# LOAD RATING REPORT
# BRIDGE NO: BEL-00040-23.265, SFN 0701599

Blaine Hill Viaduct

U.S. 40 over Wheeling Creek, C.R. 10, and Abandoned R.R.

**Blaine, Ohio**

**Submitted: October 5, 2023**

**LOAD RATING REPORT**

BEL-40-23.37, SFN 0701599

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## INTRODUCTION

Michael Baker International (Michael Baker) was tasked by ODOT District 11 to perform a load rating of the Blaine Hill Viaduct Bridge (BEL-40-23.37). Constructed in 1932 as open spandrel concrete arch bridge, the bridge underwent a major rehabilitation in 1982 that replaced the integral concrete deck and floorbeams with a composite, adjacent box-beam superstructure atop new floorbeams. The approximately 754' long structure is composed of four unique arch spans which support slab beams, one slab span, and six box beam spans which together with the slab span comprise the approach spans. The elements of the superstructure which required load ratings include the box beams and slab beams, the slab span, the floorbeams, spandrel columns, and arch ribs. The arch ribs and spandrel columns date to the original structure, built in 1932, whereas the floorbeams, box beams, and slab beams and slab span were constructed in 1982. Both the slab beams and box beams are composite with a 5" thick reinforced concrete deck. The cross section of the bridge deck consists of a 36" wide slab beam or box beam at the center of the deck with five 48" wide box beams or slab beams on each side (see Figures A and B).

Per the ODOT Bridge Design Manual (BDM) 919.3.1(H), if the load rating indicates posting is necessary, then the bridge shall be analyzed by both LFR and LRFR and the larger rating factor used to determine if posting can be avoided. Typically, this process involves rating first using the LRFR method, and if legal vehicles' LRFR ratings indicate a need for load posting, then switching to the (LFR) to compute rating factors. This procedure of initially rating in LRFR and using LFR if needed, was followed.

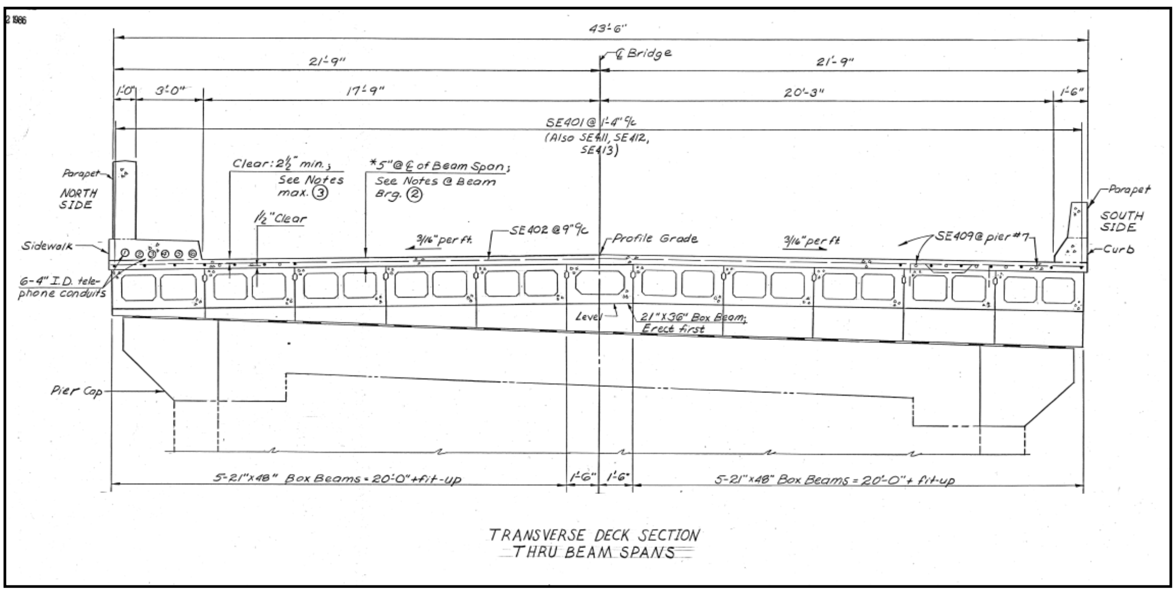


**Figure A. Typical Section of Box Beam Approach Spans**

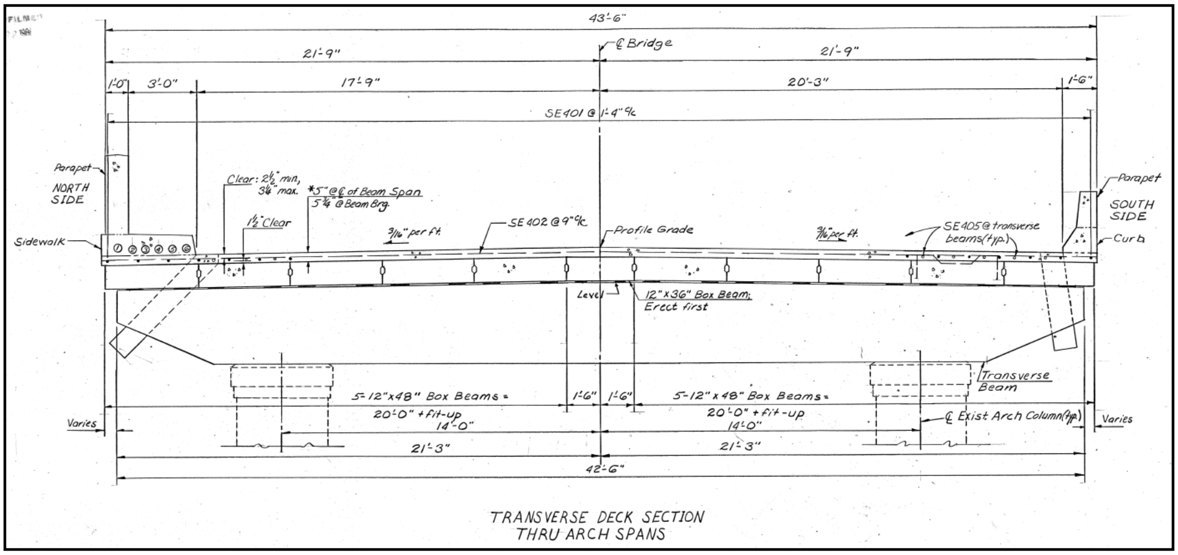


**Figure B. Typical Section of Slab Beam Arch Spans**

## DESCRIPTION OF MODELING ANALYSIS AND APPROACH

Per BDM 920.1, AASHTOWare BrR (BrR) is used for load rating purposes whenever possible. While the slab span, slab beams and box beams on the bridge can be efficiently rated using BrR, the software is not capable of rating the floorbeams, spandrel columns, and arch ribs. To accurately compute load ratings for these members, a 3D finite element model (FEM) was created using Midas Civil to generate forces and moments in the arch ribs, spandrel columns, and floorbeams which were then analyzed using spreadsheets to calculate the members' capacity and then rating factors.

The material strengths provided in the 1982 rehabilitation plans were used for load rating the box beams, slab beams, and floorbeams. However, the concrete strength of the arch ribs and spandrel columns, cast in 1932, was unknown. Typical practice is to consult the ODOT Bridge Design Manual (BDM) Table 926-1, which provides a material strength based upon year of construction. While these material strengths provided are conservative and intended to keep engineers from overestimating the capacity of older structures, modeling a lower concrete compressive strength in the 3D FEM model may underestimate members' moments and forces. This is because FEM models distribute load based upon the relative stiffness of each member in the model, and the modulus of elasticity for concrete is dependent upon its compressive strength. Thus, cores from the original bridge piers were tested to better match both the stiffness of the concrete and its compressive strength. Concrete cores were taken from pier 3, 4 and 5 bases on July 19, 2023. Refer to observed compressive strengths in Figure C. With a small sample size (3 cores), it can be unconservative to draw statistical conclusions assuming the data follows a normal distribution. This is because a normal distribution treats sample statistics with the same confidence, whether derived from 3 samples or 30. Another distribution, Student's t-distribution, is more appropriate to use with small sample sizes (common rule of thumb is less than 30 samples). Using the 3 core samples and t-distribution, conservative sample statistics could be determined. It was estimated with 90% confidence that the true mean of compressive strength was equal to or greater than 5400 psi. In other words, if we were to take many more core samples, we're very confident that the new mean calculated from the new data would be greater than 5400 psi. This lower bound estimate of compressive strength was assigned to arch rib and spandrel column material properties thereby more accurately modeling the relative stiffness of the structure.

| Core No. | Location on Bridge | Unconfined Concrete Compressive Strength (psi) |
|---|---|---|
| C-1 | Pier 3 Base | 5,418 |
| C-2 | Pier 4 Base | 7,028 |
| C-3 | Pier 5 Base | 7,822 |

**Figure C. Concrete Core Sample Data**

**Box Beam, Slab Beam, and Slab Span Analysis and Load Rating**

For efficiency, the box beam spans were separated into groups to focus on controlling BrR rating model cases. The six box beam spans are composed of two unique box beam configurations: Beam Spans B, F, and G utilize the same construction as do Beam Spans C, D, E. Because Beam Span B has no skew, it was modeled separately from Beam Spans F and G which have significant skew. Since Beam Span F is slightly longer than Beam Span G, only Beam Span F was modeled and any deterioration found during the inspection in Beam Span G was included in the Beam Span F model, which is conservative. Likewise, for Beam Spans C, D, E, only the longest span was modeled (Beam Span E) and all deterioration from these three spans noted during the inspection was included in the Beam Span E model. See the Appendix for a color coded map identifying and grouping the similar box beam and slab beam spans for analysis.

The deck above the arch spans consists of slab beams composite with the reinforced concrete deck. In general, as the spandrel column spacing is consistent throughout the four arches, the span lengths for the slab beams are also consistent, and only two different prestressed strand patterns are used in the spans. Since the loading does not change along the length of the bridge, only the longest two slab beam spans were rated. Field noted deterioration was accounted for by deducting strands as necessary. Since the slab beams nearest the expansion joints were the longest spanning slab beams and had the worst deterioration, those slab beam spans were rated in each span.

The field inspection found numerous prestressed strands exposed or broken, typically at expansion joint locations. Because the damage was always confined to a beam end and based upon Michael Baker's previous prestressed beam rating experience, it was decided to deduct from the beam cross section broken strands and estimate a debonded length for strands exposed at beam ends.

The slab span has a high skew (~48 degrees) at the rear abutment and no skew at pier 1. Due to this difference in skew, it was primarily detailed as a triangular slab, however it was necessary to modify the triangular shape of the slab to allow BrR to rate it. A rectangular slab was modeled using the longest length of the triangular slab beam within the vehicular travelway as the span length of the slab. As the 1982 plans show that the rebar size and spacing varies along the width of the triangular slab, the total reinforcing area was added together and then evenly distributed across the BrR modeled rectangular slab.

**Arch Rib, Spandrel Column, and Floorbeam Analysis and Load Rating**

From previous experience with similar arch bridges, modeling the construction sequence is important to accurately capturing dead load effects throughout the bridge. This was accomplished using Midas Civil's Construction Sequencing, which closely followed the actual construction procedure utilized when the bridge was constructed and rehabilitated. The construction sequencing allows the bridge elements to deflect together as additional elements and loads are applied prior to the deck curing, which then adds rigidity to the structure. If construction sequencing was not considered, the model would assume the deck was cast simultaneously with the arch ribs and spandrel columns, which could cause erroneous dead load moments in these supporting elements. Thus, details of the original construction sequence and the 1982 rehabilitation were included in the analysis approach.

As mentioned above, the 3D FEM model is sensitive to the relative stiffness of the defined bridge members. One aspect that affects the relative stiffness is the 7% longitudinal grade. Along this grade, tapered spandrel column heights differ, so modeling included this grade to better represent column stiffnesses. The model also accounts for each span's arch ribs having a unique span length, radius, and tapering thickness. The pier bases, to which the arch ribs are anchored, were also included in the model with fixed supports at their footings.

The load path from the deck to the spandrel column was modeled as shown in Figure D.

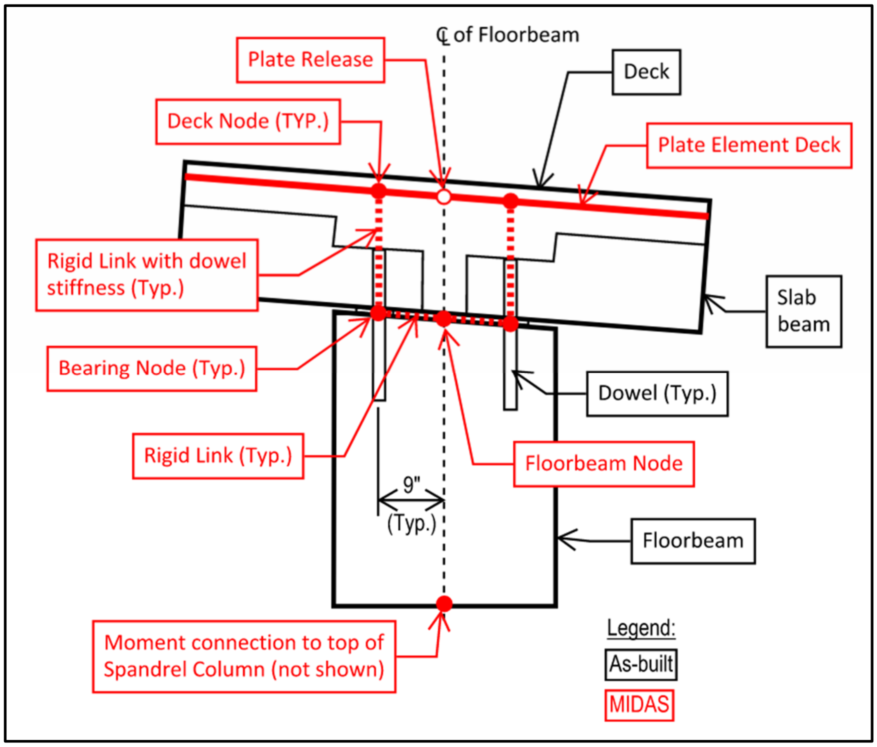


**Figure D. Elevation View of Deck to Floorbeam Connection**

The floorbeams were modeled as beam elements with a moment connection to the top of the spandrel columns to match the details shown in the plans. Since the slab beams were rated using BrR and independent of the Midas model, the slab beams and deck were modeled as a single plane of plate elements with a defined thickness equal to the deck thickness for accurate transverse load distribution. The plate element deck included releases at the expansion joint locations to simulate deck discontinuity at the joints.

Based upon the 1982 rehabilitation plans and 2010 rehabilitation plans, there is only about 5” of deck concrete, reinforced longitudinally by a single row of #4’s at 7” spacing. It is realistic to expect that beyond deck discontinuity at joints, the deck will also crack in negative moment over its supports. Observed transverse deck cracking at pier 3 supports this assumption. Accordingly, plate element releases were also assigned to the deck elements over the floorbeams to allow the deck to hinge.

As seen in Figure D, the slab beams are simply supported and their ℄ of bearing is located 9” away from the ℄ of the floorbeam. To replicate this condition in the model, nodes were placed 9” from the ℄ of the floorbeam to receive the slab beam reaction, and rigid links were used to connect these nodes to the ℄ of the floorbeam. Per the 1982 rehabilitation plans, the slab beams are anchored to the floorbeam using a single dowel, so a dowel stiffness was computed and assigned to the rigid link connecting the slab beam plate elements to the floorbeam.

