	0.000	-37.674
		15:DL + LL5	-284.540	14.966	0.000	0.000	0.000	-57.074
		16:DL + LL6	-262.315	17.348	0.000	0.000	0.000	-44.180
		17:DL + LL7	-296.977	18.075	0.000	0.000	0.000	-59.606
6	6	1:DEAD LOAD	250.202	-6.393	0.000	0.000	0.000	41.649
		11:DL + LL1	270.350	-7.725	0.000	0.000	0.000	61.049
		12:DL + LL2	277.250	-7.418	0.000	0.000	0.000	40.205
		13:DL + LL3	282.878	-10.442	0.000	0.000	0.000	63.581
		14:DL + LL4	264.722	-4.701	0.000	0.000	0.000	37.674
		15:DL + LL5	284.870	-6.032	0.000	0.000	0.000	57.074
		16:DL + LL6	262.730	-9.111	0.000	0.000	0.000	44.180

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip`ft)	My (kip`ft)	Mz (kip`ft)
		17:DL + LL7	297.398	-8.749	0.000	0.000	0.000	59.606
	7	1:DEAD LOAD	-249.146	8.732	0.000	0.000	0.000	-66.543
		11:DL + LL1	-269.294	10.063	0.000	0.000	0.000	-90.327
		12:DL + LL2	-276.194	9.756	0.000	0.000	0.000	-68.472
		13:DL + LL3	-281.822	12.780	0.000	0.000	0.000	-101.803
		14:DL + LL4	-263.666	7.039	0.000	0.000	0.000	-56.996
		15:DL + LL5	-283.814	8.370	0.000	0.000	0.000	-80.779
		16:DL + LL6	-261.674	11.449	0.000	0.000	0.000	-78.020
		17:DL + LL7	-296.342	11.088	0.000	0.000	0.000	-92.256
7	7	1:DEAD LOAD	249.297	-1.038	0.000	0.000	0.000	66.543
		11:DL + LL1	269.476	-1.747	0.000	0.000	0.000	90.327
		12:DL + LL2	276.363	-1.227	0.000	0.000	0.000	68.472
		13:DL + LL3	282.082	-4.076	0.000	0.000	0.000	101.803
		14:DL + LL4	263.758	1.102	0.000	0.000	0.000	56.996
		15:DL + LL5	283.937	0.393	0.000	0.000	0.000	80.779
		16:DL + LL6	261.903	-3.367	0.000	0.000	0.000	78.020
		17:DL + LL7	296.543	-1.936	0.000	0.000	0.000	92.256
	8	1:DEAD LOAD	-248.327	3.377	0.000	0.000	0.000	-73.713
		11:DL + LL1	-268.506	4.085	0.000	0.000	0.000	-99.799
		12:DL + LL2	-275.393	3.566	0.000	0.000	0.000	-76.256
		13:DL + LL3	-281.111	6.415	0.000	0.000	0.000	-118.841
		14:DL + LL4	-262.787	1.236	0.000	0.000	0.000	-57.214
		15:DL + LL5	-282.966	1.945	0.000	0.000	0.000	-83.300
		16:DL + LL6	-260.932	5.706	0.000	0.000	0.000	-92.755
		17:DL + LL7	-295.572	4.275	0.000	0.000	0.000	-102.342
8	8	1:DEAD LOAD	248.313	4.274	0.000	0.000	0.000	73.713
		11:DL + LL1	268.504	4.187	0.000	0.000	0.000	99.799
		12:DL + LL2	275.372	4.918	0.000	0.000	0.000	76.256
		13:DL + LL3	281.176	2.247	0.000	0.000	0.000	118.841
		14:DL + LL4	262.701	6.858	0.000	0.000	0.000	57.214
		15:DL + LL5	282.892	6.771	0.000	0.000	0.000	83.300
		16:DL + LL6	260.984	2.334	0.000	0.000	0.000	92.755
		17:DL + LL7	295.563	4.831	0.000	0.000	0.000	102.342
	9	1:DEAD LOAD	-247.426	-1.935	0.000	0.000	0.000	-63.752
		11:DL + LL1	-267.617	-1.848	0.000	0.000	0.000	-90.117
		12:DL + LL2	-274.485	-2.580	0.000	0.000	0.000	-64.227
		13:DL + LL3	-280.288	0.092	0.000	0.000	0.000	-115.384
		14:DL + LL4	-261.813	-4.520	0.000	0.000	0.000	-38.961
		15:DL + LL5	-282.005	-4.433	0.000	0.000	0.000	-65.325
		16:DL + LL6	-260.097	0.005	0.000	0.000	0.000	-89.019
		17:DL + LL7	-294.676	-2.493	0.000	0.000	0.000	-90.592
9	9	1:DEAD LOAD	246.864	8.235	0.000	0.000	0.000	63.752
		11:DL + LL1	267.049	8.755	0.000	0.000	0.000	90.117
		12:DL + LL2	273.892	9.692	0.000	0.000	0.000	64.227

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
		13:DL + LL3	279.773	7.196	0.000	0.000	0.000	115.384
		14:DL + LL4	261.168	11.251	0.000	0.000	0.000	38.961
		15:DL + LL5	281.353	11.770	0.000	0.000	0.000	65.325
		16:DL + LL6	259.588	6.676	0.000	0.000	0.000	89.019
		17:DL + LL7	294.077	10.212	0.000	0.000	0.000	90.592
	10	1:DEAD LOAD	-246.056	-5.896	0.000	0.000	0.000	-41.328
		11:DL + LL1	-266.241	-6.416	0.000	0.000	0.000	-66.042
		12:DL + LL2	-273.084	-7.354	0.000	0.000	0.000	-37.177
		13:DL + LL3	-278.965	-4.858	0.000	0.000	0.000	-96.255
		14:DL + LL4	-260.360	-8.912	0.000	0.000	0.000	-6.964
		15:DL + LL5	-280.545	-9.432	0.000	0.000	0.000	-31.679
		16:DL + LL6	-258.781	-4.338	0.000	0.000	0.000	-71.540
		17:DL + LL7	-293.269	-7.873	0.000	0.000	0.000	-61.892
10	10	1:DEAD LOAD	245.772	13.220	0.000	0.000	0.000	41.328
		11:DL + LL1	265.932	14.340	0.000	0.000	0.000	66.042
		12:DL + LL2	272.744	15.481	0.000	0.000	0.000	37.177
		13:DL + LL3	278.697	13.161	0.000	0.000	0.000	96.255
		14:DL + LL4	259.979	16.660	0.000	0.000	0.000	6.964
		15:DL + LL5	280.140	17.780	0.000	0.000	0.000	31.679
		16:DL + LL6	258.537	12.041	0.000	0.000	0.000	71.540
		17:DL + LL7	292.905	16.602	0.000	0.000	0.000	61.892
	11	1:DEAD LOAD	-245.041	-10.881	0.000	0.000	0.000	-3.454
		11:DL + LL1	-265.202	-12.002	0.000	0.000	0.000	-24.647
		12:DL + LL2	-272.014	-13.143	0.000	0.000	0.000	7.804
		13:DL + LL3	-277.967	-10.823	0.000	0.000	0.000	-58.564
		14:DL + LL4	-259.249	-14.321	0.000	0.000	0.000	41.722
		15:DL + LL5	-279.409	-15.442	0.000	0.000	0.000	20.529
		16:DL + LL6	-257.806	-9.702	0.000	0.000	0.000	-37.371
		17:DL + LL7	-292.174	-14.263	0.000	0.000	0.000	-13.389
11	11	1:DEAD LOAD	244.610	18.159	0.000	0.000	0.000	3.454
		11:DL + LL1	264.728	19.878	0.000	0.000	0.000	24.647
		12:DL + LL2	271.503	21.221	0.000	0.000	0.000	-7.804
		13:DL + LL3	277.522	19.079	0.000	0.000	0.000	58.564
		14:DL + LL4	258.709	22.020	0.000	0.000	0.000	-41.722
		15:DL + LL5	278.827	23.739	0.000	0.000	0.000	-20.529
		16:DL + LL6	257.404	17.360	0.000	0.000	0.000	37.371
		17:DL + LL7	291.621	22.940	0.000	0.000	0.000	13.389
	12	1:DEAD LOAD	-243.955	-15.820	0.000	0.000	0.000	49.475
		11:DL + LL1	-264.073	-17.540	0.000	0.000	0.000	33.638
		12:DL + LL2	-270.848	-18.883	0.000	0.000	0.000	70.273
		13:DL + LL3	-276.867	-16.741	0.000	0.000	0.000	-2.768
		14:DL + LL4	-258.054	-19.681	0.000	0.000	0.000	106.679
		15:DL + LL5	-278.172	-21.401	0.000	0.000	0.000	90.842
		16:DL + LL6	-256.749	-15.022	0.000	0.000	0.000	13.069

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip˘ft)	My (kip˘ft)	Mz (kip˘ft)
		17:DL + LL7	-290.967	-20.602	0.000	0.000	0.000	54.436
12	12	1:DEAD LOAD	243.387	22.957	0.000	0.000	0.000	-49.475
		11:DL + LL1	263.446	25.264	0.000	0.000	0.000	-33.638
		12:DL + LL2	270.179	26.805	0.000	0.000	0.000	-70.273
		13:DL + LL3	276.259	24.841	0.000	0.000	0.000	2.768
		14:DL + LL4	257.367	27.229	0.000	0.000	0.000	-106.679
		15:DL + LL5	277.426	29.537	0.000	0.000	0.000	-90.842
		16:DL + LL6	256.199	22.533	0.000	0.000	0.000	-13.069
		17:DL + LL7	290.239	29.113	0.000	0.000	0.000	-54.436
	13	1:DEAD LOAD	-242.806	-20.618	0.000	0.000	0.000	116.829
		11:DL + LL1	-262.865	-22.926	0.000	0.000	0.000	108.125
		12:DL + LL2	-269.598	-24.467	0.000	0.000	0.000	149.523
		13:DL + LL3	-275.677	-22.502	0.000	0.000	0.000	70.409
		14:DL + LL4	-256.786	-24.891	0.000	0.000	0.000	187.240
		15:DL + LL5	-276.845	-27.198	0.000	0.000	0.000	178.536
		16:DL + LL6	-255.618	-20.195	0.000	0.000	0.000	79.112
		17:DL + LL7	-289.657	-26.774	0.000	0.000	0.000	140.820
13	13	1:DEAD LOAD	229.422	-7.279	0.000	0.000	0.000	-116.829
		11:DL + LL1	248.670	-5.136	0.000	0.000	0.000	-108.125
		12:DL + LL2	253.855	-10.674	0.000	0.000	0.000	-149.523
		13:DL + LL3	261.999	-3.443	0.000	0.000	0.000	-70.409
		14:DL + LL4	240.527	-12.367	0.000	0.000	0.000	-187.240
		15:DL + LL5	259.775	-10.224	0.000	0.000	0.000	-178.536
		16:DL + LL6	242.751	-5.586	0.000	0.000	0.000	-79.112
		17:DL + LL7	273.103	-8.531	0.000	0.000	0.000	-140.820
	14	1:DEAD LOAD	-228.912	9.618	0.000	0.000	0.000	90.888
		11:DL + LL1	-248.160	7.475	0.000	0.000	0.000	88.765
		12:DL + LL2	-253.345	13.012	0.000	0.000	0.000	113.160
		13:DL + LL3	-261.489	5.782	0.000	0.000	0.000	56.246
		14:DL + LL4	-240.017	14.705	0.000	0.000	0.000	145.679
		15:DL + LL5	-259.265	12.562	0.000	0.000	0.000	143.555
		16:DL + LL6	-242.241	7.925	0.000	0.000	0.000	58.369
		17:DL + LL7	-272.593	10.869	0.000	0.000	0.000	111.036
14	14	1:DEAD LOAD	229.096	-2.946	0.000	0.000	0.000	-90.888
		11:DL + LL1	248.273	-0.243	0.000	0.000	0.000	-88.765
		12:DL + LL2	253.617	-5.627	0.000	0.000	0.000	-113.160
		13:DL + LL3	261.546	1.837	0.000	0.000	0.000	-56.246
		14:DL + LL4	240.343	-7.707	0.000	0.000	0.000	-145.679
		15:DL + LL5	259.521	-5.005	0.000	0.000	0.000	-143.555
		16:DL + LL6	242.369	-0.865	0.000	0.000	0.000	-58.369
		17:DL + LL7	272.794	-2.924	0.000	0.000	0.000	-111.036
	15	1:DEAD LOAD	-228.657	5.284	0.000	0.000	0.000	78.328
		11:DL + LL1	-247.834	2.581	0.000	0.000	0.000	84.454
		12:DL + LL2	-253.178	7.966	0.000	0.000	0.000	92.415

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
		13:DL + LL3	-261.108	0.501	0.000	0.000	0.000	58.285
		14:DL + LL4	-239.905	10.046	0.000	0.000	0.000	118.584
		15:DL + LL5	-259.082	7.343	0.000	0.000	0.000	124.710
		16:DL + LL6	-241.930	3.204	0.000	0.000	0.000	52.159
		17:DL + LL7	-272.355	5.263	0.000	0.000	0.000	98.541
15	15	1:DEAD LOAD	217.294	-1.769	0.000	0.000	0.000	-78.328
		11:DL + LL1	234.829	0.677	0.000	0.000	0.000	-84.454
		12:DL + LL2	240.852	-4.636	0.000	0.000	0.000	-92.415
		13:DL + LL3	247.973	1.049	0.000	0.000	0.000	-58.285
		14:DL + LL4	227.709	-5.009	0.000	0.000	0.000	-118.584
		15:DL + LL5	245.244	-2.563	0.000	0.000	0.000	-124.710
		16:DL + LL6	230.437	-1.397	0.000	0.000	0.000	-52.159
		17:DL + LL7	258.388	-2.190	0.000	0.000	0.000	-98.541
	16	1:DEAD LOAD	-216.924	4.108	0.000	0.000	0.000	69.403
		11:DL + LL1	-234.460	1.662	0.000	0.000	0.000	82.959
		12:DL + LL2	-240.483	6.975	0.000	0.000	0.000	74.783
		13:DL + LL3	-247.603	1.289	0.000	0.000	0.000	57.922
		14:DL + LL4	-227.339	7.347	0.000	0.000	0.000	99.820
		15:DL + LL5	-244.875	4.901	0.000	0.000	0.000	113.375
		16:DL + LL6	-230.068	3.735	0.000	0.000	0.000	44.366
		17:DL + LL7	-258.018	4.529	0.000	0.000	0.000	88.338
16	16	1:DEAD LOAD	216.953	2.057	0.000	0.000	0.000	-69.403
		11:DL + LL1	234.412	5.000	0.000	0.000	0.000	-82.959
		12:DL + LL2	240.584	-0.140	0.000	0.000	0.000	-74.783
		13:DL + LL3	247.540	5.746	0.000	0.000	0.000	-57.922
		14:DL + LL4	227.456	-0.886	0.000	0.000	0.000	-99.820
		15:DL + LL5	244.915	2.058	0.000	0.000	0.000	-113.375
		16:DL + LL6	230.081	2.802	0.000	0.000	0.000	-44.366
		17:DL + LL7	258.043	2.803	0.000	0.000	0.000	-88.338
	17	1:DEAD LOAD	-216.652	0.282	0.000	0.000	0.000	72.087
		11:DL + LL1	-234.110	-2.661	0.000	0.000	0.000	94.546
		12:DL + LL2	-240.282	2.478	0.000	0.000	0.000	70.823
		13:DL + LL3	-247.238	-3.407	0.000	0.000	0.000	71.765
		14:DL + LL4	-227.155	3.224	0.000	0.000	0.000	93.604
		15:DL + LL5	-244.613	0.281	0.000	0.000	0.000	116.063
		16:DL + LL6	-229.779	-0.464	0.000	0.000	0.000	49.306
		17:DL + LL7	-257.741	-0.465	0.000	0.000	0.000	93.282
17	17	1:DEAD LOAD	196.905	-1.434	0.000	0.000	0.000	-72.087
		11:DL + LL1	211.215	0.170	0.000	0.000	0.000	-94.546
		12:DL + LL2	219.385	-3.416	0.000	0.000	0.000	-70.823
		13:DL + LL3	224.110	1.040	0.000	0.000	0.000	-71.765
		14:DL + LL4	206.490	-4.285	0.000	0.000	0.000	-93.604
		15:DL + LL5	220.799	-2.682	0.000	0.000	0.000	-116.063
		16:DL + LL6	209.801	-0.564	0.000	0.000	0.000	-49.306

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
		17:DL + LL7	233.694	-1.812	0.000	0.000	0.000	-93.282
	18	1:DEAD LOAD	-196.672	3.772	0.000	0.000	0.000	64.240
		11:DL + LL1	-210.982	2.169	0.000	0.000	0.000	91.533
		12:DL + LL2	-219.152	5.754	0.000	0.000	0.000	57.000
		13:DL + LL3	-223.877	1.299	0.000	0.000	0.000	71.374
		14:DL + LL4	-206.257	6.624	0.000	0.000	0.000	77.159
		15:DL + LL5	-220.566	5.020	0.000	0.000	0.000	104.453
		16:DL + LL6	-209.567	2.903	0.000	0.000	0.000	44.081
		17:DL + LL7	-233.461	4.150	0.000	0.000	0.000	84.294
18	18	1:DEAD LOAD	196.701	1.760	0.000	0.000	0.000	-64.240
		11:DL + LL1	210.959	3.766	0.000	0.000	0.000	-91.533
		12:DL + LL2	219.227	0.411	0.000	0.000	0.000	-57.000
		13:DL + LL3	223.825	4.998	0.000	0.000	0.000	-71.374
		14:DL + LL4	206.361	-0.821	0.000	0.000	0.000	-77.159
		15:DL + LL5	220.620	1.185	0.000	0.000	0.000	-104.453
		16:DL + LL6	209.566	2.992	0.000	0.000	0.000	-44.081
		17:DL + LL7	233.486	2.417	0.000	0.000	0.000	-84.294
	19	1:DEAD LOAD	-196.534	0.579	0.000	0.000	0.000	66.016
		11:DL + LL1	-210.792	-1.427	0.000	0.000	0.000	99.342
		12:DL + LL2	-219.060	1.927	0.000	0.000	0.000	54.720
		13:DL + LL3	-223.658	-2.659	0.000	0.000	0.000	82.888
		14:DL + LL4	-206.195	3.160	0.000	0.000	0.000	71.173
		15:DL + LL5	-220.453	1.154	0.000	0.000	0.000	104.498
		16:DL + LL6	-209.399	-0.653	0.000	0.000	0.000	49.563
		17:DL + LL7	-233.319	-0.078	0.000	0.000	0.000	88.045
19	19	1:DEAD LOAD	176.064	0.123	0.000	0.000	0.000	-66.016
		11:DL + LL1	187.032	0.565	0.000	0.000	0.000	-99.342
		12:DL + LL2	197.486	-0.713	0.000	0.000	0.000	-54.720
		13:DL + LL3	198.834	2.014	0.000	0.000	0.000	-82.888
		14:DL + LL4	185.684	-2.162	0.000	0.000	0.000	-71.173
		15:DL + LL5	196.652	-1.720	0.000	0.000	0.000	-104.498
		16:DL + LL6	187.866	1.572	0.000	0.000	0.000	-49.563
		17:DL + LL7	208.454	-0.271	0.000	0.000	0.000	-88.045
	20	1:DEAD LOAD	-175.965	2.215	0.000	0.000	0.000	62.875
		11:DL + LL1	-186.933	1.773	0.000	0.000	0.000	97.528
		12:DL + LL2	-197.387	3.052	0.000	0.000	0.000	49.067
		13:DL + LL3	-198.735	0.324	0.000	0.000	0.000	85.425
		14:DL + LL4	-185.585	4.501	0.000	0.000	0.000	61.170
		15:DL + LL5	-196.553	4.059	0.000	0.000	0.000	95.822
		16:DL + LL6	-187.767	0.766	0.000	0.000	0.000	50.773
		17:DL + LL7	-208.355	2.610	0.000	0.000	0.000	83.720
20	20	1:DEAD LOAD	175.958	2.707	0.000	0.000	0.000	-62.875
		11:DL + LL1	186.909	3.456	0.000	0.000	0.000	-97.528
		12:DL + LL2	197.395	2.471	0.000	0.000	0.000	-49.067

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip˙ft)	My (kip˙ft)	Mz (kip˙ft)
		13:DL + LL3	198.666	5.235	0.000	0.000	0.000	-85.425
		14:DL + LL4	185.638	0.692	0.000	0.000	0.000	-61.170
		15:DL + LL5	196.590	1.441	0.000	0.000	0.000	-95.822
		16:DL + LL6	187.715	4.486	0.000	0.000	0.000	-50.773
		17:DL + LL7	208.347	3.219	0.000	0.000	0.000	-83.720
	21	1:DEAD LOAD	-175.924	-0.369	0.000	0.000	0.000	67.490
		11:DL + LL1	-186.876	-1.118	0.000	0.000	0.000	104.389
		12:DL + LL2	-197.362	-0.132	0.000	0.000	0.000	52.972
		13:DL + LL3	-198.633	-2.896	0.000	0.000	0.000	97.623
		14:DL + LL4	-185.605	1.646	0.000	0.000	0.000	59.738
		15:DL + LL5	-196.556	0.898	0.000	0.000	0.000	96.637
		16:DL + LL6	-187.681	-2.148	0.000	0.000	0.000	60.724
		17:DL + LL7	-208.313	-0.881	0.000	0.000	0.000	89.871

LAKE PARK ARCH BRIDGE - LOAD RATING

ARCH RIB CAPACITY

$p = A_s/A$;
P = strength of plain concrete column;
P'' = " " reinforced column;
f_c = unit stress in concrete;
f_s = " " " steel (not exceeding its elastic limit);
f_{el} = elastic-limit strength of steel;
f = average unit stress for entire cross-section;
p' = steel ratio of the hoops of hooped columns.

Formulas.

For short columns; ratio of length to least width not exceeding 20:

$$f_s = nf_c, \quad (54)$$

$$P'' = f_cA_c + f_sA_s, \quad (55)$$

$$P'' = f_cA[1+(n-1)p], \quad (56)$$

$$\frac{P''}{P} = 1+(n-1)p. \quad (57)$$

If nf_c is greater than the elastic-limit strength of the steel, then

$$P'' = f_cA_c + f_{el}A_s. \quad (58)$$

Considère's formula for hooped columns:

$$P'' = f_cA_c + f_{el}(p+2.4p')A. \quad (59)$$

For long columns:

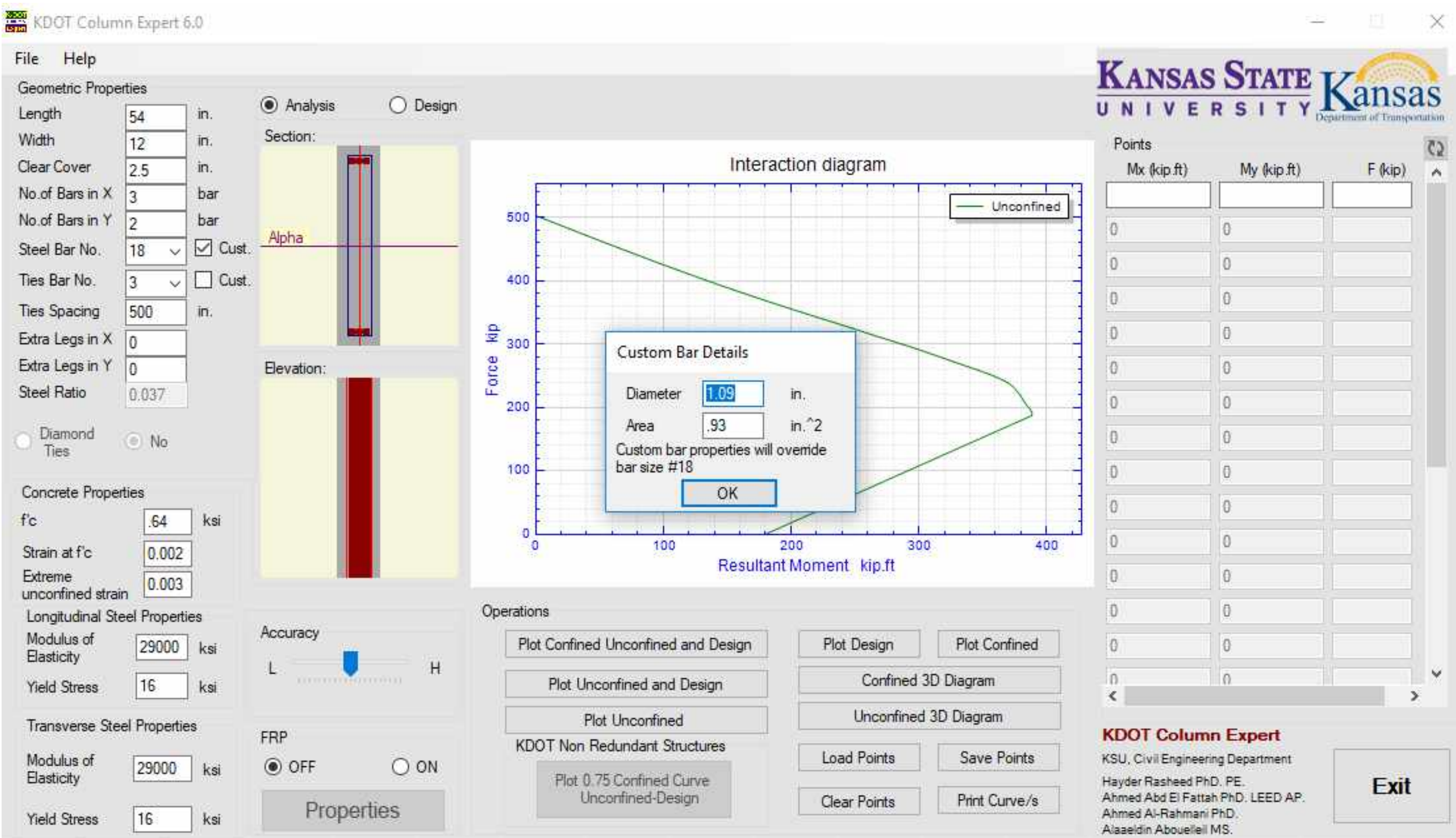
$$f = \frac{f_c[1+(n-1)p]}{1+\frac{1}{10,000}\left(\frac{l}{r}\right)^2}. \quad (60)$$

— Turneaure and Maurer, 1907 -#6

TranSystems	Made By	DWC	Date	7/25/2018	Job No. P402180060
	Checked By	SFH	Date	7/26/2018	

Calculations For: ***Lake Park Arch Bridge - Arch Rib Analysis***

LOWER ARCH - AS-BUILT / AS-CONFIGURED
AXIAL-MOMENT INTERACTION DIAGRAM



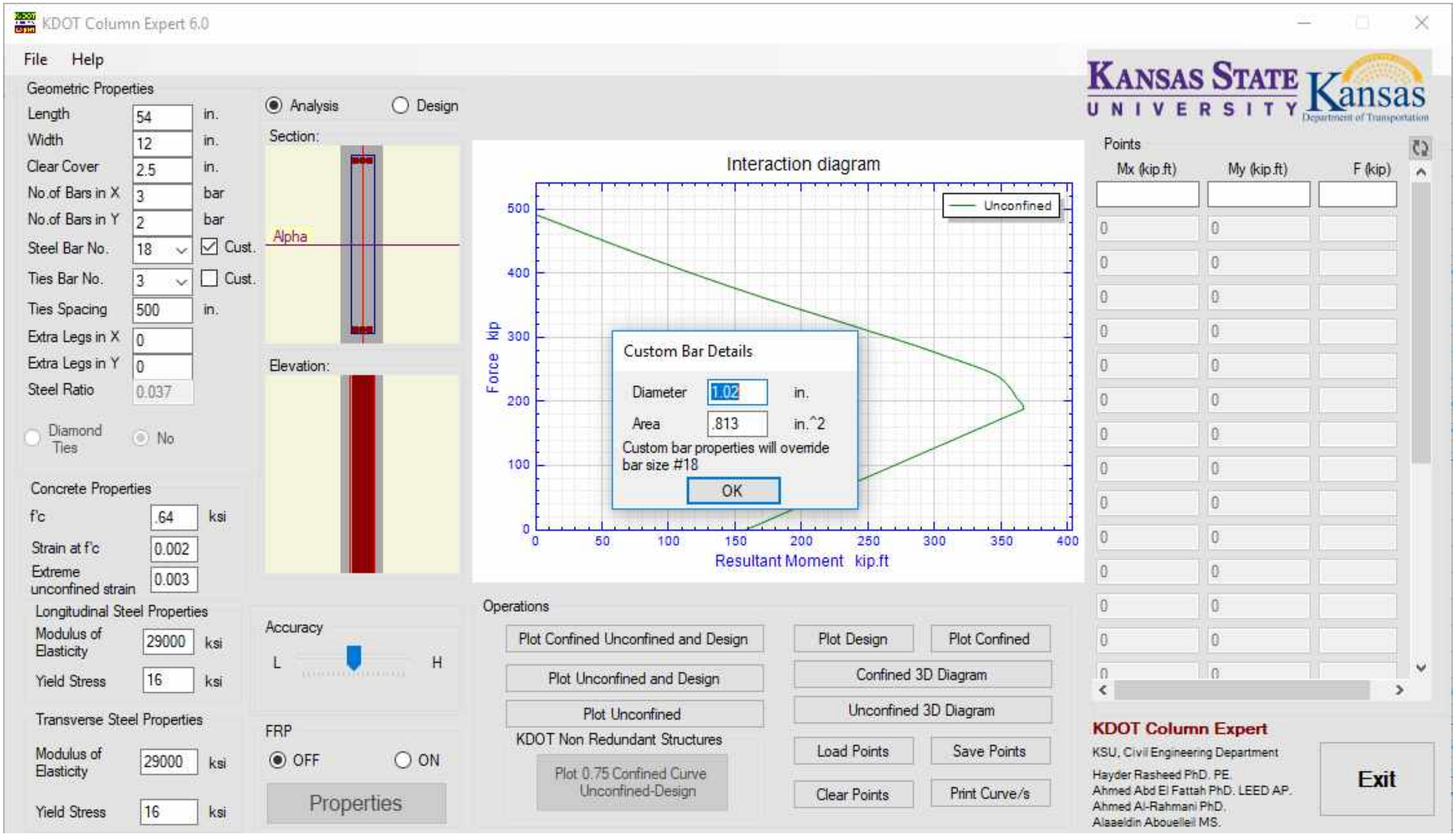
TranSystems

Made By	DWC	Date	7/25/2018	Job No.	P402180060
Checked By	SFH	Date	7/26/2018		

Calculations For: ***Lake Park Arch Bridge - Arch Rib Analysis***

MIDDLE ARCH - AS-BUILT / AS-CONFIGURED

AXIAL-MOMENT INTERACTION DIAGRAM

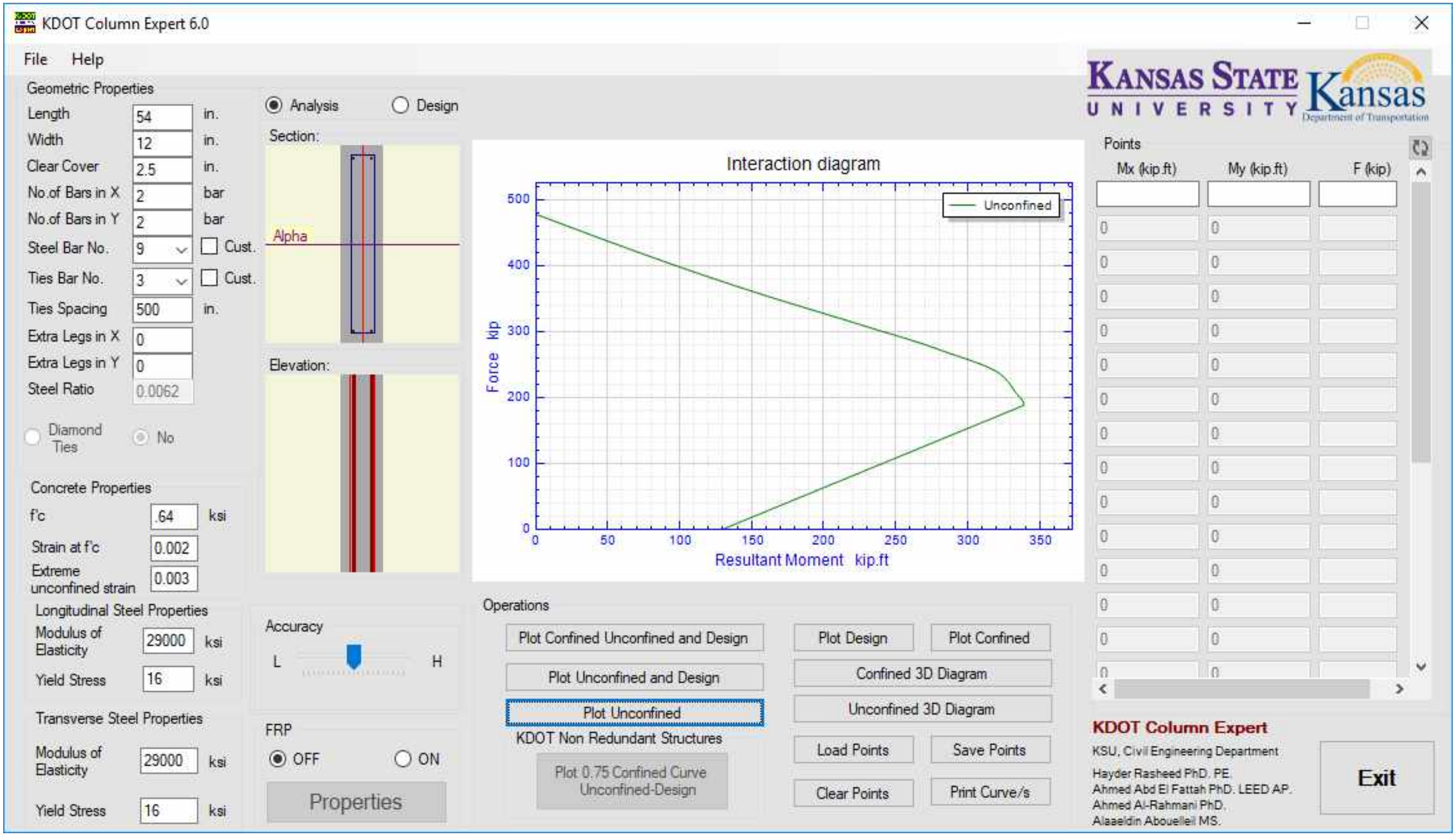


TranSystems	Made By	DWC	Date	7/25/2018	Job No. P402180060
	Checked By	SFH	Date	7/26/2018	