Midas Civil's live load function can operate on either surface lanes or line lanes. Since creation of a permit tool is part of Michael Baker's scope of services, the line lanes were chosen to allow for generation of influence lines which will be used to create the permit tool. All live loads shown on the BR100 rating form were input into the model, and appropriate impact and multiple presence factors assigned. Since the bridge deck width can fit up to three lanes of vehicles at a time, lanes were assigned that maximized load on either edge of the bridge deck or the center of the deck using Midas Civil's Moving Load Cases to determine the governing loading on each element of the bridge. The Moving Load Cases use multiple presences factors in conjunction with varying numbers of vehicles to produce that governing load. Particularly for the arch rib and spandrel column elements, which are governed by a combination of axial force and flexure, the concurrent force option was activated in Midas Civil so that concurrent forces, instead of a force envelope, could be used for generating rating factors. The 3D model is shown in isometric view in Figure E.

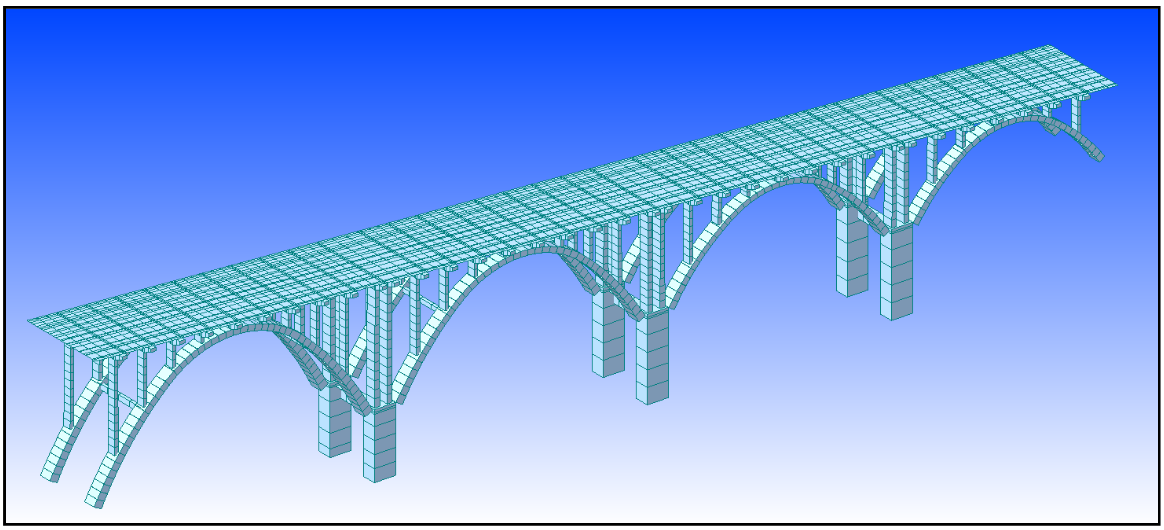

**Figure E. Isometric View of Arch Spans' 3D Model**

## LOAD RATING RESULTS

Given the complexity of the bridge, different approaches were taken to load rate various elements of the bridge. See Figure F for a visual representation of what was load rated and for which force effects. Further explanation of the process is provided below.

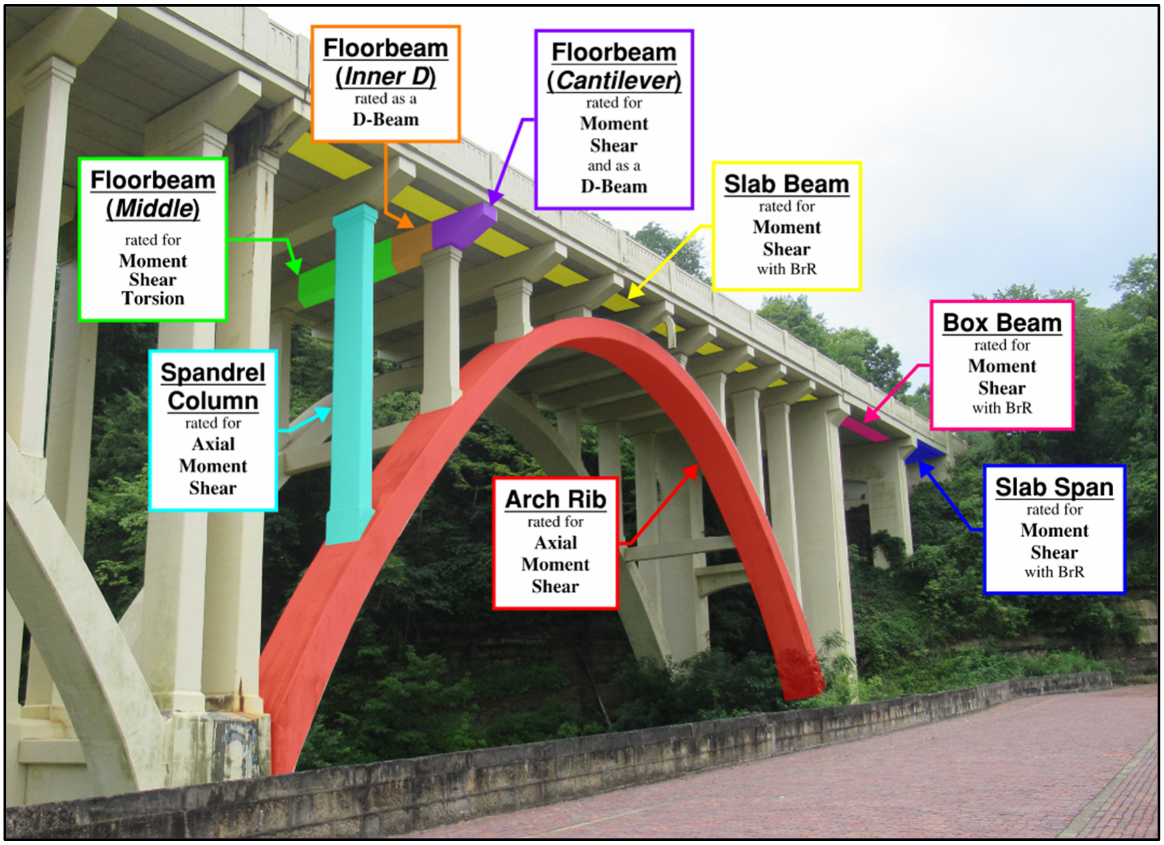


**Figure F. Representative Graphic with Location and Type of Load Ratings**

### Slab Span

As mentioned previously, the triangular slab span was modeled as a rectangle to allow for BrR input and load rating. The slab is governed by midspan flexure for all vehicles, and the LRFR rating factors shown in Figure G were obtained from the BrR slab model.

| SLAB SPAN | |
|---|---|
| **TRUCK** | **RATING FACTORS** |
| HL-93 INV | **0.801** |
| HL-93 OP | **1.039** |
| 2F1 | **2.354** |
| 3F1 | **1.691** |
| 5C1 | **1.804** |
| Type 3 | **1.804** |
| Type 3-3 | **2.19** |
| Type 3S2 | **1.923** |
| SU4 | **1.519** |
| SU5 | **1.409** |
| SU6 | **1.315** |
| SU7 | **1.315** |
| EV2 | **1.522** |
| EV3 | **1.169** |
| RPL 60T | **1.729** |
| RPL 65T | **1.625** |

**Figure G. Summary of Slab Span Load Rating Factors**

## Box and Slab Beams

For clarity, box beams without voids are referred to as "slab beams" for this report. Following the creation of the BrR models for the box beam and slab beam spans, rating factors were compiled. In Arch Span B, slab beam B126 was found to have very little capacity remaining after deduction of broken strands and debonding of exposed strands. Since ODOT took action to restrict traffic from this portion of the bridge, slab beam B126 is not included in the rating results. See the Appendix for slab beam B126 calculations and memo. The following tables in Figure H show governing rating factors for the remaining slab beams and box beams which continue to see live load following the lane closure.

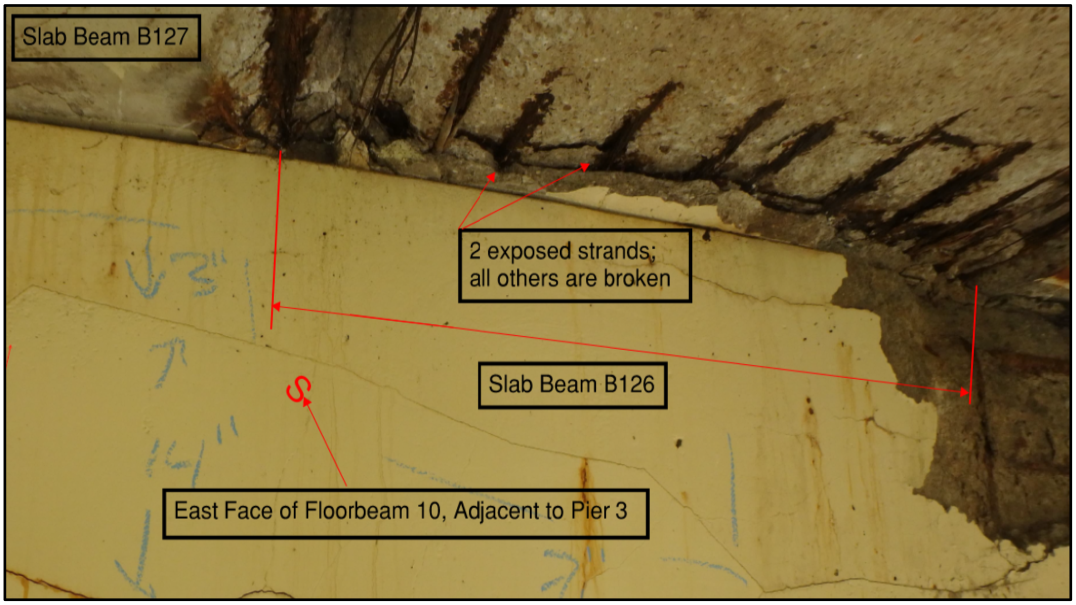


**Photo: Western End of Slab Beam B126 at Pier 3**

| BOX BEAM SPAN | | |
|---|---|---|
| TRUCK | RATING FACTORS | GOVERNING LOCATION |
| HL-93 INV | **1.401** | Beam Span C, B396 |
| HL-93 OP | **1.955** | Beam Span F, B429 |
| 2F1 | **4.591** | Beam Span C, B396 |
| 3F1 | **3.156** | Beam Span C, B396 |
| 5C1 | **3.280** | Beam Span C, B396 |
| Type 3 | **3.235** | Beam Span F, B429 |
| Type 3-3 | **3.616** | Beam Span F, B429 |
| Type 3S2 | **3.397** | Beam Span F, B429 |
| SU4 | **2.870** | Beam Span C, B396 |
| SU5 | **2.649** | Beam Span C, B396 |
| SU6 | **2.476** | Beam Span C, B396 |
| SU7 | **2.339** | Beam Span C, B396 |
| EV2 | **2.714** | Beam Span F, B429 |
| EV3 | **2.240** | Beam Span F, B429 |
| RPL 60T | **3.240** | Beam Span F, B429 |
| RPL 65T | **2.716** | Beam Span F, B429 |

| SLAB BEAM SPAN | | |
|---|---|---|
| TRUCK | RATING FACTORS | GOVERNING LOCATION |
| HL-93 INV | **1.090** | Arch C, B227 |
| HL-93 OP | **1.413** | Arch C, B227 |
| 2F1 | **3.171** | Arch C, B227 |
| 3F1 | **2.394** | Arch C, B227 |
| 5C1 | **2.394** | Arch C, B227 |
| Type 3 | **2.394** | Arch C, B227 |
| Type 3-3 | **2.908** | Arch C, B227 |
| Type 3S2 | **2.626** | Arch C, B227 |
| SU4 | **2.074** | Arch C, B227 |
| SU5 | **2.000** | Arch C, B227 |
| SU6 | **1.930** | Arch C, B227 |
| SU7 | **1.930** | Arch C, B227 |
| EV2 | **1.940** | Arch C, B227 |
| EV3 | **1.862** | Arch C, B227 |
| RPL 60T | **2.124** | Arch C, B227 |
| RPL 65T | **1.968** | Arch C, B227 |

**Figure H: Summary of Box Beam and Slab Beam Rating Factors**

**Spandrel Columns and Arch Ribs**

The load rating of spandrel columns and arch ribs was performed as an iterative process, evaluating relative stiffnesses in the 3D FEM model. Michael Baker initially rated the spandrel columns' axial forces and moments assuming uncracked section properties to establish a baseline load rating. Low rating factors were initially calculated for many of the shorter, stiffer spandrel columns near the crown of the arch. This was predictable as these modeled sections attracted load as a stiff uncracked section, but had capacity defined by a fully cracked section. In experience, these stiff elements crack, lowering their relative stiffness, and load is redistributed to other elements.

The next step in the analysis is to develop a relationship between load and stiffness to account for the cycle of high load, cracking and load redistribution per updated relative stiffnesses. This nonlinear relationship between load and stiffness can be defined using moment curvature analysis. For reinforced concrete, this relationship has important points such as first cracking, first yield of tension steel, complete yield of steel and hinging. While bridges of this type should not be posted for first cracking, it was decided the columns should not be allowed to hinge either, as this presents a serviceability concern. Therefore, an effective stiffness representing 30% of the uncracked spandrel column section was chosen, limiting deformations to just beyond first yield of the tension steel. Note that this partially cracked stiffness reduces the load, while capacity is still conservatively calculated assuming the section is fully cracked. This partially cracked stiffness results in all legal and permit loads to pass rating, while also limiting serviceability issues, i.e. the extreme level of cracking associated with hinging. Michael Baker utilized Midas General Section Designer's Moment Curvature function to perform this nonlinear analysis. For spandrel columns failing in the baseline model, this reduction in stiffness was applied to the FEM model using the Section Stiffness Scale function in Midas. The updated model was reanalyzed, and results were used to compute new rating factors for the spandrel columns. As predicted, the reduction in stiffness of the shorter spandrel columns, which had produced low rating factors in the uncracked baseline model, resulted in these members attracting less load as some of their load was distributed to stiffer elements. A comparison of these short column elements' rating factors in the baseline condition vs. the cracked condition simulated using the moment curvature analysis is shown in Figure I.

| SPANDREL COLUMNS Baseline (Uncracked) Analysis | | |
|---|---|---|
| Truck | Controlling Rating Factors | Element |
| HL-93 INV | 0.378 | 25_I |
| HL-93 OP | 0.489 | 25_I |
| Controlling Legal (SU7) | 0.591 | 25_I |
| EV3 | 0.574 | 25_I |
| RPL 60T | 0.612 | 25_I |
| RPL 65T | 0.562 | 25_I |

| SPANDREL COLUMNS Refined (Cracked) Analysis | | |
|---|---|---|
| Truck | Controlling Rating Factors | Element |
| HL-93 INV | 0.701 | 25_I |
| HL-93 OP | 0.909 | 25_I |
| Controlling Legal (SU7) | 1.116 | 25_I |
| EV3 | 1.081 | 25_I |
| RPL 60T | 1.129 | 25_I |
| RPL 65T | 1.060 | 25_I |

**Figure I. Comparison of Uncracked and Cracked Spandrel Column Rating Factors**
Note: Element 25_I is a short column in Arch Span A

Following the reduced stiffness for select spandrel columns, the redistribution of spandrel column forces resulted in legal and permit load rating factors greater than 1.0. However, there was one arch rib element, near the crown of arch span B, that had legal and permit load rating factors below 1.0. Moment curvature analysis was performed on this arch rib element, and it was determined that the effective stiffness of this arch rib element could be bounded by using 36% of the uncracked arch rib section stiffness without serviceability concerns. Once this arch rib element's stiffness had been updated in the FEM model using the Section Stiffness Scale, the model was reanalyzed, and rating factors were generated for both the arch ribs and the spandrel columns. Through this iterative approach to evaluate cracking, simulated using stiffness reduction of select members, LRFR rating factors were above 1.0 for all legal vehicles and it was not necessary to simulate further cracking.