Calculations For: ***Lake Park Arch Bridge - Arch Rib Analysis***

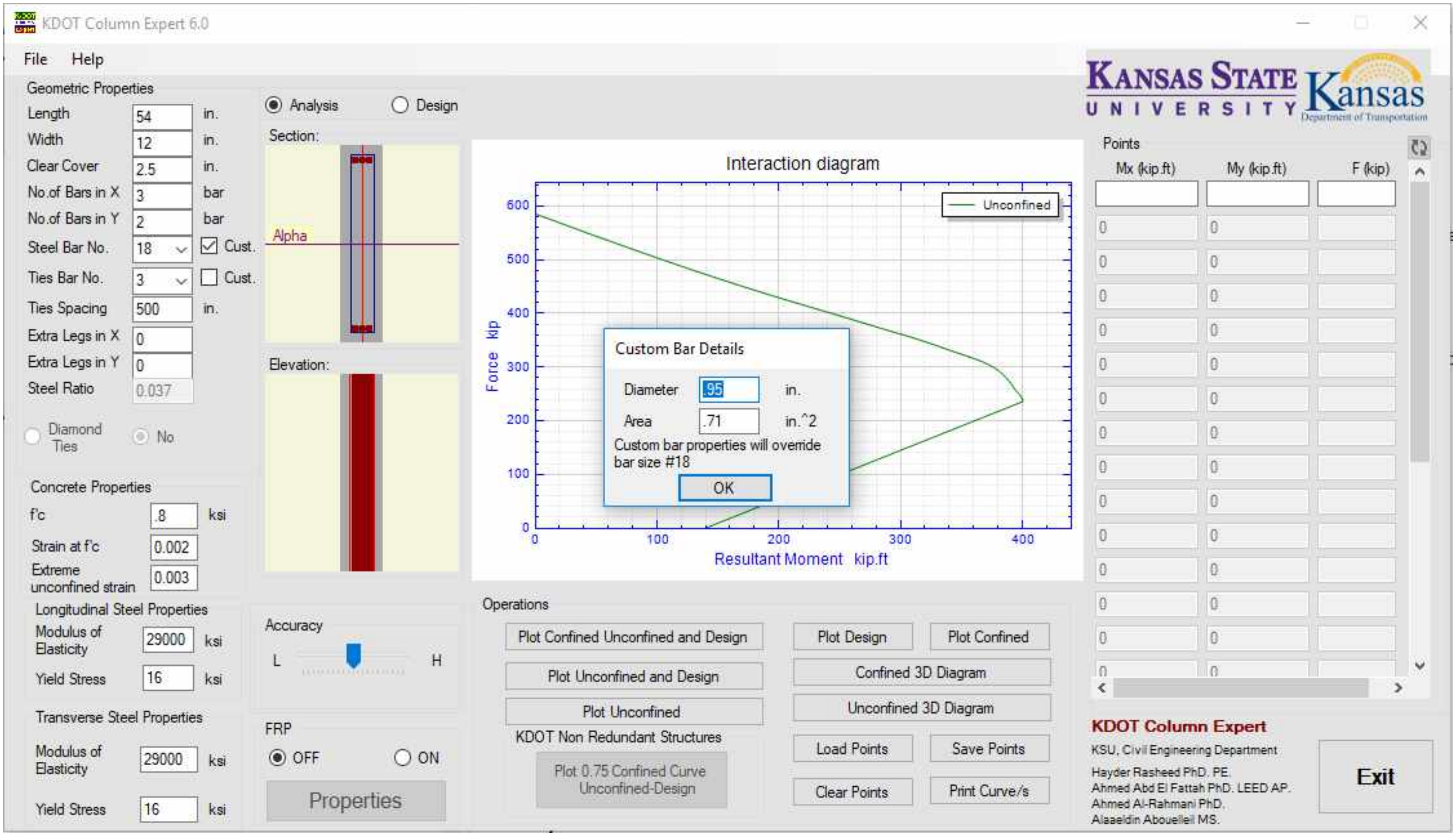
TOP ARCH - AS-BUILT / AS-CONFIGURED

AXIAL-MOMENT INTERACTION DIAGRAM



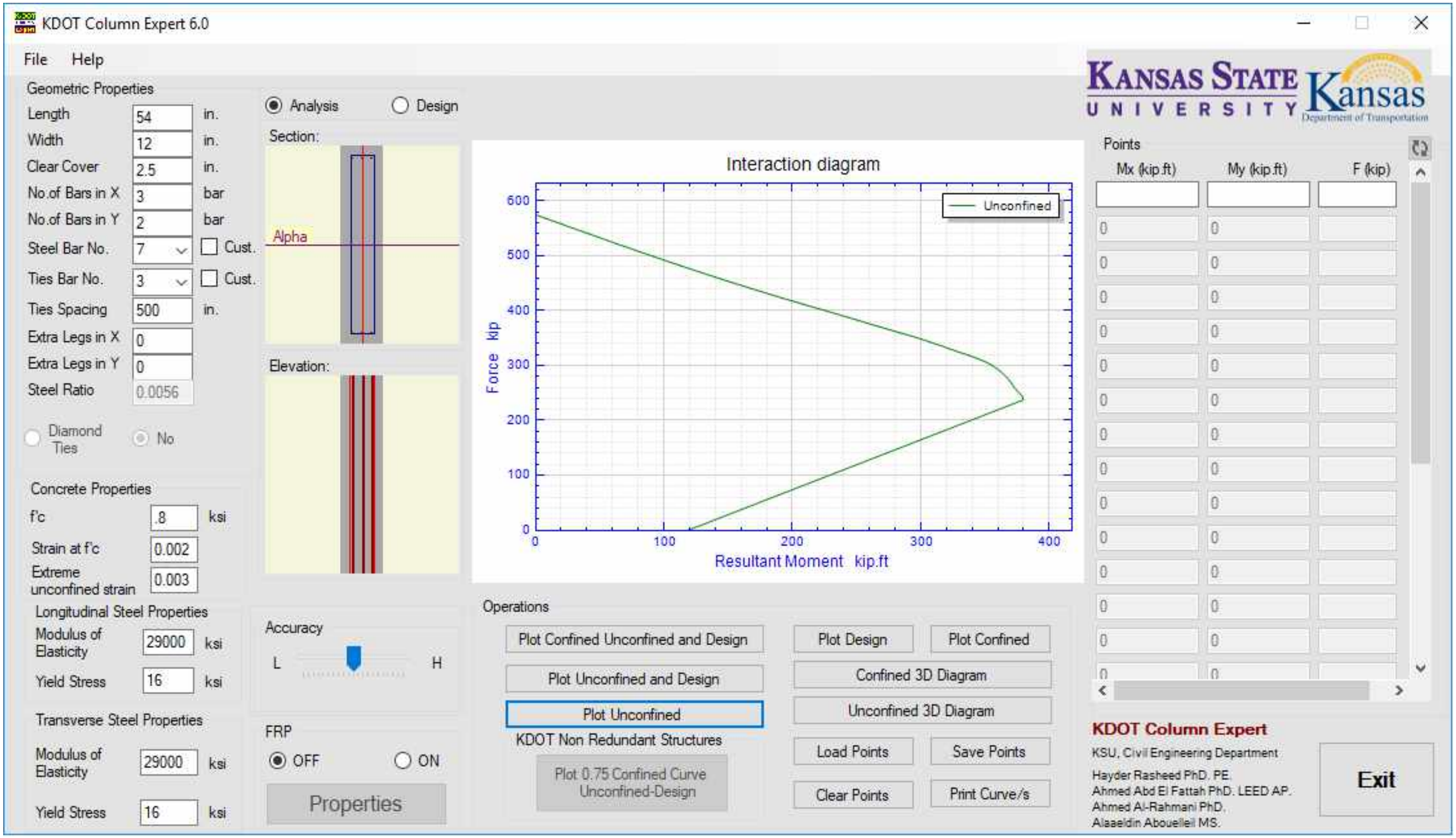
LOWER ARCH - AS-INSPECTED

AXIAL-MOMENT INTERACTION DIAGRAM



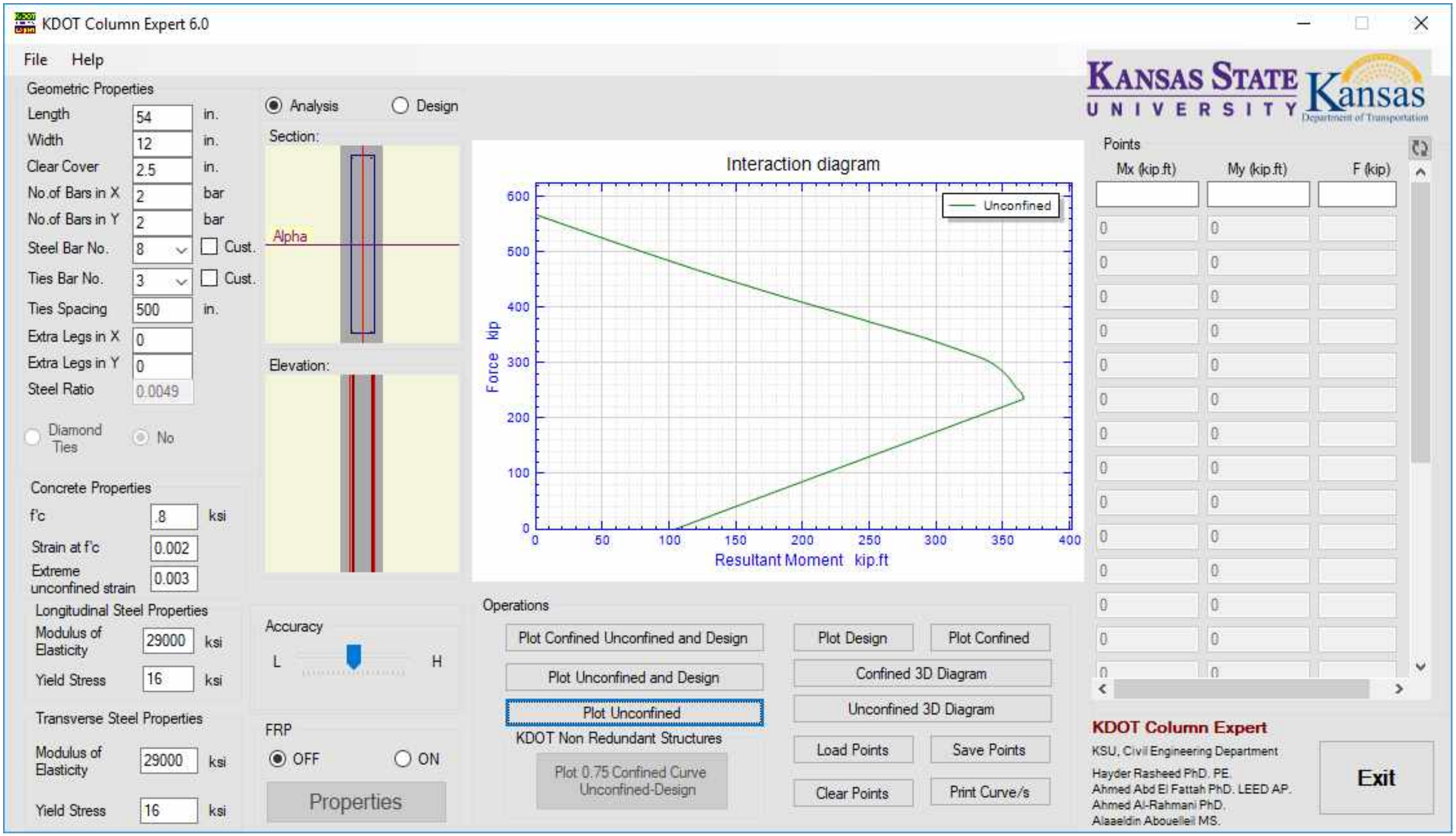
MIDDLE ARCH - AS-INSPECTED

AXIAL-MOMENT INTERACTION DIAGRAM



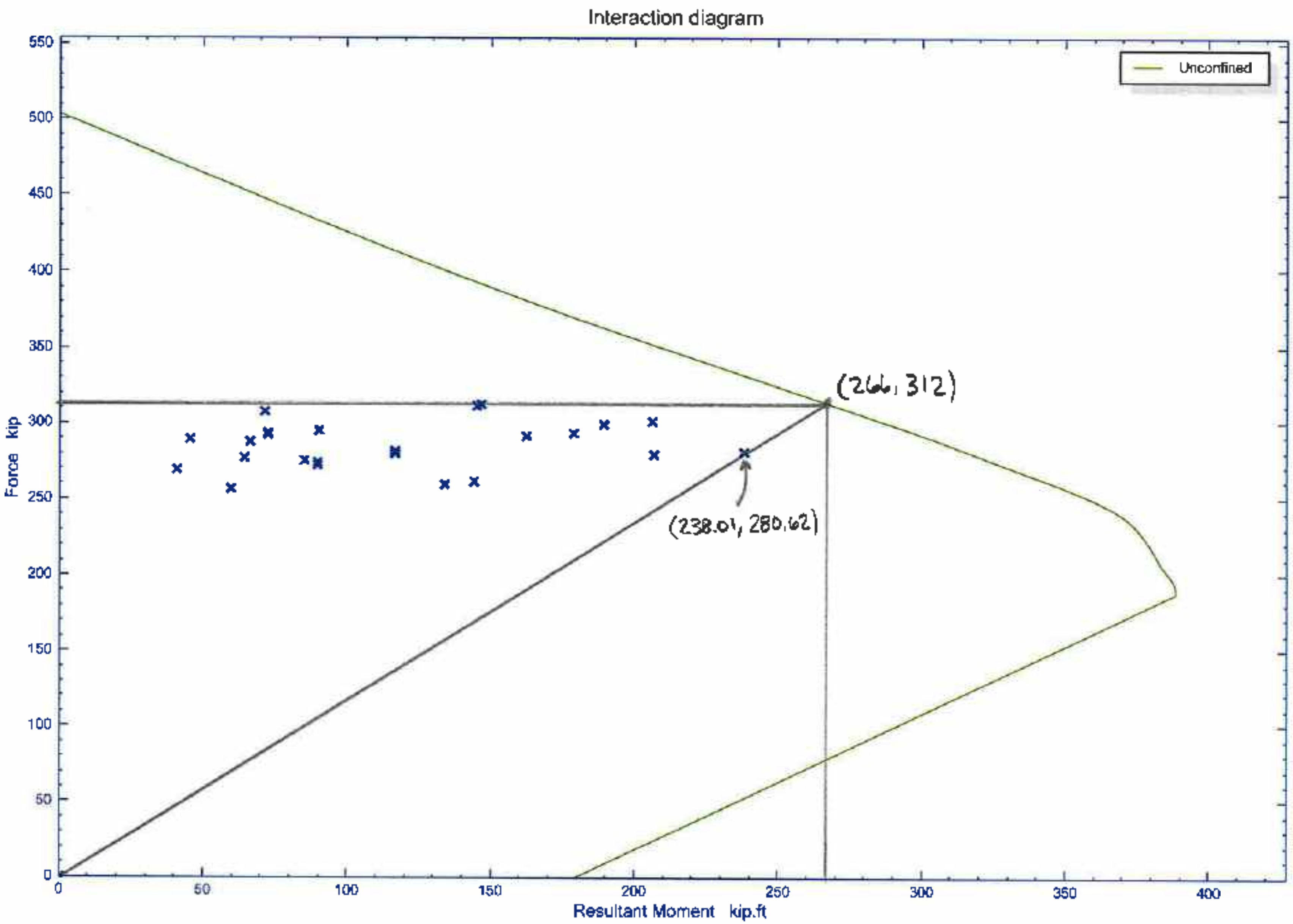
TOP ARCH - AS-INSPECTED

AXIAL-MOMENT INTERACTION DIAGRAM



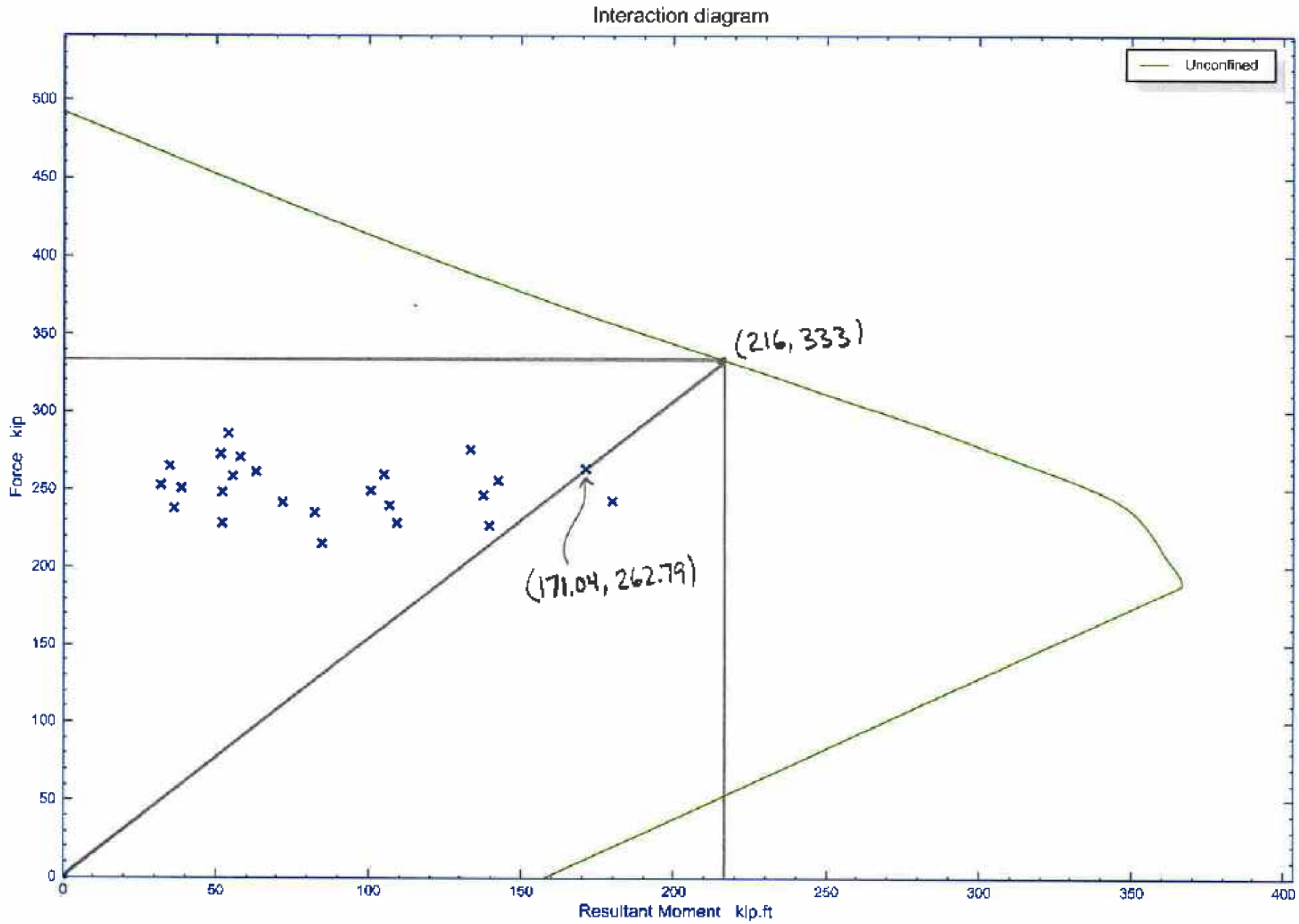
AS-BUILT - LOWER ARCH

Beams 1, 2, and 4



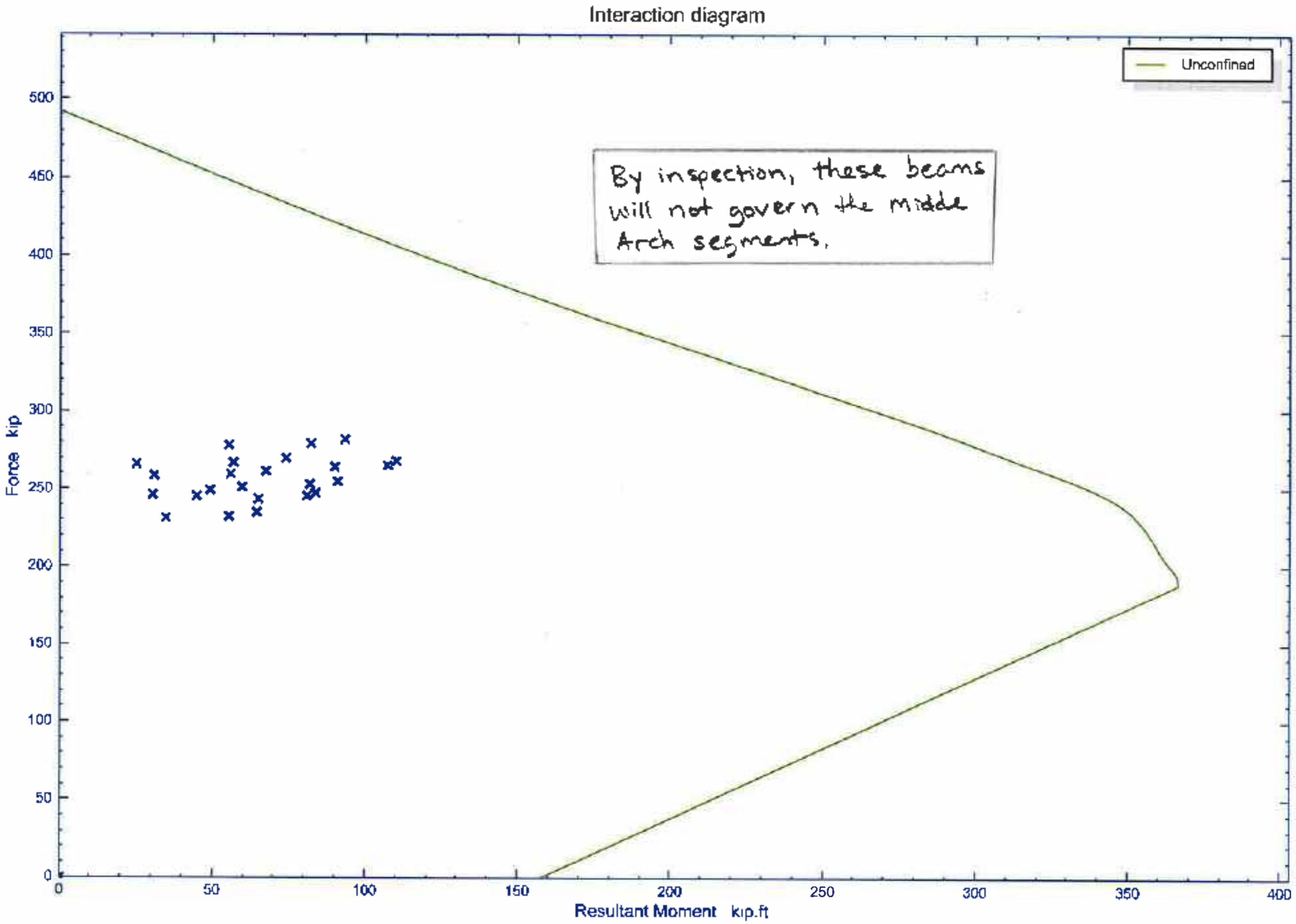
AS-BUILT - MIDDLE ARCH

Beams 5, 12, 14

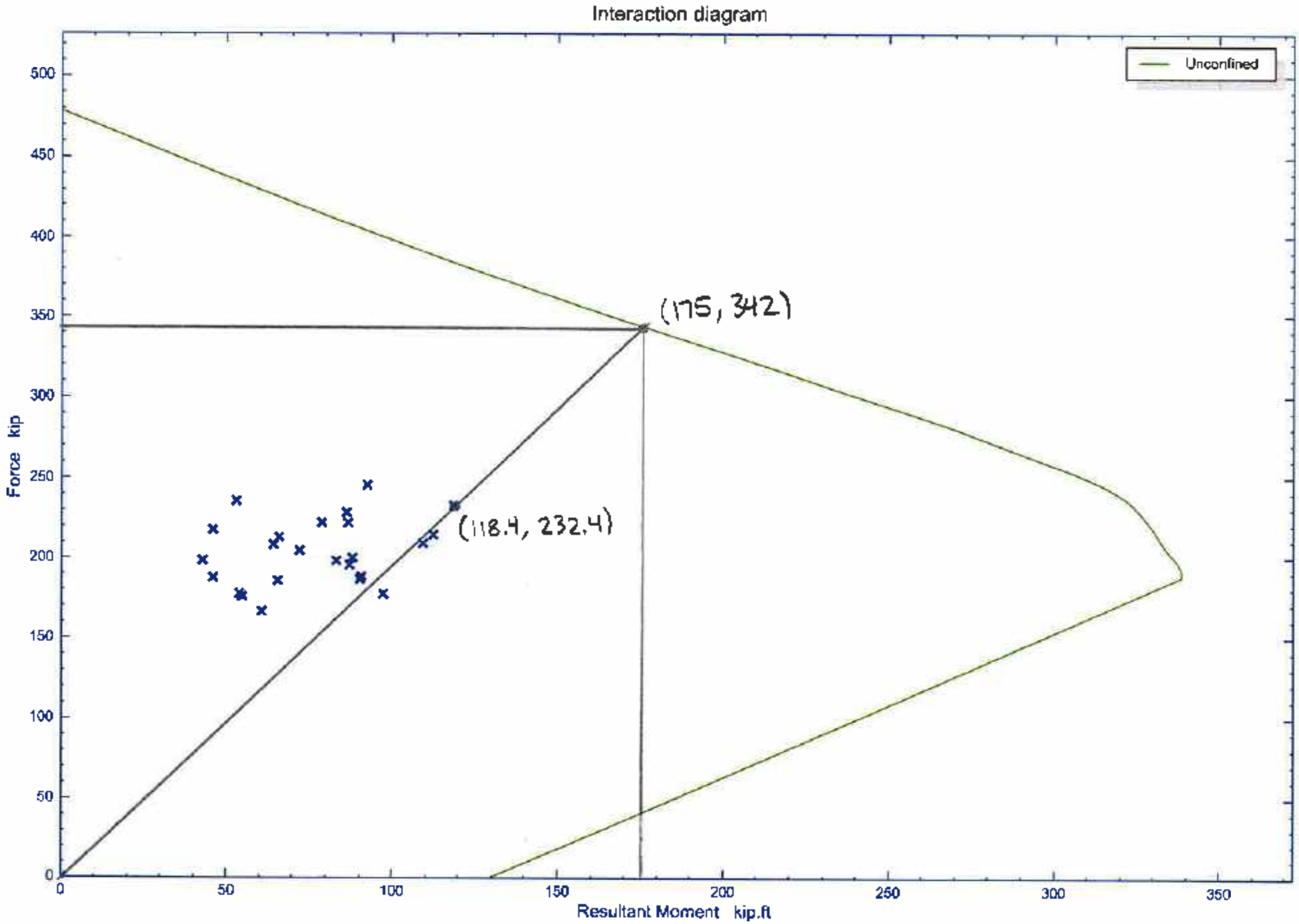


AS-BUILT - MIDDLE ARCH

Beams 7, 9, 10

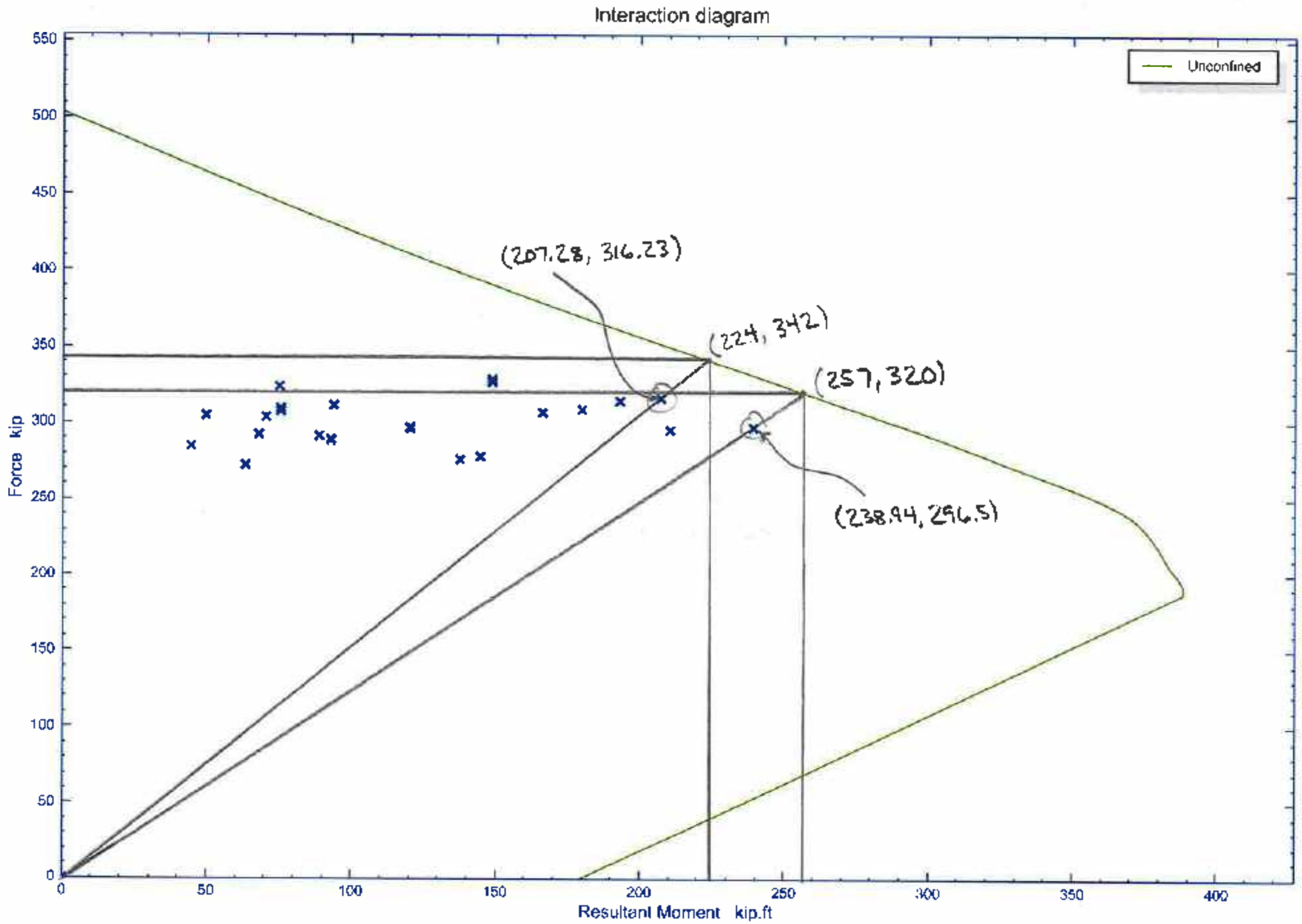


AS-BUILT - TOP ARCH
Beams 15, 17, 20



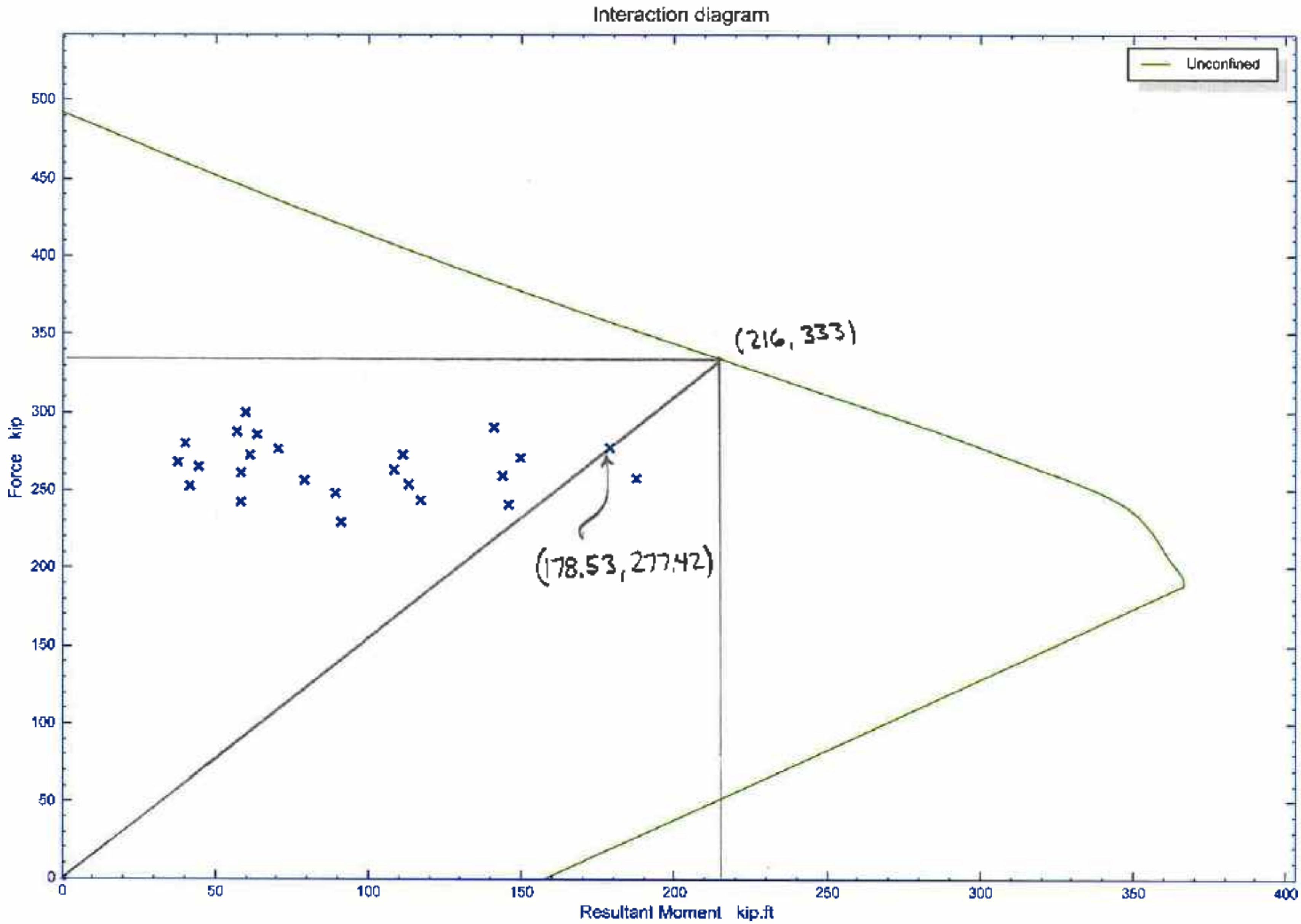
AS-CONFIGURED - LOWER ARCH

Beams 1, 2, 4



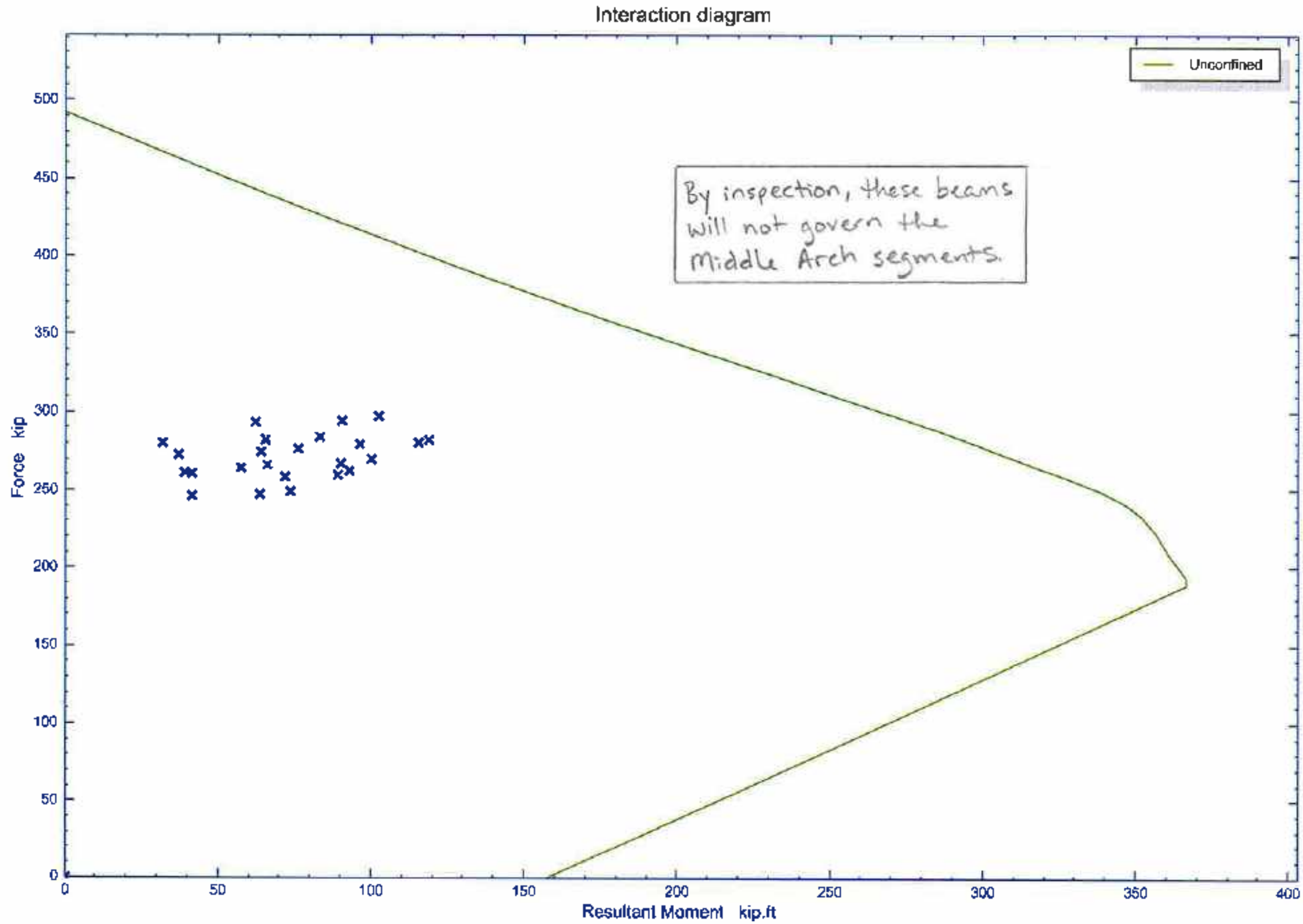
AS-CONFIGURED - MIDDLE ARCH

Beams 5, 12, 14



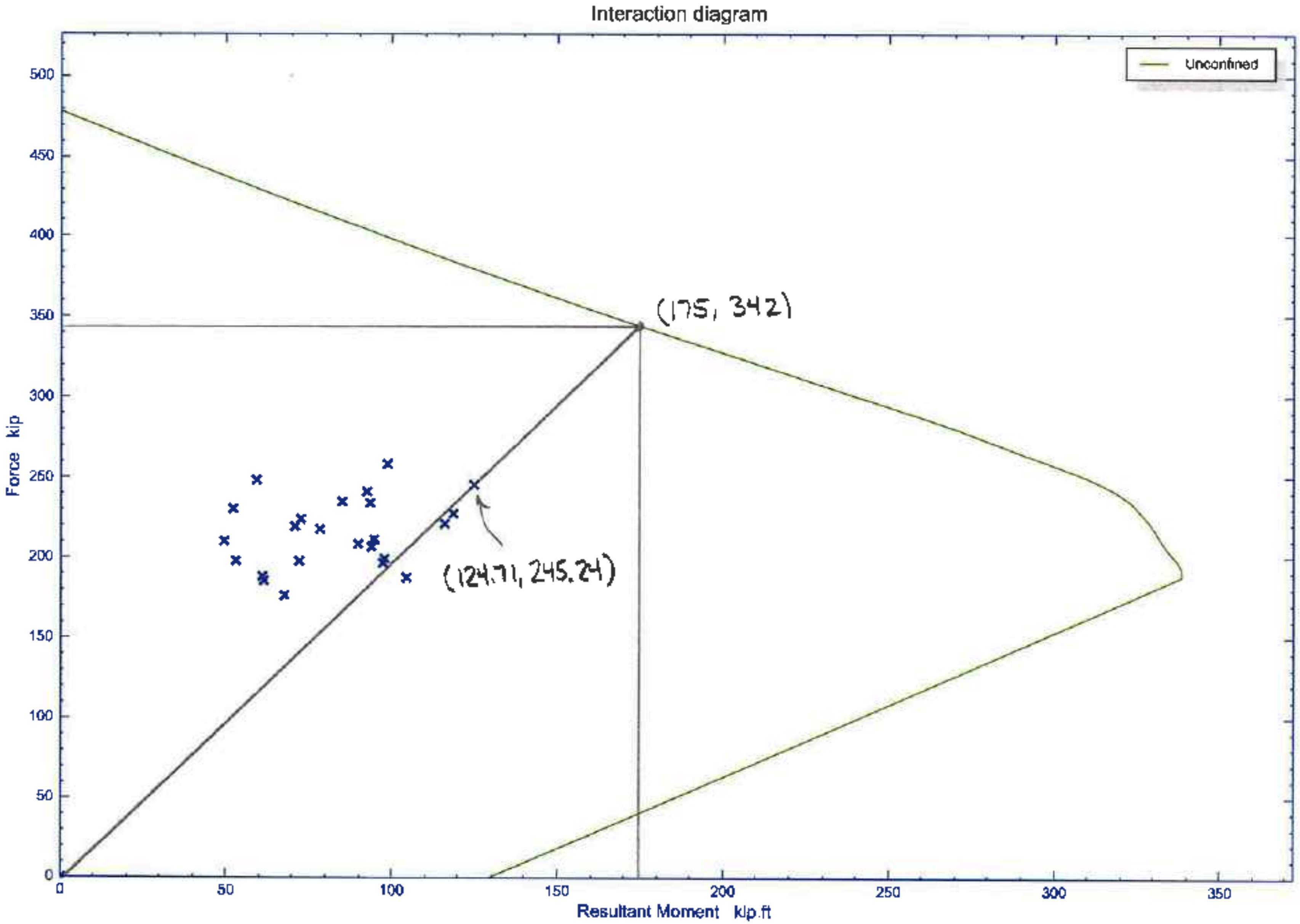
AS-CONFIGURED - MIDDLE ARCH

Beams 7, 9, 10



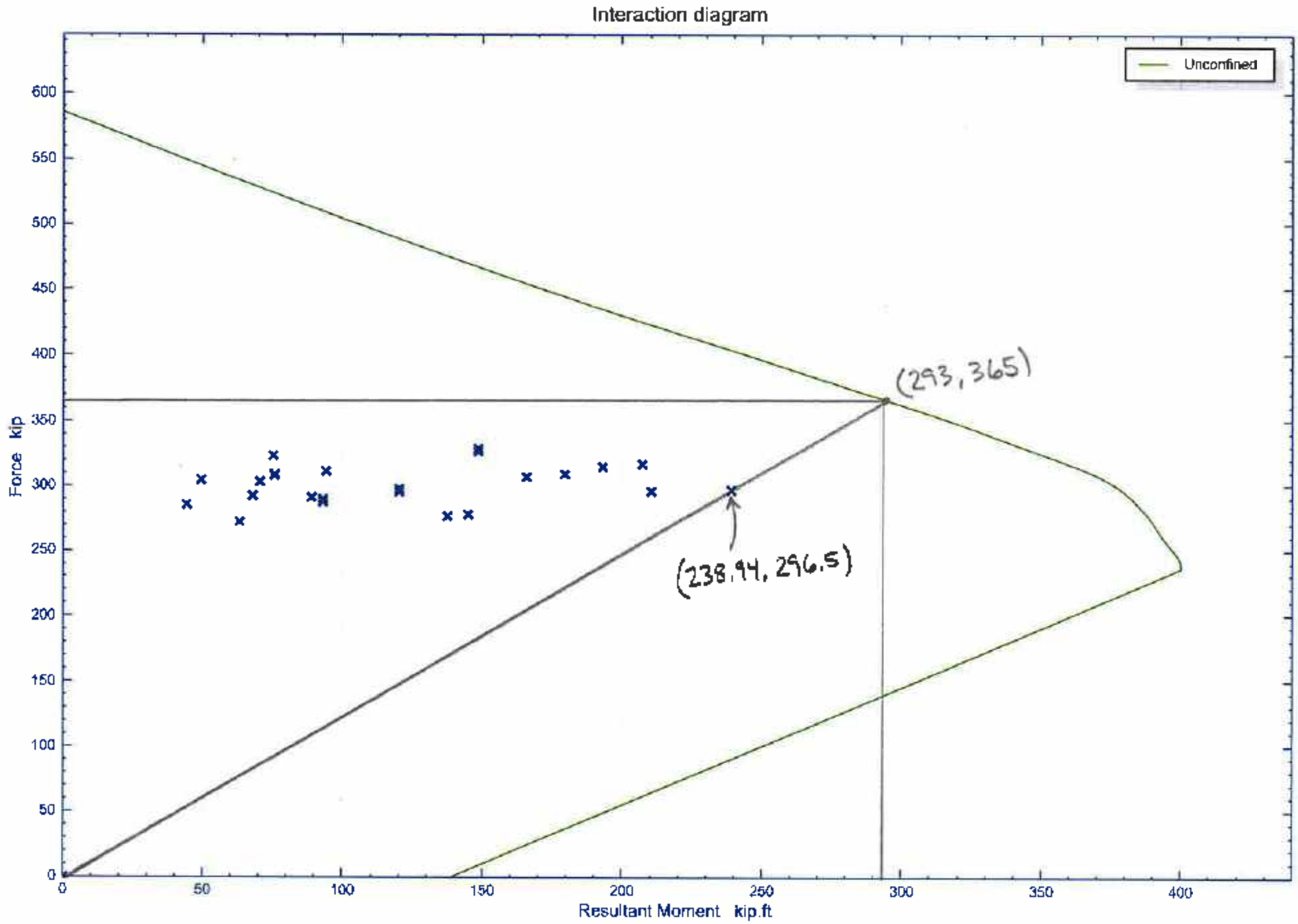
AS-CONFIGURED - TOP ARCH

Beams 15, 17, 20



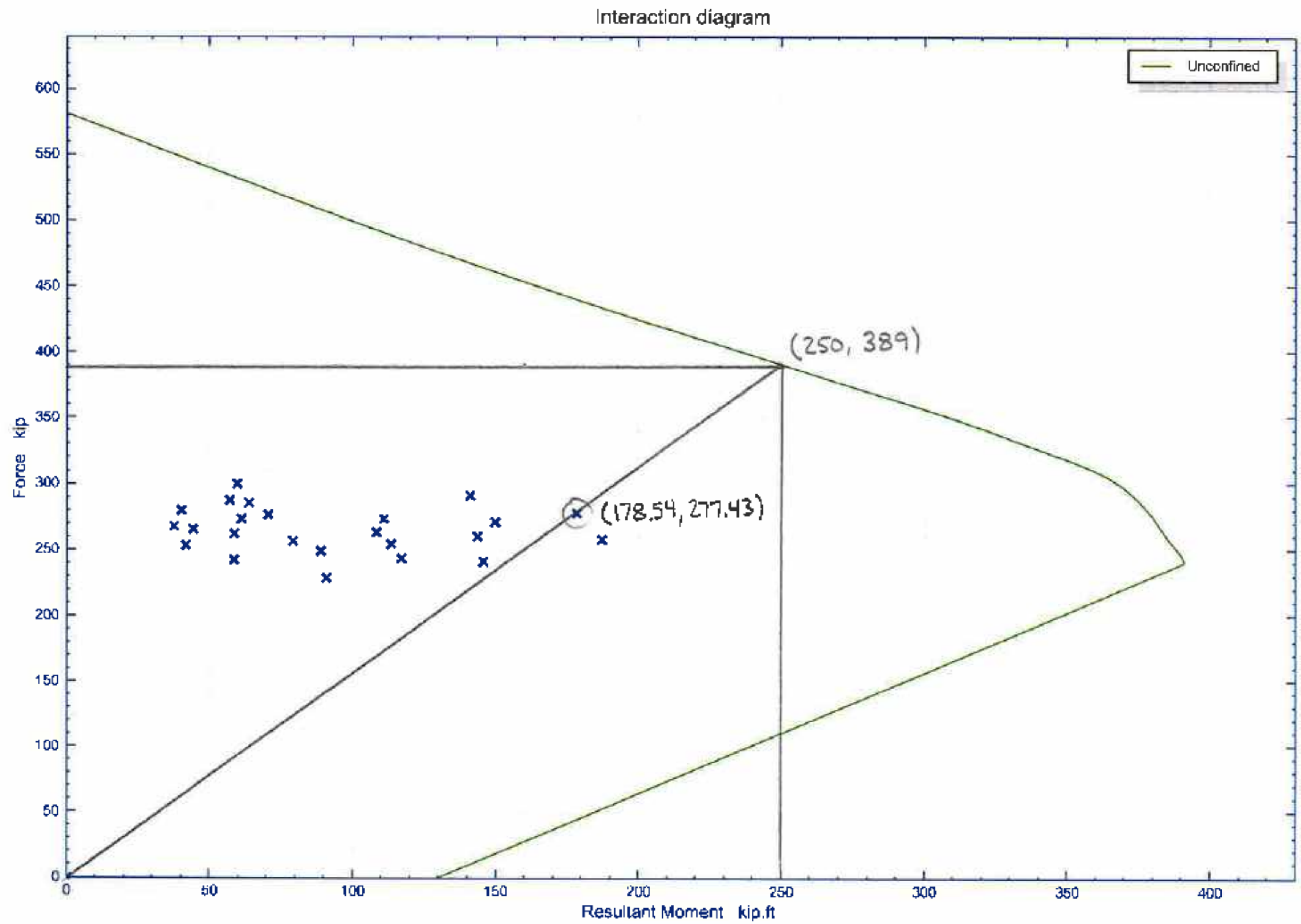
AS-INSPECTED - LOWER ARCH

Beams 1, 2, 4



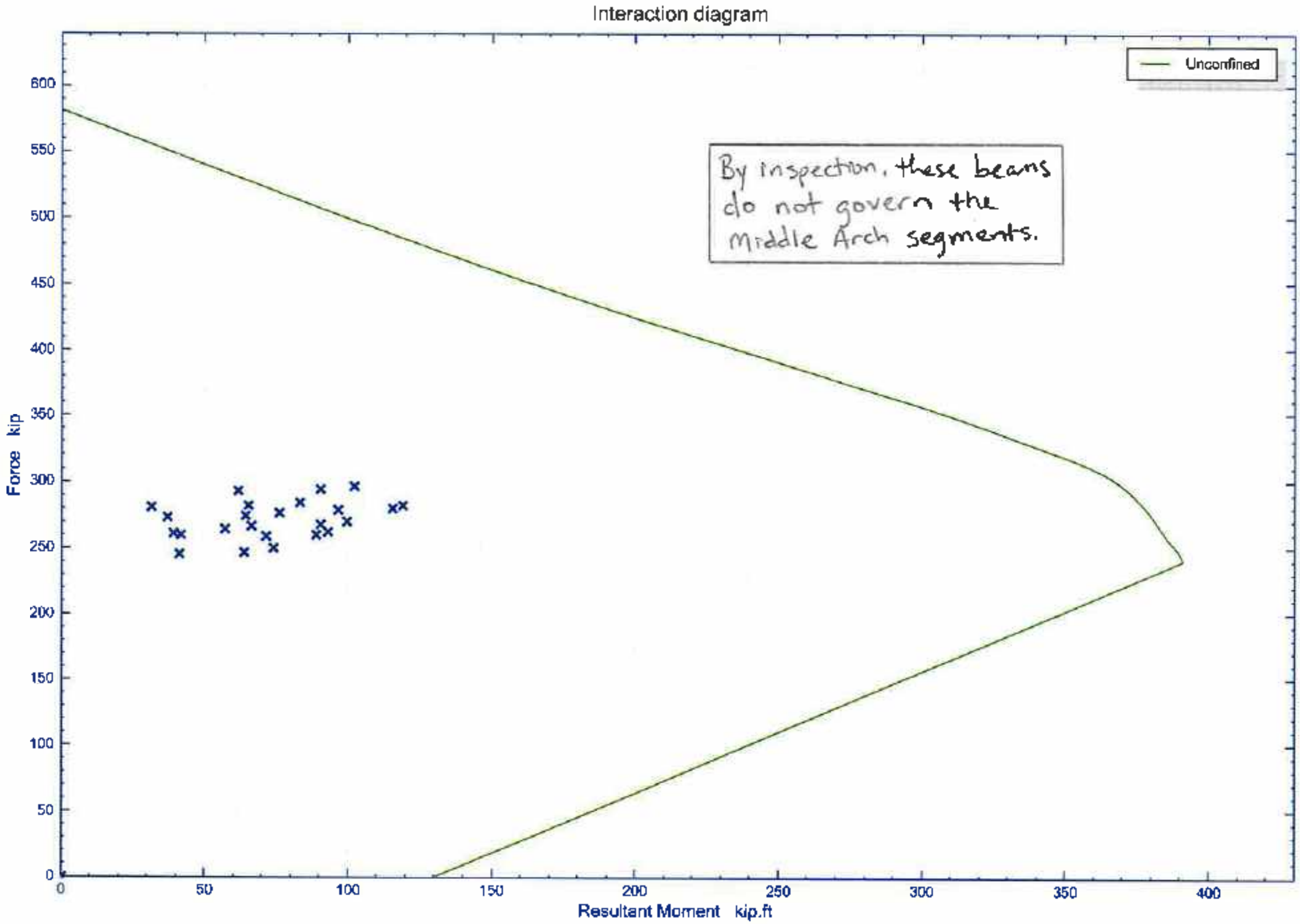
AS-INSPECTED - MIDDLE ARCH

Beams 5, 12, 14



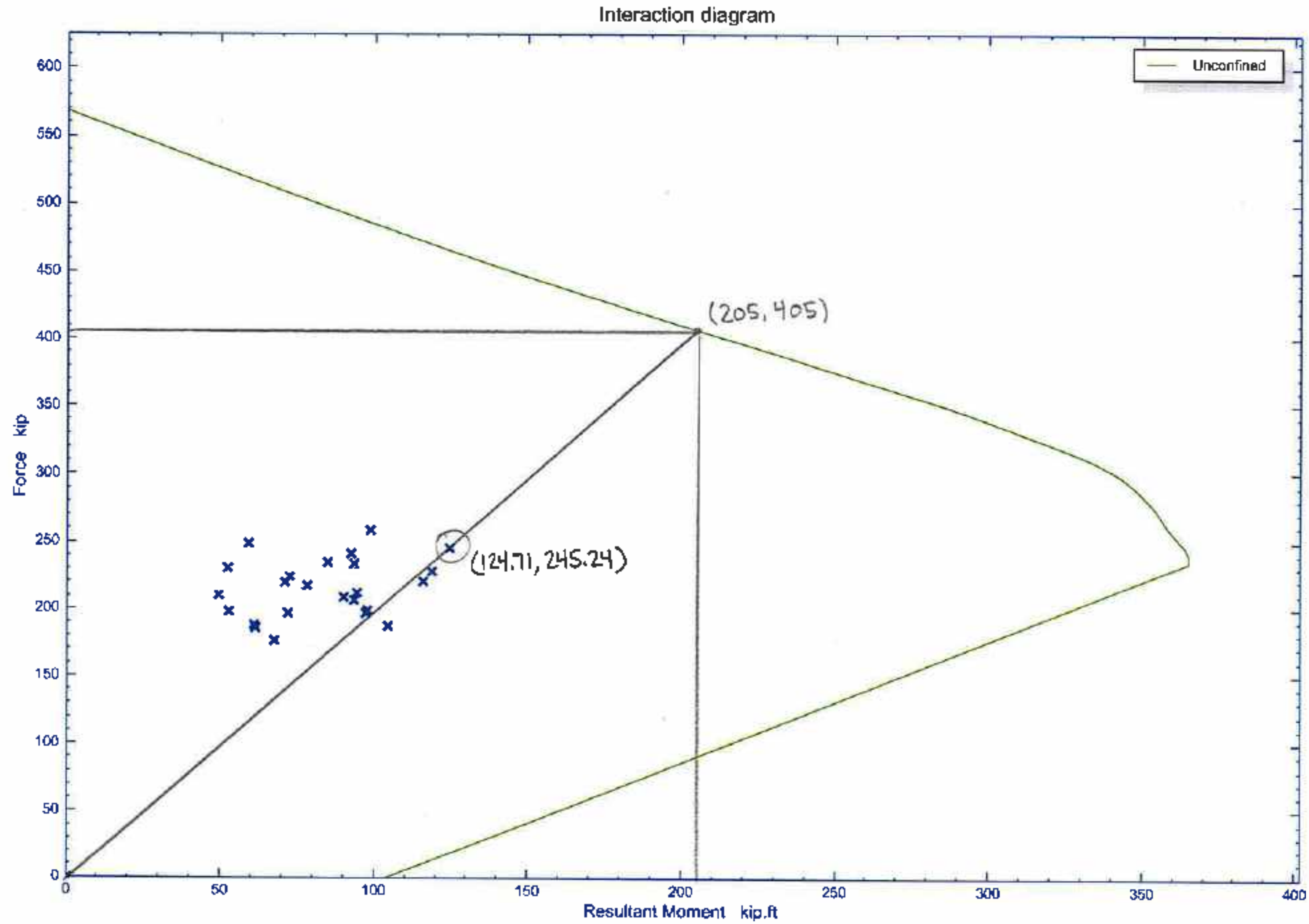
AS-INSPECTED - MIDDLE ARCH

Beams 7, 9, 10



AS-INSPECTED - TOP ARCH

Beams 15, 17, 20



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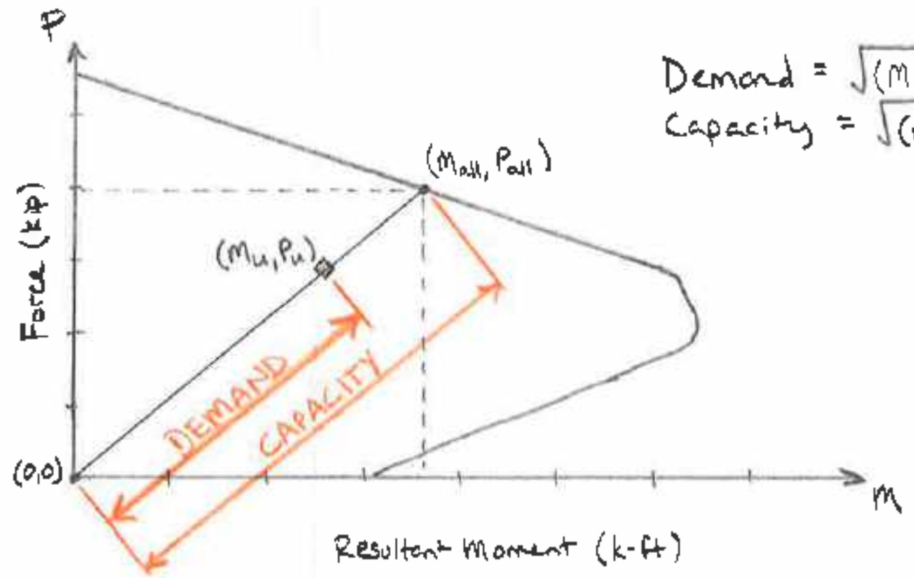
Checked By: SFH	Date: 7/25/18	Sheet No.

LAKE PARK ARCH BRIDGE - LOAD RATING

CAPACITY TO DEMAND RATIOS - ARCH RIBS



$$\text{Demand} = \sqrt{(M_u)^2 + (P_u)^2}$$
$$\text{Capacity} = \sqrt{(M_{all})^2 + (P_{all})^2}$$

TYPICAL AXIAL-MOMENT INTERACTION DIAGRAM

To determine capacity to demand ratios for arch ribs, chart moments and axial forces from applicable load cases on interaction diagrams. If load case falls within envelope of diagram, the section has sufficient capacity for the applied load. If the load falls outside the bounds of the interaction diagram, the applied loading exceeds the capacity of the section (Capacity/Demand Ratio less than 1.0).

To calculate capacity to demand ratios, calculate the scale factor necessary to make the governing load case intersect the axial-moment interaction curve.

By inspection, lower arch segments govern for all analysis alternatives.