Results from the baseline and refined model were also used to calculate shear rating factors for spandrel columns and arch ribs but were found not to control in any case. Torsion was not rated for arch ribs nor spandrel columns. These elements were not likely explicitly designed for torsion. Given the minimal stirrup reinforcement (#4s @ 1'-6" in columns, #5s @2'-0" in arch ribs), they likely would fail the current AASHTO LRFD capacity equations for design. The current design equations assume the section has significantly cracked and only steel resists torsion. For bridges with two ribs, like this bridge, papers such as "Arch Bridges" by Douglas A. Nettleton, note that "live load eccentricity is carried by an increase in vertical load to the ribs on the side of the eccentricity and a decrease to the other ribs" and torsion is not of concern in this region. With no obvious torsional cracking visible, the assumption that only steel resists torsion is considered overly conservative.

Figure J below shows an elevation view Arch Span B with the results from the refined analysis. Spandrel column and arch rib elements that were modeled to be cracked are highlighted in red. The three other arch spans also have cracking at similar spandrel column locations where the column frames into the arch rib.

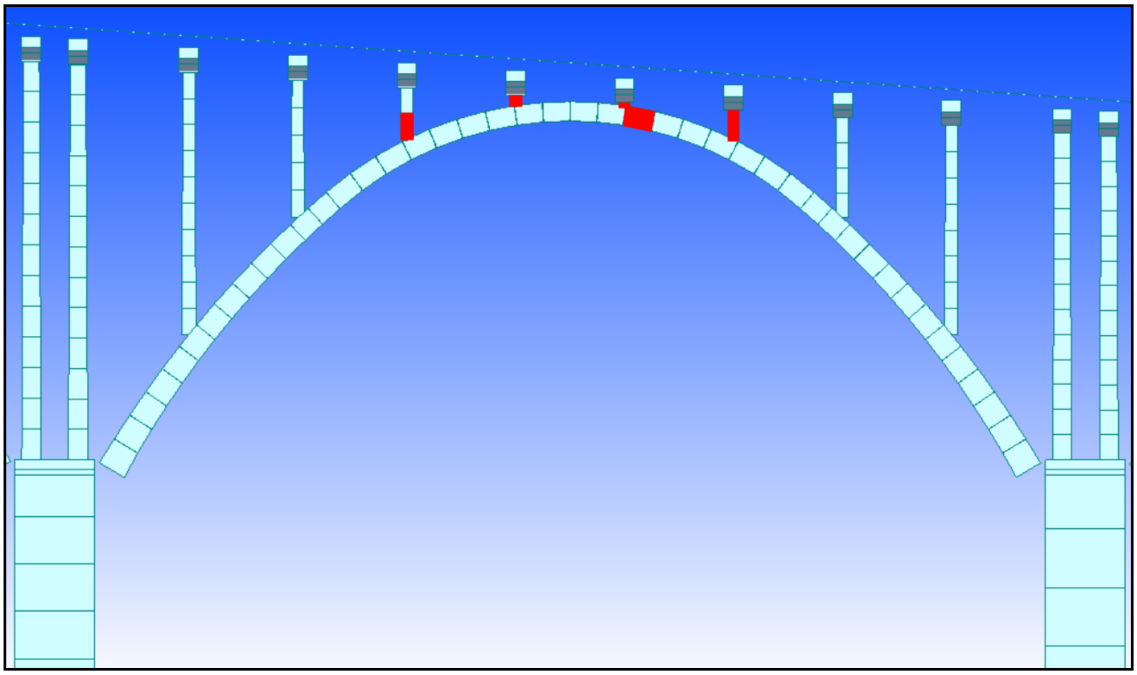

Figure J. Arch Span B with cracked arch rib and spandrel column elements in red

Due to the bridge's general appraisal rating of 5, a condition factor of 0.95 was applied to all spandrel column and arch rib capacities for all load effects. The summary of rating factors for the governing spandrel columns and arch ribs are shown in Figure K.

| SPANDREL COLUMNS | | |
|---|---|---|
| TRUCK | RATING FACTORS | |
| | P-M | SHEAR |
| HL-93 INV | 0.701 | 1.540 |
| HL-93 OP | 0.909 | 2.332 |
| 2F1 | 2.649 | 5.656 |
| 3F1 | 1.403 | 4.518 |
| 5C1 | 1.442 | 4.462 |
| Type 3 | 1.461 | 4.664 |
| Type 3-3 | 1.765 | 5.491 |
| Type 3S2 | 1.579 | 4.673 |
| SU4 | 1.320 | 4.048 |
| SU5 | 1.316 | 3.770 |
| SU6 | 1.212 | 3.608 |
| SU7 | 1.116 | 3.448 |
| EV2 | 1.135 | 3.672 |
| EV3 | 1.081 | 3.413 |
| RPL 60T | 1.129 | 3.229 |
| RPL 65T | 1.060 | 2.474 |

| ARCH RIB | | |
|---|---|---|
| TRUCK | RATING FACTORS | |
| | P-M | SHEAR |
| HL-93 INV | 0.738 | 3.712 |
| HL-93 OP | 0.960 | 4.811 |
| 2F1 | 1.986 | 9.793 |
| 3F1 | 1.446 | 6.955 |
| 5C1 | 1.471 | 7.099 |
| Type 3 | 1.454 | 7.125 |
| Type 3-3 | 1.745 | 8.533 |
| Type 3S2 | 1.603 | 7.802 |
| SU4 | 1.287 | 6.193 |
| SU5 | 1.189 | 5.670 |
| SU6 | 1.096 | 5.263 |
| SU7 | 1.071 | 4.940 |
| EV2 | 1.252 | 5.964 |
| EV3 | 1.066 | 5.268 |
| RPL 60T | 1.050 | 5.324 |
| RPL 65T | 0.969 | 4.732 |

**Figure K. Summary of Spandrel Column and Arch Rib Rating Factors**

While the arch ribs and spandrel columns do not meet HL-93 design loadings, they do rate for all Ohio legal loads as well as all AASHTO vehicles. Only the RPL 65T doesn't pass, which was rated to 97% of demand.

### Floorbeams

As can be seen in Figure F, the analysis of the floorbeam was divided into three regions. The first region is labeled the "cantilever" region. During the 8/24 post-inspection meeting with ODOT, there was concern over the condition of the floorbeam at Pier 5 (pictured below) and rating this region was elevated to a top priority.

**Photo: Southeast face of Floorbeam at Pier 5**

Floorbeams, such as the one identified in the above photograph, are located at deck joints. The floorbeams at deck joints exhibit high levels of deterioration compared to other floorbeams. Deterioration was most extreme at the cantilever region. Section and reinforcement loss was modeled in this location and a condition factor of 0.85 was applied. This region was first modeled as a B-Beam (traditional beam which assumes linear strain profile) and was found to have adequate capacity for STR I moment, shear, and torsion. The region was then modeled as a D-Beam (beam regions where a linear strain profile is inappropriate to assume) and rated using a Strut-and-Tie model. Due to the geometry and loading in this area, as discussed in AASHTO 5.5.1.2.1, this region of the floorbeam is a D-Beam and use of Strut-and-Tie in this region is appropriate. Ratings for all legal loads pass. Given its location and severe degradation, this cantilever was considered the worst case. Therefore, this floorbeam cantilever was considered to envelope the behavior for all floorbeam cantilevers for load rating purposes, as documented in Pier 5 Floorbeam Cantilever Load Rating Memo.

After the cantilever was deemed not to be an immediate concern, a second "inner D" model was created. This model captured the D-Beam behavior of the floorbeam to the inside of the support for a distance approximately equal to the depth of the floorbeam beyond the face of support. Since this region was generally in better condition than the cantilever, no section or reinforcement loss was assumed. However, a condition factor of 0.95 was applied to acknowledge deterioration that had occurred in this region.

Finally, the middle section of the floorbeam was modeled as a B-beam since it was sufficiently far away from the supports. Shear, moment, and torsion was rated for this region and a condition factor of 0.95 was applied. The summary of rating factors for the floorbeams are shown in Figure L.

| FLOORBEAM RATINGS | | | | | |
|---|---|---|---|---|---|
| **Truck** | **Cantilever S&T** | **Inner D S&T** | **Middle B-Beam** | **Controlling Model** | **Controlling RF** |
| HL-93 INV | **0.926** | **0.690** | **0.645** | Middle B-Beam | **0.645** |
| HL-93 OP | **1.200** | **0.897** | **0.944** | Inner D S&T | **0.897** |
| 2F1 | **2.717** | **1.972** | **2.619** | Inner D S&T | **1.972** |
| 3F1 | **1.821** | **1.353** | **1.521** | Inner D S&T | **1.353** |
| 5C1 | **1.913** | **1.322** | **1.565** | Inner D S&T | **1.322** |
| Type 3 | **1.945** | **1.419** | **1.631** | Inner D S&T | **1.419** |
| Type 3-3 | **2.089** | **1.657** | **2.059** | Inner D S&T | **1.657** |
| Type 3S2 | **2.366** | **1.606** | **1.776** | Inner D S&T | **1.606** |
| SU4 | **1.639** | **1.224** | **1.437** | Inner D S&T | **1.224** |
| SU5 | **1.538** | **1.168** | **1.430** | Inner D S&T | **1.168** |
| SU6 | **1.390** | **1.089** | **1.287** | Inner D S&T | **1.089** |
| SU7 | **1.287** | **1.007** | **1.203** | Inner D S&T | **1.007** |
| EV2 | **1.582** | **1.140** | **1.352** | Inner D S&T | **1.140** |
| EV3 | **1.458** | **1.006** | **1.054** | Inner D S&T | **1.006** |
| RPL 60T | **1.363** | **0.986** | **1.071** | Inner D S&T | **0.986** |
| RPL 65T | **1.253** | **0.878** | **0.972** | Inner D S&T | **0.878** |

**Figure L: Summary of Floorbeam Rating Factors**

While the floorbeams do not meet HL-93 design loadings, they do rate for all Ohio legal loads as well as all AASHTO vehicles. Both routine permit loads don't pass, which rate to 88% of demand.

## LINK SLAB EVALUATION

As part of the arch analysis and load rating, Michael Baker evaluated the potential impacts of incorporating link slabs which might be included in rehabilitation strategies to eliminate deck joints. Use of link slabs are increasing across multiple states to cost effectively connect previously discontinuous bridge deck slabs and eliminate expansion joints on bridges. This elimination of expansion joints can prevent premature corrosion to superstructure elements underneath the deck. Generally, link slabs are designed to support wheel loads and the bending moment due to girder end rotations without transmitting live load effects from one span to another. This discontinuity between girders is often accompanied by debonding the link slab from the ends of the girders.

As most of the bridge deterioration noted during the bridge inspection was located directly below the expansion joints, Michael Baker was tasked with investigating the potential consequences of installing link slabs to protect the prestressed beams and precast floorbeams from future deterioration.

Link slabs do not create continuous girders but do increase the length of bridge superstructure that will expand and contract due to temperature change or other loading. Ensuring that the bridge unit, formerly consisting of simple spans and now consisting of a single, joined unit, will behave without bridge damage, is paramount. During the iterative load rating process for the spandrel columns and arch ribs, it was noted that the spandrel columns were sensitive to moments induced from horizontal loads applied at the deck level. To examine the effect that the link slabs would have on these spandrel columns, link slabs were simulated at the three expansion joints between the four arch spans by removing the plate end releases assigned at each deck interface between the arch spans. For the comparison, the final iteration of the model used for load rating the arch ribs and spandrel columns was ‘saved as’ and the link slabs simulated. The model was then run, and the spandrel column rating factors were compared between the two models. The spandrel columns’ ratings plummeted, as can be seen in the below Figure M. From this investigation it is likely that the implementation of link slabs would cause significant loading changes to the spandrel columns which could result in new cracking and deformations. Some of the loading changes could possibly be mitigated through use of Teflon sliding bearings, but the bridge’s steep longitudinal slope presents additional challenges. Therefore, the feasibility of using link slabs will be dependent on the rehabilitation approach selected and will require additional analysis to determine how they can be incorporated without negative effects to the arch and spandrel ratings.

| AXIAL MOMENT RATINGS OF SPANDREL COLUMNS | | | |
|---|---|---|---|
| | *Without Link Slab* | | *With Link Slab* |
| **Truck** | **Controlling RF** | | **Controlling RF** |
| HL-93 INV | **0.701** | → | **0.240** |
| HL-93 OP | **0.909** | → | **0.324** |
| 2F1 | **2.649** | → | **0.764** |
| 3F1 | **1.403** | → | **0.520** |
| 5C1 | **1.442** | → | **0.520** |
| Type 3 | **1.461** | → | **0.520** |
| Type 3-3 | **1.765** | → | **0.635** |
| Type 3S2 | **1.579** | → | **0.572** |
| SU4 | **1.320** | → | **0.445** |
| SU5 | **1.316** | → | **0.416** |
| SU6 | **1.212** | → | **0.388** |
| SU7 | **1.116** | → | **0.388** |
| EV2 | **1.135** | → | **0.502** |
| EV3 | **1.081** | → | **0.367** |
| RPL 60T | **1.129** | → | **0.339** |
| RPL 65T | **1.060** | → | **0.319** |

**Figure M: Summary of Lower Rating Factors with Introduction of Link Slabs**

## SUMMARY

Initial load ratings identified an individual box beam with substandard load carrying capacity due to broken and corroded prestressing strands. ODOT has subsequently restricted traffic access from these portions of the bridge with temporary traffic control devices. Thus, in evaluation of the remainder of elements subject to current traffic, the Blaine Hill Viaduct rates satisfactorily for all Ohio legal loads, specialized hauling vehicles, and emergency vehicles. The bridge does not satisfy the HL-93 Inventory or Operating ratings; however, this is a modern notional load (design case) that didn't exist when the original structure was designed and doesn't affect consideration of load-carrying capacity or posting. Since traffic control has been implemented to restrict vehicles from driving atop the deteriorated box beam, the Blaine Hill Viaduct has sufficient capacity for all Ohio legal loads, specialized hauling vehicles, and emergency vehicles, and does not require any load posting at this time. The controlling ratings for each vehicle are summarized in Figure N.

| Truck | Governing Bridge Rating Factors | Controlling Location |
|---|---|---|
| HL-93 INV | **0.645** | Floorbeam |
| HL-93 OP | **0.897** | Floorbeam |
| 2F1 | **1.972** | Floorbeam |
| 3F1 | **1.353** | Floorbeam |
| 5C1 | **1.322** | Floorbeam |
| Type 3 | **1.419** | Floorbeam |
| Type 3-3 | **1.657** | Floorbeam |
| Type 3S2 | **1.579** | Spandrel Column |
| SU4 | **1.224** | Floorbeam |
| SU5 | **1.168** | Floorbeam |
| SU6 | **1.089** | Floorbeam |
| SU7 | **1.007** | Floorbeam |
| EV2 | **1.135** | Spandrel Column |
| EV3 | **1.006** | Floorbeam |
| RPL 60T | **0.986** | Floorbeam |
| RPL 65T | **0.878** | Floorbeam |

**Figure N: Summary of Controlling Rating Factors for Each Vehicle**

# ODOT BR-100 BRIDGE LOAD RATING SUMMARY REPORT

# BRIDGE LOAD RATING SUMMARY REPORT
## OFFICE OF STRUCTURAL ENGINEERING
## OHIO DEPARTMENT OF TRANSPORTATION

| SFN | Bridge Number | DISTRICT | GPS COORDINATES LATITUDE: | GPS COORDINATES LONGITUDE: |
|---|---|---|---|---|
| 0701599 | BEL-00040-23265 | 11 | 40.066619 | -80.821422 |

| ORIGINAL YEAR BUILT | YEAR REBUILT | TOTAL BRIDGE LENGTH | FEATURE INTERSECTED (Below) |
|---|---|---|---|
| 1932 | 1982 | 754 ft | WHLNG CR,CR.10 & ABND.RR |