LAKE PARK ARCH BRIDGE - LOAD RATING

CAPACITY TO DEMAND RATIOS - ARCH RIBS

AS-BUILT : $\dfrac{\text{Capacity}}{\text{Demand}} = \dfrac{\sqrt{(266)^2+(312)^2}}{\sqrt{(238.01)^2+(280.62)^2}} = \dfrac{410}{367.96} = \underline{\underline{1.11}}$

AS-CONFIGURED : Check two cases that potentially govern

$$\frac{\text{Capacity}}{\text{Demand}} = \frac{\sqrt{(224)^2+(342)^2}}{\sqrt{(207.28)^2+(316.23)^2}} = \frac{408.8}{378.1} = 1.08$$

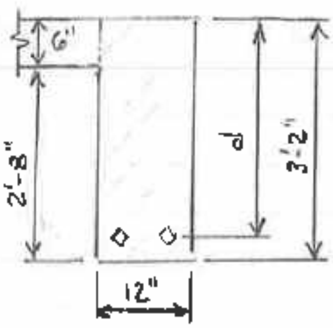
$$\frac{\text{Capacity}}{\text{Demand}} = \frac{\sqrt{(257)^2+(320)^2}}{\sqrt{(238.94)^2+(296.5)^2}} = \frac{410.4}{380.8} = \underline{\underline{1.07}} \leftarrow \text{GOVERNS}$$

AS-INSPECTED : $\dfrac{\text{Capacity}}{\text{Demand}} = \dfrac{\sqrt{(293)^2+(365)^2}}{\sqrt{(238.94)^2+(296.5)^2}} = \underline{\underline{1.22}}$

LAKE PARK ARCH BRIDGE - LOAD RATING

LONGITUDINAL SPANDREL MEMBER

Perform analysis of longitudinal spandrel beam over teardrop opening. Because deck was poured monolithic with spandrel beam, include depth from bottom of beam to top of deck.



BEAM SECTION

$h = 3'\text{-}2"$ $\quad b = 12"$

$L = 20'$ (simply supported, per plans)

Reinforcement: Two (2) 1"x3" Khan bars
($A_{bar} = 1.42\ in^2$, $A_{core} = (1.0)(1.0) = 1.00\ in^2$)

Assume reinforcement is placed in such a manner as to have the strength of the full bars near midspan, but trussed shear bars are bent up 45° in shear zones near end, per recommendations of historic Khan system. Assume full bars for middle half of span (10'), only core where bars are bent for shear (5' on each end for 1/4 points).

$\therefore A_s = 2(1.42\ in^2) = 2.84\ in^2$ (midspan)
$A_s = 2(1.00\ in^2) = 2.00\ in^2$ (1/4 POINT)

Dead Loads:

Deck $= (0.15\ kcf)(15'/2)(6"/12) = 0.56\ k/ft$ [As-Built]
$= (0.15\ kcf)(15'/2)\left(\frac{6"+1"}{12}\right) = 0.66\ k/ft$ [As-Configured / As-Inspected]

Parapets $= 0.42\ k/ft$ [As-Built]
$= 0.47\ k/ft$ [As-Configured / As-Inspected]
} see Arch Dead Load calculations

Beam Self Weight $= (0.15\ kcf)(32"/12)(12"/12) = 0.40\ k/ft$

$\therefore g_{DL} = 0.56 + 0.42 + 0.40 = 1.38\ k/ft$ [As-Built]
$g_{DL} = 0.66 + 0.47 + 0.40 = 1.53\ k/ft$ [As-Configured / As-Inspected]

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LAKE PARK ARCH BRIDGE - LOAD RATING

Live Load: $g_{LL} = (0.080\ ksf)(12'/2) = 0.48\ k/ft$

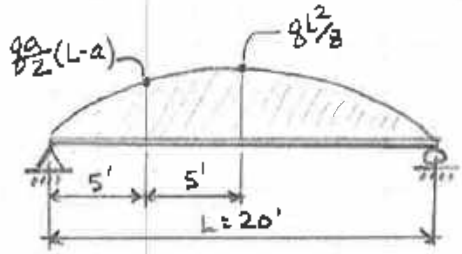
Total Loads: $g_{TOT} = g_{DL} + g_{LL}$

$g_{TOT} = 1.38\ k/ft + 0.48\ k/ft = 1.86\ k/ft$ [As-Built]
$g_{TOT} = 1.53\ k/ft + 0.48\ k/ft = 2.01\ k/ft$ [As-Configured / As-Inspected]

Moments: Calculate maximum moment at midspan and at 1/4 PT to check both reinforcing configurations

$M_{MAX} = gL^2/8$

$M_{1/4PT} = \frac{ga}{2}(L-a)$



As-Built

$M_{MAX} = (1.86\ k/ft)\frac{(20')^2}{8} = 93.0\ k\text{-}ft$

$M_{1/4PT} = (1.86\ k/ft)(5'/2)(20'-5') = 69.75\ k\text{-}ft$

As-Configured / As-Inspected

$M_{MAX} = (2.01\ k/ft)\frac{(20')^2}{8} = 100.5\ k\text{-}ft$

$M_{1/4PT} = (2.01\ k/ft)(5'/2)(20'-5') = 75.38\ k\text{-}ft$

Moment Capacity

Midspan: $a = \frac{A_s f_y}{0.85 f_c b} = \frac{(2.84)(16)}{0.85(.64)(12)} = 6.96''$

$M_{all} = A_s f_y (d - a/2) = (2.84)(16)(35.29 - \frac{6.96}{2}) = 1445.4\ k\text{-}in$
$= 120.4\ k\text{-}ft$

1/4 POINT: $a = \frac{A_s f_y}{0.85 f_c b} = \frac{(2)(16)}{0.85(.64)(12)} = 4.90''$

$M_{all} = (2.0)(16)(35.29 - \frac{4.90}{2}) = 1050.9\ k\text{-}in = 87.57\ k\text{-}ft$

Calculations for 1/4 US Grid

LAKE PARK ARCH BRIDGE - LOAD RATING

As-Inspected Moment Capacity

For As-Inspected analysis, need to also consider concrete testing (use $f'_c = 2000$ psi $\longrightarrow f_c = 0.4(2000) = 800$ psi) and account for section loss on reinforcement.

Based on photographs of spalled areas on spandrel beam, section loss appears to be 1/16" deep average. Calculate loss percentage based on 1/16" loss on both sides of the core and apply to total bar area:

$$\% \text{ Remaining} = \frac{(15/16)^2}{(1.0)^2} = 0.879 = 87.9\%$$

$$\therefore A_s = (0.879)(2.84 \text{ in}^2) = 2.49 \text{ in}^2 \quad \text{(midspan)}$$

$$A_s = (0.879)(2.00 \text{ in}^2) = 1.76 \text{ in}^2 \quad \text{(1/4 POINT)}$$

Midspan: $$a = \frac{A_s f_y}{0.85 f_c b} = \frac{(2.49)(16)}{0.85(0.8)(12)} = 4.88"$$

$$M_{all} = A_s f_y \left(d - \frac{a}{2}\right) = (2.49)(16)\left(35.29 - \frac{4.88}{2}\right) = 1308.7^{k\text{-}in} = 109.0^{k\text{-}ft}$$

1/4 POINT: $$a = \frac{A_s f_y}{0.85 f_c b} = \frac{(1.76)(16)}{0.85(0.8)(12)} = 3.45"$$

$$M_{all} = A_s f_y \left(d - \frac{a}{2}\right) = (1.76)(16)\left(35.29 - \frac{3.45}{2}\right) = 945.2^{k\text{-}in} = 78.77^{k\text{-}ft}$$

Capacity to Demand Ratios:

Check capacity to demand ratios at midspan and 1/4 points for all analysis alternatives

$$\frac{\text{Capacity}}{\text{Demand}} = \frac{M_{all}}{M_{max}} = \frac{M_{all}}{M_{DL} + M_{LL}}$$

LAKE PARK ARCH BRIDGE - LOAD RATING

AS-BUILT

Midspan: $C/D = \frac{120.4 \text{ k-ft}}{93.0 \text{ k-ft}} = 1.29$

1/4 Point: $\frac{C}{D} = \frac{87.57 \text{ k-ft}}{69.75 \text{ k-ft}} = 1.25$ ← GOVERNS

AS-CONFIGURED

Midspan: $\frac{C}{D} = \frac{120.4 \text{ k-ft}}{100.5 \text{ k-ft}} = 1.19$

1/4 Point: $\frac{C}{D} = \frac{87.57 \text{ k-ft}}{75.38 \text{ k-ft}} = 1.16$ ← GOVERNS

AS-INSPECTED

Midspan: $C/D = \frac{109.0 \text{ k-ft}}{100.5 \text{ k-ft}} = 1.08$

1/4 POINT: $C/D = \frac{78.77 \text{ k-ft}}{75.38 \text{ k-ft}} = 1.04$ ← GOVERNS

SHEAR CAPACITY

Also check shear capacity of longitudinal spandrel member to determine whether it may govern. Check As-Built case and compare to others to see if calculations need to be performed.

Max shear $= \frac{\omega L}{2} = (1.86 \text{ k/ft})(20'/2) = 18.6$ kips

Max shear stress $= v = \frac{V}{b_w d} = \frac{(18.6^k)(1000)}{(12")(35.29")} = 43.92$ psi [AASHTO EQN 8-3]

Concrete Capacity Only

$V_c = 0.95\sqrt{f'_c} = 0.95\sqrt{1600 \text{ psi}} = 38.0$ psi [AASHTO 8.15.5.2.1]

LAKE PARK ARCH BRIDGE - LOAD RATING

Shear Reinforcement

$$A_v = \frac{(V-V_c)b_w s}{f_s(\sin\alpha + \cos\alpha)}$$ [AASHTO 8.15.5.8.3 - Inclined Stirrups]

where $A_v = 2(0.795")(0.25") = 0.3975\ in^2$

$\alpha = 45°$ (assumed from design plans + Khan standards)

$b_w = 12"$

$s = 24"$ (scaled from plans)

$f_s = 16$ ksi

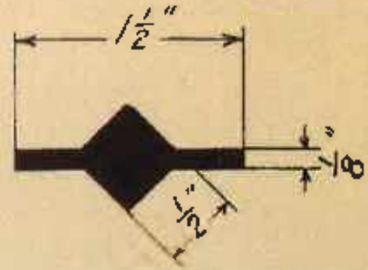
$$\frac{A_v f_s(\sin\alpha + \cos\alpha)}{b_w s} = (V - V_c) = V_s$$

$$\frac{(0.3975\ in^2)(16\ ksi)(\sin 45° + \cos 45°)}{(12")(24")} \times \frac{1000\ psi}{ksi} = 31.23\ psi$$

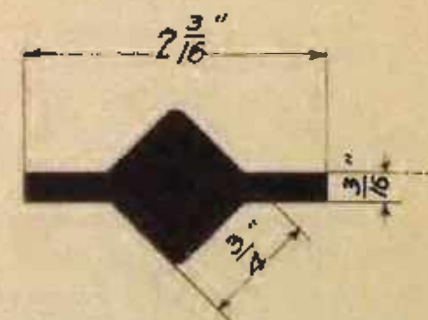
$$\therefore V_{all} = V_s + V_c = 31.23\ psi + 38.0\ psi = 69.23\ psi$$

$$\therefore \frac{Capacity}{Demand} = \frac{V_{all}}{V_{DL} + V_{LL}} = \frac{69.23\ psi}{43.92\ psi} = \underline{\underline{1.57}}$$

Because capacity to demand ratio for shear is higher than the ratio for moment and embedded shear reinforcement likely exhibits no significant section loss, assume shear capacity does not govern the longitudinal spandrel member.

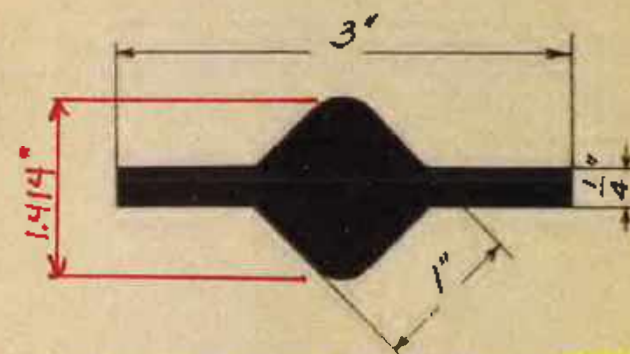


$1\frac{1}{2}'' \times \frac{1}{2}''$. Area .38 □" Weight 1.4# per ft.



These bars can have any standard size cuts as shown in Figure 11 and will be sent in lengths as ordered. In making calculations for strength of reinforced beam, assume the area of the entire cross section as here given.

$2\frac{3}{16}'' \times \frac{3}{4}''$. Area .78 □" Weight 2.7# per ft.



$3'' \times 1''$. Area 1.42 □" Weight 4.8# per ft.

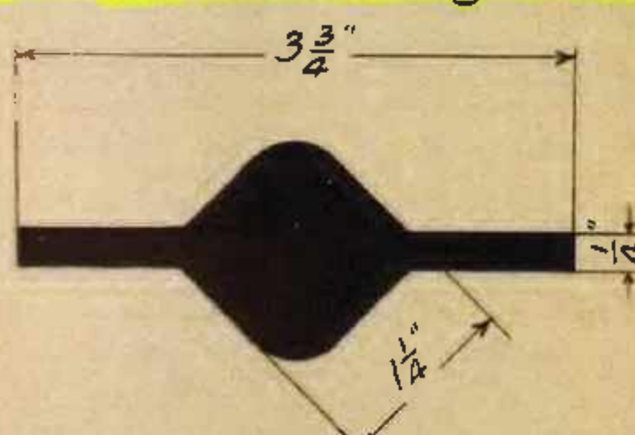


Fig. 13. $3\frac{3}{4}'' \times 1\frac{1}{4}''$. Area 2.0 □" Weight 6.9# per ft.

8.14.3.6 Walls exceeding 8 feet in height on filled spandrel arches shall be laterally supported by transverse diaphragms or counterforts with a slope greater than 45 degrees with the vertical to reduce transverse stresses in the arch barrel. The top of the arch barrel and interior faces of the spandrel walls shall be waterproofed and a drainage system provided for the fill.

8.15 SERVICE LOAD DESIGN METHOD (ALLOWABLE STRESS DESIGN)

8.15.1 General Requirements

8.15.1.1 Service load stresses shall not exceed the values given in Article 8.15.2.

8.15.1.2 Development and splices of reinforcement shall be as required in Articles 8.24 through 8.32.

8.15.2 Allowable Stresses

8.15.2.1 Concrete

Stresses in concrete shall not exceed the following:

8.15.2.1.1 Flexure

Extreme fiber stress in compression, f_c .... $0.40f'_c$
Extreme fiber stress in tension for plain concrete, f_t .... $0.21f_r$

Modulus of rupture, f_r, from tests, or, if data are not available:

Normal weight concrete .... $7.5\sqrt{f'_c}$
"Sand-lightweight" concrete .... $6.3\sqrt{f'_c}$
"All-lightweight" concrete .... $5.5\sqrt{f'_c}$

8.15.2.1.2 Shear

For detailed summary of allowable shear stress, v_c, see Article 8.15.5.2.

8.15.2.1.3 Bearing Stress

The bearing stress, f_b, on loaded area shall not exceed $0.30 f'_c$.

When the supporting surface is wider on all sides than the loaded area, the allowable bearing stress on the loaded area may be multiplied by $\sqrt{A_2/A_1}$, but not by more than 2.

When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustrum of the right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

When the loaded area is subjected to high-edge stresses due to deflection or eccentric loading, the allowable bearing stress on the loaded area, including any increase due to the supporting surface being larger than the loaded area, shall be multiplied by a factor of 0.75.

8.15.2.2 Reinforcement

The tensile stress in the reinforcement, f_s, shall not exceed the following:

Grade 40 reinforcement .... 20,000 psi
Grade 60 reinforcement .... 24,000 psi

In straight reinforcement, the range between the maximum tensile stress and the minimum stress caused by live load plus impact shall not exceed the value given in Article 8.16.8.3. Bends in primary reinforcement shall be avoided in regions of high-stress range.

8.15.3 Flexure

8.15.3.1 For the investigation of stresses at service loads, the straight-line theory of stress and strain in flexure shall be used with the following assumptions.

8.15.3.2 The strain in reinforcement and concrete is directly proportional to the distance from the neutral axis, except that for deep flexural members with overall depth to span ratios greater than ⅖ for continuous spans and ⅘ for simple spans, a nonlinear distribution of strain shall be considered.

8.15.3.3 In reinforced concrete members, concrete resists no tension.

8.15.3.4 The modular ratio, $n = E_s/E_c$, may be taken as the nearest whole number (but not less than 6). Except in calculations for deflections, the value of n for lightweight concrete shall be assumed to be the same as for normal weight concrete of the same strength.

8.15.3.5 In doubly reinforced flexural members, an effective modular ratio of $2E_s/E_c$ shall be used to transform the compression reinforcement for stress computations. The compressive stress in such reinforcement shall not be greater than the allowable tensile stress.

8.15.4 Compression Members

The combined flexural and axial load capacity of compression members shall be taken as 35% of that computed in accordance with the provisions of Article 8.16.4. Slenderness effects shall be included according to the requirements of Article 8.16.5. The term P_u in Equation (8-41) shall be replaced by 2.5 times the design axial load. In using the provisions of Articles 8.16.4 and 8.16.5, ϕ shall be taken as 1.0.

8.15.5 Shear

8.15.5.1 Shear Stress

8.15.5.1.1 Design shear stress, v, shall be computed by:

$$v = \frac{V}{b_w d} \qquad (8\text{-}3)$$

where V is design shear force at section considered, b_w is the width of web, and d is the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement. Whenever applicable, effects of torsion* shall be included.

8.15.5.1.2 For a circular section, b_w shall be the diameter and d need not be less than the distance from the extreme compression fiber to the centroid of the longitudinal reinforcement in the opposite half of the member.

8.15.5.1.3 For tapered webs, b_w shall be the average width or 1.2 times the minimum width, whichever is smaller.

8.15.5.1.4 When the reaction, in the direction of the applied shear, introduces compression into the end regions of a member, sections located less than a distance d from the face of support may be designed for the same shear, V, as that computed at a distance d. An exception occurs when major concentrated loads are imposed between that point and the face of support. In that case sections closer than d to the support shall be designed for V at distance d plus the major concentrated loads.

8.15.5.2 Shear Stress Carried by Concrete

8.15.5.2.1 Shear in Beams and One-Way Slabs and Footings

For members subject to shear and flexure only, the allowable shear stress carried by the concrete, v_c, may be taken as $0.95\sqrt{f_c'}$. A more detailed calculation of the allowable shear stress can be made using:

$$v_c = 0.9\sqrt{f_c'} + 1{,}100\rho_w\left(\frac{Vd}{M}\right) \le 1.6\sqrt{f_c'} \qquad (8\text{-}4)$$

Note:

(a) M is the design moment occurring simultaneously with V at the section being considered.
(b) The quantity Vd/M shall not be taken greater than 1.0.

8.15.5.2.2 Shear in Compression Members

For members subject to axial compression, the allowable shear stress carried by the concrete, v_c, may be taken as $0.95\sqrt{f_c'}$. A more detailed calculation can be made using:

$$v_c = 0.9\left(1 + 0.0006\frac{N}{A_g}\right)\sqrt{f_c'} \qquad (8\text{-}5)$$

The quantity N/A_g shall be expressed in pounds per square inch.

8.15.5.2.3 Shear in Tension Members

For members subject to axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using

$$v_c = 0.9\left(1 + 0.004\frac{N}{A_g}\right)\sqrt{f_c'} \qquad (8\text{-}6)$$

Note:

(a) N is negative for tension.
(b) The quantity N/A_g shall be expressed in pounds per square inch.

8.15.5.2.4 Shear in Lightweight Concrete

The provisions for shear stress, v_c, carried by the concrete apply to normal weight concrete. When lightweight aggregate concretes are used, one of the following modifications shall apply:

(a) When f_{ct} is specified, the shear stress, v_c, shall be modified by substituting $f_{ct}/6.7$ for $\sqrt{f_c'}$, but the value of $f_{ct}/6.7$ used shall not exceed $\sqrt{f_c'}$.
(b) When f_{ct} is not specified, the shear stress, v_c, shall be multiplied by 0.75 for "all-lightweight" concrete, and

*The design criteria for combined torsion and shear given in "Building Code Requirements for Reinforced Concrete"—American Concrete Institute 318 Bulletin may be used.

0.85 for "sand-lightweight" concrete. Linear interpolation may be used when partial sand replacement is used.

8.15.5.3 Shear Stress Carried by Shear Reinforcement

8.15.5.3.1 Where design shear stress v exceeds shear stress carried by concrete, v_c, shear reinforcement shall be provided in accordance with this article. Shear reinforcement shall also conform to the general requirements of Article 8.19.

8.15.5.3.2 When shear reinforcement perpendicular to the axis of the member is used:

$$A_v = \frac{(v - v_c)b_w s}{f_s} \qquad (8\text{-}7)$$

8.15.5.3.3 When inclined stirrups are used:

$$A_v = \frac{(v - v_c)b_w s}{f_s(\sin\alpha + \cos\alpha)} \qquad (8\text{-}8)$$

8.15.5.3.4 When shear reinforcement consists of a single bar or a single group of parallel bars all bent up at the same distance from the support:

$$A_v = \frac{(v - v_c)b_w d}{f_s \sin\alpha} \qquad (8\text{-}9)$$

where $(v - v_c)$ shall not exceed $1.5\sqrt{f_c'}$.

8.15.5.3.5 When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, the required area shall be computed by Equation (8-8).

8.15.5.3.6 Only the center three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

8.15.5.3.7 Where more than one type of shear reinforcement is used to reinforce the same portion of the member, the required area shall be computed as the sum of the values computed for the various types separately. In such computations, v_c shall be included only once.

8.15.5.3.8 When $(v - v_c)$ exceeds $2\sqrt{f_c'}$ the maximum spacings given in Article 8.19 shall be reduced by one-half.

8.15.5.3.9 The value of $(v - v_c)$ shall not exceed $4\sqrt{f_c'}$.

8.15.5.3.10 When flexural reinforcement located within the width of a member used to compute the shear strength is terminated in a tension zone, shear reinforcement shall be provided in accordance with Article 8.24.1.4.

8.15.5.4 Shear Friction

8.15.5.4.1 Provisions for shear-friction are to be applied where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times.

8.15.5.4.2 A crack shall be assumed to occur along the shear plane considered. Required area of shear-friction reinforcement A_{vf} across the shear plane may be designed using either Article 8.15.5.4.3 or any other shear transfer design method that results in prediction of strength in substantial agreement with results of comprehensive tests. Provisions of Articles 8.15.5.4.4 through 8.15.5.4.8 shall apply for all calculations of shear transfer strength.

8.15.5.4.3 Shear-friction Design Method

(a) When shear-friction reinforcement is perpendicular to the shear plane, area of shear-friction reinforcement A_{vf} shall be computed by:

$$A_{vf} = \frac{V}{f_s\mu} \qquad (8\text{-}10)$$

where μ is the coefficient of friction in accordance with Article 8.15.5.4.3(c).

(b) When shear-friction reinforcement is inclined to the shear plane such that the shear force produces tension in shear-friction reinforcement, the area of shear-friction reinforcement A_{vf} shall be computed by:

$$A_{vf} = \frac{V}{f_s(\mu\sin\alpha_f + \cos\alpha_f)} \qquad (8\text{-}11)$$

where α_f is the angle between the shear-friction reinforcement and the shear plane.

(c) Coefficient of friction μ in Equations (8-10) and (8-11) shall be:

concrete placed monolithically 1.4λ

concrete placed against hardened concrete with surface intentionally roughened as specified in Article 8.15.5.4.7 . . . . 1.0λ

concrete placed against hardened concrete not intentionally roughened 0.6λ
concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see Article 8.15.5.4.8 . . . . 0.7λ

where λ = 1.0 for normal weight concrete; 0.85 for "sand-lightweight" concrete; and 0.75 for "all lightweight" concrete. Linear interpolation may be applied when partial sand replacement is used.

8.15.5.4.4 Shear stress v shall not exceed $0.09f'_c$ nor 360 psi.

8.15.5.4.5 Net tension across the shear plane shall be resisted by additional reinforcement. Permanent net compression across the shear plane may be taken as additive to the force in the shear-friction reinforcement $A_{vf}f_y$, when calculating required A_{vf}.

8.15.5.4.6 Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop the specified yield strength on both sides by embedment, hooks, or welding to special devices.

8.15.5.4.7 For the purpose of Article 8.15.5.4, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If μ is assumed equal to 1.0λ, the interface shall be roughened to a full amplitude of approximately ¼ inch.

8.15.5.4.8 When shear is transferred between steel beams or girders and concrete using headed studs or welded reinforcing bars, steel shall be clean and free of paint.

8.15.5.5 Horizontal Shear Design for Composite Concrete Flexural Members

8.15.5.5.1 In a composite member, full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements.

8.15.5.5.2 Design of cross sections subject to horizontal shear may be in accordance with provisions of Articles 8.15.5.5.3 or 8.15.5.5.4 or any other shear transfer design method that results in prediction of strength in substantial agreement with results of comprehensive tests.

8.15.5.5.3 Design horizontal shear stress v_{dh} at any cross section may be computed by:

$$v_{dh} = \frac{V}{b_v d} \qquad (8\text{-}11A)$$

where V is the design shear force at the section considered and d is for the entire composite section. Horizontal shear v_{dh} shall not exceed permissible horizontal shear v_h in accordance with the following:

(a) When the contact surface is clean, free of laitance, and intentionally roughened, shear stress v_h shall not exceed 36 psi.
(b) When minimum ties are provided in accordance with Article 8.15.5.5.5, and the contact surface is clean and free of laitance, but not intentionally roughened, shear stress v_h shall not exceed 36 psi.
(c) When minimum ties are provided in accordance with Article 8.15.5.5.5, and the contact surface is clean, free of laitance, and intentionally roughened to a full magnitude of approximately ¼ inch, shear stress v_h shall not exceed 160 psi.
(d) For each percent of tie reinforcement crossing the contact surface in excess of the minimum required by Article 8.15.5.5.5, permissible v_h may be increased by $72f_y/40{,}000$ psi.

8.15.5.5.4 Horizontal shear may be investigated by computing, in any segment not exceeding one-tenth of the span, the actual change in compressive or tensile force to be transferred, and provisions made to transfer that force as horizontal shear between interconnected elements. Horizontal shear shall not exceed the permissible horizontal shear stress v_h in accordance with Article 8.15.5.5.3.

8.15.5.5.5 Ties for Horizontal Shear

(a) When required, a minimum area of tie reinforcement shall be provided between interconnected elements. Tie area shall not be less than $50b_v s/f_y$, and tie spacing s shall not exceed four times the least web width of support element, nor 24 inch.
(b) Ties for horizontal shear may consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric (smooth or deformed). All ties shall be adequately anchored into interconnected elements by embedment or hooks.

8.15.5.6 Special Provisions for Slabs and Footings

8.15.5.6.1 Shear capacity of slabs and footings in the vicinity of concentrated loads or reactions shall be governed by the more severe of two conditions:

(a) Beam action for the slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from the face of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 8.15.5.1 through 8.15.5.3, except at footings supported on piles, the shear on the critical section shall be determined in accordance with Article 4.4.11.3.
(b) Two-way action for the slab or footing, with a critical section perpendicular to the plane of the member and located so that its perimeter b_o is a minimum, but not closer than d/2 to the perimeter of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 8.15.5.6.2 and 8.15.5.6.3.

8.15.5.6.2 Design shear stress, v, shall be computed by:

$$v = \frac{V}{b_o d} \qquad (8\text{-}12)$$

where V and b_o shall be taken at the critical section defined in Article 8.15.5.6.1(b).

8.15.5.6.3 Design shear stress, v, shall not exceed v_c given by Equation (8-13) unless shear reinforcement is provided in accordance with Article 8.15.5.6.4.

$$v_c = \left(0.8 + \frac{2}{\beta_c}\right)\sqrt{f'_c} \le 1.8\sqrt{f'_c} \qquad (8\text{-}13)$$

β_c is the ratio of long side to short side of concentrated load or reaction area.

8.15.5.6.4 Shear reinforcement consisting of bars or wires may be used in slabs and footings in accordance with the following provisions:

(a) Shear stresses computed by Equation (8-12) shall be investigated at the critical section defined in Article 8.15.5.6.1(b) and at successive sections more distant from the support.
(b) Shear stress v_c at any section shall not exceed 0.9 $\sqrt{f'_c}$ and v shall not exceed $3\sqrt{f'_c}$.
(c) Where v exceeds 0.9 $\sqrt{f'_c}$, shear reinforcement shall be provided in accordance with Article 8.15.5.3.

8.15.5.7 Special Provisions for Slabs of Box Culverts

For slabs of box culverts under 2 feet or more fill, shear stress v_c may be computed by:

$$v_c = \sqrt{f'_c} + 2{,}200\rho\left(\frac{Vd}{M}\right) \qquad (8\text{-}14)$$

but v_c shall not exceed $1.8\sqrt{f'_c}$. For single cell box culverts only, v_c for slabs monolithic with walls need not be taken less than $1.4\sqrt{f'_c}$, and v_c for slabs simply supported need not be taken less than $1.2\sqrt{f'_c}$. The quantity Vd/M shall not be taken greater than 1.0 where M is the moment occurring simultaneously with V at the section considered. For slabs of box culverts under less than 2 feet of fill, applicable provisions of Articles 3.24 and 6.4 should be used.

8.15.5.8 Special Provisions for Brackets and Corbels*

8.15.5.8.1 Provisions of Article 8.15.5.8 shall apply to brackets and corbels with a shear span-to-depth ratio a_v/d not greater than unity, and subject to a horizontal tensile force N_c not larger than V. Distance d shall be measured at the face of support.

8.15.5.8.2 Depth at outside edge of bearing area shall not be less than 0.5d.

8.15.5.8.3 The section at the face of support shall be designed to resist simultaneously a shear V, a moment $[Va_v + N_c(h - d)]$, and a horizontal tensile force N_c. Distance h shall be measured at the face of support.

(a) Design of shear-friction reinforcement, A_{vf}, to resist shear, V, shall be in accordance with Article 8.15.5.4. For normal weight concrete, shear stress v shall not exceed $0.09f'_c$ nor 360 psi. For "all lightweight" or "sand-lightweight" concrete, shear stress v shall not exceed $(0.09-0.03a_v/d)f'_c$ nor $(360-126a_v/d)$ psi.
(b) Reinforcement A_f to resist moment $[Va_v + N_c(h - d)]$ shall be computed in accordance with Articles 8.15.2 and 8.15.3.
(c) Reinforcement A_n to resist tensile force N_c shall be computed by $A_n = N_c/f_s$. Tensile force N_c shall not be taken less than 0.2V unless special provisions are made to avoid tensile forces.
(d) Area of primary tension reinforcement, A_s, shall be made equal to the greater of (A_f+A_n), or $(2A_{vf}/3+A_n)$.

8.15.5.8.4 Closed stirrups or ties parallel to A_s, with a total area A_h not less than $0.5(A_s-A_n)$, shall be uni-

*These provisions do not apply to beam ledges. The PCA publication, "Notes on ACI 318–83," contains an example design of beam ledges—Part 16, example 16-3.

formly distributed within two-thirds of the effective depth adjacent to A_s.

8.15.5.8.5 Ratio $\rho = A_s/bd$ shall not be taken less than $0.04(f_c'/f_y)$.

8.15.5.8.6 At the front face of a bracket or corbel, primary tension reinforcement, A_s, shall be anchored by one of the following:

(a) a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength f_y of A_s bars;
(b) bending primary tension bars A_s back to form a horizontal loop; or
(c) some other means of positive anchorage.

8.15.5.8.7 Bearing area of load on a bracket or corbel shall not project beyond the straight portion of primary tension bars A_s, nor project beyond the interior face of a transverse anchor bar (if one is provided).

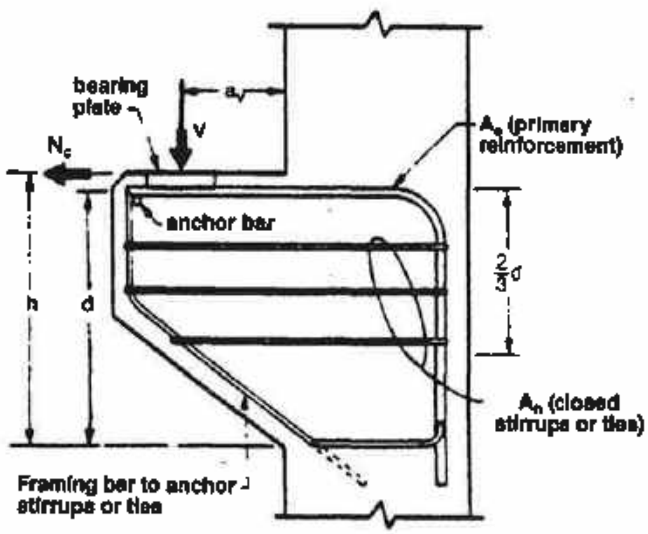


FIGURE 8.15.5.8

8.16 STRENGTH DESIGN METHOD (LOAD FACTOR DESIGN)

8.16.1 Strength Requirements

8.16.1.1 Required Strength

The required strength of a section is the strength necessary to resist the factored loads and forces applied to the structure in the combinations stipulated in Article 3.22. All sections of structures and structural members shall have design strengths at least equal to the required strength.

8.16.1.2 Design Strength

8.16.1.2.1 The design strength provided by a member or cross section in terms of load, moment, shear, or stress shall be the nominal strength calculated in accordance with the requirements and assumptions of the strength-design method, multiplied by a strength-reduction factor ϕ.*

8.16.1.2.2 The strength-reduction factors, ϕ, shall be as follows:

(a) Flexure . . . . $\phi = 0.90$
(b) Shear . . . . $\phi = 0.85$
(c) Axial compression with—
Spirals . . . . $\phi = 0.75$
Ties . . . . $\phi = 0.70$
(d) Bearing on concrete . . . . $\phi = 0.70$

The value of ϕ may be increased linearly from the value for compression members to the value for flexure as the design axial load strength, ϕP_n, decreases from $0.10f_c' A_g$ or ϕP_b, whichever is smaller, to zero.

8.16.1.2.3 The development and splice lengths of reinforcement specified in Articles 8.24 through 8.32 do not require a strength-reduction factor.

8.16.2 Design Assumptions

8.16.2.1 The strength design of members for flexure and axial loads shall be based on the assumptions given in this article, and on the satisfaction of the applicable conditions of equilibrium of internal stresses and compatibility of strains.

8.16.2.2 The strain in reinforcement and concrete is directly proportional to the distance from the neutral axis.

8.16.2.3 The maximum usable strain at the extreme concrete compression fiber is equal to 0.003.

*The coefficient ϕ provides for the possibility that small adverse variations in material strengths, workmanship, and dimensions, while individually within acceptable tolerances and limits of good practice, may combine to result in understrength.

TranSystems

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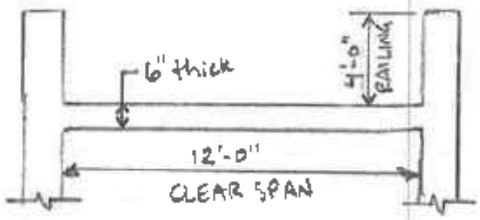
LAKE PARK ARCH BRIDGE – LOAD RATING

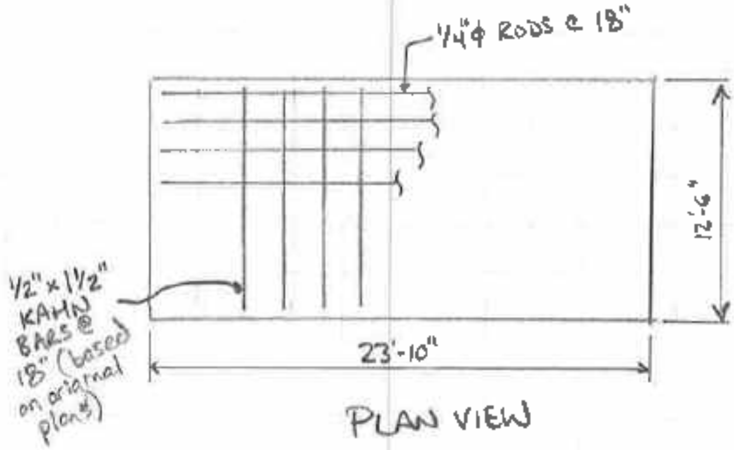
DECK

Deck consists of 6" thick reinforced concrete supported on 4 sides, reinforced both directions with bottom reinforcement only.
∴ Assume simple supports at edges, but check if two-way bending is applicable.

Dead Load: 6" thick concrete self weight
Live Load: 80 psf



CROSS SECTION



PLAN VIEW

Span length is the distance center to center of supports but shall not exceed clear span plus slab thickness [AASHTO 3.24.1]

$$L_{trans} = 12'\text{-}0" + 6" = 12'\text{-}6"$$
$$L_{long} = 24'\text{-}0" - \underbrace{8"}_{℄ wall} + 6" = 23'\text{-}10"$$

Based on unit strip, calculate moments for transverse spanning deck.

$$q_{DL} = (0.15\ kcf)(6/12)(12/12) = 0.075\ k/ft \Rightarrow M_{DL} = \frac{q_{DL}L^2}{8} = \frac{(0.075)(12.5)^2}{8} = 1.46\ k\text{-}ft$$
$$q_{LL} = (80\ psf)(1ft) \times 1/1000 = 0.080\ k/ft \Rightarrow M_{LL} = \frac{(0.080)(12.5)^2}{8} = 1.56\ k\text{-}ft$$

Also check basic finite element model for two-way bending to establish more accurate moments. Use shell elements over a 23'-10" by 12'-6" area, 6" thick, and apply pressure loading. Pinned supports along perimeter for simply supported two-way slab.

Calculations for 1/4" US Grid

LAKE PARK ARCH BRIDGE – LOAD RATING

Based on STAAD output for two-way slab model, transverse bending moment results as follows:

$M_{DL} = 1.142$ k-ft/ft $\qquad M_{LL} = 1.218$ k-ft/ft

Update: Do NOT consider 2-way bending due to very low longitudinal moment capacity

From original design plans:
Transverse reinforcement = 1/2" × 1 1/2 Kahn bars @ 18" ($A_s = 0.38$ in²/bar)
Longitudinal reinforcement = 1/4" ϕ rods at 18" ($A_s = 0.05$ in²/bar)

Transverse Moment Capacity (Per plans)
Area of steel $A_s = (0.38\text{in}^2)(12")/18" = 0.253$ in²/ft
Assume 1" clear cover $\longrightarrow$ $d = 6" - 1"_{CLR} - \frac{0.707"}{2} = 4.65"$
For concrete strength, $f'_c \le 4$ ksi, $\beta_1 = 0.85$

$$a = \frac{A_s f_y}{0.85 f_c b} = \frac{(0.253)(16)}{0.85(.640)(12)} = 0.620"$$

$$M_{all} = A_s f_y (d - a/2) = (0.253)(16)\left(4.65" - \frac{0.620"}{2}\right) = 17.56 \text{ k-in} = 1.46 \text{ k-ft}$$

Capacity/Demand Ratio: $C/D = \frac{M_{all}}{M_{DL}+M_{LL}} = \frac{1.46}{1.46+1.56} = 0.48$ based on original design plans

Transverse Moment Capacity (based on Field Investigation)

Based on photos of exposed reinforcement on the deck underside, transverse reinforcement is clearly not spaced at 18." From scaling multiple photos, reinforcement seems to vary from 6" to 7 1/2". To be conservative, use reinforcement spacing of 7."

Bars appear to be ~1 1/2" wide $\Longrightarrow$ ∴ Assume 1/2" × 1 1/2" Kahn bars from plans

Area of steel: $A_s = (0.38\text{in}^2/\text{ft})(12")/7" = 0.65$ in²/ft

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LAKE PARK ARCH BRIDGE - LOAD RATING

$$a = \frac{A_s f_y}{0.85 f_c b} = \frac{(0.65)(16)}{0.85(0.640)(12)} = 1.54''$$

$$M_{all} = A_s f_y (d - a/2) = (0.65)(16)\left(4.65'' - \frac{1.54''}{2}\right) = 40.07 \text{ k-in} = 3.339 \text{ k-ft}$$

Because transverse steel provided is much greater than longitudinal reinforcing steel, assume deck is simply supported transversely as implied by original design plans. ∴ Use loads from transverse unit strip:

$$M_{DL} = 1.46 \text{ k-ft} \qquad M_{LL} = 1.56 \text{ k-ft}$$

$$\therefore \text{Capacity / Demand} = \frac{M_{all}}{M_{DL} + M_{LL}} = \frac{3.339}{1.46 + 1.56} = \underline{1.10} \quad \text{[As-Built]}$$

AS-CONFIGURED

For bridge in AS-CONFIGURED condition, add the dead load due to 1" concrete wearing surface. Additional railing load does not apply since it is not transferred through deck:

$$g_{DL} = (0.15 \text{ kcf})\left(\frac{6''+1''}{12}\right)(12''/12) = 0.0875 \text{ k/ft}$$

$$M_{DL} = gL^2/8 = (0.0875)\left(\frac{12.5^2}{8}\right) = 1.71 \text{ k-ft}$$

$$M_{TOT} = M_{DL} + M_{LL} = 1.71 \text{ k-ft} + 1.56 \text{ k-ft} = 3.27 \text{ k-ft}$$

$$C/D = \frac{M_{all}}{M_{TOT}} = \frac{3.339}{3.27} = \underline{1.02} \quad \text{[As-Configured]}$$

AS-INSPECTED

Use higher f'_c = 2000 psi based on concrete testing with As-Configured loads. No significant section loss is noted on rebar in governing areas, therefore use as-built properties for steel.

$$a = \frac{A_s f_y}{0.85 f_c b} = \frac{(0.65)(16)}{0.85(0.80)(12)} = 1.275'' \quad \text{, where } f_c = 0.4 f'_c$$

$$M_{all} = A_s f_y (d - a/2) = (0.65)(16)\left(4.65 - \frac{1.275}{2}\right) = 41.73 \text{ k-in} = 3.478 \text{ k-ft}$$

$$\therefore C/D = \frac{M_{all}}{M_{TOT}} = \frac{3.478}{3.27} = \underline{1.06} \quad \text{[As-Inspected]}$$

LAKE PARK ARCH BRIDGE – LOAD RATING

SHEAR CAPACITY

Also check shear capacity on concrete deck to determine if it may govern rating. Examine As-Built condition first:

$$\text{MAX SHEAR} = \frac{\gamma L}{2} = (0.075\ \text{k/ft} + 0.08\ \text{k/ft}) \times \frac{12.5'}{2} = 0.969\ \text{kips}$$

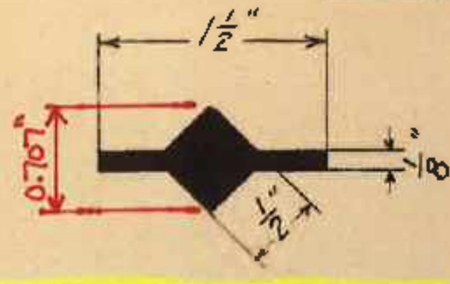
$$\text{MAX SHEAR STRESS} = v = \frac{V}{b_w d} = \frac{(0.969\ \text{k})(1000)}{(12'')(4.65'')} = 17.37\ \text{psi}$$

First check allowable shear stress based on concrete only:

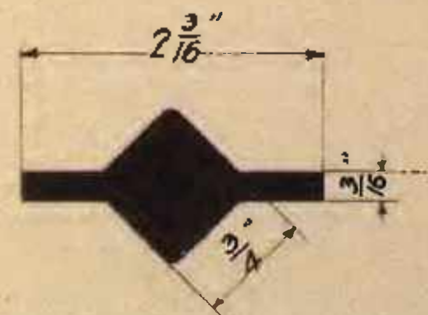
$$v_c = 0.95\sqrt{f'_c} = 0.95\sqrt{1600\ \text{psi}} = 38.0\ \text{psi} \gg 17.37\ \text{psi}$$

∴ Capacity/Demand ratio is greater than 2 without considering contribution from inclined stirrups

⇒ **SHEAR WILL NOT GOVERN DECK ANALYSIS**

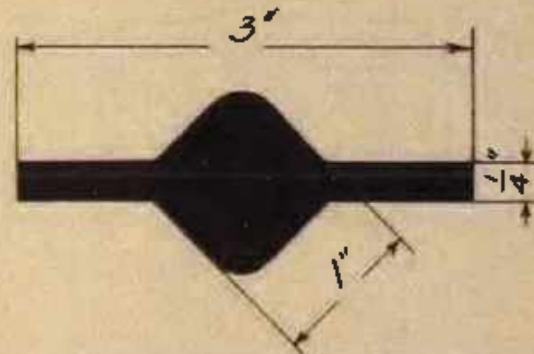


$1\frac{1}{2}'' \times \frac{1}{2}''$. Area .38 □" Weight 1.4# per ft.



$2\frac{3}{16}'' \times \frac{3}{4}''$. Area .78 □" Weight 2.7# per ft.

These bars can have any standard size cuts as shown in Figure 14 and will be sent in lengths as ordered. In making calculations for strength of reinforced beam, assume the area of the entire cross section as here given.



$3'' \times 1''$. Area 1.42 □" Weight 4.8# per ft.

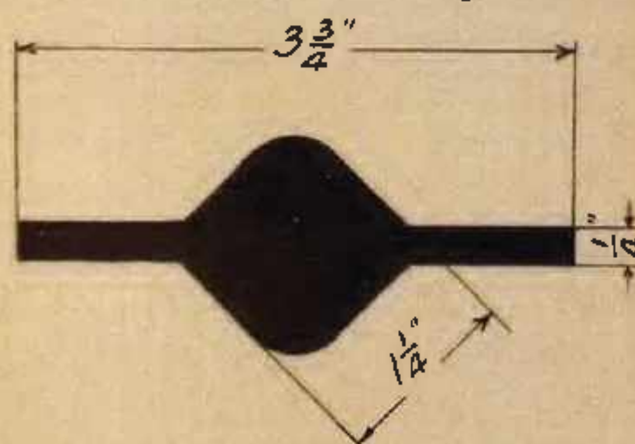


FIG. 13. $3\frac{3}{4}'' \times 1\frac{1}{4}''$. Area 2.0 □" Weight 6.9# per ft.