**SPECIAL ASSUMPTIONS & COMMENTS**

Modeled in October 2023 from original plans (1932) and rehabilitation plans (1981, 2010). AASHTOWare BrR and Midas Civil were used for load rating of slab span, box beams, slab beams, floorbeams, spandrel columns, and arch ribs. The deck is 5" thick and 43'-6" O/O. The roadway is 38'-0" F/F of curb and has a 3'-0" sidewalk on the left and a deflector parapet on the right. The bridge is on a tangent alignment and has a skew that varies from 0 to 34.65 degress RF. The wearing surface is 1.25" of microsilica concrete overlay per 2010 rehabilitation plans. The controlling location for legal loads is the floorbeam above Pier 5 in arch C above the left spandrel column. The rating is controlled by the strut to node interface limit state. ODOT closed the center lane of the bridge on 8/25/23 based on a memo from Michael Baker dated 8/24/23. Ratings for members within this lane closure are not included.

| | Please type or select on right using drop down arrow |
|---|---|
| LOAD RATING PURPOSE: | 7 - Not Applicable |
| GENERAL APPRAISAL (0-9): | 5 |
| LOAD RATING SOFTWARE: | 7 - Combination |
| SOFTWARE VERSION: | AASHTOWare BrR 7.4.1.3001 and Midas Civil 2022 v1.2 |
| ROUTINE PERMIT LOAD (RPL): | N - Agency doesnot issue routine permits |
| RATING SOURCE: | 1 - Plan information available for load rating analysis |
| LOAD RATING METHOD: | LRFR - Load & Resistance Factor Rating (RF) - Code 8 |
| DESIGN LOADING: | 5 - HS20 |

STATE OF OHIO
EDWARD T. BAZNIK
78469
REGISTERED
PROFESSIONAL ENGINEER

## STRUCTURE RATING SUMMARY

**OHIO & AASHTO LEGAL VEHICLES**

| Legal Load | GVW (Tons) | No of Axles | Rating Factor RF | Safe Weight (Tons) |
|---|---|---|---|---|
| 2F1 | 15 | 2 | 1.972 | 15.00 |
| 3F1 | 23 | 3 | 1.353 | 23.00 |
| 5C1 | 40 | 5 | 1.322 | 40.00 |
| Type 3 | 25 | 3 | 1.419 | 25.00 |
| Type 3-3 | 40 | 6 | 1.657 | 40.00 |
| Type 3S2 | 36 | 5 | 1.579 | 36.00 |
| **SPECIALIZED HAULING VEHICLES (SHV)** | | | | |
| SU4/4F1 | 27 | 4 | 1.224 | 27.00 |
| SU5 | 31 | 5 | 1.168 | 31.00 |
| SU6 | 34.75 | 6 | 1.089 | 34.75 |
| SU7 | 38.75 | 7 | 1.007 | 38.75 |

**EMERGENCY VEHICLES (EV)**

*Check box if rating for EV3* ☑

| | | | | |
|---|---|---|---|---|
| EV2 | 28.75 | 2 | 1.135 | 28.75 |
| EV3 | 43 | 3 | 1.006 | 43.00 |

| Controlling Legal Load RF | 100% | 1.00 |
|---|---|---|

**Design Inventory and Operating Ratings**

| Loading Type | Rating by RF: Inventory | Rating by RF: Operating |
|---|---|---|
| HL93 Loading | 0.645 | 0.897 |
| Recommendation | No Load Posting is Recommended | |
| Sign Posting Recommendation: | | |

**Permit Load (PL) Analysis (optional)**

| Loading Type | GVW (Tons) | No of | Rating Factor | Safe Load (Tons) |
|---|---|---|---|---|
| PL 60T | 60 | 6 | 0.986 | 59.16 |
| PL 65T | 65 | 7 | 0.878 | 57.07 |

| PL Analysis Method | Load & Resistance Factor Rating (LRFR) |
|---|---|

| AGENCY/FIRM/OFFICE | Michael Baker International | | | | |
|---|---|---|---|---|---|
| | Name | PE Number | Phone Number | Email | Report Date: 2023-10-05 |
| Rated By | John Carey | 81773 | 216-776-6638 | John.Carey@mbakerintl.com | |
| Reviewed By | Edward Baznik | 78469 | 216-776-6637 | ebaznik@mbakerintl.com | |

# COLOR CODED SELECTION OF BOX BEAM AND SLAB BEAM SPANS FOR LOAD RATING

BEAM SPAN B MODELED
ARCH SPAN A MODELED
ARCH SPAN B MODELED
BEL-40-2338
FHWA REGION
STATE
PROJECT
5
OHIO
19/74
MICROFILMED
JAN 22 1985
Beam Span "B" = 45'-6"
23'-3"
℄ Diaphragm
℄ Beam Span
ARCH SPAN "A" = 145'-6"
16'-3"
14'-0"
℄ Bridge
Sta. 106+85.00
℄ Expansion Joint
℄ Tie Rods (typ.);
℄ Expansion Joint
℄ Precast transverse beams
Match line A
6" (typ)
3"
N
Match line A
ARCH SPAN "A" = 145'-6"
℄ Pier #3
ARCH SPAN "B" = 132'-6"
14'-3"
20'-3"
3'-0"
℄ Expansion Joint & Trans. Beam
PRESTRESSED CONCRETE BEAM PLAN
NOTES:
For Box Beam details see sheet 21/72
For Superstructure details see sheets 22/72 thru 38/72 and 46/72 thru 50/72
For Transverse Beam plan see sheet 14/72
Match line B; see sheet 19/72
NOTES: (con't)
Box beams shall have the same identification marks on the shop drawings as on the project plans.
STATE OF OHIO
DEPARTMENT OF TRANSPORTATION
BUREAU OF BRIDGES AND STRUCTURAL DESIGN
18/72
BOX BEAM LAYOUT
BRIDGE NO. BEL-40-2338
OVER THE B. & O. RAILROAD
AND WHEELING CREEK
DESIGNED J.A.M.
DRAWN J.A.M.
TRACED
CHECKED R.L.D.
REVIEWED WJJ
DATE 12-1-80
REVISED

MICROFILMED
JAN 22 1986
ARCH SPAN C MODELED
ARCH SPAN "B" = 132'-6"
℄ PIER #4
ARCH SPAN "C" = 118'-6"
14'-0"
14'-3"
20'-3"
14'-0"
14'-0"
14'-0"
14'-0"
14'-0"
14'-0"
14'-3"
FHWA REGION
5
STATE
OHIO
PROJECT
20
74
BEL-40-23.38
Match line B; see sheet 18/72
℄ Pier # 5
N
℄ Bridge
1'-6"
1'-6"
B199
B200
B201
B202
B203
B204
B205
B206
B207
B208
B209
B221
B222
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B302
B303
B304
B305
B306
B307
B308
Match line C
℄ Expansion Joint and Transverse beam
3'-0"
6" (typ.)
℄ Precast Trans. Beam (typ.)
℄ Expansion Joint & Trans Beam
3'-0"
℄ Pier #5
℄ Pier #6
ARCH SPAN "D" = 103'-6"
20'-3"
14'-0"
14'-0"
14'-0"
14'-0"
14'-0"
15'-3"
1'-0"
Match line C
℄ Bridge
Match line D; see sheet 20/72
B309
B310
B311
B312
B313
B314
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B375
B376
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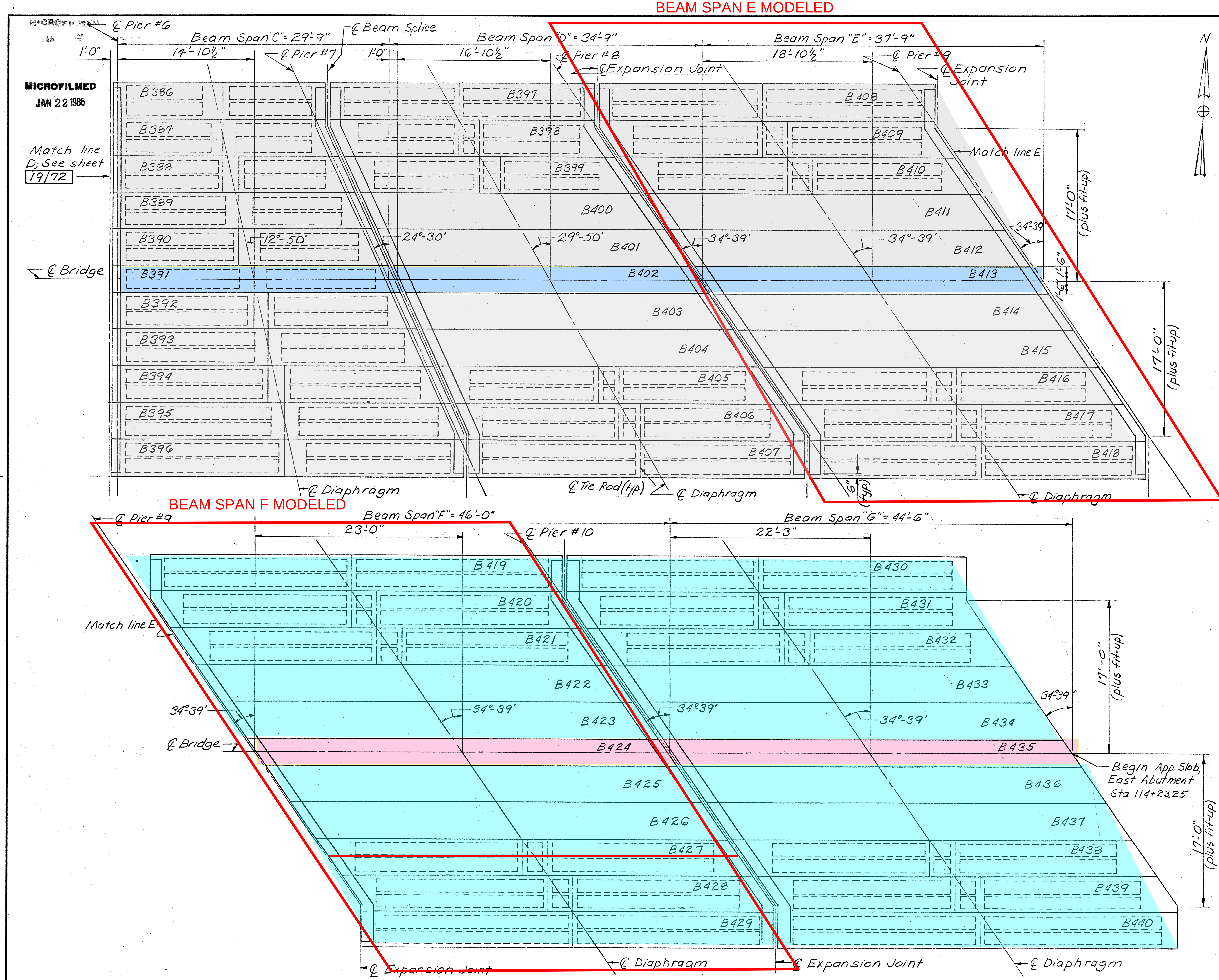
B378
B379
B380
B381
B382
B383
B384
B385
3'-0"
ARCH SPAN D MODELED
℄ Expansion Joint
PRESTRESSED CONCRETE BEAM PLAN
FOR notes see sheet 18/72
STATE OF OHIO
DEPARTMENT OF TRANSPORTATION
BUREAU OF BRIDGES AND STRUCTURAL DESIGN
19/72
BOX BEAM LAYOUT
BRIDGE NO. BEL-40-2338
OVER THE B.&O. RAILROAD
AND WHEELING CREEK
DESIGNED J.A.M.
DRAWN J.A.M.
TRACED
CHECKED R.L.D.
REVIEWED WJJ
DATE 12-1-80
REVISED




PRESTRESSED CONCRETE BEAM PLAN

| FHWA REGION | STATE | PROJECT | |
|---|---|---|---|
| 5 | OHIO | | |

BEL-40-23.38

For notes see sheet 18/72

STATE OF OHIO
DEPARTMENT OF TRANSPORTATION
BUREAU OF BRIDGES AND STRUCTURAL DESIGN

20/72

BOX BEAM LAYOUT

BRIDGE NO. BEL-40-23.38
OVER THE B. & O. RAILROAD
AND WHEELING CREEK

| DESIGNED | DRAWN | TRACED | CHECKED | REVIEWED | DATE |
|---|---|---|---|---|---|
| J.A.M. | J.A.M. | | R.L.D. | WJJ | 12-1 |

# SLAB BEAM B126 LOAD RATING MEMO

| | | | |
|---|---|---|---|
| PROJECT : *BEL-40-23.38 over Wheeling Creek (SFN 0701599)* | | Michael Baker INTERNATIONAL | |
| TASK : *As-Inspected Rating* | | PROJECT NO : *195987* | |
| SUBJECT : *Slab Beams B122-B132* | | | |
| CALCULATED BY : *ETB* | DATE : *8/24/2023* | CHECKED BY : *CDC* | DATE : *8/25/2027* |

**BEL-40-23.38 B122-B132 RATING**

## DESCRIPTION:

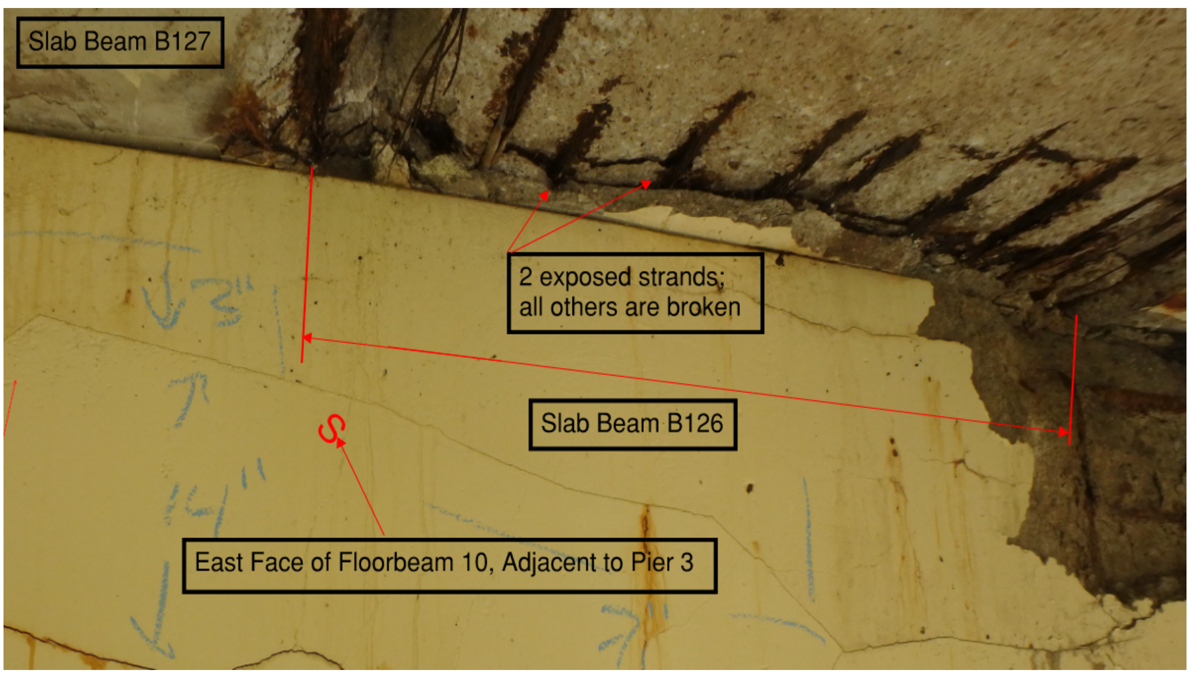
LRFR (and LFR where necessary) Load Rating of Bridge BEL-40-23.38 Slab Beams B122 - B132 and supporting documentation. For the purpose of identifying the slab beams, their numbering designation, as provided in the 1980 rehab plans, is used throughout the calculations.

## REFERENCES:

| Title | | |
|---|---|---|
| AASHTO LRFD Design Specifications, 9th Edition, 2020 | | |
| AASHTO Manual for Bridge Evaluation, 3rd Edition, 2018 | | |
| ODOT Bridge Design Manual 2020 Edition, July 2023 | | |
| "D11-22815-BEL-00040-23.38-2010-00.pdf" (2010 rehabilitation plans) | | |
| "BEL 40 2338 1981 Box Beam Install.pdf" (1980 rehabilitation plans) | | |
| PSBD-1-71 | | |

## CRITICAL FIND

The composite slab beam B126, which was originally cast with 10 prestressing strands, currently has 8 broken strands and the remaining two strands are exposed per the July 2023 inspection.