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 12-Jul-18
END JOB INFORMATION
INPUT WIDTH 79
UNIT FEET KIP
JOINT COORDINATES
1 0 0 0; 2 23.8333 0 0; 3 23.8333 0 12.5; 4 0 0 12.5; 5 0.993054 0 0;
6 0.993054 0 1.04167; 7 0 0 1.04167; 8 1.98611 0 0; 9 1.98611 0 1.04167;
10 2.97916 0 0; 11 2.97916 0 1.04167; 12 3.97222 0 0; 13 3.97222 0 1.04167;
14 4.96527 0 0; 15 4.96527 0 1.04167; 16 5.95833 0 0; 17 5.95833 0 1.04167;
18 6.95138 0 0; 19 6.95138 0 1.04167; 20 7.94443 0 0; 21 7.94443 0 1.04167;
22 8.93749 0 0; 23 8.93749 0 1.04167; 24 9.93054 0 0; 25 9.93054 0 1.04167;
26 10.9236 0 0; 27 10.9236 0 1.04167; 28 11.9167 0 0; 29 11.9167 0 1.04167;
30 12.9097 0 0; 31 12.9097 0 1.04167; 32 13.9028 0 0; 33 13.9028 0 1.04167;
34 14.8958 0 0; 35 14.8958 0 1.04167; 36 15.8889 0 0; 37 15.8889 0 1.04167;
38 16.8819 0 0; 39 16.8819 0 1.04167; 40 17.875 0 0; 41 17.875 0 1.04167;
42 18.868 0 0; 43 18.868 0 1.04167; 44 19.8611 0 0; 45 19.8611 0 1.04167;
46 20.8541 0 0; 47 20.8541 0 1.04167; 48 21.8472 0 0; 49 21.8472 0 1.04167;
50 22.8402 0 0; 51 22.8402 0 1.04167; 52 23.8333 0 1.04167;
53 0.993054 0 2.08333; 54 0 0 2.08333; 55 1.98611 0 2.08333;
56 2.97916 0 2.08333; 57 3.97222 0 2.08333; 58 4.96527 0 2.08333;
59 5.95833 0 2.08333; 60 6.95138 0 2.08333; 61 7.94443 0 2.08333;
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68 14.8958 0 2.08333; 69 15.8889 0 2.08333; 70 16.8819 0 2.08333;
71 17.875 0 2.08333; 72 18.868 0 2.08333; 73 19.8611 0 2.08333;
74 20.8541 0 2.08333; 75 21.8472 0 2.08333; 76 22.8402 0 2.08333;
77 23.8333 0 2.08333; 78 0.993054 0 3.125; 79 0 0 3.125; 80 1.98611 0 3.125;
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85 6.95138 0 3.125; 86 7.94443 0 3.125; 87 8.93749 0 3.125; 88 9.93054 0 3.125;
89 10.9236 0 3.125; 90 11.9167 0 3.125; 91 12.9097 0 3.125; 92 13.9028 0 3.125;
93 14.8958 0 3.125; 94 15.8889 0 3.125; 95 16.8819 0 3.125; 96 17.875 0 3.125;
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PERFORM ANALYSIS PRINT LOAD DATA
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KAHN SYSTEM of REINFORCED CONCRETE

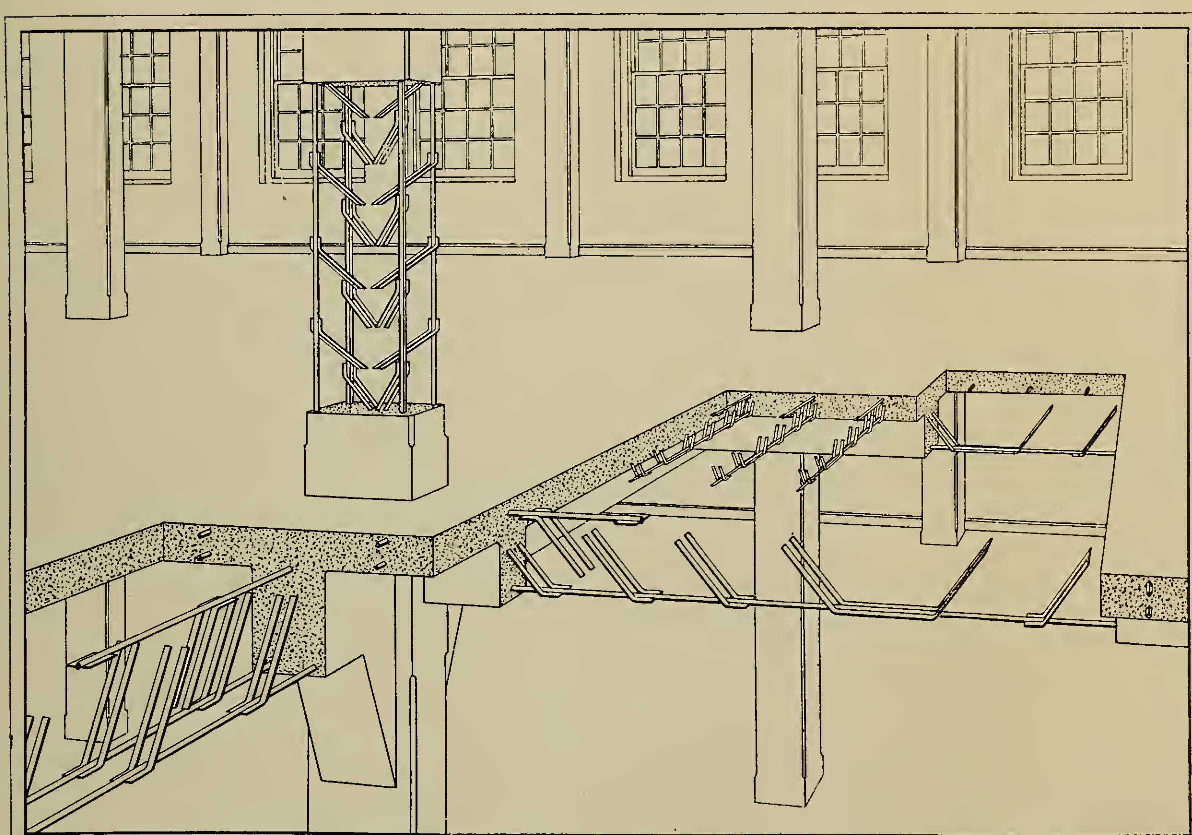
PATENTED

General Catalogue D

The Trussed Con-
crete-Steel Co.
Union Trust Bldg. Detroit.

Truscon steel company

Kahn System of Reinforced Concrete



Perspective of general adaptation.

[redacted]

Trussed Concrete Steel Co.,
Union Trust Building
Detroit, = Michigan.

HOME OFFICE
UNION TRUST BUILDING,
DETROIT, MICHIGAN.

Representatives:

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ERIE, PA.

STEEL WORKS AT DETROIT AND PITTSBURG.
TILE WORKS AT AKRON, OHIO.

The Kahn Trussed Bar.

NOTE.—This handbook is revised in accordance with the most recent practice of the Trussed Concrete Steel Co., and should be given preference to all previous issues.

Reinforced Concrete Bridge designed in accordance with "Kahn System" of Reinforced Concrete.

Kahn System of Reinforced Concrete

So much actual work is being done at the present time with reinforced concrete, and in general, the subject is receiving such intense interest by those taking part in buildings, bridges, or other constructions, that the new method of steel reinforcement herein described, it is believed, will be of interest.

The advantages of reinforced concrete above steel, masonry, or wood, are so well known, that it is hardly necessary to enter into comparison here. Reinforced concrete is absolutely free of any of the serious objections which exist in the use of these other materials. It is fire proof, and rust proof, but what is most advantageous about this type of construction, is the fact that its strength continually increases with age.

Reinforced concrete lends itself admirably to the construction of walls, columns, floors, roofs, and all parts of buildings; to bridges, arches, culverts, abutments, retaining walls, tunnels, foundations, railway ties, and in general, it replaces, to advantage, all masonry or steel construction.

The Kahn trussed bar consists of a half truss, struck up directly from a single rolled section, and provides the tensional members only. Concrete within itself is an excellent material to take up compressive strains, but is comparatively weak for resisting tensile strains. The Kahn bar, when imbedded in a mass of concrete, therefore, supplies strength to the latter where this is

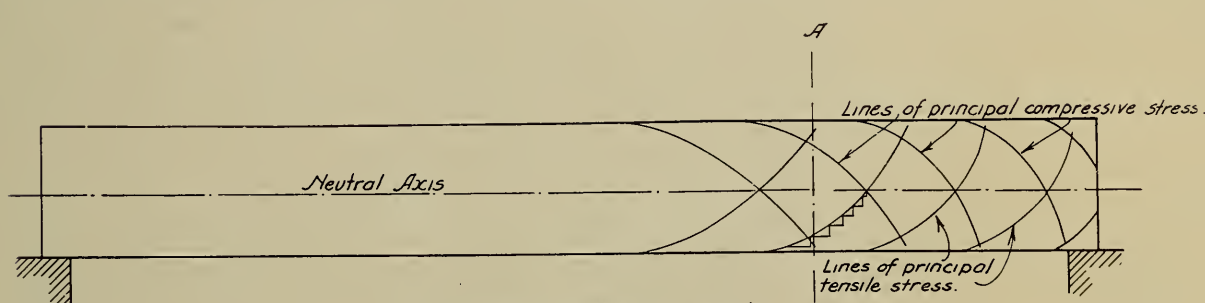


Fig. 1. Showing lines of principal stress in a uniformly loaded beam supported at ends.

most necessary, and the combination of the two materials, forms a complete truss. The main virtue of this trussed bar lies in the fact that concrete is reinforced wherever it is deemed necessary, that the steel extends upwardly into the mass, as well as lying merely along its bottom edge. This, then, in short, is the essence of this new type of construction, and a further reading of this pamphlet will show the large number of its applications.

It is fairly well recognized among engineers, that vertical reinforcement for concrete beams is just as essential as the horizontal reinforcement, and in many cases to accomplish this purpose, the horizontal rods are surrounded by U shaped stirrups of band or twisted iron. It was noticed at first by European engineers that a concrete beam, when tested to destruction under uniform loading, invariably failed by shear at the ends, the lines of rupture corresponding closely to the lines of principal compressive stress for such a beam, as is shown in Figure 1. In this country engineers were apparently very slow to realize the importance of such vertical reinforcement. In fact, upon its strong recommendation by one of the U. S. Engineer Corps in a leading Engineering Journal, a number of engineers argued the matter strongly and pointed out tests which they had actually made, where apparently the break did not occur at the ends of the beam. Without one exception, however, these tests, when investigated, proved to be beams which had been loaded either unfairly, so as not to develop strains actually occurring in building practice, or they referred to beams so abnormally proportioned that they could not possibly be used.

The Trussed Concrete Steel Company has made a number of tests on beams reinforced with plain and deformed rods on the bottom, and without one exception, all such beams, when tested to destruction under uniform loading, failed suddenly by vertical cracking or shear through the concrete, or longitudinal shear along the end of the rod.

This matter of vertical reinforcement is certainly of more importance than some American Engineers have been willing to grant. It seems most natural

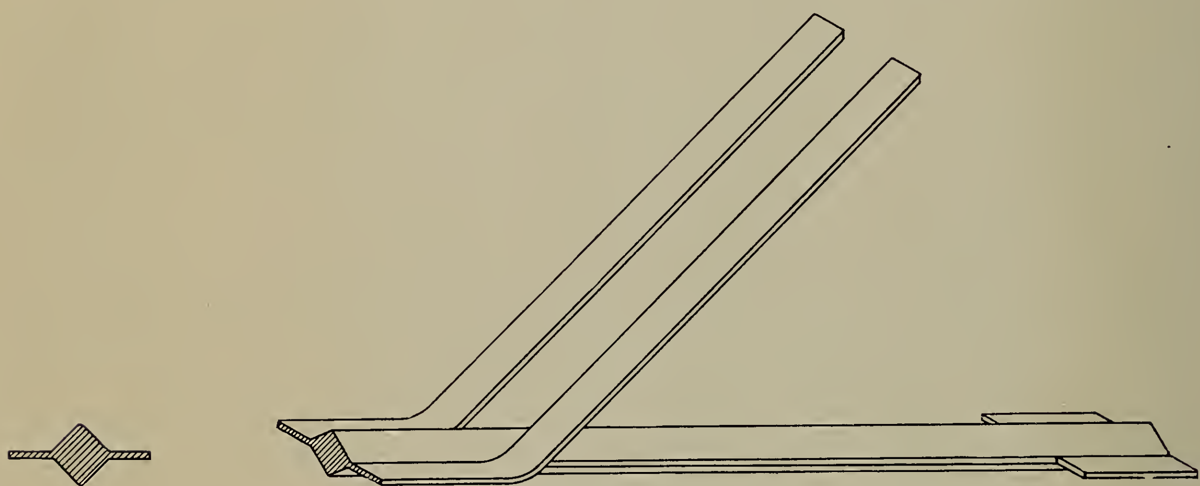


Fig. 2.

that rupture should occur in this manner. In fact, one can hardly conceive of its occurring in any other way. It must, of course, be remembered that a beam, when tested for both shear and bending moment, should be subjected to a uniformly distributed load, not to a concentrated center load; for, a beam loaded according to this latter method would only develop one-half the shear which exists in a uniformly loaded beam for a given bending moment.

Take, for example, a certain beam, as shown in Figure 1, and consider the cross section "AA."

The tension strain on each fibre below the neutral axis, varies in proportion to its distance therefrom. The vertical shearing is, however, practically constant. The resultant strain on any particle should therefore be a combination of these two components, producing a line of principal tensile stress, which is one of variable curvature from the bottom of the beam to the top.

If, then, lines of principal tensile stress exist throughout a beam, it is most natural that the concrete, being weak in tension, should open at right angles to these lines, and this is what has occurred in all the tests which the writer has observed in well proportioned concrete steel beams, when tested to destruction under uniform load, and where the metal reinforcement was horizontal only.

As has already been noted, European engineers endeavored to overcome the difficulty by placing stirrups throughout the beam, their distances apart varying, of course, in the inverse ratio of the shear. There seems no doubt whatever in the writer's mind that such stirrups accomplish a great deal of good, as they cross the lines of rupture at an angle, and tend to hold the material together. If, however, they are placed in a beam, they should be placed in a

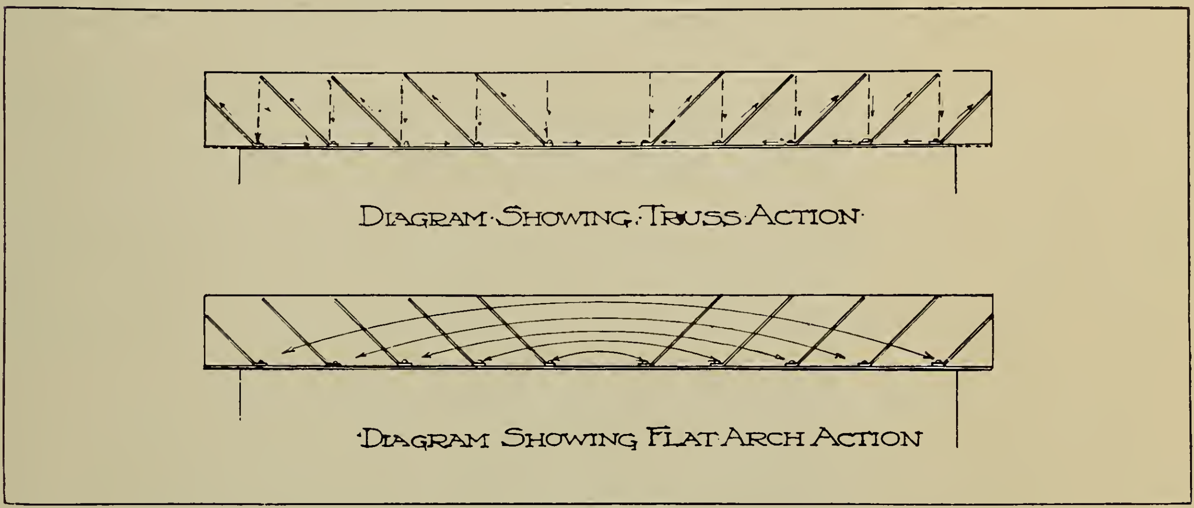


Fig. 3.

direction inclined to the horizontal, so as to lie more closely along the line of principal tensile stress, for if they lie in exactly this line, they also cut the actual line of rupture at right angles, and are therefore of maximum efficiency in holding together the concrete where its natural tendency is to open up. Furthermore, if such stirrups are to carry stress, they should carry it into some member capable of receiving it, and the bottom chord member or the horizontal reinforcement is there for that purpose. In the first place, then, it seems to the writer that stirrups should be inclined to the vertical and preferably bent to a curvature to approximate the line of principal tensile stress, and secondly, these stirrups should be rigidly connected to the main horizontal reinforcing bar.

There is still another matter in connection with the steel reinforcement for concrete beams, which is also of great importance, in so far that it affects economy in the use of steel. In a uniformly loaded beam, the maximum bending moment occurs at the center, whereas the maximum shear occurs at the ends, and if the same quantity of steel reinforcement is therefore placed along the bottom of the beam and extends the full length of it, it does seem to the writer that steel has been wasted so far as bending moment alone is concerned, and certainly the beam has been neglected so far as shear is concerned. A steel I beam in this manner is not an economical construction for uniform loading; its top and bottom flanges are only required at the center and at this place only a very thin web, whereas at the ends the stress is almost altogether shear, and web alone is required with very little of top and bottom flanges.

In the system of concrete reinforcement, which it is the purpose of this pamphlet to describe, these two matters have been carefully considered. The fundamental principles of this type of reinforcement are:

1st. Concrete should be reinforced in a vertical plane, as well as in a horizontal one.

2nd. The reinforcement should be inclined to the vertical, preferably with varying upward curvature, approximating the line of principal tensile stress.

3rd. The metal should be distributed in proportion to the strains existing at any place.

4th. The shear members should be rigidly connected to the horizontal reinforcement steel.

It has been endeavored to accomplish all of these results by taking a bar of cross section, as shown in Figure 2, and shearing upwards into an inclined position the web on both sides of the main body, thereby forming substantially the tension members of the ordinary Pratt Truss. When such a structural member is embedded within a body of concrete, the latter unites firmly to the steel, and the combination of the two forms a trussed beam wherein the tensional members are made up of steel, and the missing compression members supplanted by the concrete. Concrete is excellent in compression; steel, in tension; and, thanks to the property of strong adhesion between the two, in their combination is made a most ideal beam.

Neglecting for a moment the matter of vertical reinforcement, it is very evident that a bar sheared up as above described, can not possibly slip through the concrete. The writer has actually taken blocks of concrete, moulded to form the voussoirs of a flat arch, and then set them between the prongs. Such a beam, though set up without a particle of mortar between the joints, will carry a very heavy weight, and were it not for the large deflection which is caused by the poorly fitting surfaces between the prongs and blocks, such a beam would carry weights to the same extent and on the same principle as when steel and concrete are actually united together.

And this presents another way of looking at the reasons why this method of reinforcement is so efficient. As soon as a load is applied on top of the beam, the concrete tends to arch itself, and a series of internal arches immediately set themselves up within the material, each arch finding its abutment in a set of prongs for which the bottom chord acts as a tie. The prongs receive the weight and carry it upwards, distributing it on the other arches of larger span, the horizontal reinforcement serving as a common tensional member. It is plainly evident that with this construction the horizontal member might actually be placed entirely outside of the concrete, and the adhesion of the concrete to it entirely neglected, the strains coming into it being so largely the horizontal components of the inclined members. Of course, for fire proofing purposes, and to prevent rusting, it is more advisable to imbed the steel within the concrete, and when this is done, advantage may be taken of both the adhesion of the concrete to the main bar and to the sheared up members. In fact, with a given amount of concrete, a maximum amount of steel may be used, since the strains which it takes up are due to the direct adhesion of the concrete to it, plus the horizontal component of the inclined members. When such a beam fails, assuming that good material has been used for its construction, one of two things must happen,—either the steel pull in two, or the concrete crush on top. The top portion of a concrete beam when used in floor construction, is largely the floor itself, and it is generally impossible for this to fail in compression. It would seem, therefore, that a very large quantity of steel could be placed in the bottom of the beam to balance the compression. In fact, in all tests which the writer has made up to date, he has pulled the steel in two at the center of the beam.

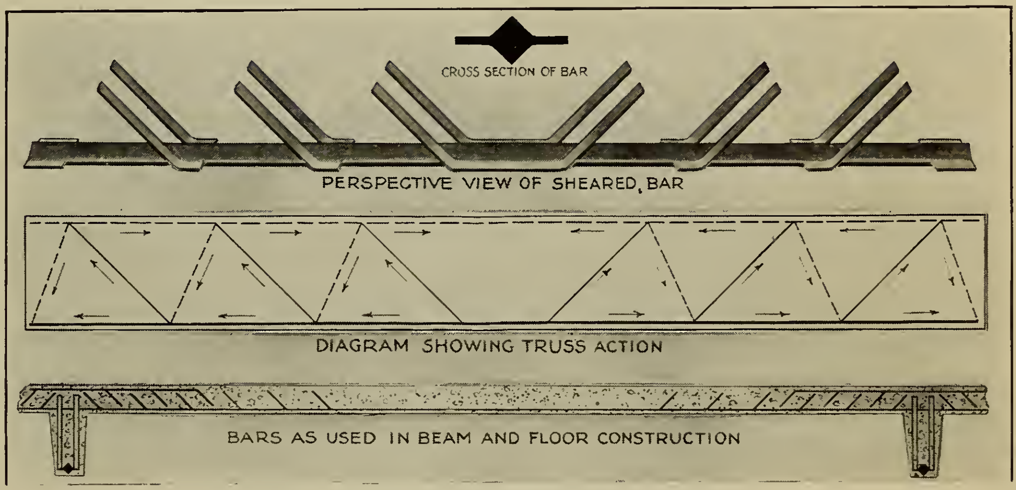


Fig. 3a.

Another point of great advantage of this construction is the fact that a beam need not necessarily be very wide to carry a given load; depth alone counts to advantage. The steel reinforcement, depending entirely upon the stresses coming into it from the sheared up members, may be one large bar. This is practically impossible with constructions wherein the stresses coming into the steel are due to adhesion only of the concrete to it. Where such adhesion is depended upon, a large body of concrete must surround the steel to be able to transmit all of the strain which the bar is capable of taking. Whatever strain exists in the steel must be transmitted into the upper portion of the concrete immediately surrounding it, and any one can readily perceive the enormity of the horizontal shear, which must therefore exist throughout the body of the concrete, and the necessity of giving this great width. With this new method of concrete reinforcement, however, the beam may be comparatively narrow; in fact, at the bottom it needs only to be sufficiently wide to encase the steel. It should taper upwards, however, widening towards the top, so that sufficient area may be given to the concrete to receive the compression. This, of course, makes a remarkable saving in the amount of this material used.

The strength of steel is, of course, a definitely determined matter. As for the concrete, it is not very expensive, and it would be advisable in all cases to give a small surplus of this material on the top of a beam, so that it will not fail by compression. With shear thus properly cared for, there is only one way in which the beam can possibly fail, and that is by the parting of the steel. Where this result can be assured with certainty, a concrete beam need no longer be subjected to a factor of safety of "ten": the ordinarily adopted factor of "four" is sufficient, as such a beam is entirely dependent upon the steel and should be subject to close calculation in the same manner as a steel I beam or truss. When a concrete beam fails by shear, as has occurred almost without exception in tests up to date, then indeed, the engineer stands more or less in mystery. In general it seems to the writer that whenever concrete is depended upon to carry other strains than direct compression. more or less risk is being assumed by the designing engineer, and a large factor of safety is strongly recommended.

Some photographs are submitted herewith of tests made on two reinforced concrete beams, of twenty-six feet span, center to center of supports, with a four-inch thick concrete slab five feet wide on top to receive the load. The concrete was made of Portland cement, sand, and crushed stone, proportioned one, two and five. Loading was done with pig iron. Deflections measured at the center. In one of the photographs, an outline is shown of the actual cross sections of the beams. The ends, it will be noted, are built up solid to give better bearing on the supporting timbers. The area of metal in the bottom of each beam was two square inches. No deflection whatever could be observed in the beams until the load had reached 48,000 pounds. When 84,000 pounds of pig iron had been loaded on the beams, making a total weight of 93,000 pounds thereon, the floor slab, weighing about 9,000 pounds, the actual deflection was five-eighths of an inch. It was evident that the elastic limit of the steel had been well exceeded by this time. With 101,100 pounds of pig iron, plus 9,300 pounds for weight of slab, making a total load of 110,400 pounds, the beam failed, breaking at the center, and pulling the steel in two at this point. Not a sign of a crack was to be seen throughout the beam at any other place than at the point of failure. This seems to the writer a very remarkable test. The absolute lack of even a hair-like crack throughout any portion of the beam, except at the place of failure, is clear evidence that shear was properly provided for.

As has already been explained, with this method of reinforcement, the adhesion of the concrete to the horizontal steel member is not essential; in fact, if the latter were placed entirely outside of the concrete, the beam would be very nearly as efficient, as the strain which comes into this lower chord is so largely the summation of the horizontal components of the inclined members.

This principle is utilized in the Kahn patented trussed lintel, drawings and photographs of which are presented herewith. In the old system of lintels, an I beam or built-up girder was figured on to carry the weight of the superimposed load and a 12x¼ inch or other similar plate was riveted to the bottom flanges of the beam to give bearing for the wall above, but the plate was counted upon as rendering little or no service in strengthening the lintel. In the new system this bearing plate not only supports the brick-work directly, but also acts as the bottom flange of a masonry beam, in which the masonry takes up the compression or thrust of a flat arch, while the steel plate takes up the tension. Diagonals, riveted to the base plate, form abutments for a series of arches of stress, which set themselves up within the masonry, and for these the base plate serves as the bottom chord or tie. Each diagonal carries its weight upwards on the principle of the ordinary truss and spreads it on other arches of larger span, each of which has its corresponding abutment in a set of diagonals.

Another way of looking at the steel reinforcement for such a masonry beam, is to regard it as a half truss, made up of tension members only, the masonry supplying the missing compression members, and the two being firmly united to each other through the cement, which forms a perfect bond between them.

One of the photographs submitted herewith shows such a lintel, consisting of a 12″x¼″ steel plate, to which 1″x¼″ diagonal members were riveted. The span was twelve feet, height of lintel eleven inches, breadth thirteen inches. Steel billets weighing 110 to 170 pounds were loaded on the beam until a total weight of 40,720 pounds was reached, equal to 3,400 pounds per linear foot of beam. The deflection was ¼ inch. Loading was stopped at this point, as the beam was beginning to be very top heavy, and it was feared might turn over and injure the workmen.

The above systems of concrete reinforcement which have been described are controlled by patents granted and now pending, which are held by the Trussed Concrete Steel Company, Union Trust Building, Detroit, Mich.

Fig. 4.

Showing method of failure for concrete, reinforced in accordance with old systems, using twisted rods. Span 18 feet.

Figures 4, 5, and 6, show tests made at Washington by the United States Engineers, on reinforced concrete beams and slabs, wherein twisted steel rods had been placed along the bottom of the floor. The methods of failure and reasons for it will at once become apparent to the engineer or architect. No matter how much horizontal reinforcement might have been placed in these floors, their strength would not have been increased. The probability is that their strength would have been greatly decreased, as the multiplicity of rods

Fig. 5.

Failure of concrete by shear, reinforcement horizontal only, using deformed rods.

would only have cut up the concrete at the bottom, wherein the enormous shearing strain existed, to which attention has already been called. The floors failed by longitudinal shear along the ends of the rods where this is maximum. All the twisting in the world would not have prevented it, nor would this twisting, to the slightest extent, have decreased the vertical shear, which, it is very apparent, was fundamental in the cause of failure. It is unscientific to neglect this matter of shear, and to imagine that concrete

Fig. 6.

Note failure of concrete when horizontal reinforcement only is used. Lines of failure correspond to lines of principal compressive stress.

within itself is capable of taking this strain. Tests for shear have developed strengths remarkably low. The writer has never been able to secure results of more than 200 to 400 lbs. per square inch. Why, therefore, assume such risk in reinforced concrete? There is only one way to prevent failures such as have been shown in these photographs, and that is by strengthening the floors both longitudinally and horizontally for shear, as well as bending moment; and this, it is believed, has been well accomplished by the Kahn system of Trussed Reinforcement.

FIG. 7.

Beams reinforced with Kahn System. Span 26 feet. Load, pig iron.

Figure 7, 8, 9 and 10 show tests of the same nature, made on two beams strengthened in accordance with the Kahn system of reinforcement These beams were 26 feet span. Please note the comparison of loadings between them and the floors of 18 feet span with twisted rods. When failure occurred in these beams, the rupture was absolutely central. The steel pulled in two. Not a sign of a crack was to be observed throughout the beam at any other point. Maximum efficiency was, therefore, given to the strength of the beam. The accomplishing of this result is of especial interest to the engineer, from the fact that he can design with certainty. If the steel pulls in two, he can calculate the strength of the concrete beam with the same accuracy as the steel I beam. Even more so; for the I beam, under test to ultimate destruction, will buckle in its top flange long before the bottom flange pulls in two.

Fig. 8.

Two beams reinforced with Kahn System. Span 26 feet.

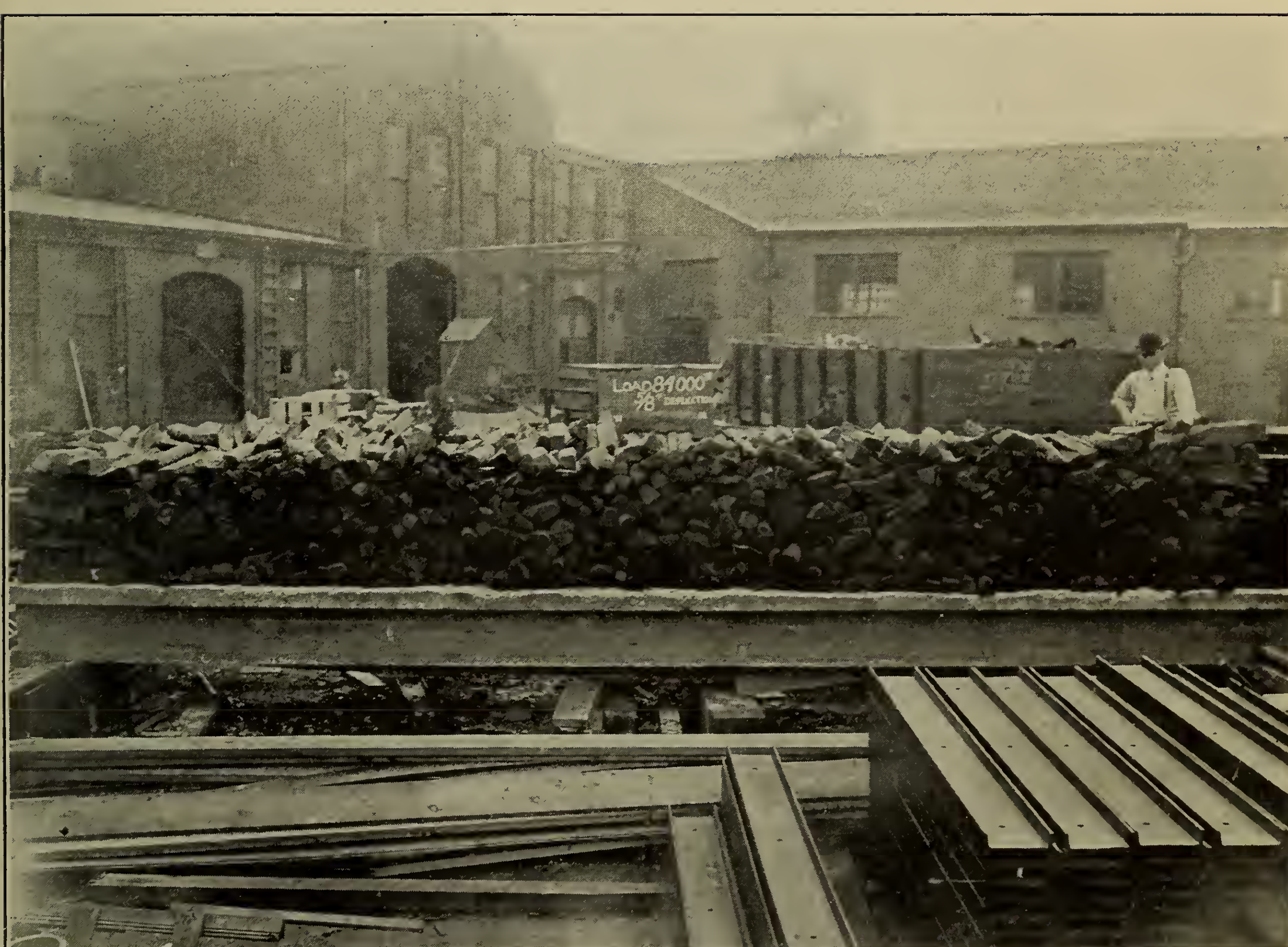


Fig. 9.

Load 84000 lbs. pig iron on two Kahn reinforced beams. Compare these with Fig. 6 where span is only 18 feet.

FIG. 10.

Failure of two Kahn reinforced beams

Load: Pig iron 101100 lbs.
Weight of floor slab 9300 lbs.

Total weight on beams 110400 lbs.

Beam failed in center pulling four bars of steel in two. Compare with Fig. 6.

Fig. 11.

Figure 11 shows the Kahn patented Trussed Bar. It is very interesting to note how readily it adapts itself to all types of construction. Its application to columns, walls, latticed girders and trusses is fully as simple as its application to beams. Where a column is to be constructed, the bars are set in the corners of the concrete, and the shear members extend across the body, forming practically a latticed column. The reasons for the efficiency of such a column will be very apparent. Under ordinary circumstances, a steel bar is steadied at points very closely together, then the entire strength of the steel can practically be developed. This result is accomplished in the steel reinforcement of a column, due to the hold of the concrete on the prongs. Furthermore, when the concrete tends to buckle, the steel comes into play on the principle of the ordinary latticed girder. In other words, the steel and concrete mutually reinforce each other.

Where moving loads are to be taken into account, it is best to place Kahn Trussed Bars in both the bottom and top of the beams, thereby producing practically a latticed girder.

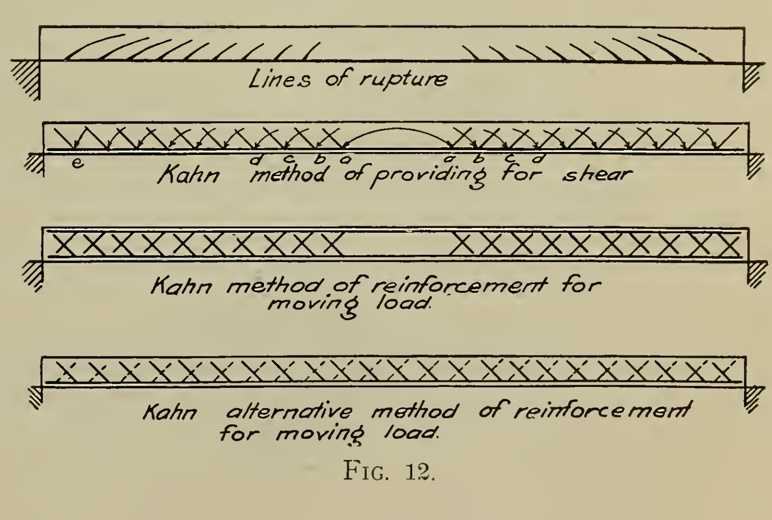
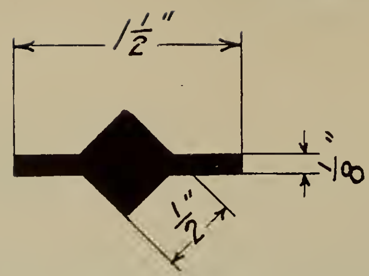
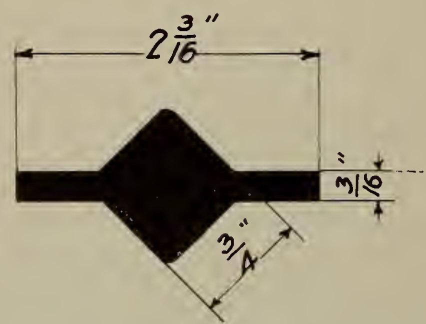


Fig. 12.

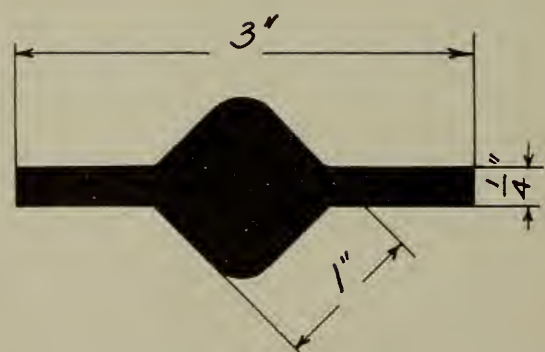


$1\frac{1}{2}'' \times \frac{1}{2}''$. Area .38 □" Weight 1.4# per ft.



These bars can have any standard size cuts as shown in Figure 14 and will be sent in lengths as ordered. In making calculations for strength of reinforced beam, assume the area of the entire cross section as here given.

$2\frac{3}{16}'' \times \frac{3}{4}''$. Area .78 □" Weight 2.7# per ft.



$3'' \times 1''$. Area 1.42 □" Weight 4.8# per ft.

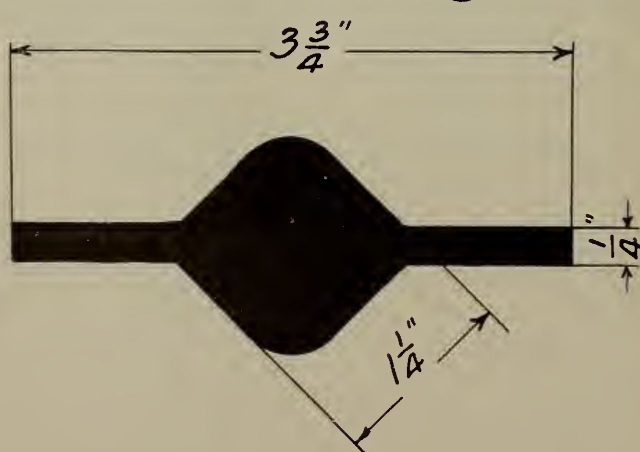
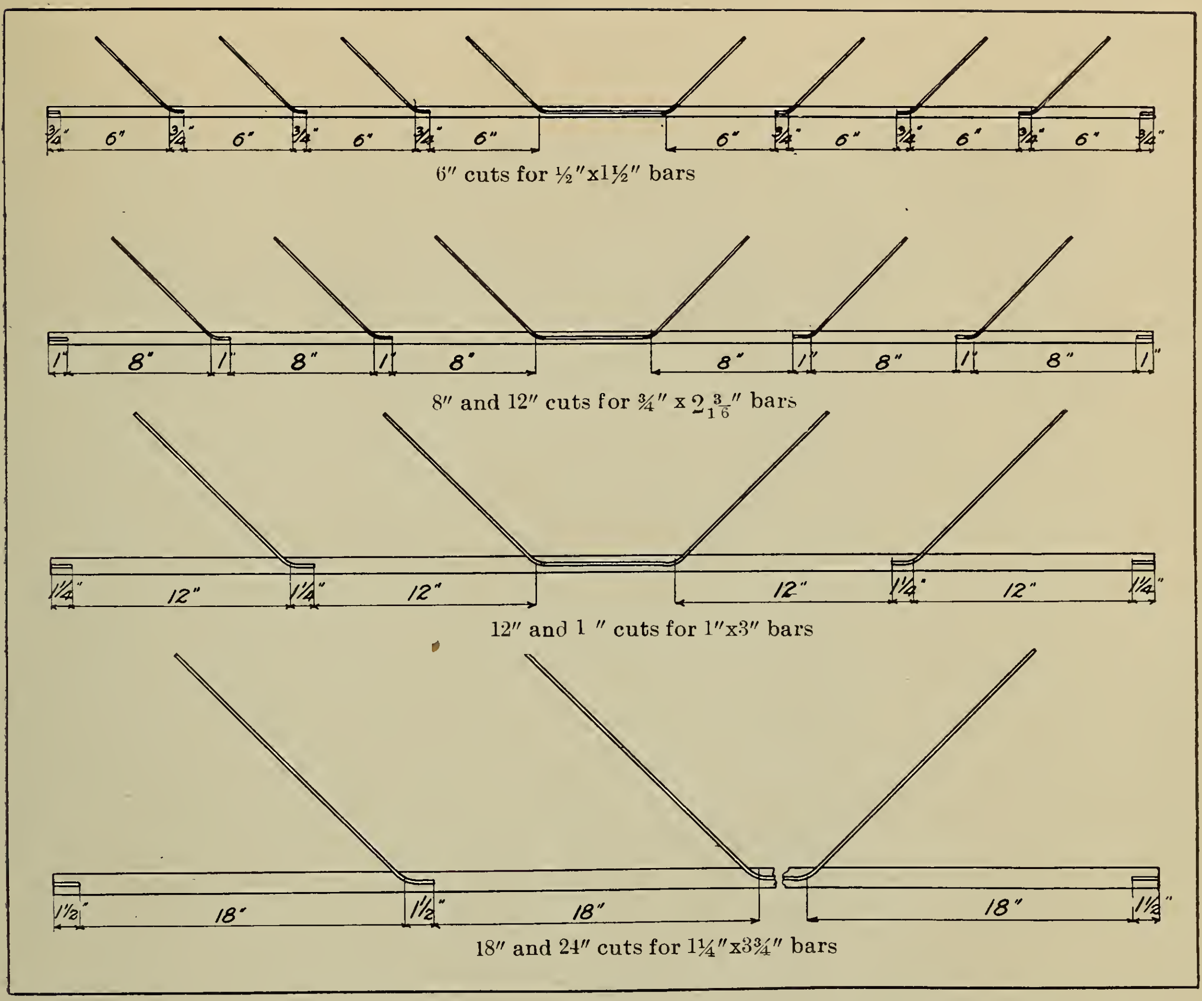


FIG. 13. $3\frac{3}{4}'' \times 1\frac{1}{4}''$. Area 2.0 □" Weight 6.9# per ft.



Bars kept in stock ready for immediate delivery, in any lengths with standard cuts.

Fig. 14.

Figure 13 shows standard sections of the Kahn Trussed Bars. Practically any construction can be built by using one of the four sizes shown and sheared up as is indicated in Figure 14. The equivalent of Steel Beams from 6 inches to 20 inches can be built up with reinforced concrete, using one or more of these bars placed in the bottom, or on the tension side.

Figure 14 shows standard cuts. It will be noticed that the largest is 18 inches. Where deeper girders are wanted, it will be well to lay some of the rods horizontally all the way along the bottom, and others slanting upwards from the bottom towards the ends of the beam, thereby distributing the shear members throughout the beam, and causing them to reach its very top.

Calculations

Modern Design Loads

LAKE PARK ARCH BRIDGE - ADDITIONAL LOAD CONSIDERATIONS

Update structural analysis to determine the capability of the bridge to resist current code prescribed pedestrian and maintenance vehicle loads.

Assumptions:

- Calculate capacity to demand ratios based on Allowable Stress Design (ASD)
- Consider the following loading cases:
  - ↳ Inventory Level: 90 psf pedestrian load
  - ↳ Operating Level: H5 Maintenance Vehicle (5-ton truck) + 90 psf Pedestrian
- Use allowable stress factors based on AASHTO Manual for Bridge Evaluation
  - ↳ Reinforcing Steel [AASHTO MBE Table 6B.5.2.3-1]
    - Inventory: f_s = 18,000 psi
    - Operating: f_s = 25,000 psi
  - ↳ Compression due to Bending [AASHTO MBE Table 6B.5.2.4.1-1]
    - Inventory: f_c = 640 psi (original), f_c = 800 psi (concrete testing)
    - Operating: f_c = 960 psi (original), f_c = 1200 psi (concrete testing)
- Based on results of original 3 analysis alternatives, shear does not govern any of the capacity to demand ratios
  - ↳ ∴ Ignore shear calculations

6B.5.2.3—Reinforcing Steel

The following are the allowable unit stresses in tension for reinforcing steel. These will ordinarily be used without reduction when the condition of the steel is unknown.

Table 6B.5.2.3-1—Allowable Unit Stresses for Reinforcing Steel

	Stresses (psi)		
	Inventory Rating	Operating Rating	Yield
Structural or unknown grade prior to 1954	18,000	25,000	33,000
Structural Grade	20,000	27,000	36,000
Grade 40 billet, intermediate, or unknown grade (after 1954)	20,000	28,000	40,000
Grade 50 rail or hard	20,000	32,500	50,000
Grade 60	24,000	36,000	60,000

6B.5.2.4—Concrete

Unit stresses in concrete may be determined in accordance with the Service Load Design Method of the AASHTO Standard Specifications (Article 8.15) or be based on the articles below. When the ultimate strength, f'_c, of the concrete is unknown and the concrete is in satisfactory condition, f'_c may be determined from Table 6B.5.2.4-1.

C6B.5.2.4

Some guidance on the ultimate strength, f'_c, of concrete may be obtained from compression testing of cores removed from the structure. (See Article 5.3.)

Guidance on considering the effects of deterioration on the load rating of concrete structures can be found in Article C6A.5.5.

Table 6B.5.2.4-1—Allowable Unit Stresses for Concrete

Year Built	f'_c (psi)
Prior to 1959	2,500
1959 and later	3,000

Ignore: Use information from existing plans and concrete testing

For prestressed concrete components, the compressive strengths shown above may be increased by 25 percent.

6B.5.2.4.1 Bending

The following maximum allowable bending unit stresses in concrete in lb/in.^{2} may be used:

Table 6B.5.2.4.1-1—Compression Due to Bending f'_c

As-Inspected based on Concrete Testing

f'_c (psi)	Compression Due to Bending f'_c (psi)		n
	Inventory Level	Operating Level	
2,000–2,400	800	1,200	15
2,500–2,900	1,000	1,500	12
3,000–3,900	1,200	1,900	10
4,000–4,900	1,600	2,400	8
5,000 or more	2,000	3,000	6

Based on values in table, use:

Inventory: $f_c = 0.4 f'_c$
Operating: $f_c = 0.6 f'_c$

2. For original concrete in As-Built and As-Configured analysis alternatives:

Inventory = 0.4 (1600) = 640 psi
Operating = 0.6 (1600) = 960 psi

mitigate the risk from vehicle collisions with the superstructure. Should the owner desire additional mitigation, the following steps may be taken:

- Increasing vertical clearance in addition to that contained in *AASHTO LRFD*
- Providing structural continuity of the superstructure, either between spans or with the substructure
- Increasing the mass of the superstructure
- Increasing the lateral resistance of the superstructure

2—PHILOSOPHY

Pedestrian bridges shall be designed for specified limit states to achieve the objectives of safety; serviceability, including comfort of the pedestrian user (vibration); and constructability with due regard to issues of inspectability, economy, and aesthetics, as specified in *AASHTO LRFD*. These Guide Specifications are based on the LRFD philosophy. Mixing provisions from specifications other than those referenced herein, even if LRFD based, should be avoided.

3—LOADS

3.1—PEDESTRIAN LOADING (PL)

Pedestrian bridges shall be designed for a uniform pedestrian loading of 90 psf. This loading shall be patterned to produce the maximum load effects. Consideration of dynamic load allowance is not required with this loading.

C3.1

This article modifies the pedestrian loading provisions of the Fourth Edition of *AASHTO LRFD*, through the 2009 Interim. The previous edition of these Guide Specifications used a base nominal loading of 85 psf, reducible to 65 psf based on influence area for the pedestrian load. With the LFD load factors, this results in factored loads of 2.17(85) = 184 psf and 2.17(65) = 141 psf. The Fourth Edition of *AASHTO LRFD* specified a constant 85 psf regardless of influence area. Multiplying by the load factor, this results in 1.75(85) = 149 psf. This falls within the range of the previous factored loading, albeit toward the lower end.

European codes appear to start with a higher nominal load (approx 105 psf), but then allow reductions based on loaded length. Additionally, the load factor applied is 1.5, resulting in a maximum factored load of (1.5)105 = 158 psf. For a long loaded length, this load can be reduced to as low as 50 psf, resulting in a factored load of (1.5)50 = 75 psf. The effect of resistance factors has not been accounted for in the above discussion of the European codes. There are,

Figure C3.1-1—Live Load of 50 psf

Figure C3.1-2—Live Load of 100 psf

Figure C3.1-3—Live Load of 150 psf

3.2—VEHICLE LOAD (LL)

Where vehicular access is not prevented by permanent physical methods, pedestrian bridges shall be designed for a maintenance vehicle load specified in Figure 1 and Table 1 for the Strength I Load Combination unless otherwise specified by the Owner.

C3.2

The vehicle loading specified is equivalent to the H-trucks shown in Article 3.6.1.6 of *AASHTO LRFD* 2009 Interim and contained in previous versions of the *AASHTO Standard Specifications for Highway Bridges*.

A single truck shall be placed to produce the maximum load effects and shall not be placed in combinations with the pedestrian load. The dynamic load allowance need not be considered for this loading.

Table 3.2-1—Design Vehicle

Clear Deck Width	Design Vehicle
7 to 10 ft	H5
Over 10 ft	H10

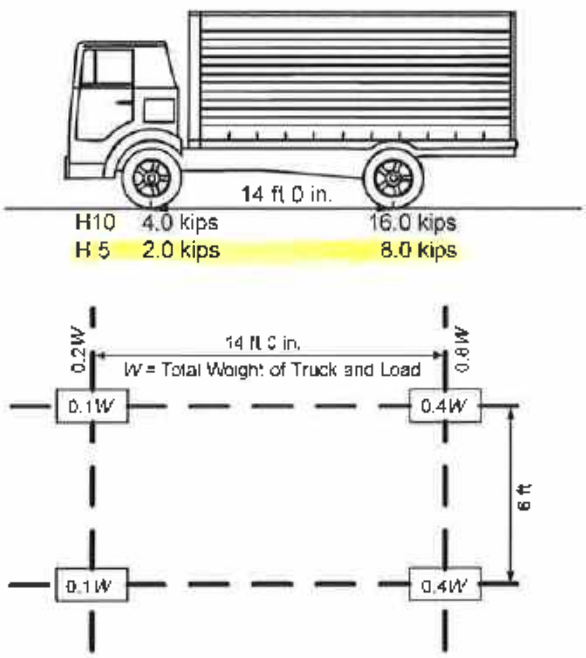


Figure 3.2-1—Maintenance Vehicle Configurations

3.3—EQUESTRIAN LOAD (LL)

Decks intended to carry equestrian loading shall be designed for a patch load of 1.00 kip over a square area measuring 4.0 in. on a side.

C3.3

The equestrian load is a live load and intended to ensure adequate punching shear capacity of pedestrian bridge decks where horses are expected. The loading was derived from hoof pressure measurements reported in Roland et. al. (2005). The worst loading occurs during a canter where the loading on one hoof approaches 100 percent of the total weight of the horse. The total factored load of 1.75 kips is approximately the maximum credible weight of a draft horse. This loading is expected to control only deck design.

3.4—WIND LOAD (WS)

Pedestrian bridges shall be designed for wind loads as specified in *AASHTO Signs*, Articles 3.8 and 3.9. Unless otherwise directed by the Owner, the Wind Importance Factor, I_r, shall be taken as 1.15. The loading shall be applied over the exposed area in front

C3.4

The wind loading is taken from *AASHTO Signs* specification rather than from *AASHTO LRFD* due to the potentially flexible nature of pedestrian bridges, and also due to the potential for traffic signs to be mounted on them.

TranSystems

Made By: DWC | Date: 8/2/18 | Job No: P402180060

Checked By: SFH | Date: 8/3/18 | Sheet No.

LAKE PARK ARCH BRIDGE – ADDITIONAL LOAD CONSIDERATIONS

ARCH RIBS

Update loads and material properties used in arch rib analysis and re-run for new numbers based on the following changes:

f_s = 18000 psi (Inventory), f_s = 25000 psi (Operating)
f_c = 640 psi (Inv, As-Built), f_c = 960 psi (Oper, As-Built)
f_c = 800 psi (Inv, As-Inspected), f_c = 1200 psi (Oper, As-Inspected)

A_s = 2.79 in² TOP + BOTTOM (As-Built)
A_s = 2.14 in² TOP + BOTTOM (As-Inspected)

Loads:

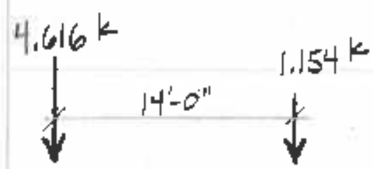
Inventory: Use 90 psf pedestrian load with position varied to maximize load effects

∴ Load in STAAD = $(90\text{ psf})(12'/2)(1/1000)$ = 0.54 k/ft

Operating: Use HS maintenance truck (5-ton truck) w/ LLDF applied (see Longitudinal Spandrel Beam calcs) plus 90 psf pedestrian load

∴ Front wheel load = LLDF × (1 kip) = 1.154 × 1ᵏ = 1.154 ᵏ

Rear wheel load = LLDF × (4 kip) = 1.154 × 4ᵏ = 4.616 ᵏ



For now, only check Lower Arch segments because they clearly governed the previous analysis.

Made By: DWC	Date: 8/3/18	Job No. P402180060
Checked By: SFH	Date: 8/3/18	Sheet No.

LAKE PARK ARCH BRIDGE - ADDITIONAL LOAD CONSIDERATIONS

ARCH RIBS

Operating Level

For Operating Level analysis, consider 90 psf pedestrian load and H5 maintenance truck acting in unison. To do this, use superposition of STAAD output results for truck (operating model) with 90 psf (Inventory Model) and Dead Load:

TOTAL = Dead Load + 90 psf + H5 Truck

To capture the worst case load effects for multiple varying load cases, sort the data for 90 psf + H5 truck load effects by the following 4 cases:

1) Maximum axial + corresponding moment
2) Minimum axial + corresponding moment
3) Maximum moment + corresponding axial
4) Minimum moment + corresponding axial

Calculations for 1/4 US Grid

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 02-Aug-18
CHECKER DATE 03-Aug-18
JOB NAME LAKE PARK ARCH BRIDGE LOAD RATING
JOB COMMENT ARCH RIBS - DEAD LOAD + 90 PSF
ENGINEER NAME DWC
CHECKER NAME SFH
END JOB INFORMATION
INPUT WIDTH 79
UNIT FEET KIP
JOINT COORDINATES
*Node X Y Z
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6 14 7.942 0; 7 17 9.297 0; 8 20 10.542 0; 9 23 11.68 0; 10 26 12.716 0;
11 29 13.653 0; 12 32 14.493 0; 13 35 15.239 0; 14 38 15.893 0;
15 41 16.456 0; 16 44 16.93 0; 17 47 17.317 0; 18 50 17.616 0;
19 53 17.83 0; 20 56 17.957 0; 21 59 18 0; 22 62 17.957 0; 23 65 17.83 0;
24 68 17.616 0; 25 71 17.317 0; 26 74 16.93 0; 27 77 16.456 0;
28 80 15.893 0; 29 83 15.239 0; 30 86 14.493 0; 31 89 13.653 0;
32 92 12.716 0; 33 95 11.68 0; 34 98 10.542 0; 35 101 9.297 0;
36 104 7.942 0; 37 107 6.472 0; 38 109.75 5.019 0; 39 112.5 3.46 0;
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54  47  19  0   ; 55    53  19  0   ;
56  59  19  0   ; 57    65  19  0   ;
58  71  19  0   ; 59    77  19  0   ;
60  83  19  0   ; 61    107 19  0   ;
62  118 19  0   ;

MEMBER INCIDENCES
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11 11 12; 12 12 13; 13 13 14; 14 14 15; 15 15 16; 16 16 17; 17 17 18; 18 18 19;
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77  57  58  ; 78    58  59  ; 79    59  60  ; 80    60  61  ;
81  61  62  ;

DEFINE MATERIAL START
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E 453600
POISSON 0.17
DENSITY 0.150336
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ALPHA 5e-006
DAMP 0.05
TYPE CONCRETE
STRENGTH FCU 576
ISOTROPIC STEEL
E 4.176e+006
POISSON 0.3
DENSITY 0.489024
ALPHA 6e-006
DAMP 0.03
TYPE STEEL
STRENGTH FY 5184 FU 8352 RY 1.5 RT 1.2
END DEFINE MATERIAL

MEMBER PROPERTY AMERICAN
1 TO 40  PRIS YD 4.5 ZD 1
50 TO 62 PRIS YD 0.6667 ZD 3
70 TO 81 PRIS YD 3.167 ZD 1

CONSTANTS
MATERIAL CONCRETE ALL

MEMBER RELEASE
50 51 61 62 BOTH MY MZ
70 71 START MY MZ
70 71 END MY MZ FX
80 81 END MY MZ
80 81 START MY MZ FX
52 TO 55 57 TO 60 START MY MZ

SUPPORTS
1 41 FIXED

LOAD 1 LOADTYPE Dead TITLE DEAD LOADS
*ARCH LOAD
SELFWEIGHT Y -1.0 LIST 1 TO 40
MEMBER LOAD
1 TO 40 UNI GY -0.103

*DECK AND PARAPET (AS-BUILT)
MEMBER LOAD
70 TO 81 UNI GY -0.98

****DECK AND PARAPET (AS-CONFIGURED/AS-INSPECTED)
***MEMBER LOAD
***70 TO 81 UNI GY -1.13

*TRANSVERSE WALLS
JOINT LOAD
5 37 FY -9.9
```

```
13 29 FY -4.5
21 FY -2.7

*STRUTS
JOINT LOAD
9 17 25 33 FY -1.2

*SPANDREL WALLS
MEMBER LOAD
70 71 80 81 UNI GY -0.40
MEMBER LOAD
72 TRAP GY -0.50 -0.40
73 TRAP GY -0.40 -0.30
74 TRAP GY -0.30 -0.20
75 TRAP GY -0.20 -0.10
76 TRAP GY -0.10 -0.20
77 TRAP GY -0.20 -0.30
78 TRAP GY -0.30 -0.40
79 TRAP GY -0.40 -0.50
MEMBER LOAD
4 5 36 37 UNI GY -1.18

LOAD 2 LOADTYPE Live TITLE LIVE LOAD 1
MEMBER LOAD
73 UNI GY -0.54 3.25 6
74 TO 77 UNI GY -0.54
78 UNI GY -0.54 0 2.75

LOAD 3 LOADTYPE Live TITLE LIVE LOAD 2
MEMBER LOAD
70 TO 72 79 TO 81 UNI GY -0.54
73 UNI GY -0.54 0 3.25
78 UNI GY -0.54 2.75 6

LOAD 4 LOADTYPE Live TITLE LIVE LOAD 3
MEMBER LOAD
73 UNI GY -0.54 3.25 6
74 TO 81 UNI GY -0.54

LOAD 5 LOADTYPE Live TITLE LIVE LOAD 4
MEMBER LOAD
70 TO 72 UNI GY -0.54
73 UNI GY -0.54 0 3.25

LOAD 6 LOADTYPE Live TITLE LIVE LOAD 5
MEMBER LOAD
70 TO 77 UNI GY -0.54
78 UNI GY -0.54 0 2.75

LOAD 7 LOADTYPE Live TITLE LIVE LOAD 6
```

```
MEMBER LOAD
78 UNI GY -0.54 2.75 6
79 TO 81 UNI GY -0.54

LOAD 8 LOADTYPE Live TITLE LIVE LOAD 7
MEMBER LOAD
70 TO 81 UNI GY -0.54

LOAD COMB 11 DL + LL1
1 1.0 2 1.0

LOAD COMB 12 DL + LL2
1 1.0 3 1.0

LOAD COMB 13 DL + LL3
1 1.0 4 1.0

LOAD COMB 14 DL + LL4
1 1.0 5 1.0

LOAD COMB 15 DL + LL5
1 1.0 6 1.0

LOAD COMB 16 DL + LL6
1 1.0 7 1.0

LOAD COMB 17 DL + LL7
1 1.0 8 1.0

PERFORM ANALYSIS
FINISH
```

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 02-Aug-18
CHECKER DATE 03-Aug-18
JOB NAME LAKE PARK ARCH BRIDGE LOAD RATING
JOB COMMENT ARCH RIBS - H5 TRUCK
ENGINEER NAME DWC
CHECKER NAME SFH
END JOB INFORMATION
INPUT WIDTH 79
UNIT FEET KIP
JOINT COORDINATES
*Node X Y Z
1 0 0 0; 2 2.75 1.789  ; 3 5.5 3.46  ; 4 8.25 5.019  ; 5 11 6.472 0;
6 14 7.942 0; 7 17 9.297 0; 8 20 10.542 0; 9 23 11.68 0; 10 26 12.716 0;
11 29 13.653 0; 12 32 14.493 0; 13 35 15.239 0; 14 38 15.893 0;
15 41 16.456 0; 16 44 16.93 0; 17 47 17.317 0; 18 50 17.616 0;
19 53 17.83 0; 20 56 17.957 0; 21 59 18 0; 22 62 17.957 0; 23 65 17.83 0;
24 68 17.616 0; 25 71 17.317 0; 26 74 16.93 0; 27 77 16.456 0;
28 80 15.893 0; 29 83 15.239 0; 30 86 14.493 0; 31 89 13.653 0;
32 92 12.716 0; 33 95 11.68 0; 34 98 10.542 0; 35 101 9.297 0;
36 104 7.942 0; 37 107 6.472 0; 38 109.75 5.019 0; 39 112.5 3.46 0;
40 115.25 1.789 0; 41 118 0 0;
50  0   19  0   ; 51    11  19  0   ;
52  35  19  0   ; 53    41  19  0   ;
54  47  19  0   ; 55    53  19  0   ;
56  59  19  0   ; 57    65  19  0   ;
58  71  19  0   ; 59    77  19  0   ;
60  83  19  0   ; 61    107 19  0   ;
62  118 19  0   ;

MEMBER INCIDENCES
1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 5 6; 6 6 7; 7 7 8; 8 8 9; 9 9 10; 10 10 11;
11 11 12; 12 12 13; 13 13 14; 14 14 15; 15 15 16; 16 16 17; 17 17 18; 18 18 19;
19 19 20; 20 20 21; 21 21 22; 22 22 23; 23 23 24; 24 24 25; 25 25 26; 26 26 27;
27 27 28; 28 28 29; 29 29 30; 30 30 31; 31 31 32; 32 32 33; 33 33 34; 34 34 35;
35 35 36; 36 36 37; 37 37 38; 38 38 39; 39 39 40; 40 40 41;
50  1   50  ; 51    5   51  ; 52    13  52  ; 53    15  53  ;
54  17  54  ; 55    19  55  ; 56    21  56  ; 57    23  57  ;
58  25  58  ; 59    27  59  ; 60    29  60  ; 61    37  61  ;
62  41  62  ; 70    50  51  ; 71    51  52  ; 72    52  53  ;
73  53  54  ; 74    54  55  ; 75    55  56  ; 76    56  57  ;
77  57  58  ; 78    58  59  ; 79    59  60  ; 80    60  61  ;
81  61  62  ;

DEFINE MATERIAL START
ISOTROPIC CONCRETE
E 453600
POISSON 0.17
DENSITY 0.150336
```

```
ALPHA 5e-006
DAMP 0.05
TYPE CONCRETE
STRENGTH FCU 576
ISOTROPIC STEEL
E 4.176e+006
POISSON 0.3
DENSITY 0.489024
ALPHA 6e-006
DAMP 0.03
TYPE STEEL
STRENGTH FY 5184 FU 8352 RY 1.5 RT 1.2
END DEFINE MATERIAL

MEMBER PROPERTY AMERICAN
1 TO 40  PRIS YD 4.5 ZD 1
50 TO 62 PRIS YD 0.6667 ZD 3
70 TO 81 PRIS YD 3.167 ZD 1

CONSTANTS
MATERIAL CONCRETE ALL

MEMBER RELEASE
50 51 61 62 BOTH MY MZ
70 71 START MY MZ
70 71 END MY MZ FX
80 81 END MY MZ
80 81 START MY MZ FX
52 TO 55 57 TO 60 START MY MZ

SUPPORTS
1 41 FIXED

DEFINE MOVING LOAD
TYPE 1 LOAD 4.616 1.154
DIST 14
TYPE 2 LOAD 1.154 4.616
DIST 14

***LOAD 1 LOADTYPE Dead TITLE DEAD LOADS
****ARCH LOAD
***SELFWEIGHT Y -1.0 LIST 1 TO 40
***MEMBER LOAD
***1 TO 40 UNI GY -0.103

*******DECK AND PARAPET (AS-BUILT)
******MEMBER LOAD
******70 TO 81 UNI GY -0.98

****DECK AND PARAPET (AS-CONFIGURED/AS-INSPECTED)
```

```
***MEMBER LOAD
***70 TO 81 UNI GY -1.13

****TRANSVERSE WALLS
***JOINT LOAD
***5 37 FY -9.9
***13 29 FY -4.5
***21 FY -2.7

****STRUTS
***JOINT LOAD
***9 17 25 33 FY -1.2

****SPANDREL WALLS
***MEMBER LOAD
***70 71 80 81 UNI GY -0.40
***MEMBER LOAD
***72 TRAP GY -0.50 -0.40
***73 TRAP GY -0.40 -0.30
***74 TRAP GY -0.30 -0.20
***75 TRAP GY -0.20 -0.10
***76 TRAP GY -0.10 -0.20
***77 TRAP GY -0.20 -0.30
***78 TRAP GY -0.30 -0.40
***79 TRAP GY -0.40 -0.50
***MEMBER LOAD
***4 5 36 37 UNI GY -1.18

LOAD GENERATION 21
TYPE 1 0 19 0 XINC 5
LOAD GENERATION 21
TYPE 2 0 19 0 XINC 5


PERFORM ANALYSIS
FINISH
```

Job Information

	Engineer	Checked	Approved
Name:	DWC	SFH	
Date:	02-Aug-18	03-Aug-18	

Project ID	
Project Name	

Comments

ARCH RIBS - DEAD LOAD + 90 PSF

Structure Type	SPACE FRAME

Number of Nodes	54	Highest Node	62
Number of Elements	65	Highest Beam	81

Number of Basic Load Cases	-2
Number of Combination Load Cases	7

Included in this printout are data for:

Beams	1 to 20

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	DEAD LOADS
Combination	11	DL + LL1
Combination	12	DL + LL2
Combination	13	DL + LL3
Combination	14	DL + LL4
Combination	15	DL + LL5
Combination	16	DL + LL6
Combination	17	DL + LL7

Beam End Forces

Sign convention is as the action of the joint on the beam.

			Axial	Shear		Torsion	Bending	
Beam	**Node**	**L/C**	**Fx** (kip)	**Fy** (kip)	**Fz** (kip)	**Mx** (kip`ft)	**My** (kip`ft)	**Mz** (kip`ft)
1	1	1:DEAD LOAD	261.428	4.086	0.000	0.000	0.000	143.871
		11:DL + LL1	283.604	-0.838	0.000	0.000	0.000	108.255
		12:DL + LL2	296.482	6.245	0.000	0.000	0.000	182.838
		13:DL + LL3	297.069	-6.001	0.000	0.000	0.000	41.316
		14:DL + LL4	283.016	11.408	0.000	0.000	0.000	249.777
		15:DL + LL5	305.191	6.484	0.000	0.000	0.000	214.161

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip`ft)	My (kip`ft)	Mz (kip`ft)
		11:DL + LL1	-282.209	2.982	0.000	0.000	0.000	-114.521
		12:DL + LL2	-295.087	-4.102	0.000	0.000	0.000	-165.865
		13:DL + LL3	-295.675	8.144	0.000	0.000	0.000	-64.519
		14:DL + LL4	-281.621	-9.264	0.000	0.000	0.000	-215.867
		15:DL + LL5	-303.797	-4.340	0.000	0.000	0.000	-196.406
		16:DL + LL6	-273.500	3.220	0.000	0.000	0.000	-83.980
		17:DL + LL7	-317.263	0.822	0.000	0.000	0.000	-146.404
2	2	1:DEAD LOAD	259.851	9.934	0.000	0.000	0.000	133.982
		11:DL + LL1	282.167	5.694	0.000	0.000	0.000	114.521
		12:DL + LL2	294.822	13.170	0.000	0.000	0.000	165.865
		13:DL + LL3	295.786	0.948	0.000	0.000	0.000	64.519
		14:DL + LL4	281.204	17.916	0.000	0.000	0.000	215.867
		15:DL + LL5	303.520	13.676	0.000	0.000	0.000	196.406
		16:DL + LL6	273.469	5.188	0.000	0.000	0.000	83.980
		17:DL + LL7	317.138	8.930	0.000	0.000	0.000	146.404
	3	1:DEAD LOAD	-258.549	-7.791	0.000	0.000	0.000	-105.464
		11:DL + LL1	-280.865	-3.551	0.000	0.000	0.000	-99.646
		12:DL + LL2	-293.519	-11.026	0.000	0.000	0.000	-126.935
		13:DL + LL3	-294.483	1.196	0.000	0.000	0.000	-64.917
		14:DL + LL4	-279.901	-15.773	0.000	0.000	0.000	-161.664
		15:DL + LL5	-302.217	-11.533	0.000	0.000	0.000	-155.846
		16:DL + LL6	-272.167	-3.044	0.000	0.000	0.000	-70.735
		17:DL + LL7	-315.835	-6.786	0.000	0.000	0.000	-121.117
3	3	1:DEAD LOAD	258.194	15.616	0.000	0.000	0.000	105.464
		11:DL + LL1	280.629	12.053	0.000	0.000	0.000	99.646
		12:DL + LL2	293.051	19.909	0.000	0.000	0.000	126.935
		13:DL + LL3	294.384	7.721	0.000	0.000	0.000	64.917
		14:DL + LL4	279.295	24.240	0.000	0.000	0.000	161.664
		15:DL + LL5	301.729	20.678	0.000	0.000	0.000	155.846
		16:DL + LL6	271.950	11.284	0.000	0.000	0.000	70.735
		17:DL + LL7	315.485	16.346	0.000	0.000	0.000	121.117
	4	1:DEAD LOAD	-256.979	-13.472	0.000	0.000	0.000	-59.489
		11:DL + LL1	-279.413	-9.910	0.000	0.000	0.000	-64.932
		12:DL + LL2	-291.836	-17.765	0.000	0.000	0.000	-67.388
		13:DL + LL3	-293.169	-5.578	0.000	0.000	0.000	-43.897
		14:DL + LL4	-278.080	-22.097	0.000	0.000	0.000	-88.424
		15:DL + LL5	-300.514	-18.534	0.000	0.000	0.000	-93.867
		16:DL + LL6	-270.735	-9.140	0.000	0.000	0.000	-38.454
		17:DL + LL7	-314.270	-14.203	0.000	0.000	0.000	-72.832
4	4	1:DEAD LOAD	256.466	21.085	0.000	0.000	0.000	59.489
		11:DL + LL1	278.997	18.189	0.000	0.000	0.000	64.932
		12:DL + LL2	291.181	26.409	0.000	0.000	0.000	67.388
		13:DL + LL3	292.875	14.267	0.000	0.000	0.000	43.897
		14:DL + LL4	277.302	30.332	0.000	0.000	0.000	88.424

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip˜ft)	My (kip˜ft)	Mz (kip˜ft)
		15:DL + LL5	299.833	27.436	0.000	0.000	0.000	93.867
		16:DL + LL6	270.345	17.163	0.000	0.000	0.000	38.454
		17:DL + LL7	313.711	23.514	0.000	0.000	0.000	72.832
	5	1:DEAD LOAD	-253.619	-15.696	0.000	0.000	0.000	-2.289
		11:DL + LL1	-276.149	-12.801	0.000	0.000	0.000	-16.739
		12:DL + LL2	-288.333	-21.021	0.000	0.000	0.000	6.372
		13:DL + LL3	-290.027	-8.878	0.000	0.000	0.000	-7.902
		14:DL + LL4	-274.455	-24.943	0.000	0.000	0.000	-2.465
		15:DL + LL5	-296.985	-22.047	0.000	0.000	0.000	-16.914
		16:DL + LL6	-267.497	-11.774	0.000	0.000	0.000	6.547
		17:DL + LL7	-310.863	-18.125	0.000	0.000	0.000	-8.078
5	5	1:DEAD LOAD	238.041	-7.162	0.000	0.000	0.000	2.289
		11:DL + LL1	260.649	-9.370	0.000	0.000	0.000	16.739
		12:DL + LL2	268.419	-9.269	0.000	0.000	0.000	-6.372
		13:DL + LL3	274.640	-12.868	0.000	0.000	0.000	7.902
		14:DL + LL4	254.428	-5.771	0.000	0.000	0.000	2.465
		15:DL + LL5	277.035	-7.979	0.000	0.000	0.000	16.914
		16:DL + LL6	252.032	-10.660	0.000	0.000	0.000	-6.547
		17:DL + LL7	291.026	-11.477	0.000	0.000	0.000	8.078
	6	1:DEAD LOAD	-235.161	13.041	0.000	0.000	0.000	-36.037
		11:DL + LL1	-257.768	15.249	0.000	0.000	0.000	-57.863
		12:DL + LL2	-265.538	15.147	0.000	0.000	0.000	-34.413
		13:DL + LL3	-271.759	18.747	0.000	0.000	0.000	-60.711
		14:DL + LL4	-251.547	11.650	0.000	0.000	0.000	-31.565
		15:DL + LL5	-274.155	13.858	0.000	0.000	0.000	-53.391
		16:DL + LL6	-249.152	16.539	0.000	0.000	0.000	-38.885
		17:DL + LL7	-288.146	17.355	0.000	0.000	0.000	-56.239
6	6	1:DEAD LOAD	235.454	-5.657	0.000	0.000	0.000	36.037
		11:DL + LL1	258.120	-7.155	0.000	0.000	0.000	57.863
		12:DL + LL2	265.883	-6.810	0.000	0.000	0.000	34.413
		13:DL + LL3	272.214	-10.212	0.000	0.000	0.000	60.711
		14:DL + LL4	251.789	-3.753	0.000	0.000	0.000	31.565
		15:DL + LL5	274.455	-5.250	0.000	0.000	0.000	53.391
		16:DL + LL6	249.548	-8.714	0.000	0.000	0.000	38.885
		17:DL + LL7	288.549	-8.307	0.000	0.000	0.000	56.239
	7	1:DEAD LOAD	-234.398	7.996	0.000	0.000	0.000	-58.508
		11:DL + LL1	-257.064	9.493	0.000	0.000	0.000	-85.265
		12:DL + LL2	-264.826	9.148	0.000	0.000	0.000	-60.678
		13:DL + LL3	-271.158	12.550	0.000	0.000	0.000	-98.176
		14:DL + LL4	-250.732	6.091	0.000	0.000	0.000	-47.767
		15:DL + LL5	-273.398	7.589	0.000	0.000	0.000	-74.524
		16:DL + LL6	-248.492	11.053	0.000	0.000	0.000	-71.419
		17:DL + LL7	-287.492	10.646	0.000	0.000	0.000	-87.435
7	7	1:DEAD LOAD	234.533	-0.757	0.000	0.000	0.000	58.508

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
		11:DL + LL1	257.234	-1.555	0.000	0.000	0.000	85.265
		12:DL + LL2	264.983	-0.970	0.000	0.000	0.000	60.678
		13:DL + LL3	271.416	-4.175	0.000	0.000	0.000	98.176
		14:DL + LL4	250.801	1.650	0.000	0.000	0.000	47.767
		15:DL + LL5	273.502	0.853	0.000	0.000	0.000	74.524
		16:DL + LL6	248.714	-3.378	0.000	0.000	0.000	71.419
		17:DL + LL7	287.684	-1.768	0.000	0.000	0.000	87.435
	8	1:DEAD LOAD	-233.562	3.096	0.000	0.000	0.000	-64.767
		11:DL + LL1	-256.264	3.893	0.000	0.000	0.000	-94.113
		12:DL + LL2	-264.012	3.309	0.000	0.000	0.000	-67.628
		13:DL + LL3	-270.445	6.514	0.000	0.000	0.000	-115.535
		14:DL + LL4	-249.831	0.688	0.000	0.000	0.000	-46.205
		15:DL + LL5	-272.532	1.486	0.000	0.000	0.000	-75.551
		16:DL + LL6	-247.744	5.716	0.000	0.000	0.000	-86.189
		17:DL + LL7	-286.714	4.106	0.000	0.000	0.000	-96.974
8	8	1:DEAD LOAD	233.547	4.099	0.000	0.000	0.000	64.767
		11:DL + LL1	256.262	4.002	0.000	0.000	0.000	94.113
		12:DL + LL2	263.989	4.825	0.000	0.000	0.000	67.628
		13:DL + LL3	270.518	1.819	0.000	0.000	0.000	115.535
		14:DL + LL4	249.733	7.007	0.000	0.000	0.000	46.205
		15:DL + LL5	272.448	6.909	0.000	0.000	0.000	75.551
		16:DL + LL6	247.802	1.917	0.000	0.000	0.000	86.189
		17:DL + LL7	286.704	4.727	0.000	0.000	0.000	96.974
	9	1:DEAD LOAD	-232.660	-1.761	0.000	0.000	0.000	-55.365
		11:DL + LL1	-255.375	-1.663	0.000	0.000	0.000	-85.025
		12:DL + LL2	-263.102	-2.486	0.000	0.000	0.000	-55.899
		13:DL + LL3	-269.630	0.519	0.000	0.000	0.000	-113.450
		14:DL + LL4	-248.846	-4.668	0.000	0.000	0.000	-27.474
		15:DL + LL5	-271.561	-4.571	0.000	0.000	0.000	-57.135
		16:DL + LL6	-246.915	0.422	0.000	0.000	0.000	-83.790
		17:DL + LL7	-285.817	-2.388	0.000	0.000	0.000	-85.559
9	9	1:DEAD LOAD	232.110	7.617	0.000	0.000	0.000	55.365
		11:DL + LL1	254.818	8.202	0.000	0.000	0.000	85.025
		12:DL + LL2	262.516	9.257	0.000	0.000	0.000	55.899
		13:DL + LL3	269.133	6.449	0.000	0.000	0.000	113.450
		14:DL + LL4	248.202	11.010	0.000	0.000	0.000	27.474
		15:DL + LL5	270.910	11.594	0.000	0.000	0.000	57.135
		16:DL + LL6	246.425	5.864	0.000	0.000	0.000	83.790
		17:DL + LL7	285.224	9.841	0.000	0.000	0.000	85.559
	10	1:DEAD LOAD	-231.302	-5.278	0.000	0.000	0.000	-34.901
		11:DL + LL1	-254.010	-5.863	0.000	0.000	0.000	-62.706
		12:DL + LL2	-261.709	-6.918	0.000	0.000	0.000	-30.232
		13:DL + LL3	-268.325	-4.110	0.000	0.000	0.000	-96.694
		14:DL + LL4	-247.394	-8.671	0.000	0.000	0.000	3.757

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
		15:DL + LL5	-270.102	-9.256	0.000	0.000	0.000	-24.047
		16:DL + LL6	-245.617	-3.525	0.000	0.000	0.000	-68.890
		17:DL + LL7	-284.417	-7.503	0.000	0.000	0.000	-58.036
10	10	1:DEAD LOAD	231.043	12.163	0.000	0.000	0.000	34.901
		11:DL + LL1	253.723	13.423	0.000	0.000	0.000	62.706
		12:DL + LL2	261.387	14.707	0.000	0.000	0.000	30.232
		13:DL + LL3	268.084	12.097	0.000	0.000	0.000	96.694
		14:DL + LL4	247.026	16.033	0.000	0.000	0.000	-3.757
		15:DL + LL5	269.707	17.294	0.000	0.000	0.000	24.047
		16:DL + LL6	245.403	10.837	0.000	0.000	0.000	68.890
		17:DL + LL7	284.067	15.968	0.000	0.000	0.000	58.036
	11	1:DEAD LOAD	-230.312	-9.824	0.000	0.000	0.000	-0.350
		11:DL + LL1	-252.993	-11.085	0.000	0.000	0.000	-24.192
		12:DL + LL2	-260.656	-12.368	0.000	0.000	0.000	12.316
		13:DL + LL3	-267.353	-9.759	0.000	0.000	0.000	-62.349
		14:DL + LL4	-246.296	-13.695	0.000	0.000	0.000	50.473
		15:DL + LL5	-268.976	-14.955	0.000	0.000	0.000	26.631
		16:DL + LL6	-244.673	-8.498	0.000	0.000	0.000	-38.506
		17:DL + LL7	-283.337	-13.629	0.000	0.000	0.000	-11.526
11	11	1:DEAD LOAD	229.919	16.665	0.000	0.000	0.000	0.350
		11:DL + LL1	252.552	18.599	0.000	0.000	0.000	24.192
		12:DL + LL2	260.174	20.110	0.000	0.000	0.000	-12.316
		13:DL + LL3	266.945	17.700	0.000	0.000	0.000	62.349
		14:DL + LL4	245.780	21.008	0.000	0.000	0.000	-50.473
		15:DL + LL5	268.413	22.942	0.000	0.000	0.000	-26.631
		16:DL + LL6	244.312	15.766	0.000	0.000	0.000	38.506
		17:DL + LL7	282.807	22.044	0.000	0.000	0.000	11.526
	12	1:DEAD LOAD	-229.264	-14.326	0.000	0.000	0.000	47.925
		11:DL + LL1	-251.897	-16.260	0.000	0.000	0.000	30.108
		12:DL + LL2	-259.519	-17.771	0.000	0.000	0.000	71.323
		13:DL + LL3	-266.290	-15.362	0.000	0.000	0.000	-10.849
		14:DL + LL4	-245.125	-18.670	0.000	0.000	0.000	112.279
		15:DL + LL5	-267.758	-20.604	0.000	0.000	0.000	94.463
		16:DL + LL6	-243.658	-13.427	0.000	0.000	0.000	6.968
		17:DL + LL7	-282.152	-19.705	0.000	0.000	0.000	53.506
12	12	1:DEAD LOAD	228.746	21.033	0.000	0.000	0.000	-47.925
		11:DL + LL1	251.313	23.629	0.000	0.000	0.000	-30.108
		12:DL + LL2	258.887	25.363	0.000	0.000	0.000	-71.323
		13:DL + LL3	265.726	23.152	0.000	0.000	0.000	10.849
		14:DL + LL4	244.474	25.839	0.000	0.000	0.000	-112.279
		15:DL + LL5	267.040	28.435	0.000	0.000	0.000	-94.463
		16:DL + LL6	243.160	20.556	0.000	0.000	0.000	-6.968
		17:DL + LL7	281.454	27.959	0.000	0.000	0.000	-53.506
	13	1:DEAD LOAD	-228.165	-18.695	0.000	0.000	0.000	109.331

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
		11:DL + LL1	-250.731	-21.291	0.000	0.000	0.000	99.540
		12:DL + LL2	-258.306	-23.024	0.000	0.000	0.000	146.113
		13:DL + LL3	-265.145	-20.814	0.000	0.000	0.000	57.109
		14:DL + LL4	-243.892	-23.501	0.000	0.000	0.000	188.544
		15:DL + LL5	-266.459	-26.097	0.000	0.000	0.000	178.752
		16:DL + LL6	-242.578	-18.218	0.000	0.000	0.000	66.900
		17:DL + LL7	-280.872	-25.620	0.000	0.000	0.000	136.322
13	13	1:DEAD LOAD	215.772	-6.888	0.000	0.000	0.000	-109.331
		11:DL + LL1	237.426	-4.477	0.000	0.000	0.000	-99.540
		12:DL + LL2	243.259	-10.707	0.000	0.000	0.000	-146.113
		13:DL + LL3	252.421	-2.573	0.000	0.000	0.000	-57.109
		14:DL + LL4	228.264	-12.611	0.000	0.000	0.000	-188.544
		15:DL + LL5	249.918	-10.200	0.000	0.000	0.000	-178.752
		16:DL + LL6	230.767	-4.984	0.000	0.000	0.000	-66.900
		17:DL + LL7	264.913	-8.296	0.000	0.000	0.000	-136.322
	14	1:DEAD LOAD	-215.262	9.226	0.000	0.000	0.000	84.592
		11:DL + LL1	-236.916	6.816	0.000	0.000	0.000	82.203
		12:DL + LL2	-242.749	13.045	0.000	0.000	0.000	109.648
		13:DL + LL3	-251.911	4.911	0.000	0.000	0.000	45.619
		14:DL + LL4	-227.755	14.950	0.000	0.000	0.000	146.231
		15:DL + LL5	-249.409	12.539	0.000	0.000	0.000	143.842
		16:DL + LL6	-230.257	7.322	0.000	0.000	0.000	48.008
		17:DL + LL7	-264.403	10.635	0.000	0.000	0.000	107.258
14	14	1:DEAD LOAD	215.440	-2.952	0.000	0.000	0.000	-84.592
		11:DL + LL1	237.014	0.088	0.000	0.000	0.000	-82.203
		12:DL + LL2	243.026	-5.969	0.000	0.000	0.000	-109.648
		13:DL + LL3	251.947	2.429	0.000	0.000	0.000	-45.619
		14:DL + LL4	228.094	-8.309	0.000	0.000	0.000	-146.231
		15:DL + LL5	249.668	-5.269	0.000	0.000	0.000	-143.842
		16:DL + LL6	230.372	-0.612	0.000	0.000	0.000	-48.008
		17:DL + LL7	264.601	-2.928	0.000	0.000	0.000	-107.258
	15	1:DEAD LOAD	-215.001	5.291	0.000	0.000	0.000	72.012
		11:DL + LL1	-236.575	2.250	0.000	0.000	0.000	78.904
		12:DL + LL2	-242.587	8.307	0.000	0.000	0.000	87.859
		13:DL + LL3	-251.508	-0.090	0.000	0.000	0.000	49.464
		14:DL + LL4	-227.655	10.648	0.000	0.000	0.000	117.299
		15:DL + LL5	-249.229	7.607	0.000	0.000	0.000	124.191
		16:DL + LL6	-229.933	2.950	0.000	0.000	0.000	42.572
		17:DL + LL7	-264.162	5.267	0.000	0.000	0.000	94.751
15	15	1:DEAD LOAD	204.452	-1.638	0.000	0.000	0.000	-72.012
		11:DL + LL1	224.179	1.114	0.000	0.000	0.000	-78.904
		12:DL + LL2	230.955	-4.863	0.000	0.000	0.000	-87.859
		13:DL + LL3	238.966	1.533	0.000	0.000	0.000	-49.464
		14:DL + LL4	216.169	-5.282	0.000	0.000	0.000	-117.299

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
		15:DL + LL5	235.896	-2.530	0.000	0.000	0.000	-124.191
		16:DL + LL6	219.238	-1.219	0.000	0.000	0.000	-42.572
		17:DL + LL7	250.683	-2.111	0.000	0.000	0.000	-94.751
	16	1:DEAD LOAD	-204.082	3.976	0.000	0.000	0.000	63.486
		11:DL + LL1	-223.810	1.224	0.000	0.000	0.000	78.736
		12:DL + LL2	-230.586	7.201	0.000	0.000	0.000	69.538
		13:DL + LL3	-238.596	0.805	0.000	0.000	0.000	50.569
		14:DL + LL4	-215.799	7.621	0.000	0.000	0.000	97.705
		15:DL + LL5	-235.526	4.869	0.000	0.000	0.000	112.955
		16:DL + LL6	-218.869	3.557	0.000	0.000	0.000	35.319
		17:DL + LL7	-250.313	4.450	0.000	0.000	0.000	84.788
16	16	1:DEAD LOAD	204.113	1.823	0.000	0.000	0.000	-63.486
		11:DL + LL1	223.754	5.134	0.000	0.000	0.000	-78.736
		12:DL + LL2	230.697	-0.648	0.000	0.000	0.000	-69.538
		13:DL + LL3	238.523	5.973	0.000	0.000	0.000	-50.569
		14:DL + LL4	215.929	-1.487	0.000	0.000	0.000	-97.705
		15:DL + LL5	235.570	1.824	0.000	0.000	0.000	-112.955
		16:DL + LL6	218.882	2.662	0.000	0.000	0.000	-35.319
		17:DL + LL7	250.339	2.663	0.000	0.000	0.000	-84.788
	17	1:DEAD LOAD	-203.811	0.515	0.000	0.000	0.000	65.464
		11:DL + LL1	-223.452	-2.796	0.000	0.000	0.000	90.730
		12:DL + LL2	-230.396	2.986	0.000	0.000	0.000	64.042
		13:DL + LL3	-238.221	-3.635	0.000	0.000	0.000	65.101
		14:DL + LL4	-215.627	3.825	0.000	0.000	0.000	89.671
		15:DL + LL5	-235.268	0.514	0.000	0.000	0.000	114.936
		16:DL + LL6	-218.580	-0.324	0.000	0.000	0.000	39.835
		17:DL + LL7	-250.037	-0.325	0.000	0.000	0.000	89.308
17	17	1:DEAD LOAD	185.409	-1.316	0.000	0.000	0.000	-65.464
		11:DL + LL1	201.507	0.489	0.000	0.000	0.000	-90.730
		12:DL + LL2	210.698	-3.545	0.000	0.000	0.000	-64.042
		13:DL + LL3	216.014	1.467	0.000	0.000	0.000	-65.101
		14:DL + LL4	196.191	-4.524	0.000	0.000	0.000	-89.671
		15:DL + LL5	212.289	-2.719	0.000	0.000	0.000	-114.936
		16:DL + LL6	199.916	-0.337	0.000	0.000	0.000	-39.835
		17:DL + LL7	226.796	-1.741	0.000	0.000	0.000	-89.308
	18	1:DEAD LOAD	-185.176	3.654	0.000	0.000	0.000	57.973
		11:DL + LL1	-201.274	1.850	0.000	0.000	0.000	88.678
		12:DL + LL2	-210.465	5.884	0.000	0.000	0.000	49.828
		13:DL + LL3	-215.781	0.871	0.000	0.000	0.000	65.999
		14:DL + LL4	-195.958	6.862	0.000	0.000	0.000	72.507
		15:DL + LL5	-212.056	5.058	0.000	0.000	0.000	103.212
		16:DL + LL6	-199.683	2.676	0.000	0.000	0.000	35.294
		17:DL + LL7	-226.563	4.080	0.000	0.000	0.000	80.534
18	18	1:DEAD LOAD	185.205	1.555	0.000	0.000	0.000	-57.973

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
		11:DL + LL1	201.246	3.811	0.000	0.000	0.000	-88.678
		12:DL + LL2	210.547	0.037	0.000	0.000	0.000	-49.828
		13:DL + LL3	215.720	5.197	0.000	0.000	0.000	-65.999
		14:DL + LL4	196.074	-1.349	0.000	0.000	0.000	-72.507
		15:DL + LL5	212.115	0.907	0.000	0.000	0.000	-103.212
		16:DL + LL6	199.679	2.941	0.000	0.000	0.000	-35.294
		17:DL + LL7	226.588	2.293	0.000	0.000	0.000	-80.534
	19	1:DEAD LOAD	-185.038	0.784	0.000	0.000	0.000	59.132
		11:DL + LL1	-201.079	-1.473	0.000	0.000	0.000	96.623
		12:DL + LL2	-210.381	2.301	0.000	0.000	0.000	46.424
		13:DL + LL3	-215.553	-2.859	0.000	0.000	0.000	78.113
		14:DL + LL4	-195.907	3.687	0.000	0.000	0.000	64.934
		15:DL + LL5	-211.948	1.431	0.000	0.000	0.000	102.425
		16:DL + LL6	-199.512	-0.602	0.000	0.000	0.000	40.622
		17:DL + LL7	-226.422	0.045	0.000	0.000	0.000	83.915
19	19	1:DEAD LOAD	165.942	0.246	0.000	0.000	0.000	-59.132
		11:DL + LL1	178.281	0.744	0.000	0.000	0.000	-96.623
		12:DL + LL2	190.042	-0.695	0.000	0.000	0.000	-46.424
		13:DL + LL3	191.558	2.374	0.000	0.000	0.000	-78.113
		14:DL + LL4	176.765	-2.325	0.000	0.000	0.000	-64.934
		15:DL + LL5	189.104	-1.827	0.000	0.000	0.000	-102.425
		16:DL + LL6	179.219	1.877	0.000	0.000	0.000	-40.622
		17:DL + LL7	202.381	-0.197	0.000	0.000	0.000	-83.915
	20	1:DEAD LOAD	-165.843	2.092	0.000	0.000	0.000	56.361
		11:DL + LL1	-178.182	1.595	0.000	0.000	0.000	95.345
		12:DL + LL2	-189.943	3.033	0.000	0.000	0.000	40.827
		13:DL + LL3	-191.459	-0.035	0.000	0.000	0.000	81.730
		14:DL + LL4	-176.666	4.663	0.000	0.000	0.000	54.443
		15:DL + LL5	-189.005	4.166	0.000	0.000	0.000	93.427
		16:DL + LL6	-179.120	0.462	0.000	0.000	0.000	42.746
		17:DL + LL7	-202.282	2.536	0.000	0.000	0.000	79.811
20	20	1:DEAD LOAD	165.836	2.548	0.000	0.000	0.000	-56.361
		11:DL + LL1	178.157	3.390	0.000	0.000	0.000	-95.345
		12:DL + LL2	189.953	2.281	0.000	0.000	0.000	-40.827
		13:DL + LL3	191.383	5.391	0.000	0.000	0.000	-81.730
		14:DL + LL4	176.727	0.280	0.000	0.000	0.000	-54.443
		15:DL + LL5	189.047	1.122	0.000	0.000	0.000	-93.427
		16:DL + LL6	179.063	4.548	0.000	0.000	0.000	-42.746
		17:DL + LL7	202.274	3.123	0.000	0.000	0.000	-79.811
	21	1:DEAD LOAD	-165.803	-0.209	0.000	0.000	0.000	60.496
		11:DL + LL1	-178.123	-1.051	0.000	0.000	0.000	102.007
		12:DL + LL2	-189.920	0.057	0.000	0.000	0.000	44.163
		13:DL + LL3	-191.350	-3.052	0.000	0.000	0.000	94.396
		14:DL + LL4	-176.693	2.058	0.000	0.000	0.000	51.775

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip¯ft)	My (kip¯ft)	Mz (kip¯ft)
		15:DL + LL5	-189.014	1.216	0.000	0.000	0.000	93.286
		16:DL + LL6	-179.029	-2.210	0.000	0.000	0.000	52.884
		17:DL + LL7	-202.240	-0.785	0.000	0.000	0.000	85.674

Software licensed to TranSystems	Job No	Sheet No 1	Rev
	Part		
Job Title LAKE PARK ARCH BRIDGE LOAD RATING	Ref		
	By DWC	Date 02-Aug-18	Chd SFH
Client	File Lake Park Arch - H5 Truck	Date/Time 07-Aug-2018 07:51	

Job Information

	Engineer	Checked	Approved
Name:	DWC	SFH	
Date:	02-Aug-18	03-Aug-18	

Project ID	
Project Name	

Comments

ARCH RIBS - H5 TRUCK

Structure Type	SPACE FRAME

Number of Nodes	54	Highest Node	62
Number of Elements	65	Highest Beam	81

Number of Basic Load Cases	-2
Number of Combination Load Cases	0

Included in this printout are data for:

Beams	1 to 4

Included in this printout are results for load cases:

Type	L/C	Name
Generation	1	LOAD GENERATION, LOAD #1, (1 of 21)
Generation	2	LOAD GENERATION, LOAD #2, (2 of 21)
Generation	3	LOAD GENERATION, LOAD #3, (3 of 21)
Generation	4	LOAD GENERATION, LOAD #4, (4 of 21)
Generation	5	LOAD GENERATION, LOAD #5, (5 of 21)
Generation	6	LOAD GENERATION, LOAD #6, (6 of 21)
Generation	7	LOAD GENERATION, LOAD #7, (7 of 21)
Generation	8	LOAD GENERATION, LOAD #8, (8 of 21)
Generation	9	LOAD GENERATION, LOAD #9, (9 of 21)
Generation	10	LOAD GENERATION, LOAD #10, (10 of 2
Generation	11	LOAD GENERATION, LOAD #11, (11 of 21
Generation	12	LOAD GENERATION, LOAD #12, (12 of 2
Generation	13	LOAD GENERATION, LOAD #13, (13 of 2
Generation	14	LOAD GENERATION, LOAD #14, (14 of 2
Generation	15	LOAD GENERATION, LOAD #15, (15 of 2
Generation	16	LOAD GENERATION, LOAD #16, (16 of 2
Generation	17	LOAD GENERATION, LOAD #17, (17 of 2
Generation	18	LOAD GENERATION, LOAD #18, (18 of 2
Generation	19	LOAD GENERATION, LOAD #19, (19 of 2
Generation	20	LOAD GENERATION, LOAD #20, (20 of 2
Generation	21	LOAD GENERATION, LOAD #21, (21 of 2
Generation	22	LOAD GENERATION, LOAD #22, (1 of 21)

Job Information Cont...