## ASSUMPTIONS

-Rating is initially performed using Load and Resistance Factor Rating (LRFR) to conform to current ODOT Rating Practices. If any legal vehicle's rating factor is found to be less than 1.0, the rating is then re-calculated using Load Factor Rating (LFR) to determine if the member can avoid posting.

-Based on discussion with Baker load rating staff, and ODOT precedent, the exposed (but unbroken) strands were analyzed as debonded strands and the broken strands were deducted from the beam. To calculate a debonding length, 36" was added to the exposed strand length to estimate the loss of prestressing force in those exposed strands.

-The slab beams are modeled as composite with the reinforced concrete deck as per the 1980 rehabilitation plans. These plans show a deck thickness varying between 5" and 5.75". Additionally, the 2010 rehab plans show a removal of 1/2" of deck concrete prior to placement of the overlay. The deck thickness used for composite action was calculated as: 5.375" - 1/2" = 4.875"

-It is a typical ODOT practice to code concrete type overlays as DC2 instead of DW. The 2010 rehab plans show a microsilica overlay thickness of 1.25". Therefore this overlay is applied as a DC2 load.

-For any slab beam information not shown in the plans, ODOT Standard Drawing PSBD-1-71 was consulted and used per the reference to this standard drawing in the "Box Beam Details" sheet in the 1980 rehab plans.

-It is typical practice to include a 5% "Additional Self Load" factor in the BrR model in the "Member Alternative Description" to account for unknowns. However, this conservatism has been removed to obtain a more accurate load rating.

-All dead loads are calculated by BrR except for the sidewalk weight, which is shown below.

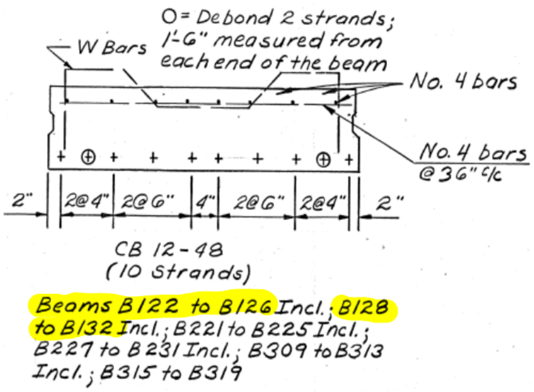
-Typically, the DC2 dead loads are distributed evenly to all beams. Since the beams with the worst strand deterioration are in the center of the cross section, furthest from the sidewalks and parapets, the Stage 2 DL Distribution function in BrR has been changed from "Evenly to all girders" to "By tributary area" to remove that DC2 load from the center beams with the most deterioration.

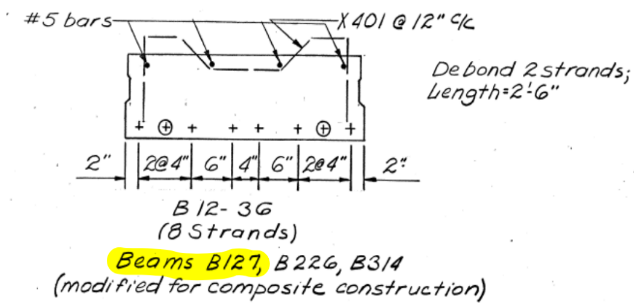
## DEAD LOAD CALCULATIONS

**Sidewalk Parapet Weight**

| | | |
|---|---|---|
| Concrete density: | **150.00** | pcf |
| Total Height: | **42.00** | in |
| Total Width: | **12.00** | in |
| Area of concrete: | 504.00 | in |
| Weight of parapet: | 0.525 | kip/ft |

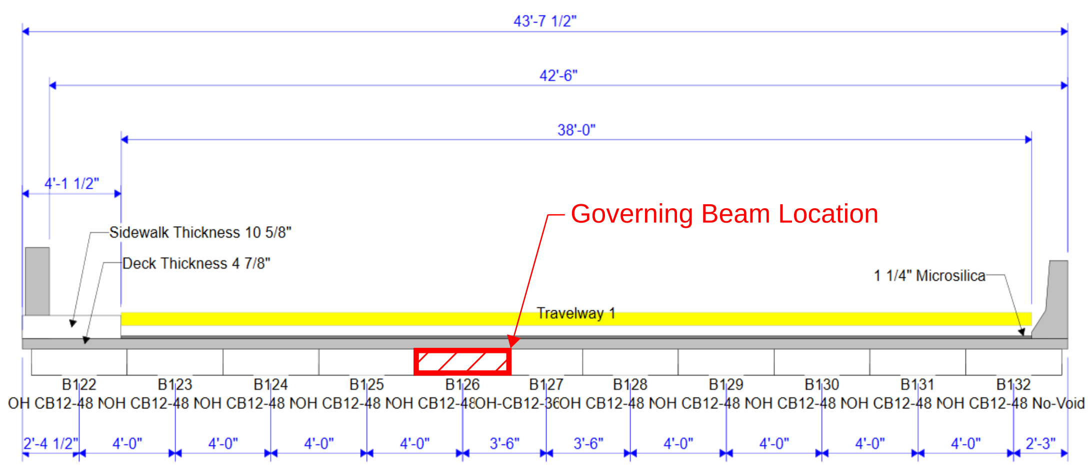
## ORIGINAL SLAB BEAM PRESTRESSING PATTERNS




## STRAND DETERIORATION BY SLAB BEAM

| Beam | Slab Beams B122-B132 Strand Deterioration (Observed at West End of Slab Beam) |
|---|---|
| B122 | - |
| B123 | 1 exterior strand broken (removed) |
| B124 | 1 exterior strand exposed (debonded for 3 ft + 36'') |
| B125 | - |
| B126 | 2 interior strands exposed (debonded for 3 ft + 36'') & remaining 8 strands broken (removed) |
| B127 | 2 exterior strands broken (removed) and one interior strand broken (removed) |
| B128 | - |
| B129 | - |
| B130 | - |
| B131 | - |
| B132 | - |



| PROJECT : BEL-40-23.38 over Wheeling Creek (SFN 0701599) | | Michael Baker INTERNATIONAL |
|---|---|---|
| TASK : As-Inspected Rating | PROJECT NO : 195987 | |
| SUBJECT : Slab Beams B122-B132 | | |
| CALCULATED BY : ETB DATE : 8/24/2023 | CHECKED BY : CDC | DATE : 8/25/2027 |

## BEL-40-23.38 B122-B132 RATING

**AS-INSPECTED RATING FACTORS BY BEAM**

-LRFR rating factors are provided for each beam (grouped where appropriate). If any of a beam's legal load rating factors are below 1.0, an LRF rating is also provided.

| B122 | LRFR RF | LFR RF |
|---|---|---|
| HL-93 INV | 18.999 | - |
| HL-93 OPR | 24.628 | - |
| 2F1 | 55.269 | - |
| 3F1 | 41.737 | - |
| 5C1 | 41.737 | - |
| RPL 60T | 36.729 | - |
| RPL 65T | 34.031 | - |
| SU4 | 35.869 | - |
| SU5 | 34.575 | - |
| SU6 | 33.370 | - |
| SU7 | 33.370 | - |
| Type 3 | 41.737 | - |
| Type 3-3 | 50.681 | - |
| Type 3S2 | 45.776 | - |
| EV2 | 33.549 | - |
| EV3 | 32.460 | - |

| B123 | LRFR RF | LFR RF |
|---|---|---|
| HL-93 INV | 1.230 | - |
| HL-93 OPR | 1.595 | - |
| 2F1 | 3.580 | - |
| 3F1 | 2.703 | - |
| 5C1 | 2.703 | - |
| RPL 60T | 2.394 | - |
| RPL 65T | 2.218 | - |
| SU4 | 2.338 | - |
| SU5 | 2.254 | - |
| SU6 | 2.175 | - |
| SU7 | 2.175 | - |
| Type 3 | 2.703 | - |
| Type 3-3 | 3.282 | - |
| Type 3S2 | 2.965 | - |
| EV2 | 2.187 | - |
| EV3 | 2.102 | - |

| B124 | LRFR RF | LFR RF |
|---|---|---|
| HL-93 INV | 1.300 | - |
| HL-93 OPR | 1.686 | - |
| 2F1 | 3.783 | - |
| 3F1 | 2.856 | - |
| 5C1 | 2.856 | - |
| RPL 60T | 2.560 | - |
| RPL 65T | 2.372 | - |
| SU4 | 2.501 | - |
| SU5 | 2.410 | - |
| SU6 | 2.326 | - |
| SU7 | 2.326 | - |
| Type 3 | 2.856 | - |
| Type 3-3 | 3.469 | - |
| Type 3S2 | 3.133 | - |
| EV2 | 2.339 | - |
| EV3 | 2.222 | - |

| B125, B129-31 | LRFR RF | LFR RF |
|---|---|---|
| HL-93 INV | 1.357 | - |
| HL-93 OPR | 1.758 | - |
| 2F1 | 3.946 | - |
| 3F1 | 2.980 | - |
| 5C1 | 2.980 | - |
| RPL 60T | 2.637 | - |
| RPL 65T | 2.443 | - |
| SU4 | 2.575 | - |
| SU5 | 2.482 | - |
| SU6 | 2.395 | - |
| SU7 | 2.395 | - |
| Type 3 | 2.980 | - |
| Type 3-3 | 3.619 | - |
| Type 3S2 | 3.269 | - |
| EV2 | 2.408 | - |
| EV3 | 2.318 | - |

| B126 | LRFR RF | LFR RF |
|---|---|---|
| HL-93 INV | 0.000 | 0.000 |
| HL-93 OPR | 0.000 | 0.096 |
| 2F1 | 0.000 | 0.102 |
| 3F1 | 0.000 | 0.079 |
| 5C1 | 0.000 | 0.079 |
| RPL 60T | 0.000 | 0.056 |
| RPL 65T | 0.000 | 0.054 |
| SU4 | 0.000 | 0.071 |
| SU5 | 0.000 | 0.069 |
| SU6 | 0.000 | 0.066 |
| SU7 | 0.000 | 0.066 |
| Type 3 | 0.000 | 0.079 |
| Type 3-3 | 0.000 | 0.096 |
| Type 3S2 | 0.000 | 0.087 |
| EV2 | 0.000 | 0.068 |
| EV3 | 0.000 | 0.051 |

| B127 | LRFR RF | LFR RF |
|---|---|---|
| HL-93 INV | 0.720 | - |
| HL-93 OPR | 0.934 | - |
| 2F1 | 2.096 | - |
| 3F1 | 1.583 | - |
| 5C1 | 1.583 | - |
| RPL 60T | 1.398 | - |
| RPL 65T | 1.295 | - |
| SU4 | 1.365 | - |
| SU5 | 1.316 | - |
| SU6 | 1.270 | - |
| SU7 | 1.270 | - |
| Type 3 | 1.583 | - |
| Type 3-3 | 1.922 | - |
| Type 3S2 | 1.736 | - |
| EV2 | 1.277 | - |
| EV3 | 1.231 | - |

| B128 | LRFR RF | LFR RF |
|---|---|---|
| HL-93 INV | 1.341 | - |
| HL-93 OPR | 1.738 | - |
| 2F1 | 3.900 | - |
| 3F1 | 2.945 | - |
| 5C1 | 2.945 | - |
| RPL 60T | 2.602 | - |
| RPL 65T | 2.411 | - |
| SU4 | 2.541 | - |
| SU5 | 2.449 | - |
| SU6 | 2.364 | - |
| SU7 | 2.364 | - |
| Type 3 | 2.945 | - |
| Type 3-3 | 3.577 | - |
| Type 3S2 | 3.231 | - |
| EV2 | 2.376 | - |
| EV3 | 2.291 | - |

| B132 | LRFR RF | LFR RF |
|---|---|---|
| HL-93 INV | 1.876 | - |
| HL-93 OPR | 2.432 | - |
| 2F1 | 5.459 | - |
| 3F1 | 4.122 | - |
| 5C1 | 4.122 | - |
| RPL 60T | 3.639 | - |
| RPL 65T | 3.372 | - |
| SU4 | 3.554 | - |
| SU5 | 3.426 | - |
| SU6 | 3.306 | - |
| SU7 | 3.306 | - |
| Type 3 | 4.122 | - |
| Type 3-3 | 5.006 | - |
| Type 3S2 | 4.521 | - |
| EV2 | 3.324 | - |
| EV3 | 2.996 | - |

**CONCLUSIONS AND RECOMMENDATIONS**

-Beam B126's legal ratings for both LRFR and LFR are well below 1.0. It is recommended that a lane closure be implemented to keep traffic off B126.
-The controlling ratings for Beam B126 are the result of implenting best practices and recommendations from the ODOT prestressed box beam tutorials with respect to damaged strands. It is recognized that there is some conservatism built into these methods which, if removed, would increase the rating factors. Additionally, there are contributions to the capacity of this system that are not easily quantified which would also increase the rating factors. The reinforced concrete deck and tie rods provide a level of redundancy as well. Therefore, this load rating is the product of appropriate procedures but it is acknowledged that the actual capacity of the slab beam is greater than what the rating factors indicate.

# PIER 5 FLOORBEAM CANTILEVER

# LOAD RATING MEMO

| PROJECT : *BEL-40-23.37* | | Michael Baker INTERNATIONAL |
|---|---|---|
| TASK : *Rating* | PROJECT NO : *195987* | |
| SUBJECT : *Pier 5 Cap Rating* | | |
| CALCULATED BY : *JCC* DATE : *10/3/2023* | CHECKED BY : *ETB* | DATE : -- |

**GENERAL**

## DESCRIPTION:

Documentation of pier cap check with loss of concrete and exposed reinforcement.

## SUMMARY

| Vehicle | Type | RF |
|---|---|---|
| HL-93 INV | Design | 0.926 |
| HL-93 OPR | Design | 1.200 |
| 2F1 | Legal | 2.717 |
| 3F1 | Legal | 1.821 |
| 4F1 | Legal | 1.620 |
| 5C1 | Legal | 1.913 |
| Type3 | Legal | 1.945 |
| Type3S2 | Legal | 2.366 |
| Type3-3 | Legal | 2.089 |
| SU4 | Legal | 1.639 |
| SU5 | Legal | 1.538 |
| SU6 | Legal | 1.390 |
| SU7 | Legal | 1.287 |
| EV2 | Legal | 1.582 |
| EV3 | Permit | 1.458 |
| RPL 60T | Permit | 1.363 |
| RPL 65T | Permit | 1.253 |

Traffic restrictions at the pier cap is not currently required.

The controlling rating is from the vertical ties at the exposed stirrups. Ignoring more stirrup reinforcement may result in a load posting rating.