Type	L/C	Name
Generation	26	LOAD GENERATION, LOAD #26, (5 of 21)
Generation	27	LOAD GENERATION, LOAD #27, (6 of 21)
Generation	28	LOAD GENERATION, LOAD #28, (7 of 21)
Generation	29	LOAD GENERATION, LOAD #29, (8 of 21)
Generation	30	LOAD GENERATION, LOAD #30, (9 of 21)
Generation	31	LOAD GENERATION, LOAD #31, (10 of 2
Generation	32	LOAD GENERATION, LOAD #32, (11 of 21
Generation	33	LOAD GENERATION, LOAD #33, (12 of 2
Generation	34	LOAD GENERATION, LOAD #34, (13 of 2
Generation	35	LOAD GENERATION, LOAD #35, (14 of 2
Generation	36	LOAD GENERATION, LOAD #36, (15 of 2
Generation	37	LOAD GENERATION, LOAD #37, (16 of 2
Generation	38	LOAD GENERATION, LOAD #38, (17 of 2
Generation	39	LOAD GENERATION, LOAD #39, (18 of 2
Generation	40	LOAD GENERATION, LOAD #40, (19 of 2
Generation	41	LOAD GENERATION, LOAD #41, (20 of 2
Generation	42	LOAD GENERATION, LOAD #42, (21 of 2

Beam End Forces

Sign convention is as the action of the joint on the beam.

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip`ft)	My (kip`ft)	Mz (kip`ft)
1	1	1:LOAD GENE	0.881	0.734	0.000	0.000	0.000	7.372
		2:LOAD GENE	2.470	2.082	0.000	0.000	0.000	20.817
		3:LOAD GENE	4.059	3.430	0.000	0.000	0.000	34.265
		4:LOAD GENE	4.949	3.107	0.000	0.000	0.000	34.736
		5:LOAD GENE	5.664	2.367	0.000	0.000	0.000	31.963
		6:LOAD GENE	6.337	1.637	0.000	0.000	0.000	28.180
		7:LOAD GENE	6.979	0.920	0.000	0.000	0.000	24.149
		8:LOAD GENE	7.592	0.219	0.000	0.000	0.000	20.184
		9:LOAD GENE	7.955	-0.406	0.000	0.000	0.000	11.320
		10:LOAD GENI	8.191	-0.961	0.000	0.000	0.000	2.701
		11:LOAD GENI	8.275	-1.425	0.000	0.000	0.000	-5.307
		12:LOAD GENI	8.204	-1.794	0.000	0.000	0.000	-12.358
		13:LOAD GENI	7.975	-2.061	0.000	0.000	0.000	-18.201
		14:LOAD GENI	7.590	-2.223	0.000	0.000	0.000	-22.646
		15:LOAD GENI	7.056	-2.278	0.000	0.000	0.000	-25.515
		16:LOAD GENI	6.380	-2.217	0.000	0.000	0.000	-26.460
		17:LOAD GENI	5.597	-2.074	0.000	0.000	0.000	-26.107
		18:LOAD GENI	4.710	-1.824	0.000	0.000	0.000	-23.756
		19:LOAD GENI	3.743	-1.470	0.000	0.000	0.000	-19.395
		20:LOAD GENI	2.817	-1.128	0.000	0.000	0.000	-15.128
		21:LOAD GENI	1.953	-0.803	0.000	0.000	0.000	-11.001

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip·ft)	My (kip·ft)	Mz (kip·ft)
		22:LOAD GENI	3.525	2.936	0.000	0.000	0.000	29.476
		23:LOAD GENI	4.458	2.718	0.000	0.000	0.000	30.757
		24:LOAD GENI	5.392	2.499	0.000	0.000	0.000	32.040
		25:LOAD GENI	6.151	1.863	0.000	0.000	0.000	30.078
		26:LOAD GENI	6.866	1.122	0.000	0.000	0.000	27.305
		27:LOAD GENI	7.410	0.424	0.000	0.000	0.000	20.491
		28:LOAD GENI	7.834	-0.222	0.000	0.000	0.000	12.684
		29:LOAD GENI	8.140	-0.801	0.000	0.000	0.000	5.141
		30:LOAD GENI	8.269	-1.292	0.000	0.000	0.000	-3.060
		31:LOAD GENI	8.245	-1.691	0.000	0.000	0.000	-10.393
		32:LOAD GENI	8.062	-1.988	0.000	0.000	0.000	-16.588
		33:LOAD GENI	7.723	-2.181	0.000	0.000	0.000	-21.450
		34:LOAD GENI	7.234	-2.272	0.000	0.000	0.000	-24.855
		35:LOAD GENI	6.604	-2.260	0.000	0.000	0.000	-26.697
		36:LOAD GENI	5.847	-2.139	0.000	0.000	0.000	-26.682
		37:LOAD GENI	4.953	-1.858	0.000	0.000	0.000	-23.647
		38:LOAD GENI	4.032	-1.558	0.000	0.000	0.000	-20.288
		39:LOAD GENI	3.085	-1.230	0.000	0.000	0.000	-16.429
		40:LOAD GENI	2.118	-0.877	0.000	0.000	0.000	-12.068
		41:LOAD GENI	1.316	-0.568	0.000	0.000	0.000	-8.083
		42:LOAD GENI	0.760	-0.328	0.000	0.000	0.000	-4.662
	2	1:LOAD GENE	-0.881	-0.734	0.000	0.000	0.000	-4.962
		2:LOAD GENE	-2.470	-2.082	0.000	0.000	0.000	-13.986
		3:LOAD GENE	-4.059	-3.430	0.000	0.000	0.000	-23.011
		4:LOAD GENE	-4.949	-3.107	0.000	0.000	0.000	-24.542
		5:LOAD GENE	-5.664	-2.367	0.000	0.000	0.000	-24.199
		6:LOAD GENE	-6.337	-1.637	0.000	0.000	0.000	-22.811
		7:LOAD GENE	-6.979	-0.920	0.000	0.000	0.000	-21.132
		8:LOAD GENE	-7.592	-0.219	0.000	0.000	0.000	-19.464
		9:LOAD GENE	-7.955	0.406	0.000	0.000	0.000	-12.652
		10:LOAD GENI	-8.191	0.961	0.000	0.000	0.000	-5.853
		11:LOAD GENI	-8.275	1.425	0.000	0.000	0.000	0.631
		12:LOAD GENI	-8.204	1.794	0.000	0.000	0.000	6.473
		13:LOAD GENI	-7.975	2.061	0.000	0.000	0.000	11.440
		14:LOAD GENI	-7.590	2.223	0.000	0.000	0.000	15.353
		15:LOAD GENI	-7.056	2.278	0.000	0.000	0.000	18.040
		16:LOAD GENI	-6.380	2.217	0.000	0.000	0.000	19.188
		17:LOAD GENI	-5.597	2.074	0.000	0.000	0.000	19.301
		18:LOAD GENI	-4.710	1.824	0.000	0.000	0.000	17.772
		19:LOAD GENI	-3.743	1.470	0.000	0.000	0.000	14.571
		20:LOAD GENI	-2.817	1.128	0.000	0.000	0.000	11.426
		21:LOAD GENI	-1.953	0.803	0.000	0.000	0.000	8.366
		22:LOAD GENI	-3.525	-2.936	0.000	0.000	0.000	-19.842
		23:LOAD GENI	-4.458	-2.718	0.000	0.000	0.000	-21.841

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
		24:LOAD GENI	-5.392	-2.499	0.000	0.000	0.000	-23.840
		25:LOAD GENI	-6.151	-1.863	0.000	0.000	0.000	-23.966
		26:LOAD GENI	-6.866	-1.122	0.000	0.000	0.000	-23.623
		27:LOAD GENI	-7.410	-0.424	0.000	0.000	0.000	-19.099
		28:LOAD GENI	-7.834	0.222	0.000	0.000	0.000	-13.411
		29:LOAD GENI	-8.140	0.801	0.000	0.000	0.000	-7.768
		30:LOAD GENI	-8.269	1.292	0.000	0.000	0.000	-1.179
		31:LOAD GENI	-8.245	1.691	0.000	0.000	0.000	4.846
		32:LOAD GENI	-8.062	1.988	0.000	0.000	0.000	10.068
		33:LOAD GENI	-7.723	2.181	0.000	0.000	0.000	14.294
		34:LOAD GENI	-7.234	2.272	0.000	0.000	0.000	17.401
		35:LOAD GENI	-6.604	2.260	0.000	0.000	0.000	19.283
		36:LOAD GENI	-5.847	2.139	0.000	0.000	0.000	19.664
		37:LOAD GENI	-4.953	1.858	0.000	0.000	0.000	17.550
		38:LOAD GENI	-4.032	1.558	0.000	0.000	0.000	15.177
		39:LOAD GENI	-3.085	1.230	0.000	0.000	0.000	12.393
		40:LOAD GENI	-2.118	0.877	0.000	0.000	0.000	9.192
		41:LOAD GENI	-1.316	0.568	0.000	0.000	0.000	6.218
		42:LOAD GENI	-0.760	0.328	0.000	0.000	0.000	3.585
2	2	1:LOAD GENE	0.859	0.761	0.000	0.000	0.000	4.962
		2:LOAD GENE	2.405	2.157	0.000	0.000	0.000	13.986
		3:LOAD GENE	3.952	3.554	0.000	0.000	0.000	23.011
		4:LOAD GENE	4.851	3.258	0.000	0.000	0.000	24.542
		5:LOAD GENE	5.589	2.540	0.000	0.000	0.000	24.199
		6:LOAD GENE	6.283	1.831	0.000	0.000	0.000	22.811
		7:LOAD GENE	6.947	1.134	0.000	0.000	0.000	21.132
		8:LOAD GENE	7.581	0.453	0.000	0.000	0.000	19.464
		9:LOAD GENE	7.963	-0.161	0.000	0.000	0.000	12.652
		10:LOAD GENI	8.217	-0.708	0.000	0.000	0.000	5.853
		11:LOAD GENI	8.315	-1.170	0.000	0.000	0.000	-0.631
		12:LOAD GENI	8.255	-1.541	0.000	0.000	0.000	-6.473
		13:LOAD GENI	8.035	-1.815	0.000	0.000	0.000	-11.440
		14:LOAD GENI	7.655	-1.989	0.000	0.000	0.000	-15.353
		15:LOAD GENI	7.123	-2.060	0.000	0.000	0.000	-18.040
		16:LOAD GENI	6.445	-2.019	0.000	0.000	0.000	-19.188
		17:LOAD GENI	5.658	-1.901	0.000	0.000	0.000	-19.301
		18:LOAD GENI	4.763	-1.678	0.000	0.000	0.000	-17.772
		19:LOAD GENI	3.786	-1.355	0.000	0.000	0.000	-14.571
		20:LOAD GENI	2.850	-1.041	0.000	0.000	0.000	-11.426
		21:LOAD GENI	1.976	-0.743	0.000	0.000	0.000	-8.366
		22:LOAD GENI	3.433	3.043	0.000	0.000	0.000	19.842
		23:LOAD GENI	4.373	2.854	0.000	0.000	0.000	21.841
		24:LOAD GENI	5.312	2.664	0.000	0.000	0.000	23.840
		25:LOAD GENI	6.090	2.051	0.000	0.000	0.000	23.966

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
		26:LOAD GENI	6.828	1.333	0.000	0.000	0.000	23.623
		27:LOAD GENI	7.393	0.652	0.000	0.000	0.000	19.099
		28:LOAD GENI	7.837	0.019	0.000	0.000	0.000	13.411
		29:LOAD GENI	8.161	-0.550	0.000	0.000	0.000	7.768
		30:LOAD GENI	8.305	-1.037	0.000	0.000	0.000	1.179
		31:LOAD GENI	8.293	-1.436	0.000	0.000	0.000	-4.846
		32:LOAD GENI	8.120	-1.739	0.000	0.000	0.000	-10.068
		33:LOAD GENI	7.786	-1.943	0.000	0.000	0.000	-14.294
		34:LOAD GENI	7.301	-2.048	0.000	0.000	0.000	-17.401
		35:LOAD GENI	6.671	-2.056	0.000	0.000	0.000	-19.283
		36:LOAD GENI	5.910	-1.958	0.000	0.000	0.000	-19.664
		37:LOAD GENI	5.008	-1.705	0.000	0.000	0.000	-17.550
		38:LOAD GENI	4.078	-1.433	0.000	0.000	0.000	-15.177
		39:LOAD GENI	3.121	-1.135	0.000	0.000	0.000	-12.393
		40:LOAD GENI	2.144	-0.811	0.000	0.000	0.000	-9.192
		41:LOAD GENI	1.332	-0.528	0.000	0.000	0.000	-6.218
		42:LOAD GENI	0.770	-0.305	0.000	0.000	0.000	-3.585
	3	1:LOAD GENE	-0.859	-0.761	0.000	0.000	0.000	-2.513
		2:LOAD GENE	-2.405	-2.157	0.000	0.000	0.000	-7.044
		3:LOAD GENE	-3.952	-3.554	0.000	0.000	0.000	-11.576
		4:LOAD GENE	-4.851	-3.258	0.000	0.000	0.000	-14.058
		5:LOAD GENE	-5.589	-2.540	0.000	0.000	0.000	-16.027
		6:LOAD GENE	-6.283	-1.831	0.000	0.000	0.000	-16.920
		7:LOAD GENE	-6.947	-1.134	0.000	0.000	0.000	-17.484
		8:LOAD GENE	-7.581	-0.453	0.000	0.000	0.000	-18.008
		9:LOAD GENE	-7.963	0.161	0.000	0.000	0.000	-13.172
		10:LOAD GENI	-8.217	0.708	0.000	0.000	0.000	-8.132
		11:LOAD GENI	-8.315	1.170	0.000	0.000	0.000	-3.136
		12:LOAD GENI	-8.255	1.541	0.000	0.000	0.000	1.515
		13:LOAD GENI	-8.035	1.815	0.000	0.000	0.000	5.600
		14:LOAD GENI	-7.655	1.989	0.000	0.000	0.000	8.954
		15:LOAD GENI	-7.123	2.060	0.000	0.000	0.000	11.410
		16:LOAD GENI	-6.445	2.019	0.000	0.000	0.000	12.689
		17:LOAD GENI	-5.658	1.901	0.000	0.000	0.000	13.183
		18:LOAD GENI	-4.763	1.678	0.000	0.000	0.000	12.372
		19:LOAD GENI	-3.786	1.355	0.000	0.000	0.000	10.211
		20:LOAD GENI	-2.850	1.041	0.000	0.000	0.000	8.075
		21:LOAD GENI	-1.976	0.743	0.000	0.000	0.000	5.976
		22:LOAD GENI	-3.433	-3.043	0.000	0.000	0.000	-10.049
		23:LOAD GENI	-4.373	-2.854	0.000	0.000	0.000	-12.659
		24:LOAD GENI	-5.312	-2.664	0.000	0.000	0.000	-15.269
		25:LOAD GENI	-6.090	-2.051	0.000	0.000	0.000	-17.366
		26:LOAD GENI	-6.828	-1.333	0.000	0.000	0.000	-19.335
		27:LOAD GENI	-7.393	-0.652	0.000	0.000	0.000	-17.001

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip`ft)	My (kip`ft)	Mz (kip`ft)
		28:LOAD GENI	-7.837	-0.019	0.000	0.000	0.000	-13.349
		29:LOAD GENI	-8.161	0.550	0.000	0.000	0.000	-9.538
		30:LOAD GENI	-8.305	1.037	0.000	0.000	0.000	-4.517
		31:LOAD GENI	-8.293	1.436	0.000	0.000	0.000	0.225
		32:LOAD GENI	-8.120	1.739	0.000	0.000	0.000	4.473
		33:LOAD GENI	-7.786	1.943	0.000	0.000	0.000	8.043
		34:LOAD GENI	-7.301	2.048	0.000	0.000	0.000	10.810
		35:LOAD GENI	-6.671	2.056	0.000	0.000	0.000	12.667
		36:LOAD GENI	-5.910	1.958	0.000	0.000	0.000	13.363
		37:LOAD GENI	-5.008	1.705	0.000	0.000	0.000	12.062
		38:LOAD GENI	-4.078	1.433	0.000	0.000	0.000	10.565
		39:LOAD GENI	-3.121	1.135	0.000	0.000	0.000	8.742
		40:LOAD GENI	-2.144	0.811	0.000	0.000	0.000	6.581
		41:LOAD GENI	-1.332	0.528	0.000	0.000	0.000	4.519
		42:LOAD GENI	-0.770	0.305	0.000	0.000	0.000	2.605
3	3	1:LOAD GENE	0.835	0.787	0.000	0.000	0.000	2.513
		2:LOAD GENE	2.339	2.229	0.000	0.000	0.000	7.044
		3:LOAD GENE	3.843	3.672	0.000	0.000	0.000	11.576
		4:LOAD GENE	4.751	3.403	0.000	0.000	0.000	14.058
		5:LOAD GENE	5.509	2.708	0.000	0.000	0.000	16.027
		6:LOAD GENE	6.225	2.020	0.000	0.000	0.000	16.920
		7:LOAD GENE	6.910	1.343	0.000	0.000	0.000	17.484
		8:LOAD GENE	7.564	0.682	0.000	0.000	0.000	18.008
		9:LOAD GENE	7.965	0.080	0.000	0.000	0.000	13.172
		10:LOAD GENI	8.234	-0.459	0.000	0.000	0.000	8.132
		11:LOAD GENI	8.347	-0.918	0.000	0.000	0.000	3.136
		12:LOAD GENI	8.298	-1.290	0.000	0.000	0.000	-1.515
		13:LOAD GENI	8.086	-1.571	0.000	0.000	0.000	-5.600
		14:LOAD GENI	7.711	-1.756	0.000	0.000	0.000	-8.954
		15:LOAD GENI	7.182	-1.844	0.000	0.000	0.000	-11.410
		16:LOAD GENI	6.503	-1.823	0.000	0.000	0.000	-12.689
		17:LOAD GENI	5.713	-1.729	0.000	0.000	0.000	-13.183
		18:LOAD GENI	4.812	-1.533	0.000	0.000	0.000	-12.372
		19:LOAD GENI	3.825	-1.239	0.000	0.000	0.000	-10.211
		20:LOAD GENI	2.880	-0.954	0.000	0.000	0.000	-8.075
		21:LOAD GENI	1.998	-0.683	0.000	0.000	0.000	-5.976
		22:LOAD GENI	3.339	3.146	0.000	0.000	0.000	10.049
		23:LOAD GENI	4.284	2.985	0.000	0.000	0.000	12.659
		24:LOAD GENI	5.229	2.823	0.000	0.000	0.000	15.269
		25:LOAD GENI	6.025	2.235	0.000	0.000	0.000	17.366
		26:LOAD GENI	6.784	1.539	0.000	0.000	0.000	19.335
		27:LOAD GENI	7.370	0.875	0.000	0.000	0.000	17.001
		28:LOAD GENI	7.833	0.257	0.000	0.000	0.000	13.349
		29:LOAD GENI	8.174	-0.303	0.000	0.000	0.000	9.538

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
		30:LOAD GENI	8.332	-0.785	0.000	0.000	0.000	4.517
		31:LOAD GENI	8.333	-1.185	0.000	0.000	0.000	-0.225
		32:LOAD GENI	8.168	-1.492	0.000	0.000	0.000	-4.473
		33:LOAD GENI	7.842	-1.706	0.000	0.000	0.000	-8.043
		34:LOAD GENI	7.359	-1.826	0.000	0.000	0.000	-10.810
		35:LOAD GENI	6.730	-1.853	0.000	0.000	0.000	-12.667
		36:LOAD GENI	5.967	-1.778	0.000	0.000	0.000	-13.363
		37:LOAD GENI	5.057	-1.553	0.000	0.000	0.000	-12.062
		38:LOAD GENI	4.120	-1.309	0.000	0.000	0.000	-10.565
		39:LOAD GENI	3.154	-1.040	0.000	0.000	0.000	-8.742
		40:LOAD GENI	2.168	-0.746	0.000	0.000	0.000	-6.581
		41:LOAD GENI	1.348	-0.487	0.000	0.000	0.000	-4.519
		42:LOAD GENI	0.778	-0.281	0.000	0.000	0.000	-2.605
	4	1:LOAD GENE	-0.835	-0.787	0.000	0.000	0.000	-0.026
		2:LOAD GENE	-2.339	-2.229	0.000	0.000	0.000	0.002
		3:LOAD GENE	-3.843	-3.672	0.000	0.000	0.000	0.030
		4:LOAD GENE	-4.751	-3.403	0.000	0.000	0.000	-3.299
		5:LOAD GENE	-5.509	-2.708	0.000	0.000	0.000	-7.467
		6:LOAD GENE	-6.225	-2.020	0.000	0.000	0.000	-10.535
		7:LOAD GENE	-6.910	-1.343	0.000	0.000	0.000	-13.237
		8:LOAD GENE	-7.564	-0.682	0.000	0.000	0.000	-15.851
		9:LOAD GENE	-7.965	-0.080	0.000	0.000	0.000	-12.919
		10:LOAD GENI	-8.234	0.459	0.000	0.000	0.000	-9.583
		11:LOAD GENI	-8.347	0.918	0.000	0.000	0.000	-6.038
		12:LOAD GENI	-8.298	1.290	0.000	0.000	0.000	-2.564
		13:LOAD GENI	-8.086	1.571	0.000	0.000	0.000	0.635
		14:LOAD GENI	-7.711	1.756	0.000	0.000	0.000	3.403
		15:LOAD GENI	-7.182	1.844	0.000	0.000	0.000	5.582
		16:LOAD GENI	-6.503	1.823	0.000	0.000	0.000	6.925
		17:LOAD GENI	-5.713	1.729	0.000	0.000	0.000	7.717
		18:LOAD GENI	-4.812	1.533	0.000	0.000	0.000	7.525
		19:LOAD GENI	-3.825	1.239	0.000	0.000	0.000	6.293
		20:LOAD GENI	-2.880	0.954	0.000	0.000	0.000	5.058
		21:LOAD GENI	-1.998	0.683	0.000	0.000	0.000	3.819
		22:LOAD GENI	-3.339	-3.146	0.000	0.000	0.000	-0.104
		23:LOAD GENI	-4.284	-2.985	0.000	0.000	0.000	-3.224
		24:LOAD GENI	-5.229	-2.823	0.000	0.000	0.000	-6.343
		25:LOAD GENI	-6.025	-2.235	0.000	0.000	0.000	-10.302
		26:LOAD GENI	-6.784	-1.539	0.000	0.000	0.000	-14.471
		27:LOAD GENI	-7.370	-0.875	0.000	0.000	0.000	-14.233
		28:LOAD GENI	-7.833	-0.257	0.000	0.000	0.000	-12.537
		29:LOAD GENI	-8.174	0.303	0.000	0.000	0.000	-10.495
		30:LOAD GENI	-8.332	0.785	0.000	0.000	0.000	-7.000
		31:LOAD GENI	-8.333	1.185	0.000	0.000	0.000	-3.520

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip˜ft)	My (kip˜ft)	Mz (kip˜ft)
		32:LOAD GENI	-8.168	1.492	0.000	0.000	0.000	-0.244
		33:LOAD GENI	-7.842	1.706	0.000	0.000	0.000	2.650
		34:LOAD GENI	-7.359	1.826	0.000	0.000	0.000	5.036
		35:LOAD GENI	-6.730	1.853	0.000	0.000	0.000	6.809
		36:LOAD GENI	-5.967	1.778	0.000	0.000	0.000	7.741
		37:LOAD GENI	-5.057	1.553	0.000	0.000	0.000	7.153
		38:LOAD GENI	-4.120	1.309	0.000	0.000	0.000	6.427
		39:LOAD GENI	-3.154	1.040	0.000	0.000	0.000	5.455
		40:LOAD GENI	-2.168	0.746	0.000	0.000	0.000	4.224
		41:LOAD GENI	-1.348	0.487	0.000	0.000	0.000	2.979
		42:LOAD GENI	-0.778	0.281	0.000	0.000	0.000	1.717
4	4	1:LOAD GENE	0.811	0.811	0.000	0.000	0.000	0.026
		2:LOAD GENE	2.272	2.297	0.000	0.000	0.000	-0.002
		3:LOAD GENE	3.732	3.784	0.000	0.000	0.000	-0.030
		4:LOAD GENE	4.648	3.543	0.000	0.000	0.000	3.299
		5:LOAD GENE	5.427	2.870	0.000	0.000	0.000	7.467
		6:LOAD GENE	6.162	2.204	0.000	0.000	0.000	10.535
		7:LOAD GENE	6.867	1.548	0.000	0.000	0.000	13.237
		8:LOAD GENE	7.540	0.906	0.000	0.000	0.000	15.851
		9:LOAD GENE	7.959	0.316	0.000	0.000	0.000	12.919
		10:LOAD GENI	8.244	-0.215	0.000	0.000	0.000	9.583
		11:LOAD GENI	8.370	-0.670	0.000	0.000	0.000	6.038
		12:LOAD GENI	8.333	-1.044	0.000	0.000	0.000	2.564
		13:LOAD GENI	8.129	-1.330	0.000	0.000	0.000	-0.635
		14:LOAD GENI	7.760	-1.527	0.000	0.000	0.000	-3.403
		15:LOAD GENI	7.233	-1.630	0.000	0.000	0.000	-5.582
		16:LOAD GENI	6.554	-1.630	0.000	0.000	0.000	-6.925
		17:LOAD GENI	5.762	-1.559	0.000	0.000	0.000	-7.717
		18:LOAD GENI	4.855	-1.390	0.000	0.000	0.000	-7.525
		19:LOAD GENI	3.860	-1.126	0.000	0.000	0.000	-6.293
		20:LOAD GENI	2.907	-0.869	0.000	0.000	0.000	-5.058
		21:LOAD GENI	2.017	-0.623	0.000	0.000	0.000	-3.819
		22:LOAD GENI	3.244	3.244	0.000	0.000	0.000	0.104
		23:LOAD GENI	4.194	3.110	0.000	0.000	0.000	3.224
		24:LOAD GENI	5.143	2.977	0.000	0.000	0.000	6.343
		25:LOAD GENI	5.957	2.412	0.000	0.000	0.000	10.302
		26:LOAD GENI	6.736	1.739	0.000	0.000	0.000	14.471
		27:LOAD GENI	7.341	1.094	0.000	0.000	0.000	14.233
		28:LOAD GENI	7.822	0.489	0.000	0.000	0.000	12.537
		29:LOAD GENI	8.179	-0.060	0.000	0.000	0.000	10.495
		30:LOAD GENI	8.352	-0.538	0.000	0.000	0.000	7.000
		31:LOAD GENI	8.365	-0.937	0.000	0.000	0.000	3.520
		32:LOAD GENI	8.209	-1.249	0.000	0.000	0.000	0.244
		33:LOAD GENI	7.889	-1.473	0.000	0.000	0.000	-2.650

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip`ft)	My (kip`ft)	Mz (kip`ft)
		34:LOAD GENI	7.410	-1.607	0.000	0.000	0.000	-5.036
		35:LOAD GENI	6.782	-1.653	0.000	0.000	0.000	-6.809
		36:LOAD GENI	6.017	-1.601	0.000	0.000	0.000	-7.741
		37:LOAD GENI	5.101	-1.402	0.000	0.000	0.000	-7.153
		38:LOAD GENI	4.157	-1.186	0.000	0.000	0.000	-6.427
		39:LOAD GENI	3.184	-0.946	0.000	0.000	0.000	-5.455
		40:LOAD GENI	2.189	-0.681	0.000	0.000	0.000	-4.224
		41:LOAD GENI	1.362	-0.447	0.000	0.000	0.000	-2.979
		42:LOAD GENI	0.786	-0.258	0.000	0.000	0.000	-1.717
	5	1:LOAD GENE	-0.811	-0.811	0.000	0.000	0.000	2.497
		2:LOAD GENE	-2.272	-2.297	0.000	0.000	0.000	7.148
		3:LOAD GENE	-3.732	-3.784	0.000	0.000	0.000	11.799
		4:LOAD GENE	-4.648	-3.543	0.000	0.000	0.000	7.720
		5:LOAD GENE	-5.427	-2.870	0.000	0.000	0.000	1.459
		6:LOAD GENE	-6.162	-2.204	0.000	0.000	0.000	-3.680
		7:LOAD GENE	-6.867	-1.548	0.000	0.000	0.000	-8.423
		8:LOAD GENE	-7.540	-0.906	0.000	0.000	0.000	-13.034
		9:LOAD GENE	-7.959	-0.316	0.000	0.000	0.000	-11.937
		10:LOAD GENI	-8.244	0.215	0.000	0.000	0.000	-10.252
		11:LOAD GENI	-8.370	0.670	0.000	0.000	0.000	-8.123
		12:LOAD GENI	-8.333	1.044	0.000	0.000	0.000	-5.810
		13:LOAD GENI	-8.129	1.330	0.000	0.000	0.000	-3.503
		14:LOAD GENI	-7.760	1.527	0.000	0.000	0.000	-1.345
		15:LOAD GENI	-7.233	1.630	0.000	0.000	0.000	0.512
		16:LOAD GENI	-6.554	1.630	0.000	0.000	0.000	1.856
		17:LOAD GENI	-5.762	1.559	0.000	0.000	0.000	2.867
		18:LOAD GENI	-4.855	1.390	0.000	0.000	0.000	3.202
		19:LOAD GENI	-3.860	1.126	0.000	0.000	0.000	2.792
		20:LOAD GENI	-2.907	0.869	0.000	0.000	0.000	2.356
		21:LOAD GENI	-2.017	0.623	0.000	0.000	0.000	1.881
		22:LOAD GENI	-3.244	-3.244	0.000	0.000	0.000	9.984
		23:LOAD GENI	-4.194	-3.110	0.000	0.000	0.000	6.450
		24:LOAD GENI	-5.143	-2.977	0.000	0.000	0.000	2.917
		25:LOAD GENI	-5.957	-2.412	0.000	0.000	0.000	-2.799
		26:LOAD GENI	-6.736	-1.739	0.000	0.000	0.000	-9.061
		27:LOAD GENI	-7.341	-1.094	0.000	0.000	0.000	-10.832
		28:LOAD GENI	-7.822	-0.489	0.000	0.000	0.000	-11.017
		29:LOAD GENI	-8.179	0.060	0.000	0.000	0.000	-10.683
		30:LOAD GENI	-8.352	0.538	0.000	0.000	0.000	-8.674
		31:LOAD GENI	-8.365	0.937	0.000	0.000	0.000	-6.434
		32:LOAD GENI	-8.209	1.249	0.000	0.000	0.000	-4.130
		33:LOAD GENI	-7.889	1.473	0.000	0.000	0.000	-1.931
		34:LOAD GENI	-7.410	1.607	0.000	0.000	0.000	0.037
		35:LOAD GENI	-6.782	1.653	0.000	0.000	0.000	1.668

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip`ft)	My (kip`ft)	Mz (kip`ft)
		36:LOAD GENI	-6.017	1.601	0.000	0.000	0.000	2.762
		37:LOAD GENI	-5.101	1.402	0.000	0.000	0.000	2.791
		38:LOAD GENI	-4.157	1.186	0.000	0.000	0.000	2.737
		39:LOAD GENI	-3.184	0.946	0.000	0.000	0.000	2.514
		40:LOAD GENI	-2.189	0.681	0.000	0.000	0.000	2.105
		41:LOAD GENI	-1.362	0.447	0.000	0.000	0.000	1.589
		42:LOAD GENI	-0.786	0.258	0.000	0.000	0.000	0.915

As-Built

		Dead Load + 90 psf		H5 Truck		**TOTAL**		KDOT Column Expert - Input		
		F (kips)	M (k-ft)	F (kips)	M (k-ft)	**F (kips)**	**M (k-ft)**	Mx	My	F
BEAM 1	MAX AXIAL	318.657	147.222	8.275	5.307	**326.932**	**152.529**	152.529 ,	0 ,	326.932
	MIN AXIAL	261.428	143.871	0.76	4.662	**262.188**	**148.533**	148.533 ,	0 ,	262.188
	MAX MOMENT	283.016	249.777	4.949	34.736	**287.965**	**284.513**	284.513 ,	0 ,	287.965
	MIN MOMENT	297.069	64.519	8.269	3.06	**305.338**	**67.579**	67.579 ,	0 ,	305.338
BEAM 2	MAX AXIAL	317.138	146.404	8.315	3.136	**325.453**	**149.54**	149.54 ,	0 ,	325.453
	MIN AXIAL	259.851	133.982	0.77	3.585	**260.621**	**137.567**	137.567 ,	0 ,	260.621
	MAX MOMENT	281.204	215.867	4.851	24.542	**286.055**	**240.409**	240.409 ,	0 ,	286.055
	MIN MOMENT	295.786	64.917	8.315	3.136	**304.101**	**68.053**	68.053 ,	0 ,	304.101
BEAM 3	MAX AXIAL	315.485	121.117	8.347	6.038	**323.832**	**127.155**	127.155 ,	0 ,	323.832
	MIN AXIAL	258.194	105.464	0.778	2.605	**258.972**	**108.069**	108.069 ,	0 ,	258.972
	MAX MOMENT	279.295	161.664	6.784	19.335	**286.079**	**180.999**	180.999 ,	0 ,	286.079
	MIN MOMENT	294.384	64.917	0.835	2.513	**295.219**	**67.43**	67.43 ,	0 ,	295.219
BEAM 4	MAX AXIAL	313.711	72.832	8.37	8.123	**322.081**	**80.955**	80.955 ,	0 ,	322.081
	MIN AXIAL	256.466	59.489	0.786	1.717	**257.252**	**61.206**	61.206 ,	0 ,	257.252
	MAX MOMENT	299.833	93.867	7.54	15.851	**307.373**	**109.718**	109.718 ,	0 ,	307.373
	MIN MOMENT	270.345	38.454	0.786	1.717	**271.131**	**40.171**	40.171 ,	0 ,	271.131

As-Configured

		Dead Load + 90 psf		H5 Truck		**TOTAL**		KDOT Column Expert - Input		
		F (kips)	M (k-ft)	F (kips)	M (k-ft)	**F (kips)**	**M (k-ft)**	Mx	My	F
BEAM 1	MAX AXIAL	334.554	149.854	8.275	5.307	**342.829**	**155.161**	155.161 ,	0 ,	342.829
	MIN AXIAL	277.325	144.802	0.76	4.662	**278.085**	**149.464**	149.464 ,	0 ,	278.085
	MAX MOMENT	298.913	250.708	4.949	34.736	**303.862**	**285.444**	285.444 ,	0 ,	303.862
	MIN MOMENT	312.966	67.969	8.269	3.06	**321.235**	**71.029**	71.029 ,	0 ,	321.235
BEAM 2	MAX AXIAL	333.051	149.854	8.315	3.136	**341.366**	**152.99**	152.99 ,	0 ,	341.366
	MIN AXIAL	275.764	137.433	0.77	3.585	**276.534**	**141.018**	141.018 ,	0 ,	276.534
	MAX MOMENT	297.117	219.318	4.851	24.542	**301.968**	**243.86**	243.86 ,	0 ,	301.968
	MIN MOMENT	311.699	69.265	8.315	3.136	**320.014**	**72.401**	72.401 ,	0 ,	320.014
BEAM 3	MAX AXIAL	331.399	125.465	8.347	6.038	**339.746**	**131.503**	131.503 ,	0 ,	339.746
	MIN AXIAL	274.108	109.812	0.778	2.605	**274.886**	**112.417**	112.417 ,	0 ,	274.886
	MAX MOMENT	295.209	166.012	6.784	19.335	**301.993**	**185.347**	185.347 ,	0 ,	301.993
	MIN MOMENT	310.298	69.265	0.835	2.513	**311.133**	**71.778**	71.778 ,	0 ,	311.133
BEAM 4	MAX AXIAL	329.612	76.538	8.37	8.123	**337.982**	**84.661**	84.661 ,	0 ,	337.982
	MIN AXIAL	272.368	63.195	0.786	1.717	**273.154**	**64.912**	64.912 ,	0 ,	273.154
	MAX MOMENT	315.734	97.573	7.54	15.851	**323.274**	**113.424**	113.424 ,	0 ,	323.274
	MIN MOMENT	286.246	42.16	0.786	1.717	**287.032**	**43.877**	43.877 ,	0 ,	287.032

TranSystems

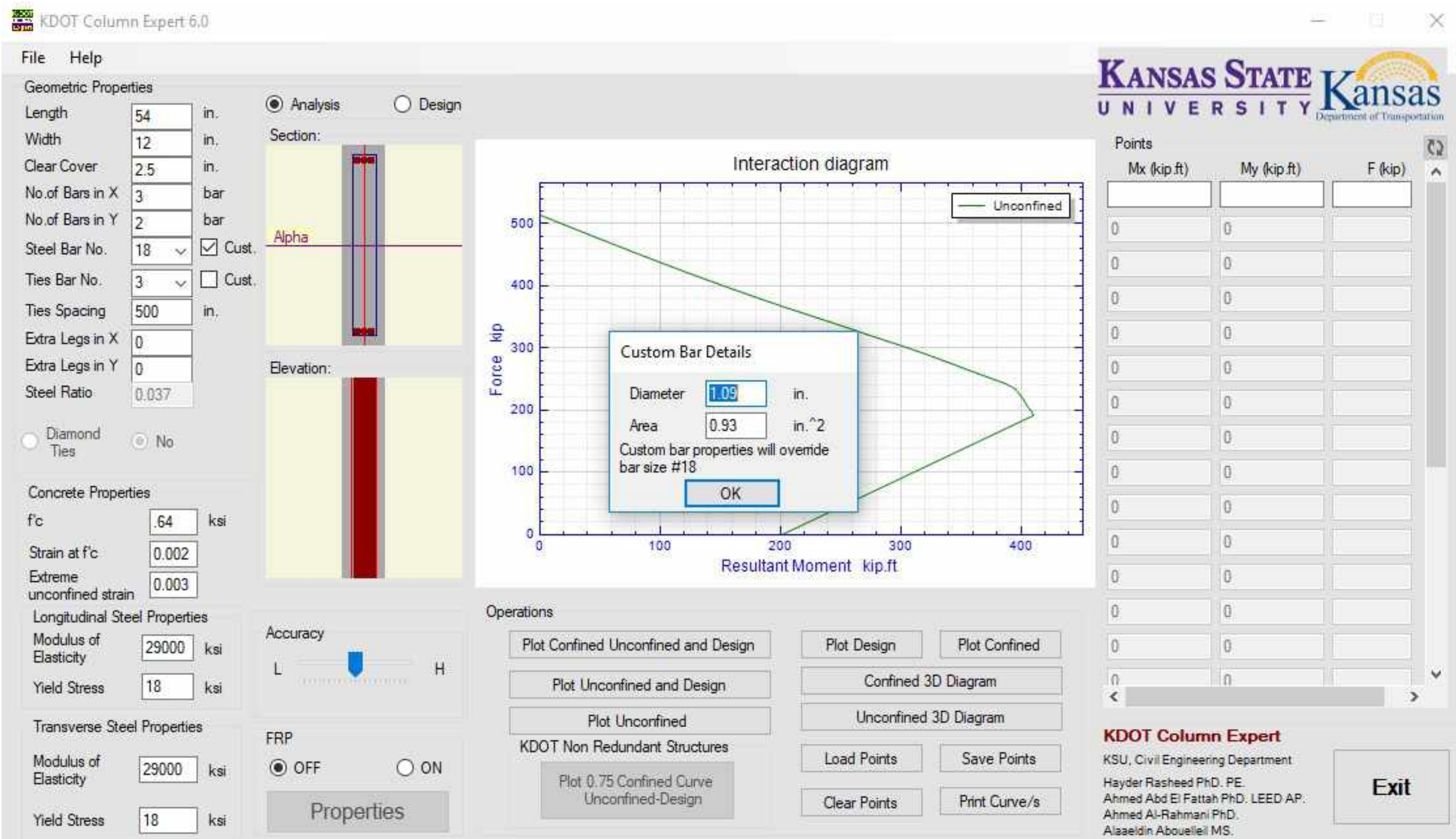
Made By	DWC	Date	8/3/2018	Job No.	P402180060
Checked By	SFH	Date	8/3/2018		

Calculations For: ***Lake Park Arch Bridge - Arch Rib Analysis***

LOWER ARCH - AS-BUILT / AS-CONFIGURED

INVENTORY LEVEL (90 PSF PEDESTRIAN LOAD)

AXIAL-MOMENT INTERACTION DIAGRAM



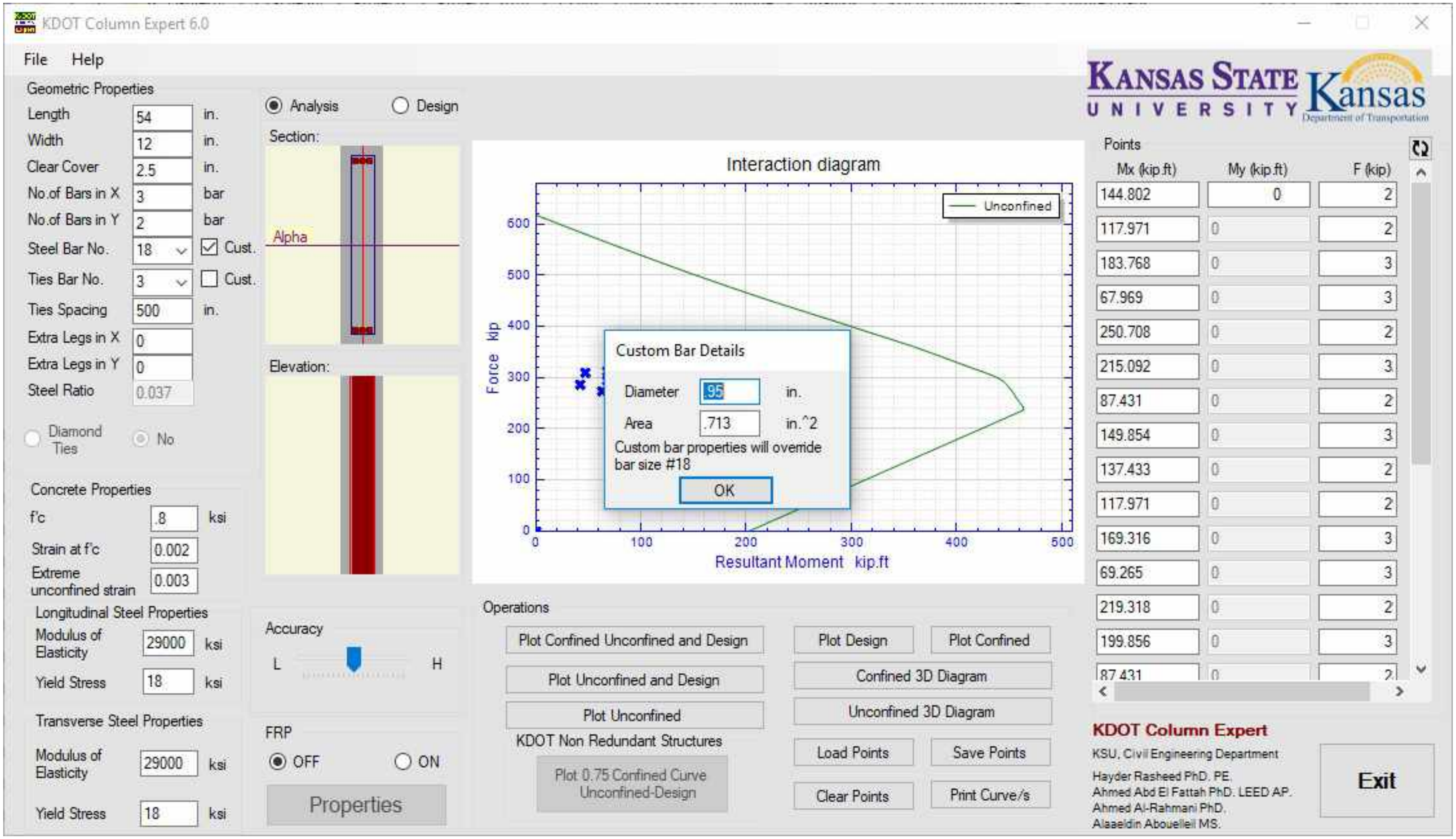
TranSystems	Made By DWC	Date 8/3/2018		Job No.	P402180060
	Checked By SFH	Date 8/3/2018			

Calculations For: ***Lake Park Arch Bridge - Arch Rib Analysis***

LOWER ARCH - AS-BUILT / AS-CONFIGURED

OPERATING LEVEL (90 PSF PEDESTRIAN LOAD + H5 TRUCK)