## REFERENCES:


(1) AASHTO LRFD 9th Ed. 2020
(2) AASHTO MBE 3rd Ed. 2018
(3) ODOT BDM 2020 Ed. July 2023 Release
(4) FHWA-NHI-17-071 Strut-and-Tie Modeling(STM) for Concrete Structures
(5) ACI 318-14 Building Code Requirements for Structural Concrete

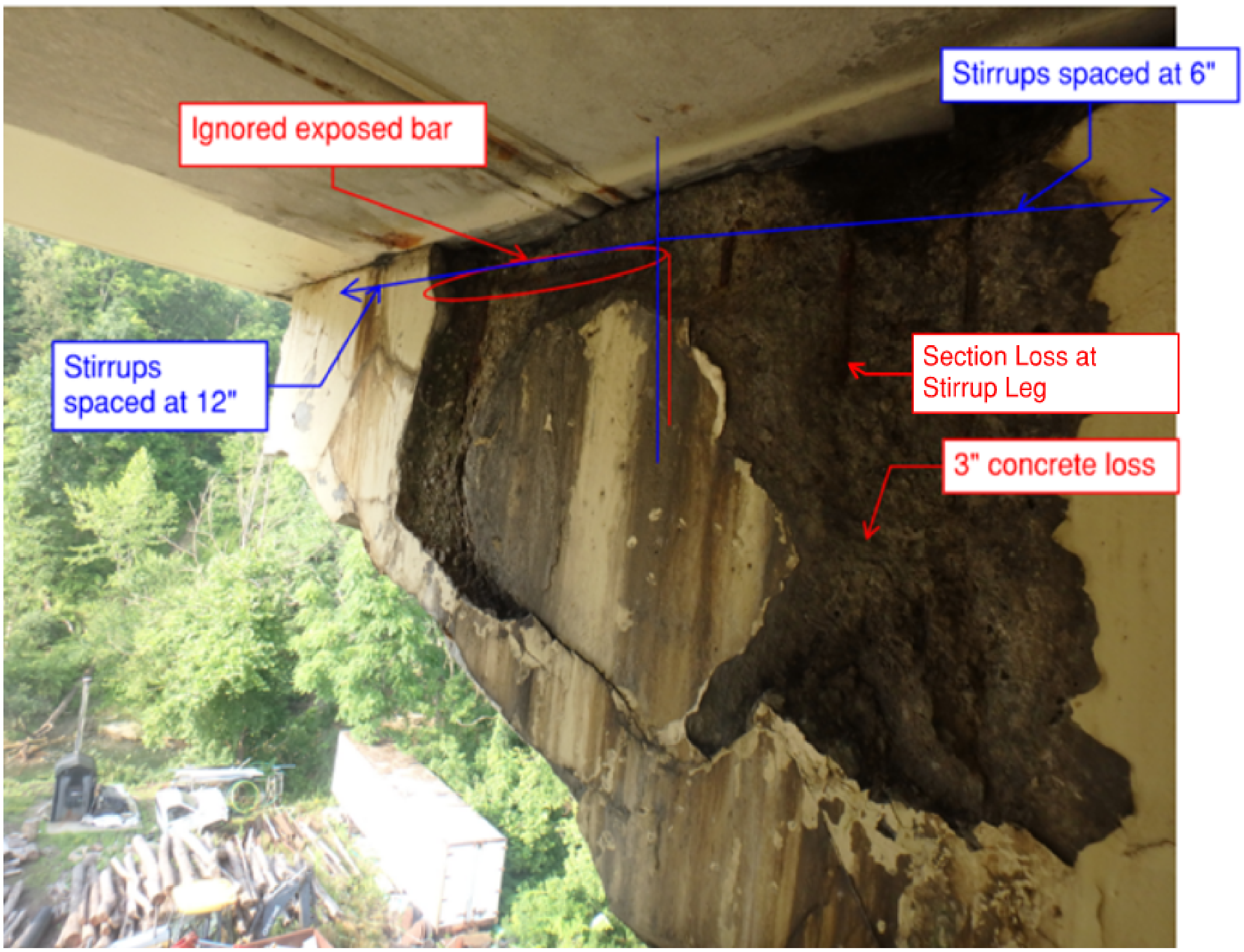

**SUPPORTING FILES**

| | |
|---|---|
| Overall Model | Belmont 40 Open Spandrel Arch Bridge Model 082923.mcb |
| Strut and Tie Model | Cantilever STM.mcb |
| STM Supporting CAD | Scaling Sketches.dgn |

## GENERAL PROCEDURE

The pier cap at pier 5, at the south end was found to have significant amounts of concrete spalled and exposed reinforcement during the recent inspection by Michael Baker International.

The below photo includes the noted section loss.




Discussion with ODOT District 11 staff indicate that this has progressed since their last inspection.

GENERAL

**CURRENT RATINGS**

This calculation will calculate rating factors given the current state of the structure by checking:
1. Check of the cantilever using beam capacity calculations.
2. Strut-and-Tie check of this region.
3. Combined shear and torsion check of this region.

As noted in AASHTO, beam theory is not accurate in regions close to supports, but was likely how this pier cap was design.
It is included for comparison with the likely design capacity and as a check of the strut-and-tie method.

These ratings are done using output from a FEM model for both Design and Legal vehicles.
While the ODOT BDM was used, additional conversations with both District 11 and Central Office staff has been used to fully define elements of this complex structure not explicitly defined in the BDM.

Rating Factors < 1.00 for legal vehicles would indicate restrictions on traffic including lane closures and load posting may be warranted.

Where the bridge is shown to be adequate for design loading, no rating factors were calculated.

**SENSITIVITY**

The most sensitive part of the cap are the stirrups that are currently exposed with some section loss.

As noted in the summary, the governing ratings come from under the interior beam bearing. Currently loss of 0.5 stirrup legs are accounted for, but additional section loss would very likely result in a posting rating.

**ACCOUNTING FOR SECTION LOSS**

The following were done to account for section loss in these calculations.
1. Assume crack control reinforcement is not provided for per AASHTO 5.8.2.6-1 which reduces concrete efficiency in the Strut-and-Tie modeling.
2. The exposed top bar was ignored in all checks.
3. 3" of width was ignored for the full height of the cap beam. bw = **27.00** in
4. One half of a stirrup was ignored for strut and tie modeling. A full stirrup was conservatively discounted for the B-beam checks
5. Development length of reinforcement was increased due to lack of confinement from exposed stirrups and concrete section loss.
6. A condition factor of 0.85 was used based on MBE 6A.4.2.3-1.
7. Confinement modification factor as defined in AASHTO 5.6.5 is not increased above 1.00 under the bearings.

**MONITORING**

While this calculation calculates rating factors for this pier cap based on recent inspection findings, the structure could continue to rapidly deteriorate.

Michael Baker International recommends a visual inspection of this beam cap at an interval of 3 months.

While the list below is not exhaustive, any of the following would be reason to update this calculation:
1. Additional spalling of the pier cap
2. Additional exposed reinforcement or section loss of the exposed reinforcement.
3. Widening of the flexure crack noted below.

## STRUT-AND-TIE MODELING

### CRACK CONTROL REINFORCEMENT

-Because of the heavy spalling and exposed reinforcement, the cap concrete is not considered to have crack control reinforceme per AASHTO 5.8.2.6.

Concrete efficiency factor v = **0.45** AASHTO 5.8.2.5.3a-1

$v$ = concrete efficiency factor:

- 0.45 for structures that do not contain crack control reinforcement as specified in Article 5.8.2.6
- as shown in Table 5.8.2.5.3a-1 for structures with crack control reinforcement as specified in Article 5.8.2.6

### CENTROID OF REINFORCEMENT

-This is used to determine both the truss node locations and back face height of CCT nodes.

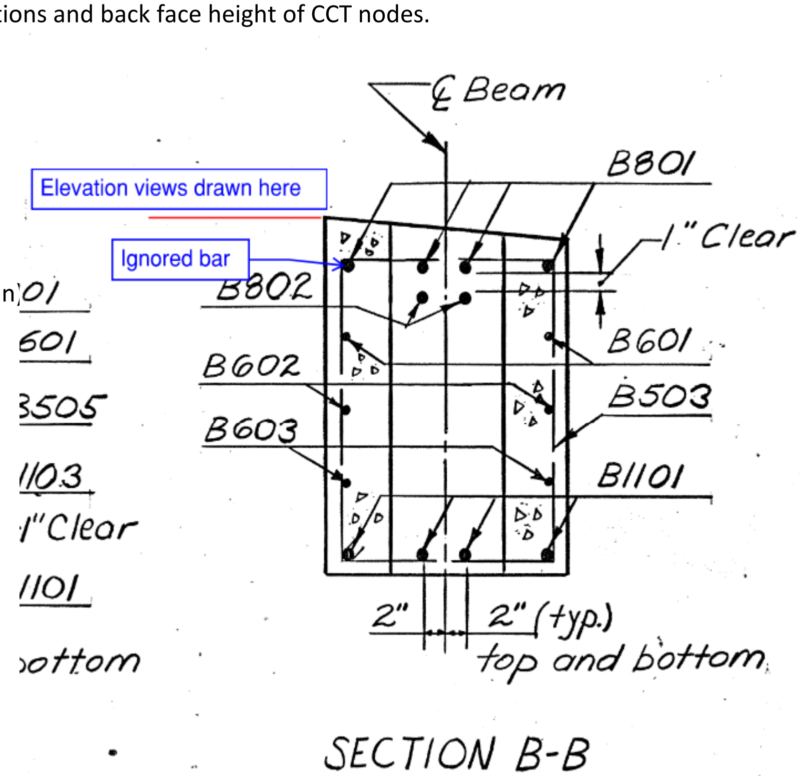
**Top Reinforcement**

| | | |
|---|---|---|
| Top cover | 2 | in |
| Stirrup diameter | 0.625 | in |
| Cross slope adjustment at center | 1.05 | in |

| | Bars | Center to Face (in) |
|---|---|---|
| Top Row | 3 | 4.175 in |
| Bottom Row | 2 | 6.175 in |
| Centroid | | 4.975 in |

**Bottom Reinforcement**

| | | |
|---|---|---|
| Bottom Row | 3.3125 | in |



**GENERAL**

Because the soundness of concrete around the bearings is deteriorated, the bearing confinement modification factor is assumed to be unity under bearings.

m = **1.00**

Bearing Area

A1 = 45 in^2

-1 Bearing pad compared against half of the applied load from MIDAS.

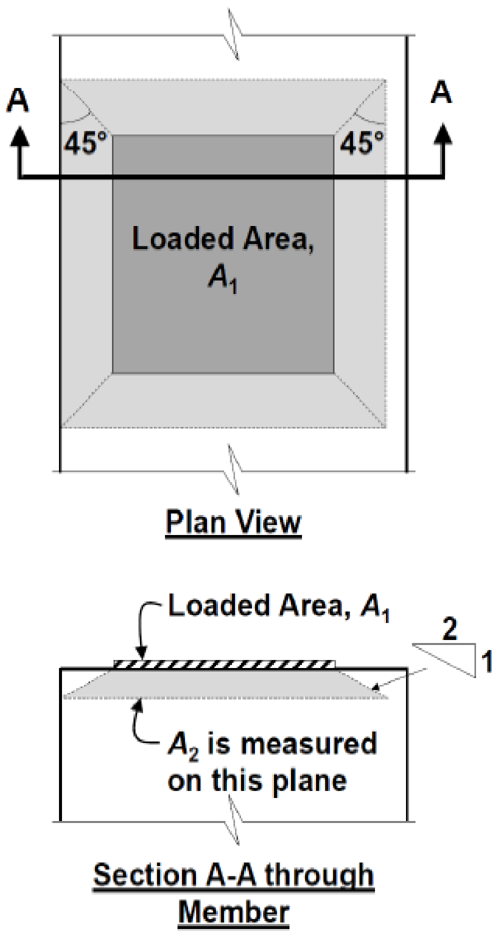



**Figure 5.6.5-1—Determination of Notional Area**

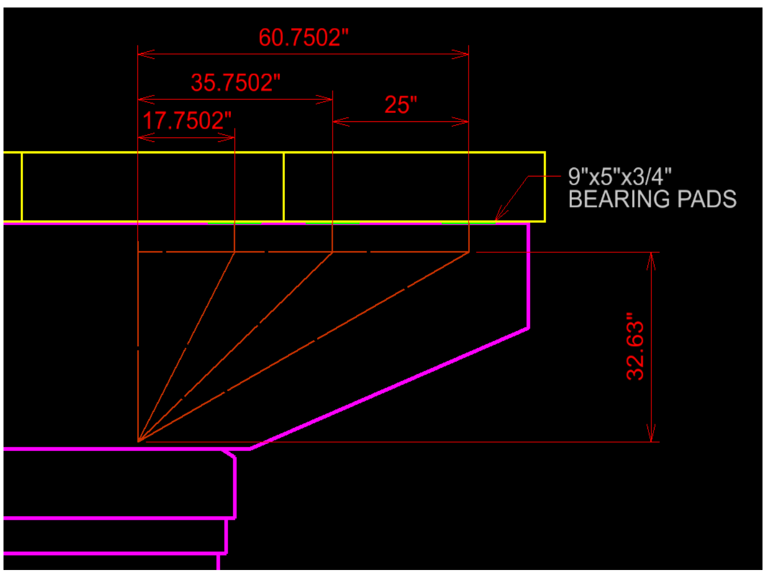
Nodes 7 & 8 are smeared nodes and do not need to be checked as described in FHWA and AASHTO C5.8.2.2.

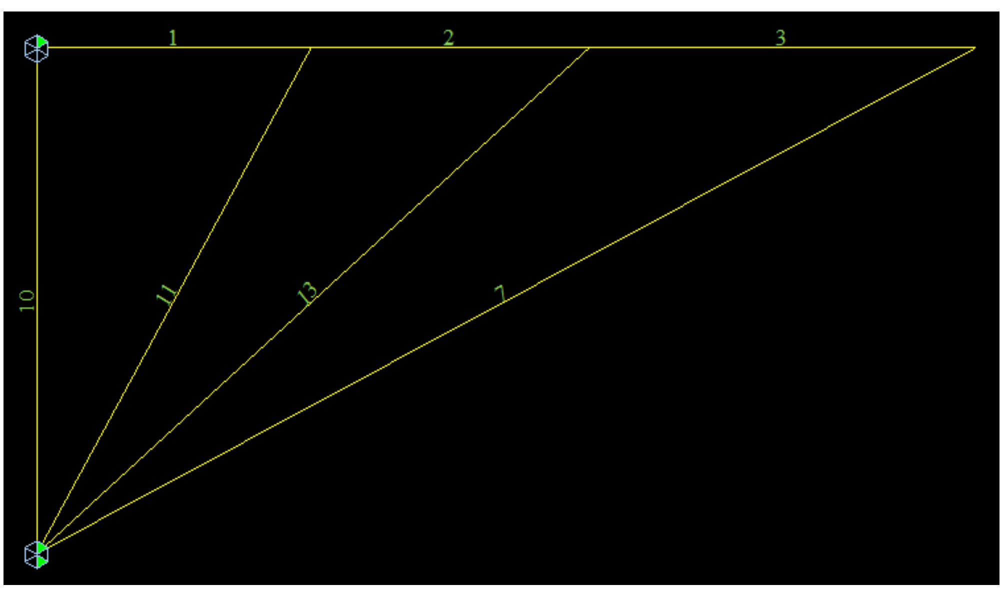
## MODELING

A model by others was previously completed of the full bridge. Reactions at the beam ends loading the cantilevered portion of this beam cap were used to load a strut and tie model.

The strut-and-tie model was created using the centroid of reinforcement calculated above and additional nodes at the bearings under each bearing.

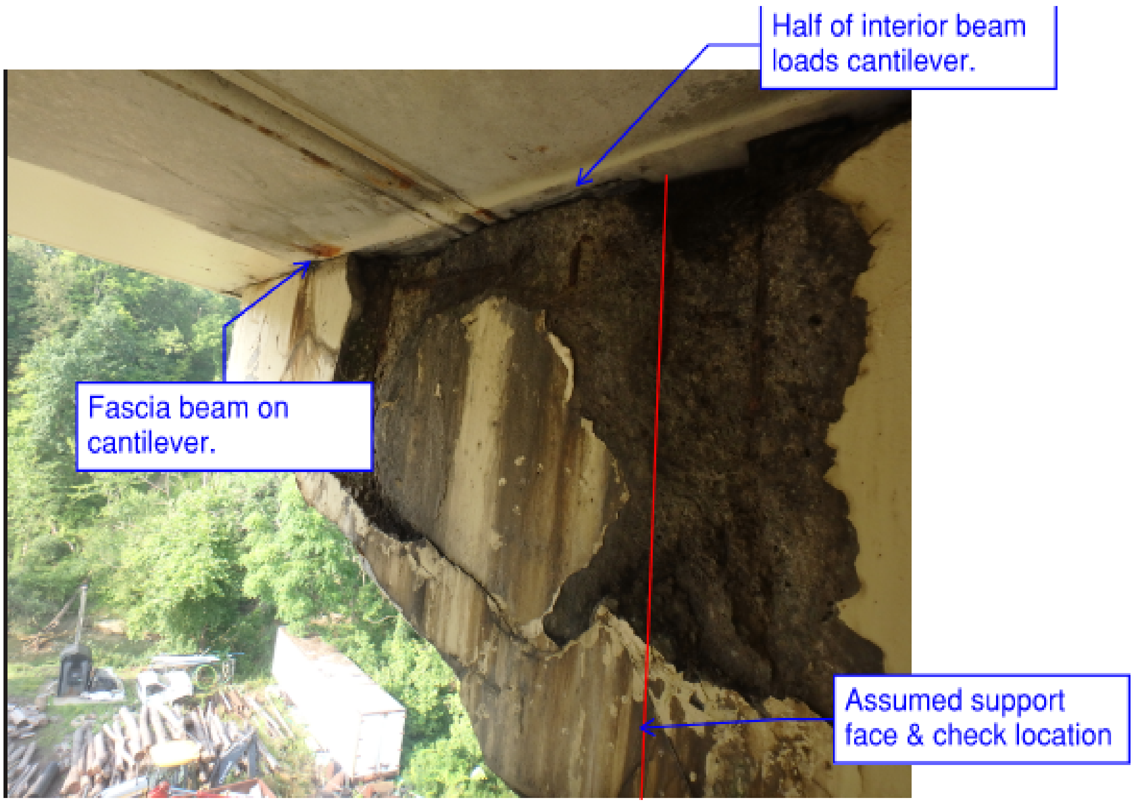
| | | | | |
|---|---|---|---|---|
| Total Beam Depth | $d_{beam}$ = | 38.75 | in | |
| Centroid of Top Bar Area | $y_{top}$ = | 4.975 | in | |
| 1/2 Depth of Compression Block | a/2 = | 1.147 | in | (calculated in Beam-Full Depth Tab) |
| Depth of Truss | $d_{truss}$ = | 32.63 | in | |



## BEAM CHECKS

-Beam checks were performed at the face of support and at the first interior beam bearing.
-Beam analysis shows that the critical points in the cap have adequate shear, moment, and torsional strength for STR I loading.
-Because this is a higher loading than Legal Posting loads, no rating factors were generated.



## DESCRIPTION:

Calculate development lengths for reinforcement in the pier cap.

## MATERIALS

| | | | | |
|---|---|---|---|---|
| Concrete strength | f'c = | **4.50** | ksi | |
| Reinforcement yield stress | fy = | **60.00** | ksi | |

## TOP HOOKED BARS - B801

| | | | | |
|---|---|---|---|---|
| Bar size | | **# 8** | | |
| Bar diameter | db = | 1.00 | in | |
| Bar area | Ab = | 0.79 | in^2 | |
| 90 degree standard hook length | lhook = 12*db = | 12 | in | AASHTO 5.10.2.1 |
| Bar hook is 13", so hook is adequate. | | | | |
| Hook development length | lhb = | 17.91 | in | AASHTO 5.10.8.2.4 |
| Normal weight concrete | Lam = | **1.00** | | |
| Cap bars are not epoxy coated | Lam.cw = | **1.00** | | |
| No confinement due to loss of cover | Lam.rc = | **1.00** | | |
| Assume need full reinforcement | Lam.er = | **1.00** | | |
| Development length | ldh = | 18.00 | in | |

**DEVELOPMENT**

*5.10.8.2.4a—Basic Hook Development Length*

The modified development length, $\ell_{dh}$, in in., for deformed bars in tension terminating in a standard hook specified in Article 5.10.2.1 shall be determined as the basic development length of standard hook in tension, $\ell_{hb}$, adjusted by the applicable modification factors specified in Article 5.10.8.2.4b, but shall not be taken less than the greater of the following:

- 8.0 bar diameters; and
- 6.0 in.

The modified development length, $\ell_{dh}$, of a standard hook in tension shall be taken as:

$$\ell_{dh} = \ell_{hb} \times \left( \frac{\lambda_{rc}\lambda_{cw}\lambda_{er}}{\lambda} \right) \tag{5.10.8.2.4a-1}$$

in which:

$$\ell_{hb} = \frac{38.0 d_b}{60.0} \left( \frac{f_y}{\sqrt{f'_c}} \right) \tag{5.10.8.2.4a-2}$$

where:

$\ell_{hb}$ = basic development length (in.)
$\lambda_{rc}$ = reinforcement confinement factor
$\lambda_{cw}$ = coating factor
$\lambda_{er}$ = excess reinforcement factor
$\lambda$ = concrete density modification factor as specified in Article 5.4.2.8

*C5.10.8.2.4a*

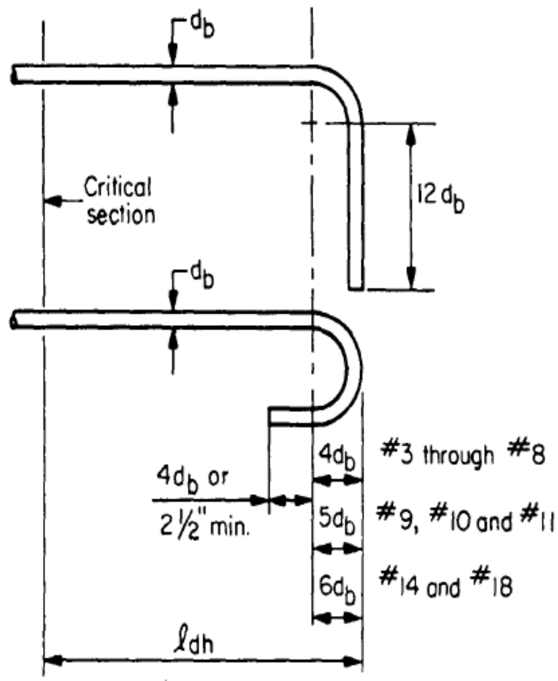



**Figure C5.10.8.2.4a-1—Hooked Bar Details for Development of Standard Hooks (ACI Committee 318 2011)**

## STRAIGHT TOP BAR - B802

| | | | |
|---|---|---|---|
| Bar size | | # 8 | |
| Bar diameter | db = | 1.00 | in |
| Bar area | Ab = | 0.79 | in^2 |
| Basic development length | ldb = | 67.88 | in |
| Normal weight concrete | Lam = | 1.00 | |
| Horizontal reinforcement | Lam.rl = | 1.30 | |
| Cap bars are not epoxy coated | Lam.cf = | 1.00 | |
| Assume need full reinforcement | Lam.er = | 1.00 | |
| Confinement factor (calculated below) | Lam.rc = | 0.50 | |
| | Ld = | 45.00 | in |

*5.10.8.2.1a—Tension Development Length*

The modified tension development length, $\ell_d$, shall not be less than the basic tension development length, $\ell_{db}$, specified herein adjusted by the modification factor or factors specified in Articles 5.10.8.2.1b and 5.10.8.2.1c. The tension development length shall not be less than 12.0 in., except for development of shear reinforcement specified in Article 5.10.8.2.6.

The modified tension development length, $\ell_d$, in in. shall be taken as:

$$\ell_d = \ell_{db} \times \left( \frac{\lambda_{rl} \times \lambda_{cf} \times \lambda_{rc} \times \lambda_{er}}{\lambda} \right) \qquad (5.10.8.2.1a\text{-}1)$$

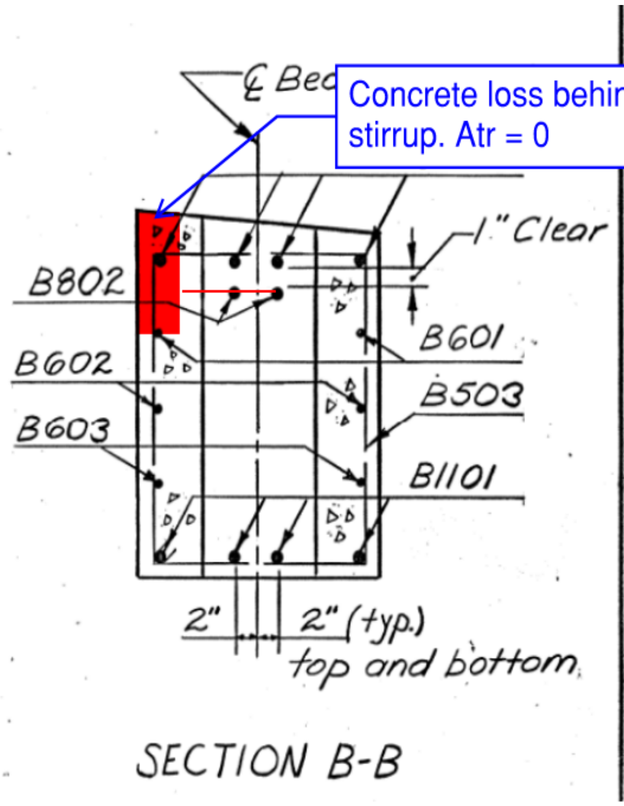
in which:

$$\ell_{db} = 2.4 d_b \frac{f_y}{\sqrt{f'_c}} \qquad (5.10.8.2.1a\text{-}2)$$

where:

$\ell_{db}$ = basic development length (in.)
$\lambda_{rl}$ = reinforcement location factor
$\lambda_{cf}$ = coating factor
$\lambda_{rc}$ = reinforcement confinement factor
$\lambda_{er}$ = excess reinforcement factor
$\lambda$ = concrete density modification factor as specified in Article 5.4.2.8
$d_b$ = nominal diameter of reinforcing bar or wire (in.)
$f_y$ = specified minimum yield strength of reinforcement (ksi)
$f'_c$ = compressive strength of concrete for use in design (ksi)

## DEVELOPMENT




Because stirrups are exposed, assume transverse reinforcement area is 0.

A vertical assumed crack would still have transverse reinforcement and require a lower development length.

- For reinforcement being developed in the length under consideration, $\lambda_{rc}$ shall satisfy the following:

$$0.4 \leq \lambda_{rc} \leq 1.0 \quad (5.10.8.2.1c\text{-}1)$$

in which:

$$\lambda_{rc} = \frac{d_b}{c_b + k_{tr}} \quad (5.10.8.2.1c\text{-}2)$$

$$k_{tr} = 40A_{tr}/(sn) \quad (5.10.8.2.1c\text{-}3)$$

where:

$c_b$ = the smaller of the distance from center of bar or wire being developed to the nearest concrete surface and one-half the center-to-center spacing of the bars or wires being developed (in.)

$k_{tr}$ = transverse reinforcement index

$A_{tr}$ = total cross-sectional area of all transverse reinforcement which is within the spacing $s$ and which crosses the potential plane of splitting through the reinforcement being developed (in.$^2$)

$s$ = maximum center-to-center spacing of transverse reinforcement within $\ell_d$ (in.)

$n$ = number of bars or wires developed along plane of splitting

| | | |
|---|---|---|
| Atr = | 0 | in^2 |
| ktr = | 0 | |
| db = | 1.00 | in^2 |
| cb = | 2 | in |

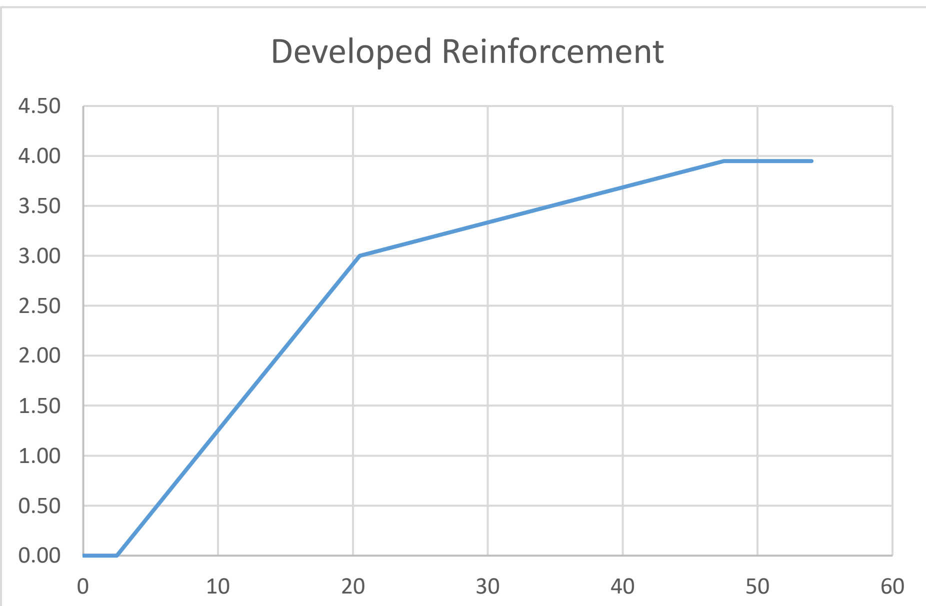
-Half of B802 spacing.

Lam.rc = 0.5

## REINFORCEMENT ALONG CANTILEVER

| | |
|---|---|
| Remaining top bars | **3.00** |
| Remaining 2nd layer bars | **2.00** |

| Location | Distance from Edge in | Description | Developed Length in | Developed Area in^2 |
|---|---|---|---|---|
| 1 | 0 | Edge | 0 | 0.00 |
| 2 | 2.5 | Start of Reinforcement | 0 | 0.00 |
| 3 | 11 | STM Node | 8.5 | 1.42 |
| 4 | 20.5 | B801 developed | 18 | 3.00 |
| 5 | 36 | STM Node | 33.5 | 3.55 |
| 6 | 47.5 | B802 developed | 45 | 3.95 |
| 7 | 54 | STM Node | 51.5 | 3.95 |



## DESCRIPTION:

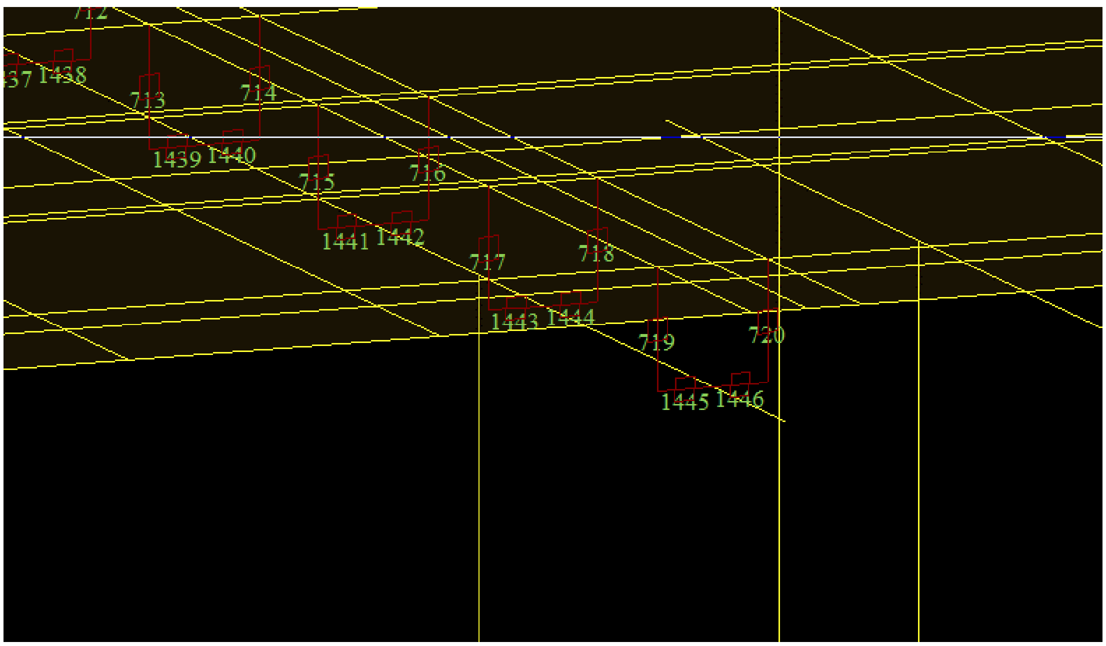
Determine loading from two outside beams on the cantilevered pier cap.