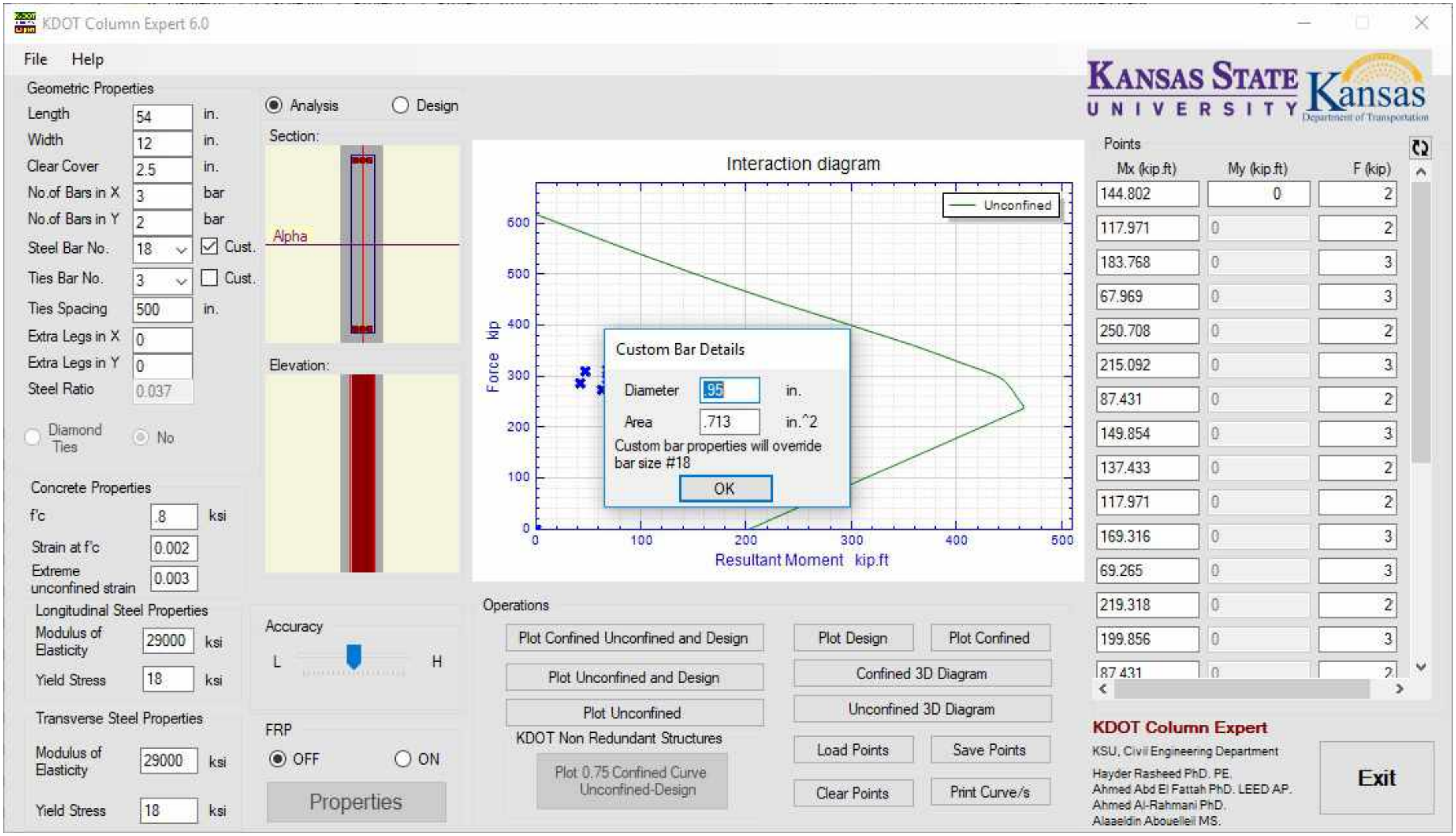
AXIAL-MOMENT INTERACTION DIAGRAM



LOWER ARCH - AS-INSPECTED

INVENTORY LEVEL (90 PSF PEDESTRIAN LOAD)

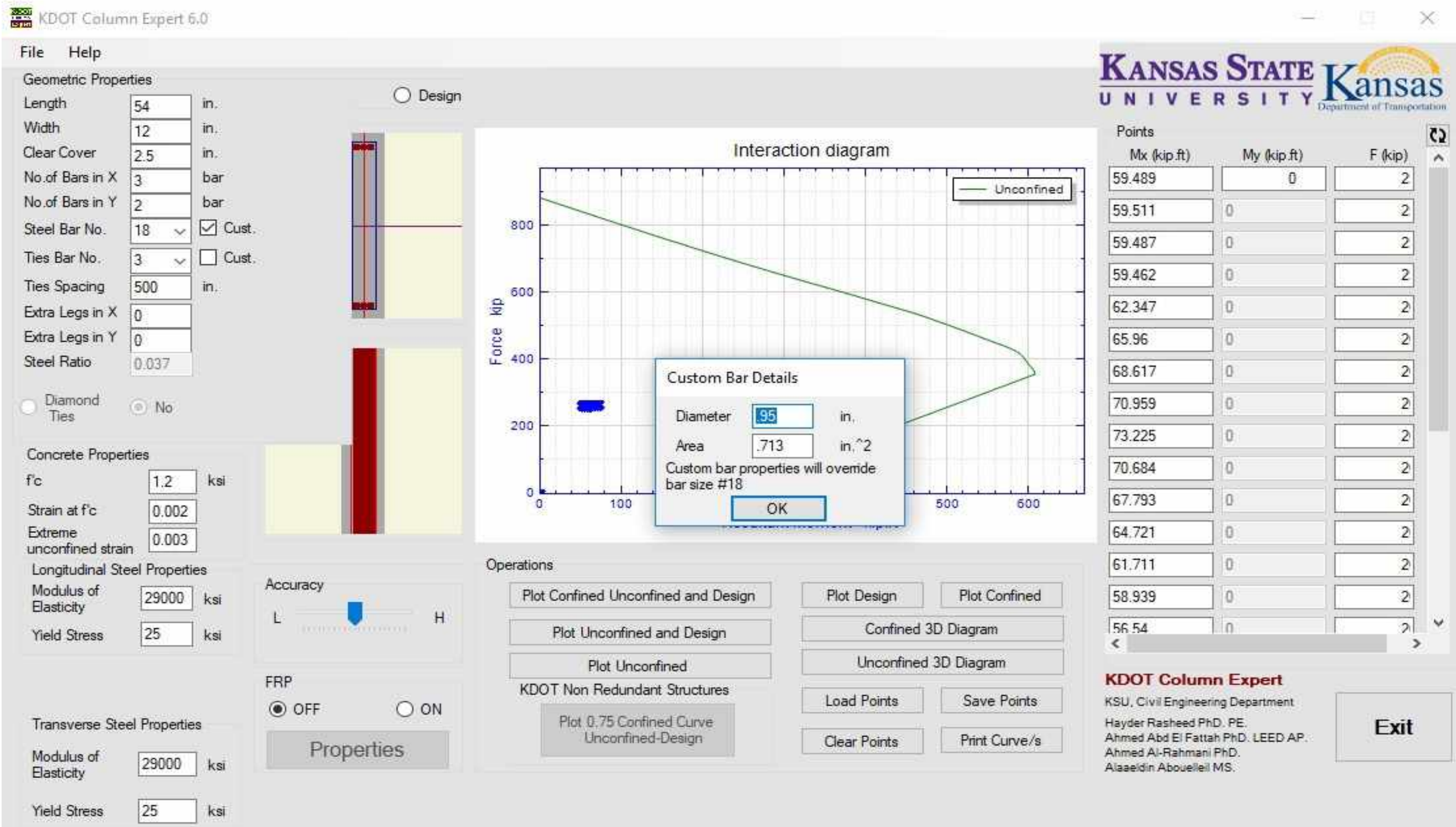
AXIAL-MOMENT INTERACTION DIAGRAM



LOWER ARCH - AS-INSPECTED

OPERATING LEVEL (90 PSF PEDESTRIAN LOAD + H5 TRUCK)

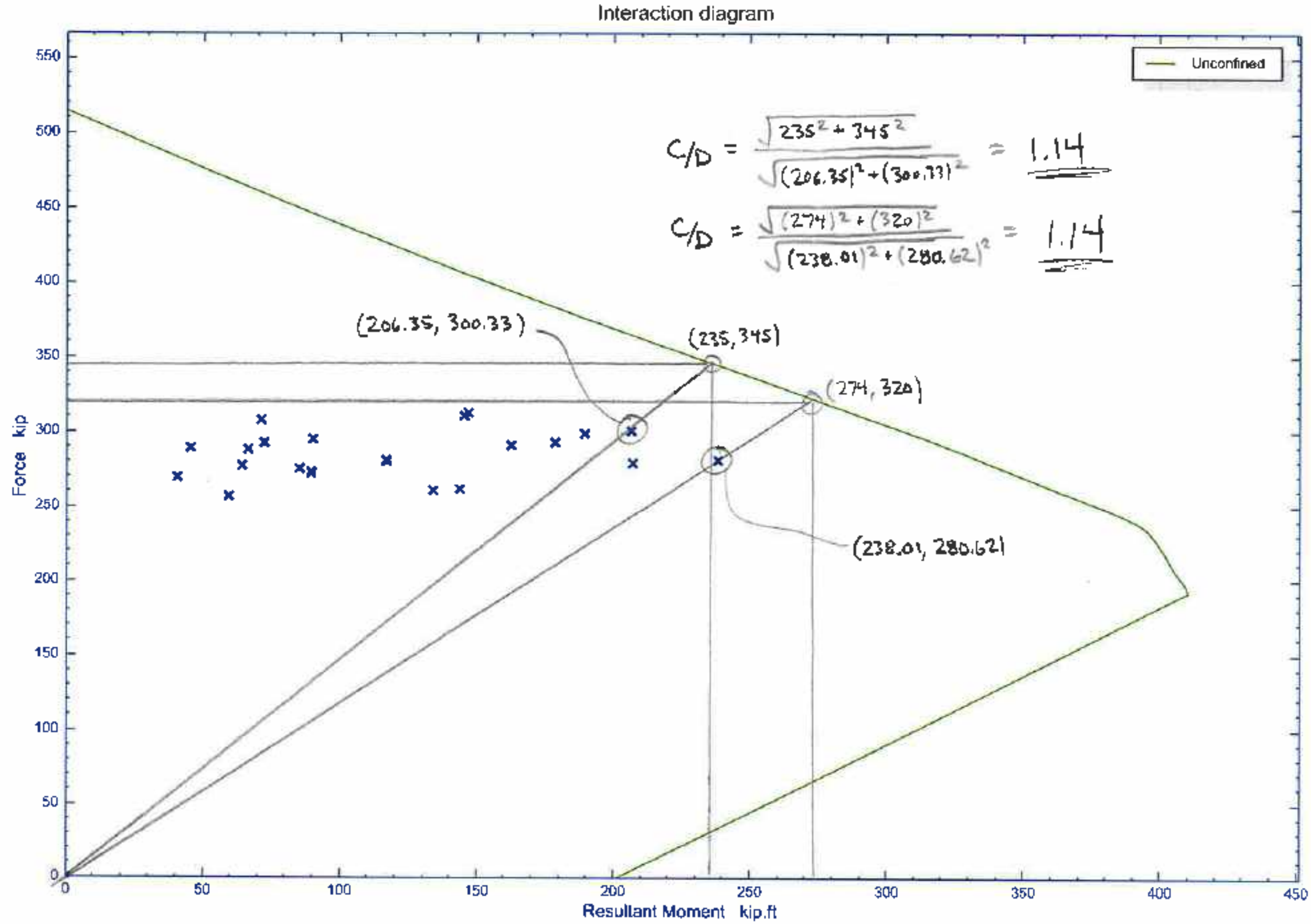
AXIAL-MOMENT INTERACTION DIAGRAM



LOWER ARCH - AS-BUILT

INVENTORY LEVEL (90 psf)

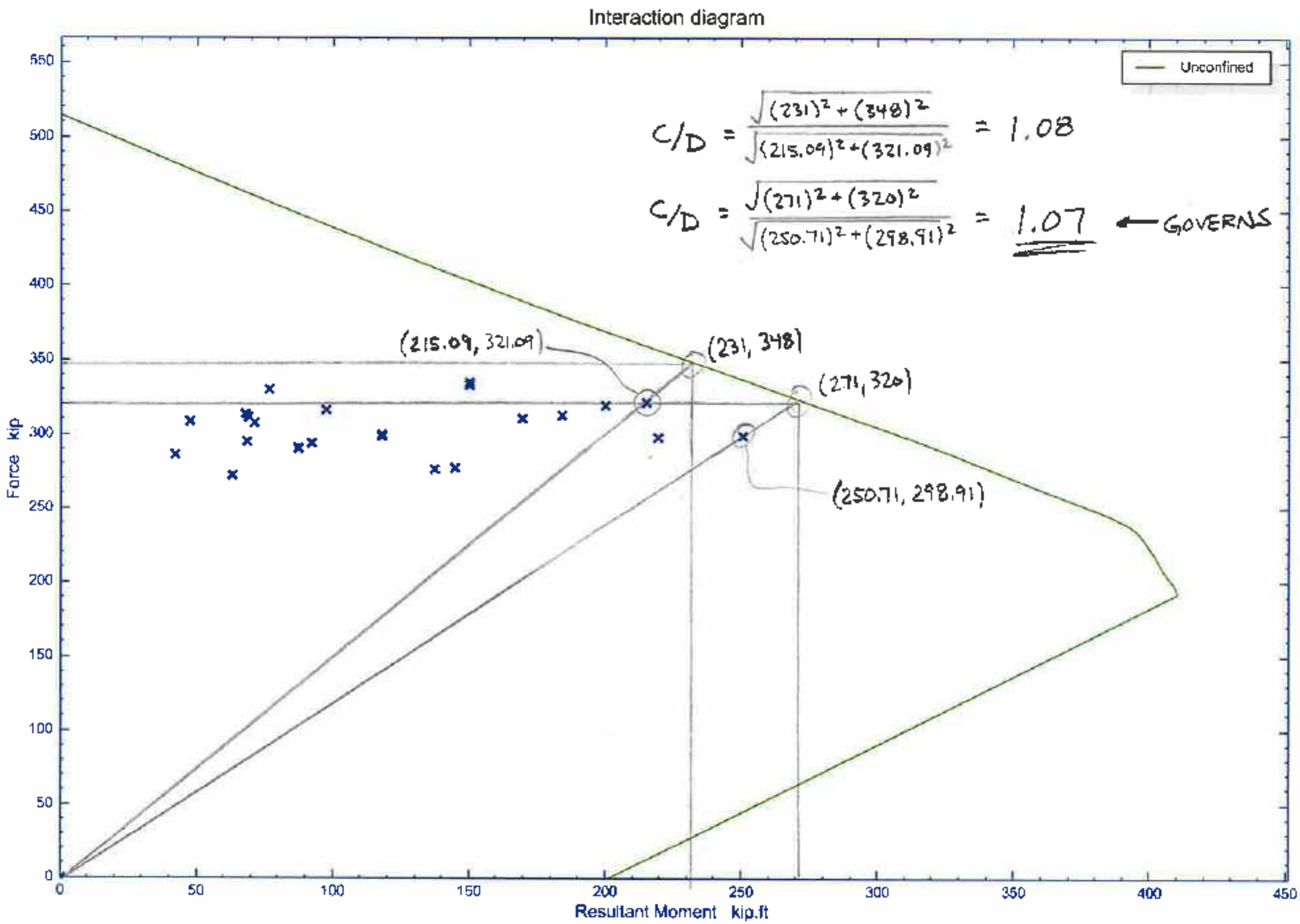
Beams 1, 2, 4



LOWER ARCH - AS-CONFIGURED

INVENTORY LEVEL (90 psf)

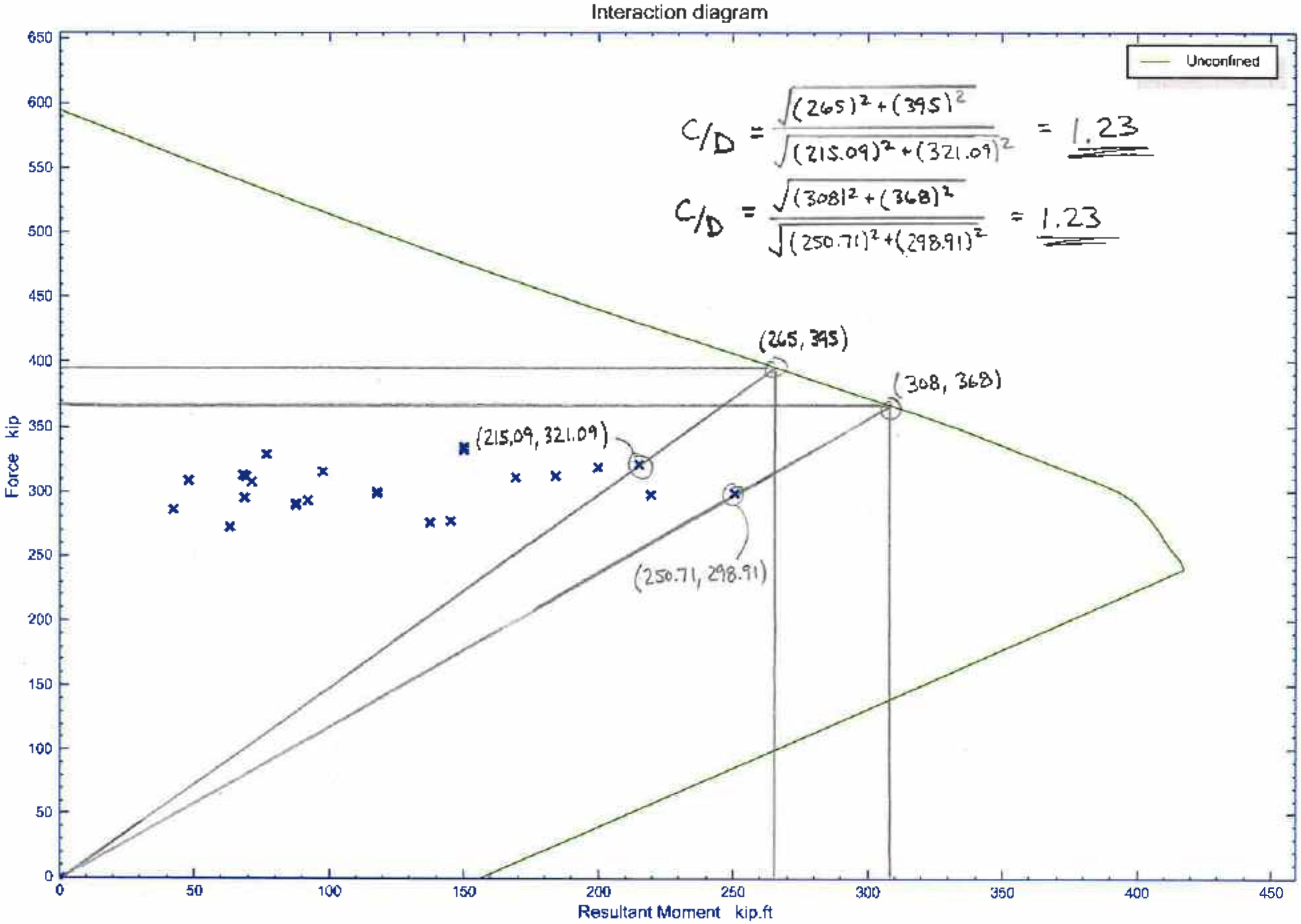
Beams 1, 2, 4



LOWER ARCH - AS-INSPECTED

INVENTORY LEVEL (90 psf)

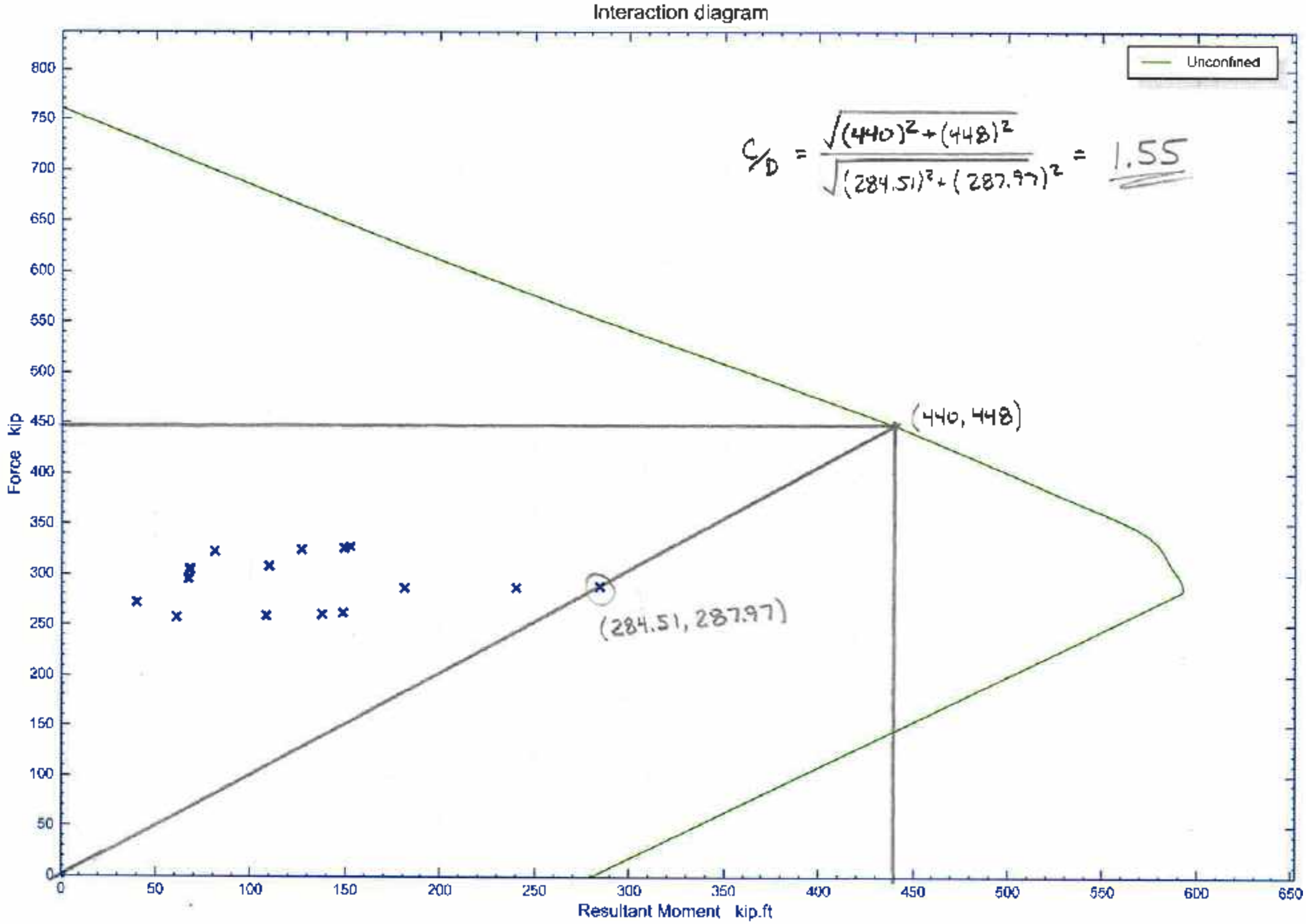
Beams 1, 2, 4



LOWER ARCH - AS-BUILT

OPERATING LEVEL (90 psf + HS truck)

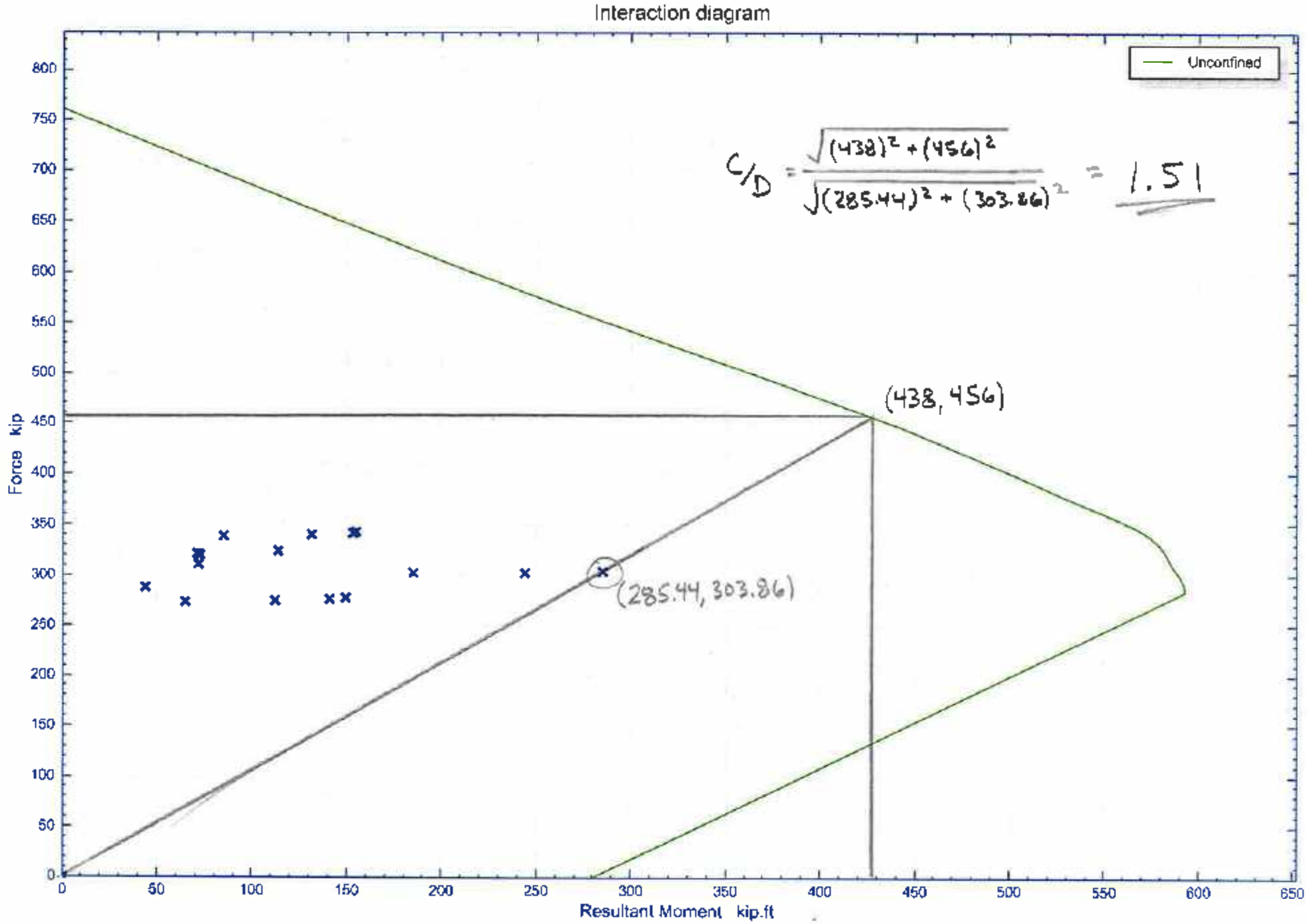
Beams 1-4



LOWER ARCH - AS-CONFIGURED

OPERATING LEVEL (90 psf + HS truck)

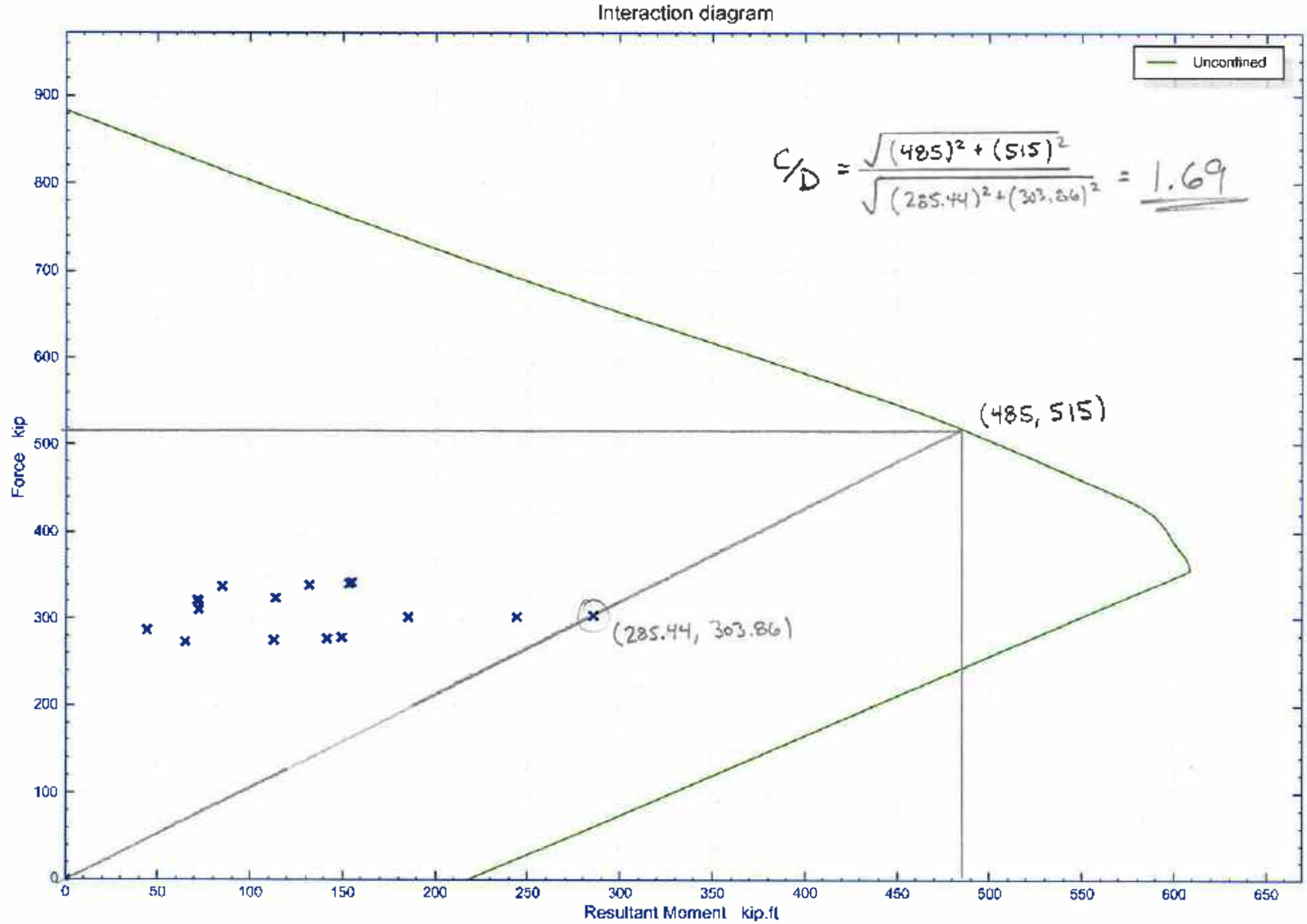
Beams 1-4



LOWER ARCH - AS-INSPECTED

OPERATING LEVEL (90 psf + HS Truck)

Beams 1-4



TranSystems

Made By: DWC Date: 8/2/18 Job No: P402180060

Checked By: SFH Date: 8/2/18 Sheet No.

LAKE PARK ARCH BRIDGE - ADDITIONAL LOAD CONSIDERATIONS

LONGITUDINAL SPANDREL MEMBER

Inventory Rating

$h = 3'\text{-}2"$, $b = 12"$, $L = 20'\text{-}0"$, $d = 35.29"$

$A_s = 2.84\ in^2$ (midspan), $A_s = 2.00\ in^2$ (1/4 POINT)

As-Built: $g_{DL} = 1.38$ k/ft
$g_{LL} = (0.090\ ksf)(12'/2) = 0.54$ k/ft } $g_{TOT} = 1.92$ k/ft

$M_{MAX} = (1.92\ k/ft)\frac{(20')^2}{8} = 96.0$ k-ft

$M_{1/4PT} = (1.92\ k/ft)(5'/2)(20'-5') = 72.0$ k-ft

Midspan: $a = \frac{A_s f_y}{0.85 f_c b} = \frac{(2.84)(18)}{(0.85)(.64)(12)} = 7.83"$

$M_{all} = A_s f_y (d - a/2) = (2.84)(18)(35.29 - \frac{7.83}{2}) = 1603.9$ k-in
$= 133.7$ k-ft

1/4 POINT: $a = \frac{A_s f_y}{0.85 f_c b} = \frac{(2.00)(18)}{0.85(.64)(12)} = 5.51"$

$M_{all} = A_s f_y (d - a/2) = (2.0)(18)(35.29 - \frac{5.51}{2}) = 1171.3$ k-in
$= 97.6$ k-ft

C/D (midspan) $= \frac{133.7\ k\text{-}ft}{96.0\ k\text{-}ft} = 1.39$

C/D (1/4 POINT) $= \frac{97.6\ k\text{-}ft}{72.0\ k\text{-}ft} = 1.35$ ← GOVERNS

As-Configured: $g_{DL} = 1.53$ k/ft
$g_{LL} = 0.54$ k/ft } $g_{TOT} = 2.07$ k/ft

$M_{MAX} = (2.07\ k/ft)\frac{(20')^2}{8} = 103.5$ k-ft

$M_{1/4PT} = (2.07\ k/ft)(5'/2)(20'-5') = 77.63$ k-ft

Capacities are same as in As-Built Condition

∴ C/D (midspan) $= \frac{133.7\ k\text{-}ft}{103.5\ k\text{-}ft} = 1.29$

C/D (1/4 POINT) $= \frac{97.6\ k\text{-}ft}{77.63\ k\text{-}ft} = 1.25$ ← GOVERNS

Calculations for 1/4 US Grid

Made By: DWC | Date: 8/2/18 | Job No: P402180060
Checked By: SFH | Date: 8/2/18 | Sheet No.

LAKE PARK ARCH BRIDGE - ADDITIONAL LOAD CONSIDERATIONS

As-Inspected: Applied loads are equal to those in the As-Configured analysis case; however, include section loss on rebar as shown in previous analysis and adjust concrete strength based on testing.

$A_s = (87.9\%)(2.84\ in^2) = 2.49\ in^2$ (midspan)
$A_s = (87.9\%)(2.00\ in^2) = 1.76\ in^2$ (1/4 POINT)

Midspan: $a = \frac{A_s f_y}{0.85 f_c b} = \frac{(2.49)(18)}{0.85(0.64)(12)} = 6.87''$

$M_{all} = A_s f_y (d - a/2) = 2.49(18)(35.29 - \frac{6.87}{2}) = 1427.7\ k\text{-}in = 119.0\ k\text{-}ft$

1/4 POINT: $a = \frac{A_s f_y}{0.85 f_c b} = \frac{(1.76)(18)}{0.85(0.64)(12)} = 4.85''$

$M_{all} = (1.76)(18)(35.29 - \frac{4.85}{2}) = 1041.2\ k\text{-}in = 86.76\ k\text{-}ft$

$C/D\ (\text{midspan}) = \frac{119.0\ k\text{-}ft}{103.5\ k\text{-}ft} = 1.15$

$C/D\ (1/4\ \text{POINT}) = \frac{86.76\ k\text{-}ft}{77.63\ k\text{-}ft} = 1.11$ ← GOVERNS

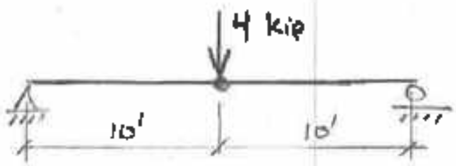
LAKE PARK ARCH BRIDGE - ADDITIONAL LOAD CONSIDERATIONS

LONGITUDINAL SPANDREL MEMBER

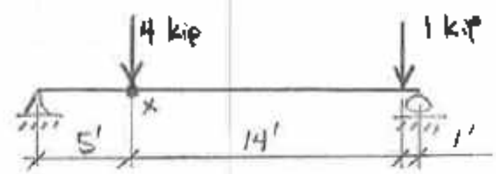
Operating Rating

For operating level, use design vehicle of HS truck + 90 psf. Calculate LLDF based on lever rule and multiply by resultant moment from varying truck position along the length of the beam.

By inspection, governing moments will occur when rear wheel is placed at point of interest



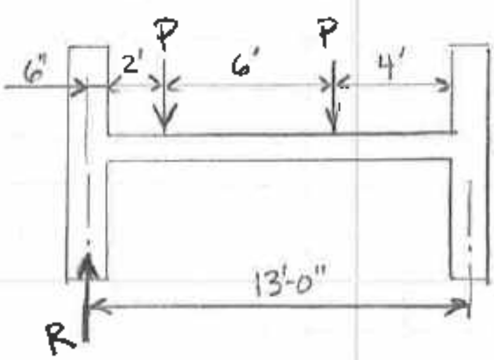
MAX MOMENT



Moment @ 1/4 POINT

$$M_{MAX} = PL/4 \qquad M_x = \frac{Pab}{L}$$

Live Load Distribution Factor:



$$R = \left(\frac{13-2.5}{13}\right)P + \left(\frac{13-8.5}{13}\right)P$$

$$= \underline{\underline{1.154}} \quad \text{(LLDF)}$$

$$\therefore M_{LL}\text{ (midspan)} = LLDF \times PL/4 = (1.154)(4\text{ kip})(20'/4) = 23.08\text{ k-ft}$$

$$M_{LL}\text{ (1/4 point)} = (1.154)\left[(4\text{ kip})\frac{(5')(15')}{20'} + (1\text{ kip})\frac{(5')(1')}{20'}\right] = 17.60\text{ k-ft}$$

TranSystems

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LAKE PARK ARCH BRIDGE - ADDITIONAL LOAD CONSIDERATIONS

As-Built : $g_{DL} = 1.38$ k/ft

$$M_{max} = M_{DL} + M_{LL} = (1.38 \text{ k/ft}) \frac{(20')^2}{8} + (0.54 \text{ k/ft}) \frac{(20')^2}{8} + 23.08 \text{ k-ft} = 119.08 \text{ k-ft}$$

$$M_{1/4 \text{ POINT}} = (1.38 \text{ k/ft})\left(\frac{5'}{2}\right)(20' - 5') + (0.54 \text{ k/ft})\left(\frac{5'}{2}\right)(20'-5') + 17.60 \text{ k-ft} = 89.60 \text{ k-ft}$$

Midspan : $a = \frac{A_s f_y}{0.85 f_c b} = \frac{(2.84)(25)}{(0.85)(0.96)(12)} = 7.25"$

$$M_{all} = A_s f_y (d - a/2) = (2.84)(25)(35.29 - \frac{7.25}{2}) = 2248.2 \text{ k-in} = 187.3 \text{ k-ft}$$

1/4 POINT : $a = \frac{A_s f_y}{0.85 f_c b} = \frac{(2.0)(25)}{(0.85)(0.96)(12)} = 5.11"$

$$M_{all} = (2.00)(25)(35.29 - \frac{5.11}{2}) = 1636.8 \text{ k-in} = 136.4 \text{ k-ft}$$

$$C/D \text{ (midspan)} = \frac{M_{all}}{M_{DL}+M_{LL}} = \frac{M_{all}}{M_{MAX}} = \frac{187.3 \text{ k-ft}}{119.08 \text{ k-ft}} = 1.57$$

$$C/D \text{ (1/4 POINT)} = \frac{M_{all}}{M_{1/4PT}} = \frac{136.4 \text{ k-ft}}{89.60 \text{ k-ft}} = \underline{1.52}$$ ← GOVERNS

As-Configured : $g_{DL} = 1.53$ k/ft

$$M_{max} = M_{DL} + M_{LL} = (1.53 \text{ k/ft}) \frac{(20')^2}{8} + 23.08 \text{ k-ft} + (0.54 \text{ k/ft}) \frac{(20')^2}{8} = 126.58 \text{ k-ft}$$

$$M_{1/4PT} = (1.53 \text{ k/ft})\left(\frac{5'}{2}\right)(20'-5') + 17.60 \text{ k-ft} + (0.54 \text{ k/ft})\left(\frac{5'}{2}\right)(20'-5') = 95.23 \text{ k-ft}$$

$$C/D \text{ (midspan)} = \frac{M_{all}}{M_{max}} = \frac{187.3 \text{ k-ft}}{126.58 \text{ k-ft}} = 1.48$$

$$C/D \text{ (1/4 POINT)} = \frac{M_{all}}{M_{1/4PT}} = \frac{136.4 \text{ k-ft}}{95.23 \text{ k-ft}} = \underline{1.43}$$ ← GOVERNS

LAKE PARK ARCH BRIDGE – ADDITIONAL LOAD CONSIDERATIONS

As-Inspected: Loads are the same as As-Configured, but use higher allowable stress in concrete due to compressive testing and include section loss on rebar.

$A_s = 2.49 \text{ in}^2$ (midspan) $\quad A_s = 1.76 \text{ in}^2$ (1/4 point)

$f_s = 25000$ psi

$f_c = 1200$ psi

Midspan: $a = \frac{A_s f_y}{0.85 f_c b} = \frac{(2.49)(25)}{0.85(1.2)(12)} = 5.09''$

$M_{all} = A_s f_y (d - a/2) = (2.49)(25)(35.29 - \frac{5.09}{2}) = 2038.4 \text{ k-in}$
$= 169.8 \text{ k-ft}$

1/4 POINT: $a = \frac{A_s f_y}{0.85 f_c b} = \frac{(1.76)(25)}{0.85(1.2)(12)} = 3.59''$

$M_{all} = (1.76)(25)(35.29 - 3.59/2) = 1473.8 \text{ k-in} = 122.8 \text{ k-ft}$

C/D (midspan) $= \frac{M_{all}}{M_{MAX}} = \frac{169.8 \text{ k-ft}}{126.58 \text{ k-ft}} = 1.34$

C/D (1/4 POINT) $= \frac{M_{all}}{M_{1/4 PT}} = \frac{122.8 \text{ k-ft}}{95.23 \text{ k-ft}} = \underline{1.29}$ ← GOVERNS

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LAKE PARK ARCH BRIDGE - ADDITIONAL LOAD CONSIDERATIONS

DECK

Inventory Rating

As-Built : $q_{DL} = 0.075$ k/ft , $M_{DL} = 1.46$ k-ft
$q_{LL} = (90\text{ psf})(1') = 0.09$ k/ft , $M_{LL} = (0.09)\frac{(12.5)^2}{8} = 1.76$ k-ft

Area of steel: $A_s = 0.65$ in²/ft (based on 7" spacing of ½" x 1½" khan bars)
Clear cover 1" ∴ $d = 4.65"$
$\beta_1 = 0.85$ for $f'_c = 1600$ psi ⟹ $f_c = 0.4(1600) = 0.64$ ksi
$f_s = 18000$ psi

$$a = \frac{A_s f_y}{0.85 f_c b} = \frac{(0.65)(18)}{0.85(.64)(12)} = 1.79"$$

$$M_{all} = A_s f_y (d - a/2) = (0.65)(18)\left(4.65 - \frac{1.79}{2}\right) = 43.93 \text{ k-in} = 3.66 \text{ k-ft}$$

$$C/D = \frac{M_{all}}{M_{DL} + M_{LL}} = \frac{3.66}{(1.46 + 1.76)} = \underline{1.13}$$

As-Configured: $q_{DL} = 0.0875$ k/ft , $M_{DL} = 1.71$ k-ft
Live Loads same as As-Built , $M_{LL} = 1.76$ k-ft

$$C/D = \frac{M_{all}}{M_{DL} + M_{LL}} = \frac{3.66}{1.71 + 1.76} = \underline{1.05}$$

As-Inspected : $M_{DL} = 1.71$ k-ft , $M_{LL} = 1.76$ k-ft

Steel exhibits no significant section loss ⟹ $A_s = 0.65$ in²/ft
Use higher concrete strength due to material testing
∴ $f'_c = 2000$ psi ⟹ $f_c = 0.4(2000) = 800$ psi

$$a = \frac{A_s f_y}{0.85 f_c b} = \frac{(0.65)(18)}{0.85(0.8)(12)} = 1.43"$$

$$M_{all} = A_s f_y (d - a/2) = (0.65)(18)\left(4.65 - \frac{1.43}{2}\right) = 46.02 \text{ k-in} = 3.83 \text{ k-ft}$$

$$C/D = \frac{M_{all}}{M_{DL} + M_{LL}} = \frac{3.83}{1.71 + 1.76} = \underline{1.10}$$

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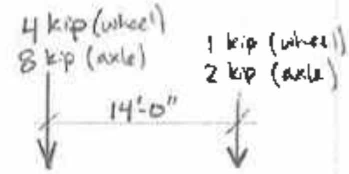
LAKE PARK ARCH BRIDGE - ADDITIONAL LOAD CONSIDERATIONS

DECK

Operating Rating



H5 truck

For operating level, use design vehicle of H5 truck.
Allowable stresses are higher for operating level:

$f_s = 25000$ psi

$f_c = 960$ psi (original), 1200 psi (concrete testing)

Determine live load moments using AASHTO 3.24.3.1 (Case A):

$M_{LL} = \left(\frac{S+2}{32}\right)P$, where $P = 4$ kips (weight of one rear wheel)

$M_{LL} = \left(\frac{12.5+2}{32}\right)(4 \text{ kips}) = 1.81$ k-ft (truck)

As-Built: $M_{DL} = 1.46$ k-ft

$M_{LL} = 1.81$ k-ft $+ 1.76$ k-ft $= 3.57$ k-ft

$$a = \frac{A_s f_y}{0.85 f_c b} = \frac{(0.65)(25)}{0.85(0.96)(12)} = 1.66''$$

$$M_{all} = A_s f_y (d - a/2) = (0.65)(25)\left(4.65 - \frac{1.66}{2}\right) = 62.08 \text{ k-in} = 5.17 \text{ k-ft}$$

$$C/D = \frac{M_{all}}{M_{DL} + M_{LL}} = \frac{5.17}{1.46 + 3.57} = \underline{1.02}$$

As-Configured: $M_{DL} = 1.71$ k-ft

$M_{LL} = 3.57$ k-ft

$$C/D = \frac{M_{all}}{M_{DL} + M_{LL}} = \frac{5.17}{1.71 + 3.57} = \underline{0.98}$$

As-Inspected: Loads same as As-Configured

$$a = \frac{A_s f_y}{0.85 f_c b} = \frac{(0.65)(25)}{0.85(1.2)(12)} = 1.33''$$

$$M_{all} = A_s f_y (d - a/2) = (0.65)(25)(4.65 - 1.33/2) = 64.76 \text{ k-in} = 5.39 \text{ k-ft}$$

$$C/D = \frac{M_{all}}{M_{DL} + M_{LL}} = \frac{5.39}{1.71 + 3.57} = \underline{1.02}$$

B, unless more exact methods are used considering tire contact area. The tire contact area needed for exact methods is given in Article 3.30.

In Cases A and B:

S = effective span length, in feet, as defined under "Span Lengths" Articles 3.24.1 and 8.8;
E = width of slab in feet over which a wheel load is distributed;
P = load on one rear wheel of truck (P_{15} or P_{20});
P_{15} = 12,000 pounds for H 15 loading;
P_{20} = 16,000 pounds for H 20 loading.

3.24.3.1 Case A—Main Reinforcement Perpendicular to Traffic (Spans 2 to 24 Feet Inclusive)

The live load moment for simple spans shall be determined by the following formulas (impact not included):

HS 20 Loading:

$$\left(\frac{S+2}{32}\right)P_{20} = \text{Moment in foot} - \text{pounds per foot} - \text{width of slab} \qquad (3\text{-}15)$$

HS 15 Loading:

$$\left(\frac{S+2}{32}\right)P_{15} = \text{Moment in foot} - \text{pounds per foot} - \text{width of slab} \qquad (3\text{-}16)$$

In slabs continuous over three or more supports, a continuity factor of 0.8 shall be applied to the above formulas for both positive and negative moment.

3.24.3.2 Case B—Main Reinforcement Parallel to Traffic

For wheel loads, the distribution width, E, shall be (4 + 0.06S) but shall not exceed 7.0 feet. Lane loads are distributed over a width of 2E. Longitudinally reinforced slabs shall be designed for the appropriate HS loading.

For simple spans, the maximum live load moment per foot width of slab, without impact, is closely approximated by the following formulas:

HS 20 Loading:

Spans up to and including 50 feet: LLM = 900S foot-pounds
Spans 50 feet to 100 feet: LLM = 1,000 (1.30S-20.0) foot-pounds

HS 15 Loading:

Use ¾ of the values obtained from the formulas for HS 20 Loading

Moments in continuous spans shall be determined by suitable analysis using the truck or appropriate lane loading.

3.24.4 Shear and Bond

Slabs designed for bending moment in accordance with Article 3.24.3 shall be considered satisfactory in bond and shear.

3.24.5 Cantilever Slabs

3.24.5.1 Truck Loads

Under the following formulas for distribution of loads on cantilever slabs, the slab is designed to support the load independently of the effects of any edge support along the end of the cantilever. The distribution given includes the effect of wheels on parallel elements.

3.24.5.1.1 Case A—Reinforcement Perpendicular to Traffic

Each wheel on the element perpendicular to traffic shall be distributed over a width according to the following formula:

$$E = 0.8X + 3.75 \qquad (3\text{-}17)$$

The moment per foot of slab shall be (P/E) X foot-pounds, in which X is the distance in feet from load to point of support.

3.24.5.1.2 Case B—Reinforcement Parallel to Traffic

The distribution width for each wheel load on the element parallel to traffic shall be as follows:

$$E = 0.35X + 3.2\text{, but shall not exceed 7.0 feet} \qquad (3\text{-}18)$$

The moment per foot of slab shall be (P/E) X foot-pounds.

3.24.5.2 Railing Loads

Railing loads shall be applied in accordance with Article 2.7. The effective length of slab resisting post loadings shall be equal to E = 0.8X + 3.75 feet where no parapet

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MEMORANDUM

TO: Karl Stave P.E., Milwaukee County Architecture, Engineering & Environmental Services

FROM: Kevin Wood, P.E.

DATE: September 21, 2018

SUBJECT: Lake Park Arch Bridge Load Calculation Review

As part part of the continued study for the Lake Park Arch Bridge over Ravine Road, the Lake Park Friends hired an independent consultant, TranSystems, to perform concrete testing and a structural analysis. Concrete testing was performed to determine if the existing in-place concrete material is capable of receiving structural repairs (such as concrete patching). A structural analysis was conducted to determine the load carrying capacity of three elements of the bridge: arch rib, spandrel beam, and deck. Milwaukee County has asked GRAEF to review the TranSystems reports and provide our opinions on their appropriateness.

Concrete Testing Results Report

TranSystems evaluated several concrete tests performed by Giles Engineer Associates, Inc. and provided their opinions in a report dated June 18, 2018. The program included testing for chloride content, petrographic/air content analysis, freeze/thaw, and review of earlier unconfined compression testing.

Overall the Concrete Testing Results Report was complete and the conclusions reasonable. There were, however, a few items to be noted:

- One of the eight concrete core samples through the deck was omitted from testing due to deterioration.
- Within the Chloride Content section of the report, one active mitigation technique mentioned is the use of galvanic anodes placed within new concrete patches. While this is a common technique to address rebar corrosion within the patch, it should be noted that reinforcement around the perimeter of the patch zone may start to corrode at an accelerated rate. This is due to pH differences between the existing concrete and new patch concrete. Placement of the galvanic anodes near the patch edges can help to mitigate this effect.
- The report Conclusions state that test results indicate rehabilitation could maintain structural integrity and load capacity for 50 years, based primarily on the lack of high chloride concentrations in the deck. While this is an important factor, our opinion continues to be that given the *overall* condition of the bridge, the life

span of a rehabilitated bridge with normal maintenance would be less than 50 years.

Structural Analysis Report
TranSystems conducted several analyses on the arch rib, spandrel beam, and deck elements of the bridge to determine their load carrying capacities and the load demands on each. For each element, three conditions were investigated:

1. As-built using original loads, section properties and material strengths
2. As-configured using current loads based on structure modifications, section properties and original material strengths
3. As-inspected using current loads based on structure modifications, current section properties based on section loss, and material strengths based on testing.

All analyses used Allowable Stress Design (ASD) methodology. For reinforced concrete, this is a method rarely used today to determine the strength of concrete components, but was the method likely used to design the bridge around 1903. This approach is still accepted by WisDOT to analyze and load rate concrete highway bridges originally designed using ASD.

Review of the TranSystem report and appendix suggests an analysis approach that generally follows GRAEF's 2005 and 2015 load ratings for Milwaukee County. However, there are several differing approaches and assumptions that strongly affect each element's load carrying capacity conclusions.

Capacity-to-Demand Ratios vs. Bridge Load Ratings
TranSystem chose to report each bridge element's load carrying capacity against the total demand of the bridge's combined dead load plus live load. Simply written, the equation is "capacity/(dead load + live load)", and values greater than 1.0 are desired. Using an ASD approach, member capacity is determined by applying a factor of safety to its calculated strength. For example, a factor of safety equal to 2 applied to a member having a strength of 100 pounds will result in an allowable usable strength of 50 pounds. Depending on the element and material type, factors of safety vary. For bridge inventory level load rating purposes, AASHTO's Manual for Condition Evaluation of Bridges uses factor of safety of 1.83 for reinforcing steel with a yield strength of 33,000 psi, and 2.5 for concrete having a yield strength of 2,000 psi.

While use of capacity-to-demand ratio is a conventional approach for building analysis, determination of load rating factors is conventional for bridges and was the method used for GRAEF's earlier load ratings. For this approach, a rating factor of a member's available capacity to resist live loads is reported. Simply written, the equation is "(capacity – dead load)/live load", and values greater than 1.0 are desired. This ratio can then be multiplied by the design live load to yield the maximum live load the member can resist.

It is emphasized that capacity-to-demand ratios are not the same as rating factors. For elements with adequate strength to resist the applied loads, capacity-to-demand ratios will be less than rating factors. For understrength elements, capacity-to-demand ratios will be greater than rating factors.

Inventory Level Ratings and Operating Level Ratings
While TranSystem correctly defines Inventory Level and Operating Level, they are incorrectly applied as load ratings in their calculations. As defined in AASHTO's Manual for Bridge Evaluation, a rating factor is defined as:

RF = (capacity – A_1 x dead load)/(A_2 x (live load + impact))

Where:
A_1 is the dead load factor
A_2 is the live load factor
(live load + impact) is a constant load (impact = 0 for pedestrian bridges)

For an ASD approach, $A_1 = A_2 = 1.0$ and the element's material capacities are varied depending on whether an inventory or operating rating is desired. This approach seems to have been used to determine element capacities within the TranSystem report. However, we see a few problems with how the inventory and operating ratings are applied.

- The live load used is inconsistent. Wheras the pedestrian loading only was used for the inventory analysis, the pedestrian + H5 service truck load was used for the operating analysis. The same level of live load should be used for comparing an inventory to operating rating.
- Pedestrian and H5 service truck live loads should not be applied simultaneously. This is an unrealistic load combination as noted in the AASHTO Guide Specifications for the Design of Pedestrian Bridges.
- Inventory and operating rating analyses are to be applied to a load rating approach as outlined in AASHTO, not to determine capacity-to-demand ratios.

Member Capacities
The arch bridge uses a proprietary steel reinforcing bar system known as Kahn bars. These bars consist of a steel square bar with thin plate projections, or "fins". The fins are cut transversely at regular intervals and cut free from the

square bar at predefined lengths. These cuts allow the fins to be bent up to provide shear reinforcement for the concrete element. The images below from Kahn's 1904 Handbook illustrate this configuration.

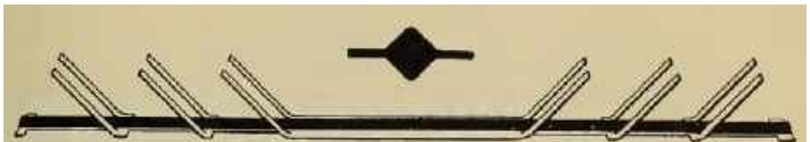


FIG. 11.

These bars can have any standard size cuts as shown in Figure 11 and will be sent in lengths as ordered. In making calculations for strength of reinforced beam, assume the area of the entire cross section as here given.

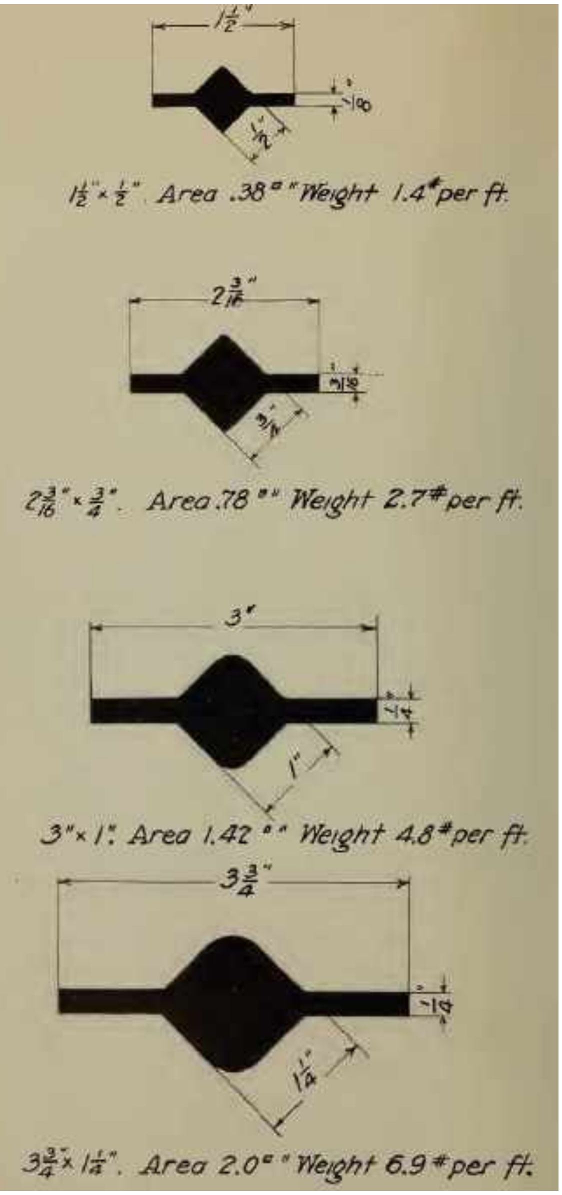
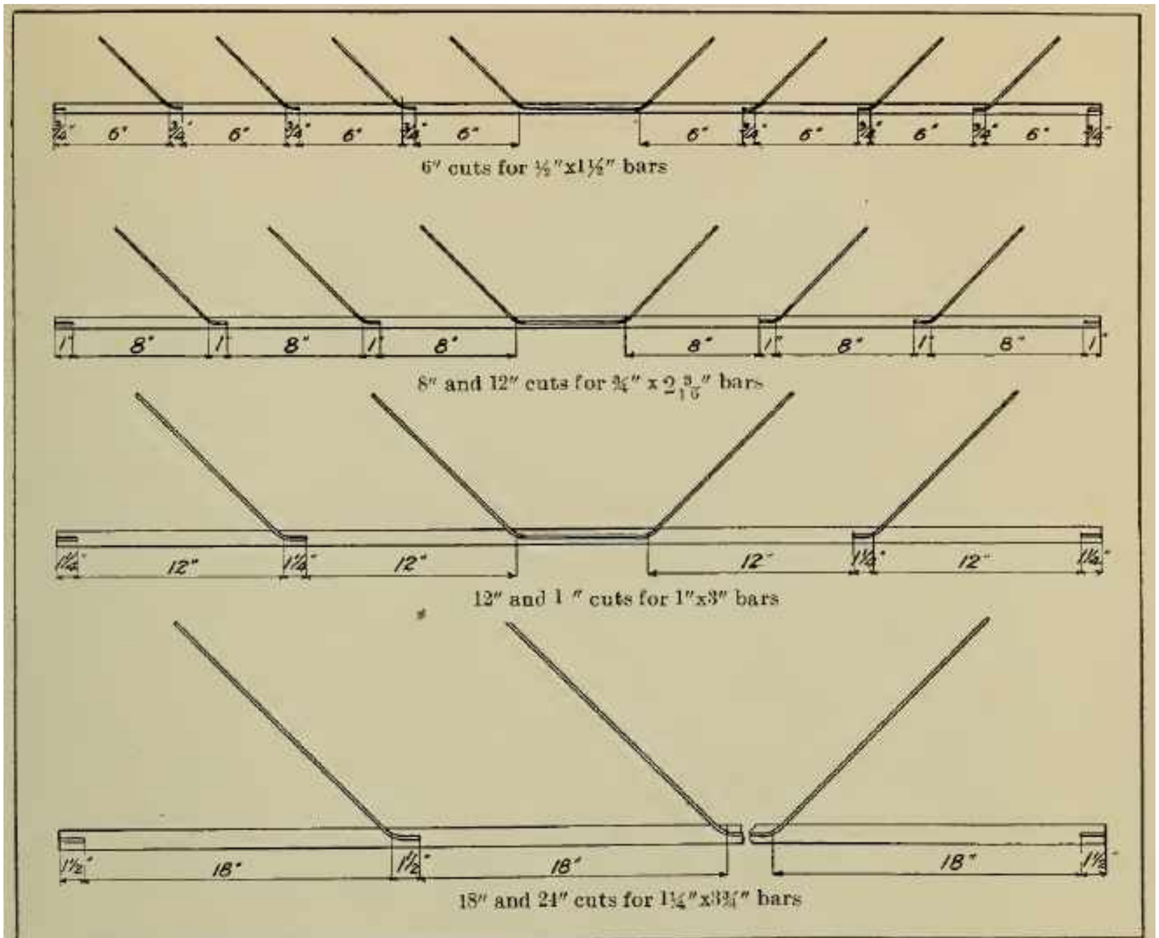


FIG. 13.



Bars kept in stock ready for immediate delivery, in any lengths with standard cuts.

FIG. 14.

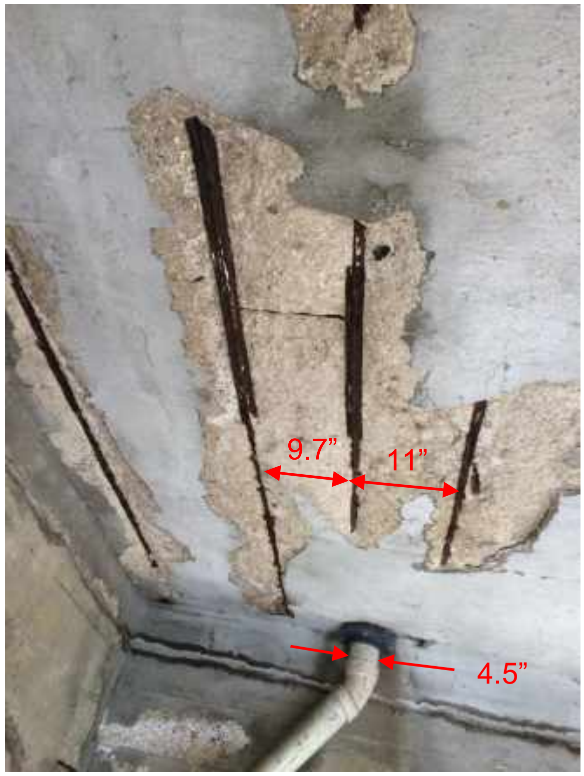
TranSystem determines the load carrying capacities of the deck, spandrel beam, and arch rib using an ASD approach. Given the limited as-built information available for this bridge, several assumptions must be made with respect to the Kahn bars. Review of TranSystem's load capacity calculations versus field observations and GRAEF's 2005 and 2015 analyses suggests an approach which in some cases is unconservative.

Deck – according to the original design drawings, ½" x 1 ½" Kahn bar reinforcing steel was to be placed transversely at 18" centers within a 6" thick deck. Using information available from a 1910 textbook, a rebar area of 0.41 in^2 spaced at 18" and a 6" thick deck was used to determine GRAEF's 2005 deck load rating factors. Straight bars with no fin cuts were assumed. The rating factors were updated in 2015 to account for higher concrete strength test results.

Scaling from photographs, TranSystem assumed a 7" rebar spacing and a rebar area of 0.38 in^2 using 1904 Kahn bar catalog information. This closer bar spacing results in a greater steel area and yields a greater deck bending capacity. Additionally, concrete cores indicate a 1" thick concrete overlay was placed on the original deck

Further review of existing information suggests deck capacity refinements are warranted in the TranSystems analysis:

- The drawings that GRAEF used to determine capacity in 2005 indicated 18 inch spacing of Kahn bars. Subsequent field investigations have revealed as-placed spacing of bars closer than 18 inches. However, these field observations are only a snapshot of areas, and are not comprehensive. We believe the assumption of a 7 inch bar spacing is not warranted, given the level of uncertainty regarding spacing, and the fact that wider spacings have been observed.
- Scaling photographs from GRAEF's 2015 inspection and 2018 site visit indicate a maximum deck rebar spacing at approximately 11" centers in the 3rd bay from the south, with an average spacing of approximately 10". The image below shows the spacing in comparison with the 4 ½" outside diameter drain pipe. Given the uncertainty that exists regarding reinforcement spacing, no less than a 10 inch spacing should be used to determine the governing bending capacity.



- Original design drawing deck cross sections and field observations of underside spalls (see image above) suggest the Kahn bar fins are bent up for most of the deck width. This will reduce the reinforcement areas assumed by both GRAEF and TranSystem.
- The 1904 Kahn bar literature suggests the full bar areas (square bar area plus bar fins) can be used to determine the strengths of reinforced beams. However, we do not believe this is an appopriate approach for two reasons. First, the bent up fins provide no bending strength. Second, even when bars are not bent up, the discontinuity of the steel where fins are transversely slit makes use of the fins questionable. From the 1904 Kahn bar literature Figure 14 and deck underside photograph earlier, only the middle 6” of the bar length has continuous uncut fin steel, and only this length should be considered effective as having the full square bar plus fin area. Regions beyond the middle 6” should consider the square bar area only.
- When using ASD to determine the capacity of reinforced concrete flexure members, AASHTO Standard Specifications 8.15.3 state that straight-line theory of stress and strain in flexure be used. TranSystems calculations appear to use working stresses in a Whitney Block approach to compute

the capacity of concrete elements, which is not a straight line method. Straight-line theory should be used for consistency with ASD methodologies.

Spandrel Beam – according to the original design drawings, (2) 3" x 1" Kahn reinforcing steel bars are placed in the bottom of the beams. Field observations and photographs of the heavily spalled southeast spandrel indicate there are no fins contributing to the reinforcement area, and approximately 1/16" surface section loss all around the 1" x 1" bars.

Further review of existing information suggests spandrel capacity refinements are warranted in the TranSystems analysis:

- Beam capacity calculations use Kahn bar areas that include the square bar plus fin areas in the middle 10-ft of the beam. Field observations of the southeast spandrel show that the fin areas should not be used because the fins are not present on the longitudal reinforcement. This is shown in the photograph below.

- From the 1904 Kahn bar literature Figure 14 for 3" x 1" bars, only the middle 18" of the bar length should be considered effective in having the full bar square bar plus fin area. Regions beyond the middle 18" should consider the square bar area only.

- The TranSystem report states that a spandrel beam depth of 3'-2" was used because the deck was poured monolithically with the beam. We believe that use of the entire 3'-2" depth is unconservative. Our field inspection and photos show cold joints between the deck and spandrel beam. Most of these joints are cracked and had been routed and filled with caulk. In addition, the original design drawings suggest the bent up spandrel beam Kahn bar fins do not project into the deck. 1904 Kahn bar literature Figure 14 shows 18" to 24" standard cuts for the 3" x 1" bar bent up fins which are not long enough to reach into the deck. In the absence of reinforcement crossing this degradated joint, we do not believe there will be sufficient shear transfer to allow for the deck and the beam to act in a composite fashion.

- When using ASD to determine the capacity of reinforced concrete flexure members, AASHTO Standard Specifications 8.15.3 state that straight-line theory of stress and strain in flexure be used. As explained above, this method had not been used in the TranSystem calculations. Straight-line theory should be used for consistency with ASD methodologies.

Arch Ribs – according to the original design drawings, various combinations of Kahn bar reinforcing steel are placed in the top and bottom of the arch ribs. Field observations and photographs of spalled regions indicate bent up fins inherent with the Kahn system. The same arch rib size and reinforcing used for GRAEF's 2005 and 2015 load ratings were used in the TranSystem calculations, however, TranSystem calculations indicates a capacity-to-demand ratio approach was used as opposed to an AASHTO based load rating approach.

- Arch rib capacities are determined using KDOT Column Expert software. Concrete and reinforcing steel strengths are input using ASD level stresses. It is unknown if the software is properly being used as an ASD tool as most reinforced concrete design software uses modern ultimate strength based equations. AASHTO Standard Specification 8.15.4 states that combined flexural and axial ASD load capacity of compression members is to be taken as 35% of that computed using the strength design methods of section 8.16.
- The demand-to-capacity approach uses a straight line methodology to determine capacity on a column's interaction diagram. The approach used in AASHTO's Manual for Bridge Evaluation uses a more refined two-line procedure that accounts for differences in the bending moment to axial load ratios of the dead loads versus the live loads.

Conclusions and Recommendations

Our largest concern with the TranSystem analysis is that the deck and spandrel beam member capacities are in some cases unconservative. Assumptions that lead to the unconservative capacities include use of reinforcing steel areas that are too large, and a spandrel beam depth that is too large. Member capacities that are too high yield capacity-to-demand ratios that are also too high, suggesting these bridge elements have strength to resist current code prescribed pedestrian live loads when they do not.

Other concerns include that member capacities should be determined using ASD methods as outlined in the AASHTO Standard Specifications for consistency with using service loading. Whereas using a capacity-to-demand ratio to check member adequacy is not wrong, load rating factors are normally used for bridge structures. Also, where inventory and operating levels of service are to be investigated, load rating factor equations are to be used, and only a single live load type (either pedestrian only or H5 service vehicle only) should be used when comparing the rating factors.

As a minimum we recommend the following refinements to the load calculations:

Deck

Recalculate the bending capacity using a bar spacing of 10" and a reinforcing steel area only considering the ½" x ½" square bar. ASD bending capacity to follow AASHTO Standard Specifications section 8.15.3.1.

Spandrel Beam

Recalculate the bending capacity using a reinforcing steel area considering the 1" x 1" square bars and 1/16" section loss all around due to corrosion. Use a maximum beam depth of 2’-8”. ASD bending capacity to follow AASHTO Standard Specifications section 8.15.3.1.

Arch Rib
ASD capacities for combined axial load and bending to follow AASHTO Standard Specifications section 8.15.4.

KGW:kgw

cc: Lori Rosenthal, P.E. (GRAEF)
John Kissinger, P.E. (GRAEF)

Meeting Minutes

Subject:	Lake Park Ravine Road Concrete Footbridge		
Prepared by:	Colleen Reilly, President, Lake Park Friends		
Location:	Conference call	**Date/Time:**	September 25, 2018 / 3:00 – 4:30 p.m. CT
Participants:	Karl Stave, Milwaukee County		Colleen Reilly, Lake Park Friends
	Kevin Wood, GRAEF		Phil Schultz, Lake Park Friends
	John Kissinger, GRAEF		Margaret DeMichele, Lake Park Friends
	Wes Weir, TranSystems		Steve Duback, Lake Park Friends
	Don Cartwright, TranSystems		

Notes

1 Overall project goal is to have a bridge that is safe, that is economically feasible (in terms of cost and longevity), and that is true to its historical value. Federal, state, and local historic preservation laws require consideration of rehabilitation first and foremost for historic structures. All agree bridge can be rehabilitated; however, differences in what is required during rehabilitation to achieve project goal. Purpose of call is to resolve those differences to help determine what is required to rehabilitate the bridge.

2 Bridge structural elements

a. Kahn bar system in deck. There is limited as-built information, so assumptions were made based on visual inspection. There remains a difference of professional opinion regarding the effective area of steel reinforcement in the deck. Kahn bar spacing is not 18” stated in the design plans and used by GRAEF but is also not consistently 7” throughout the deck as used by TranSystems. Could measure to resolve spacing, but GRAEF believes the effective area of Kahn bars is also of concern. The exposed steel on underside of bridge shows that the uncut fin steel in the transverse Kahn bars are not continuous; only the middle section appears to have the full square bar and fin area (this construction is consistent with the Kahn Manual, which states that the full bar areas can be used to determine strength). Even in middle section, GRAEF believes that Kahn bars have discontinuities which make the use of the fin steel questionable. TranSystems’ performed calculations that shows that the bridge has the capacity to support the 80 psf live load as per the original design plans based on scaled dimensions of the rebar from the underside of deck and the effective area of the Kahn bars per the Kahn design manual. All agreed that there is no evidence that the bridge is in an overstressed condition. TranSystems believes that refining these assumptions in the TranSystems calculations will result only in a change to the load ratings for vehicular traffic but would not significantly change pedestrian load capacity. GRAEF does not believe deck can support 90 psf pedestrian loading because of these as-built uncertainties related to the spacing of the deck bars and the effective area of the Kahn bars. The County wants to be conservative with the assumptions, especially given past incidents. The load rating methodologies are inherently conservative and are used on hundreds of bridges across the US. If want to retain the current deck, could resolve via a load test prior to or during design phase. May not be so important to fine tune the deck’s load rating if the County has the funds to replace the deck. If refined numbers are lower than 90 psf, could restrict the number of people and restrict vehicles on the bridge with bollards or other.

b. Spandrel beam depth. Remains a difference of professional opinion regarding the spandrel beam depth. GRAEF believes the caulked cracks on interior face of the spandrel suggest a cold joint, and that the haunch makes it difficult to see the cold joint on the exterior of the bridge. GRAEF believes the arches were poured first, then the spandrels, then the deck. TranSystems stated that even if there is a clear joint, it would not change the analysis of the entire beam; the rebar is continuous up through the deck.

Notes	
	GRAEF does not believe the rebar crosses the spandrel beam/deck plane. TranSystems proof of concept demonstrates design intent was achieved based on TranSytems analysis assumptions. All agreed that this could be resolved during the design phase and if needed, the spandrels could be strengthened.
3	Bridge longevity. Based on the additional material testing results (no evidence of ASR; low chloride levels), all agreed that the as long as the rehabilitation is conducted properly (good specifications and good quality control) and the rehabilitated bridge is maintained, the rehabilitated bridge could last at least 50 years. The concrete deterioration that is visible is due to lack of maintenance, minimal concrete cover, and the age of the bridge. Routine maintenance would include periodic application of a penetrating protective concrete sealant, inspections, etc, which would not necessarily be needed on a newly constructed bridge to achieve the same 50+ year life span..
4	Vaulted abutments. County states the vaulted abutments are in poor condition (large cracks, lack of steel reinforcement, eroding ravine slope undermining the foundation). TranSystems stated these curtain walls are architectural features (not structural) and are not connected to the main structure, but they could be replaced during a rehabilitation for low cost. GRAEF stated that these are structural elements supporting the deck, not curtain walls. At the wider overlook sections, there are concrete beam elements that help to support the deck. All agreed this could be evaluated during the design phase.
5	Call concluded at 4:30 p.m.

From: dwcartwright@transystems.com
Subject: RE: Lake Park Arch Bridge Report Review
Date: October 4, 2018 at 3:55 PM
To: ckreilly@outlook.com
Cc: Wesley.Weir@wsp.com

D

Colleen,

Following up on our discussion from earlier, TranSystems would like to provide clarification on the Khan bar reinforcement system as discussed in GRAEF's review of our analysis report. In particular, we do not agree with the assertion that the entire bar area cannot be included for flexural strength due to discontinuities in the outer fins.

The review of our report states that the Khan bars have transverse slits even where bars are not bent up, and we do not believe this to be true. The Khan bars are fabricated as one continuous bar with a diamond-shaped inner core and fins on the outside. In order to create the include shear reinforcing, the fins are cut with a small transverse slit and variable-length longitudinal cuts which then allow the bars to be bent up. The bars are not fabricated with these cuts pre-made, and the center portion of the bar is left continuous to allow for full capacity of the bars (core + fin) in flexure. This is shown in Fig 11 on Page 4 of GRAEF's review.

Furthermore, GRAEF states that only the middle 6" of the bar length is left continuous and uncut. We believe GRAEF has incorrectly interpreted the standard cut diagram (Fig 14 on Page 6 of their report). In this diagram, the center portion of the bars shown is intentionally **not** dimensioned, as this length would be customized based on the configuration of the member being reinforced. The purpose of this diagram is strictly to demonstrate the dimensions for these cuts for different bar sizes and lengths of bent shear bars. This diagram does not specify the uncut flexural length left in the middle of the bar, and the only reason it is drawn so short is likely just to fit all the detail needed on the page. In fact, the bottom diagram showing 18" and 24" cuts specifically has a break line shown in the middle of the bar.

Based on several field photographs of the deck underside, it appears very clear that the full bar is included for most of the length of the transverse deck with while the fins are bent up near the ends. This supports the concept that the Khan system was utilized to provide full bar area in the primary flexure areas and additional shear capacity near the ends. There are no photographs suggesting that the fins are cut transversely near the center of the span. As is the design intent of the Khan system, each bar is fabricated continuously and only cut specifically in areas where bars are being bent up to provide additional shear capacity.

I have attached a very brief markup which calls attention to these points. If you have any questions, please don't hesitate to let myself or Wes know.

Thanks,
Don

From: Colleen Reilly [mailto:ckreilly@outlook.com]
Sent: Friday, September 21, 2018 4:54 PM
To: Wesley.Weir@wsp.com; CL-Don Cartwright <dwcartwright@transystems.com>; margaret@demichele.com; P.Schultz@horizondbm.com; srduback@yahoo.com
Subject: Fwd: Lake Park Arch Bridge Report Review

I have not yet reviewed this, but wanted to get this to you.

Colleen Reilly, PMP
(414) 202-5730
ckreilly@outlook.com

Begin forwarded message:

From: "Stave, Karl" <Karl.Stave@milwaukeecountywi.gov>
Date: September 21, 2018 at 3:38:52 PM CDT
To: Colleen Reilly <ckreilly@outlook.com>
Subject: FW: Lake Park Arch Bridge Report Review

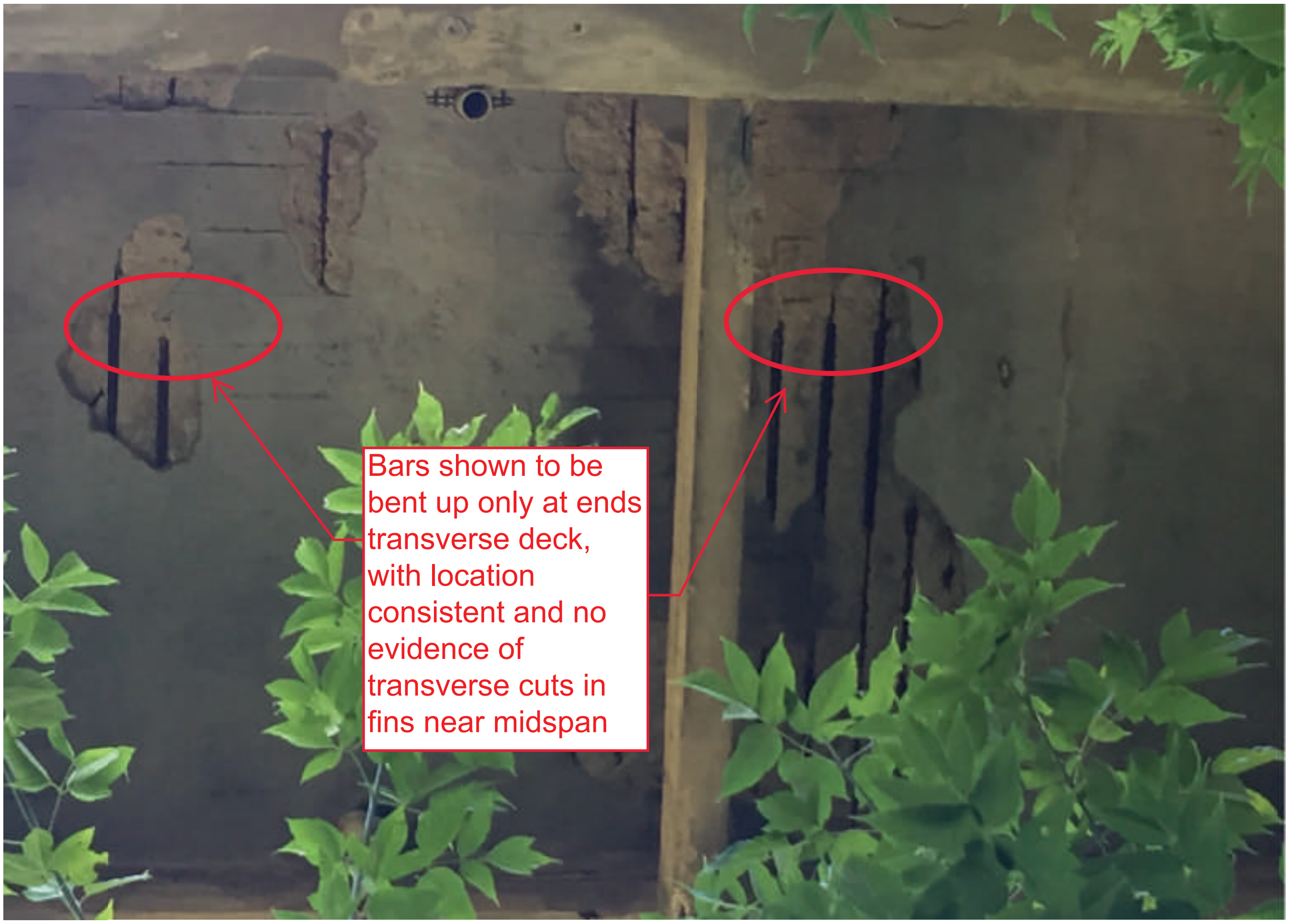
Colleen,

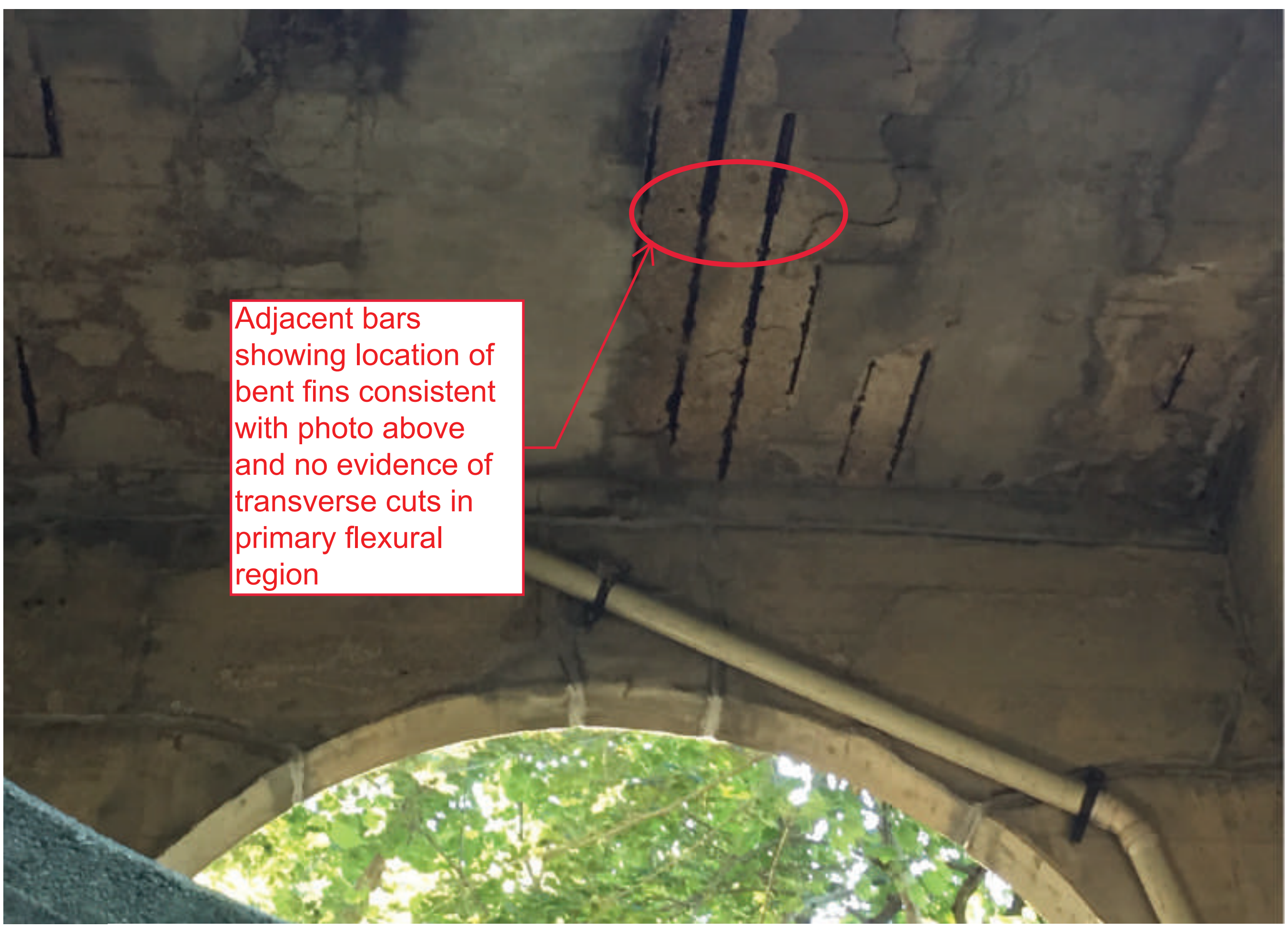
See attached review. I haven't reviewed it yet but wanted to get it to you before the weekend.

Thanks,

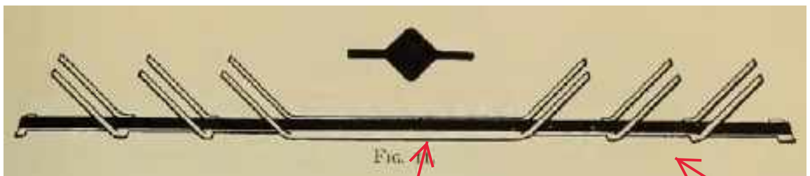
Karl Stave, P.E.
Architecture, Engineering & Environmental Services
DAS - Facilities Management Division
Milwaukee County
633 W. Wisconsin Ave.
Suite 1000
Milwaukee, WI 53203
(414) 278-4863

Lake Park Arch Bridge
Load Calcu…mments.pdf


Bars shown to be bent up only at ends transverse deck, with location consistent and no evidence of transverse cuts in fins near midspan


Adjacent bars showing location of bent fins consistent with photo above and no evidence of transverse cuts in primary flexural region

square bar at predefined lengths. These cuts allow the fins to be bent up to provide shear reinforcement for the concrete element. The images below from Kahn's 1904 Handbook illustrate this configuration.



Primary Flexure Zone: In this area, note that there are no transverse slits pre-fabricated in the Khan bar. The full bar (core plus fins) is intact as initially fabricated and is developed for flexural capacity in this region.

Primary Shear Zone (typ.): In this area, the fins are cut on each side with an L-shaped cut consisting of a small transverse slit and longitudinal cut between the fin and core. These cuts are done custom only where desired based on designer's intent with dimensions for cuts and connected material as shown in Fig 14. There are no pre-cuts made along the remaining length of the member.

These bars can have any standard size cuts as shown in Figure 11 and will be sent in lengths as ordered. In making calculations for strength of reinforced beam, assume the area of the entire cross section as here given.

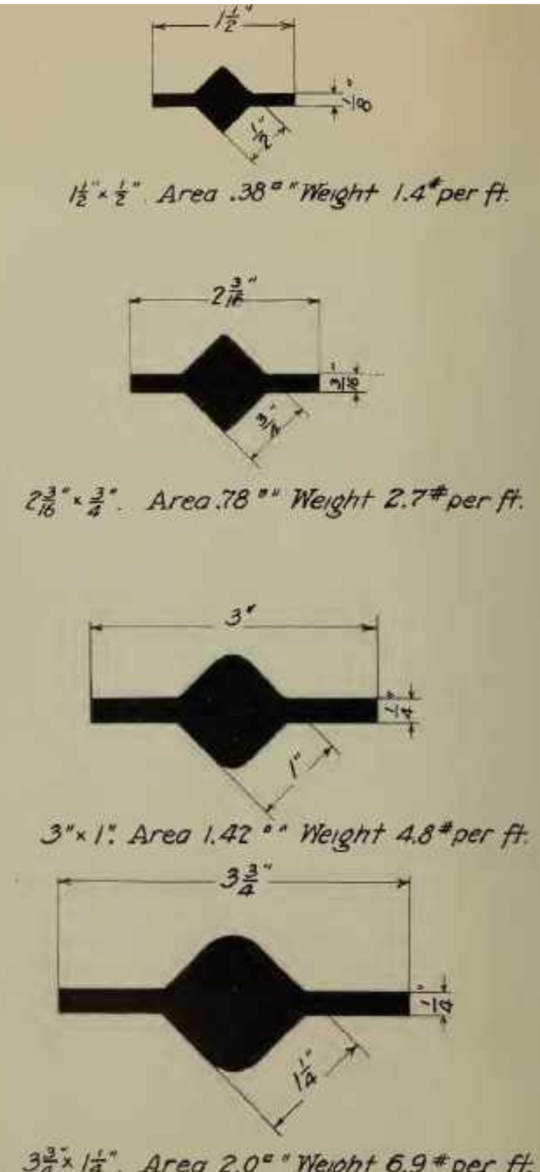
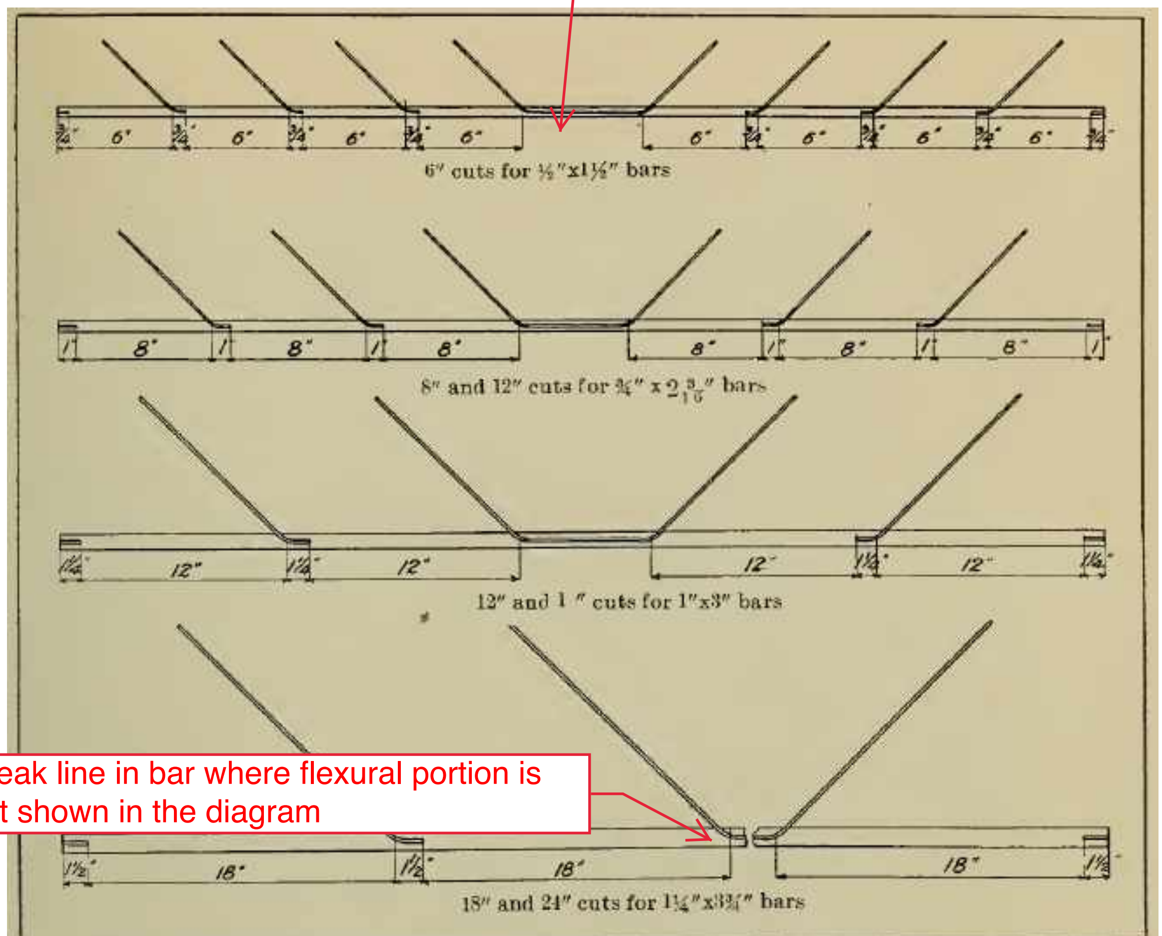


FIG. 13.

The intent of this figure is to demonstrate the typical lengths of cuts for the shear fins and the amount that is left uncut to provide connection to the inner core (3/4" for 6" cuts, 1" for 8" cuts, etc.). The length of bars of bars left uncut for flexural reinforcement are intentionally not included on this diagram, as this value would be customized based on the size and configuration of the member. (typ.)

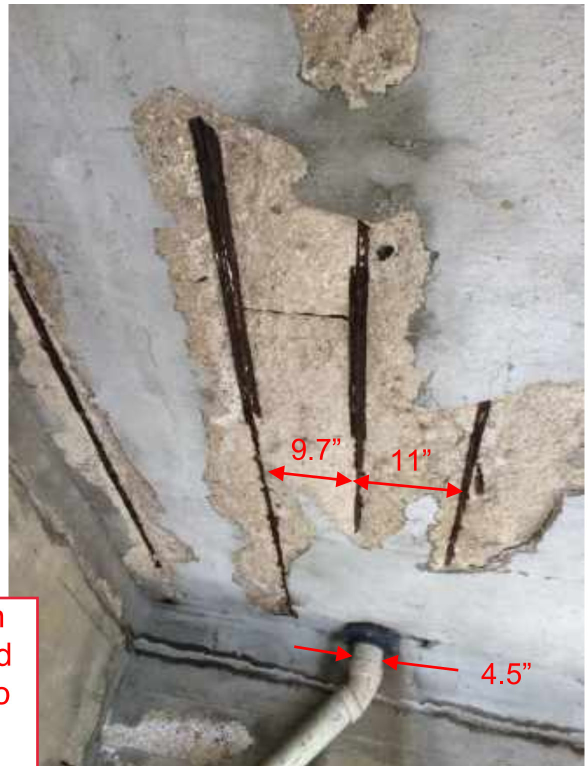


Bars kept in stock ready for immediate delivery, in any lengths with standard cuts.

FIG. 14.

TranSystem determines the load carrying capacities of the deck, spandrel beam, and arch rib using an ASD approach. Given the limited as-built information available for this bridge, several assumptions must be made with respect to the Kahn bars. Review of TranSystem's load capacity calculations versus field observations and GRAEF's 2005 and 2015 analyses suggests an approach which in some cases is unconservative.

Deck – according to the original design drawings, ½" x 1 ½" Kahn bar reinforcing steel was to be placed transversely at 18" centers within a 6" thick deck. Using information available from a 1910 textbook, a rebar area of 0.41 in^2 spaced at 18" and a 6" thick deck was used to determine GRAEF's 2005 deck load rating



This is not the case. The middle portion of the bar is specifically not dimensioned in Fig 14. The intent of the diagram is to show the standard cuts in shear zones, while the uncut portion of bar in the middle could be left at any length. It was likely drawn for a very short length in the detail to save space on the page.

esign drawing deck cross sections and field observations of spalls (see image above) suggest the Kahn bar fins are bent st of the deck width. This will reduce the reinforcement areas assumed by both GRAEF and TranSystem.

- The 1904 Kahn bar literature suggests the full bar areas (square bar area plus bar fins) can be used to determine the strengths of reinforced beams. However, we do not believe this is an appopriate approach for two reasons. First, the bent up fins provide no bending strength. Second, even when bars are not bent up, the discontinuity of the steel where fins are transversely slit makes use of the fins questionable. From the 1904 Kahn bar literature Figure 14 and deck underside photograph earlier, only the middle 6” of the bar length has continuous uncut fin steel, and only this length should be considered effective as having the full square bar plus fin area. Regions beyond the middle 6” should consider the square bar area only.
- When using ASD to determine the capacity of reinforced concrete flexure members, AASHTO Standard Specifications 8.15.3 state that straight-line theory of stress and strain in flexure be used. TranSystems calculations appear to use working stresses in a Whitney Block approach to compute