## FULL STRUCTURE MODEL

Each beam end is modeled with a single link. Forces on cantilever are taken by summing both beam ends supported by this cap beam.

Only 1 of 2 bearings from the interior beam are outside the cap support.

Model:




| | | | |
|---|---|---|---|
| Exterior links | 719 | Interior links | 717 |
| | 720 | | 718 |

## DC & DW LOADS

Taken from construction stages

| No. | Stage | Step | Load | Node | Axial (kips) | Shear-y (kips) | Shear-z (kips) | Torsion (ft*kips) | Moment-y (ft*kips) | Moment-z (ft*kips) |
|---|---|---|---|---|---|---|---|---|---|---|
| 717 | Columns Hi | 001(last) | Dead Load | 1380 | -2.9 | -1.99 | 0.01 | 0 | 0 | 0 |
| 717 | Columns Hi | 001(last) | Dead Load | 1377 | -2.9 | -1.99 | 0.01 | 0 | 0 | 0 |
| 718 | Columns Hi | 001(last) | Dead Load | 1379 | -4.78 | -1.9 | 0.05 | 0 | 0 | 0 |
| 718 | Columns Hi | 001(last) | Dead Load | 1376 | -4.78 | -1.9 | 0.05 | 0 | 0 | 0 |
| 719 | Columns Hi | 001(last) | Dead Load | 1586 | -13.43 | -4.63 | 0.03 | 0 | 0 | 0 |
| 719 | Columns Hi | 001(last) | Dead Load | 1583 | -13.43 | -4.63 | 0.03 | 0 | 0 | 0 |
| 720 | Columns Hi | 001(last) | Dead Load | 1585 | -19.42 | -4.69 | 0.06 | 0 | 0 | 0 |
| 720 | Columns Hi | 001(last) | Dead Load | 1582 | -19.42 | -4.69 | 0.06 | 0 | 0 | 0 |
| 717 | Columns Hi | 001(last) | DW | 1380 | -0.49 | -0.03 | 0 | 0 | 0 | 0 |
| 717 | Columns Hi | 001(last) | DW | 1377 | -0.49 | -0.03 | 0 | 0 | 0 | 0 |
| 718 | Columns Hi | 001(last) | DW | 1379 | -0.67 | -0.01 | 0 | 0 | 0 | 0 |
| 718 | Columns Hi | 001(last) | DW | 1376 | -0.67 | -0.01 | 0 | 0 | 0 | 0 |
| 719 | Columns Hi | 001(last) | DW | 1586 | -0.46 | -0.14 | 0 | 0 | 0 | 0 |
| 719 | Columns Hi | 001(last) | DW | 1583 | -0.46 | -0.14 | 0 | 0 | 0 | 0 |
| 720 | Columns Hi | 001(last) | DW | 1585 | -0.73 | -0.12 | 0 | 0 | 0 | 0 |
| 720 | Columns Hi | 001(last) | DW | 1582 | -0.73 | -0.12 | 0 | 0 | 0 | 0 |
| 717 | Columns Hi | 001(last) | Summation | 1380 | -3.39 | -2.02 | 0.01 | 0 | 0 | 0 |
| 717 | Columns Hi | 001(last) | Summation | 1377 | -3.39 | -2.02 | 0.01 | 0 | 0 | 0 |
| 718 | Columns Hi | 001(last) | Summation | 1379 | -5.44 | -1.91 | 0.06 | 0 | 0 | 0 |
| 718 | Columns Hi | 001(last) | Summation | 1376 | -5.44 | -1.91 | 0.06 | 0 | 0 | 0 |
| 719 | Columns Hi | 001(last) | Summation | 1586 | -13.89 | -4.77 | 0.03 | 0 | 0 | 0 |
| 719 | Columns Hi | 001(last) | Summation | 1583 | -13.89 | -4.77 | 0.03 | 0 | 0 | 0 |
| 720 | Columns Hi | 001(last) | Summation | 1585 | -20.15 | -4.8 | 0.07 | 0 | 0 | 0 |
| 720 | Columns Hi | 001(last) | Summation | 1582 | -20.15 | -4.8 | 0.07 | 0 | 0 | 0 |

| | | DC | DW | | | | | DC | DW | |
|---|---|---|---|---|---|---|---|---|---|---|
| Exterior | 719 | 13.43 | 0.46 | kip | Interior | 717 | | 2.9 | 0.49 | kip |
| | 720 | 19.42 | 0.73 | kip | | 718 | | 4.78 | 0.67 | kip |
| Total Exterior | | 32.85 | 1.19 | kip | Total Interior | | | 7.68 | 1.16 | kip |
| Exterior/bearing pad | | 16.425 | 0.595 | kip | Interior/bearing pad | | | 3.84 | 0.58 | kip |

LOADS

## LIVE LOADS

Static load case taken which maximizes negative bending on the cantilever.

| No. | Load | Node | Axial (kips) | Shear-y (kip | Shear-z (kip | Torsion (ft* | Moment-y | Moment-z (ft*kips) |
|---|---|---|---|---|---|---|---|---|
| 717 | MinHl-93_I_My1115 | 1380 | -28.28 | -4.49 | -0.09 | 0 | 0 | 0 |
| 717 | MinHl-93_I_My1115 | 1377 | -28.28 | -4.49 | -0.09 | 0 | 0 | 0 |
| 718 | MinHl-93_I_My1115 | 1379 | -6.75 | -4.56 | 0 | 0 | 0 | 0 |
| 718 | MinHl-93_I_My1115 | 1376 | -6.75 | -4.56 | 0 | 0 | 0 | 0 |
| 719 | MinHl-93_I_My1115 | 1586 | -16.96 | -6.78 | -0.07 | 0 | 0 | 0 |
| 719 | MinHl-93_I_My1115 | 1583 | -16.96 | -6.78 | -0.07 | 0 | 0 | 0 |
| 720 | MinHl-93_I_My1115 | 1585 | -20.83 | -7.14 | 0.01 | 0 | 0 | 0 |
| 720 | MinHl-93_I_My1115 | 1582 | -20.83 | -7.14 | 0.01 | 0 | 0 | 0 |
| 717 | Min2F1_I_My1115 | 1380 | -35.96 | -2.43 | -0.04 | 0 | 0 | 0 |
| 717 | Min2F1_I_My1115 | 1377 | -35.96 | -2.43 | -0.04 | 0 | 0 | 0 |
| 718 | Min2F1_I_My1115 | 1379 | -2.05 | -2.45 | -0.09 | 0 | 0 | 0 |
| 718 | Min2F1_I_My1115 | 1376 | -2.05 | -2.45 | -0.09 | 0 | 0 | 0 |
| 719 | Min2F1_I_My1115 | 1586 | -5.75 | -2.74 | -0.03 | 0 | 0 | 0 |
| 719 | Min2F1_I_My1115 | 1583 | -5.75 | -2.74 | -0.03 | 0 | 0 | 0 |
| 720 | Min2F1_I_My1115 | 1585 | -5.45 | -2.83 | -0.09 | 0 | 0 | 0 |
| 720 | Min2F1_I_My1115 | 1582 | -5.45 | -2.83 | -0.09 | 0 | 0 | 0 |
| 717 | Min3F1_I_My1115 | 1380 | -15.87 | -3.38 | 0.07 | 0 | 0 | 0 |
| 717 | Min3F1_I_My1115 | 1377 | -15.87 | -3.38 | 0.07 | 0 | 0 | 0 |
| 718 | Min3F1_I_My1115 | 1379 | -31.95 | -3.51 | 0 | 0 | 0 | 0 |
| 718 | Min3F1_I_My1115 | 1376 | -31.95 | -3.51 | 0 | 0 | 0 | 0 |
| 719 | Min3F1_I_My1115 | 1586 | -9.41 | -4.05 | 0.09 | 0 | 0 | 0 |
| 719 | Min3F1_I_My1115 | 1583 | -9.41 | -4.05 | 0.09 | 0 | 0 | 0 |
| 720 | Min3F1_I_My1115 | 1585 | -8.94 | -4.34 | 0 | 0 | 0 | 0 |
| 720 | Min3F1_I_My1115 | 1582 | -8.94 | -4.34 | 0 | 0 | 0 | 0 |
| 717 | Min4F1_I_My1115 | 1380 | -32.68 | -3.81 | 0.01 | 0 | 0 | 0 |
| 717 | Min4F1_I_My1115 | 1377 | -32.68 | -3.81 | 0.01 | 0 | 0 | 0 |
| 718 | Min4F1_I_My1115 | 1379 | -17.85 | -3.9 | -0.08 | 0 | 0 | 0 |
| 718 | Min4F1_I_My1115 | 1376 | -17.85 | -3.9 | -0.08 | 0 | 0 | 0 |
| 719 | Min4F1_I_My1115 | 1586 | -10.96 | -4.63 | 0.03 | 0 | 0 | 0 |
| 719 | Min4F1_I_My1115 | 1583 | -10.96 | -4.63 | 0.03 | 0 | 0 | 0 |
| 720 | Min4F1_I_My1115 | 1585 | -10.25 | -4.88 | -0.09 | 0 | 0 | 0 |
| 720 | Min4F1_I_My1115 | 1582 | -10.25 | -4.88 | -0.09 | 0 | 0 | 0 |
| 717 | Min5C1_I_My1115 | 1380 | -15.53 | -3.28 | 0.07 | 0 | 0 | 0 |
| 717 | Min5C1_I_My1115 | 1377 | -15.53 | -3.28 | 0.07 | 0 | 0 | 0 |
| 718 | Min5C1_I_My1115 | 1379 | -31.83 | -3.39 | 0 | 0 | 0 | 0 |
| 718 | Min5C1_I_My1115 | 1376 | -31.83 | -3.39 | 0 | 0 | 0 | 0 |
| 719 | Min5C1_I_My1115 | 1586 | -9.98 | -3.85 | 0.09 | 0 | 0 | 0 |
| 719 | Min5C1_I_My1115 | 1583 | -9.98 | -3.85 | 0.09 | 0 | 0 | 0 |
| 720 | Min5C1_I_My1115 | 1585 | -7.14 | -4.11 | -0.01 | 0 | 0 | 0 |
| 720 | Min5C1_I_My1115 | 1582 | -7.14 | -4.11 | -0.01 | 0 | 0 | 0 |
| 717 | MinEV2_I_My1115 | 1380 | -37.06 | -3.67 | -0.02 | 0 | 0 | 0 |
| 717 | MinEV2_I_My1115 | 1377 | -37.06 | -3.67 | -0.02 | 0 | 0 | 0 |
| 718 | MinEV2_I_My1115 | 1379 | -1.4 | -3.72 | -0.14 | 0 | 0 | 0 |
| 718 | MinEV2_I_My1115 | 1376 | -1.4 | -3.72 | -0.14 | 0 | 0 | 0 |
| 719 | MinEV2_I_My1115 | 1586 | -14.1 | -4.86 | 0 | 0 | 0 | 0 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 719 | MinEV2_I_My1115 | 1583 | -14.1 | -4.86 | 0 | 0 | 0 | 0 |
| 720 | MinEV2_I_My1115 | 1585 | -10.07 | -5.06 | -0.15 | 0 | 0 | 0 |
| 720 | MinEV2_I_My1115 | 1582 | -10.07 | -5.06 | -0.15 | 0 | 0 | 0 |
| 717 | MinEV3_I_My1115 | 1380 | -73.74 | -5.61 | -0.11 | 0 | 0 | 0 |
| 717 | MinEV3_I_My1115 | 1377 | -73.74 | -5.61 | -0.11 | 0 | 0 | 0 |
| 718 | MinEV3_I_My1115 | 1379 | -2.04 | -5.79 | -0.27 | 0 | 0 | 0 |
| 718 | MinEV3_I_My1115 | 1376 | -2.04 | -5.79 | -0.27 | 0 | 0 | 0 |
| 719 | MinEV3_I_My1115 | 1586 | -24 | -6.81 | -0.09 | 0 | 0 | 0 |
| 719 | MinEV3_I_My1115 | 1583 | -24 | -6.81 | -0.09 | 0 | 0 | 0 |
| 720 | MinEV3_I_My1115 | 1585 | -6.75 | -7.1 | -0.3 | 0 | 0 | 0 |
| 720 | MinEV3_I_My1115 | 1582 | -6.75 | -7.1 | -0.3 | 0 | 0 | 0 |
| 717 | MinRPL 60T_I_My1115 | 1380 | -26.38 | -4.43 | 0.05 | 0 | 0 | 0 |
| 717 | MinRPL 60T_I_My1115 | 1377 | -26.38 | -4.43 | 0.05 | 0 | 0 | 0 |
| 718 | MinRPL 60T_I_My1115 | 1379 | -32.88 | -4.66 | 0.02 | 0 | 0 | 0 |
| 718 | MinRPL 60T_I_My1115 | 1376 | -32.88 | -4.66 | 0.02 | 0 | 0 | 0 |
| 719 | MinRPL 60T_I_My1115 | 1586 | -7.26 | -5.57 | 0.08 | 0 | 0 | 0 |
| 719 | MinRPL 60T_I_My1115 | 1583 | -7.26 | -5.57 | 0.08 | 0 | 0 | 0 |
| 720 | MinRPL 60T_I_My1115 | 1585 | -19.4 | -6.05 | 0.03 | 0 | 0 | 0 |
| 720 | MinRPL 60T_I_My1115 | 1582 | -19.4 | -6.05 | 0.03 | 0 | 0 | 0 |
| 717 | MinRPL 65T_I_My1115 | 1380 | -47 | -5.12 | 0 | 0 | 0 | 0 |
| 717 | MinRPL 65T_I_My1115 | 1377 | -47 | -5.12 | 0 | 0 | 0 | 0 |
| 718 | MinRPL 65T_I_My1115 | 1379 | -25.1 | -5.31 | -0.12 | 0 | 0 | 0 |
| 718 | MinRPL 65T_I_My1115 | 1376 | -25.1 | -5.31 | -0.12 | 0 | 0 | 0 |
| 719 | MinRPL 65T_I_My1115 | 1586 | -15.54 | -6.13 | 0.02 | 0 | 0 | 0 |
| 719 | MinRPL 65T_I_My1115 | 1583 | -15.54 | -6.13 | 0.02 | 0 | 0 | 0 |
| 720 | MinRPL 65T_I_My1115 | 1585 | -12.05 | -6.51 | -0.13 | 0 | 0 | 0 |
| 720 | MinRPL 65T_I_My1115 | 1582 | -12.05 | -6.51 | -0.13 | 0 | 0 | 0 |
| 717 | MinSU4_I_My1115 | 1380 | -40.12 | -3.75 | -0.03 | 0 | 0 | 0 |
| 717 | MinSU4_I_My1115 | 1377 | -40.12 | -3.75 | -0.03 | 0 | 0 | 0 |
| 718 | MinSU4_I_My1115 | 1379 | -10.76 | -3.88 | -0.12 | 0 | 0 | 0 |
| 718 | MinSU4_I_My1115 | 1376 | -10.76 | -3.88 | -0.12 | 0 | 0 | 0 |
| 719 | MinSU4_I_My1115 | 1586 | -13.21 | -4.56 | -0.01 | 0 | 0 | 0 |
| 719 | MinSU4_I_My1115 | 1583 | -13.21 | -4.56 | -0.01 | 0 | 0 | 0 |
| 720 | MinSU4_I_My1115 | 1585 | -7.58 | -4.8 | -0.14 | 0 | 0 | 0 |
| 720 | MinSU4_I_My1115 | 1582 | -7.58 | -4.8 | -0.14 | 0 | 0 | 0 |
| 717 | MinSU5_I_My1115 | 1380 | -40.01 | -4.06 | -0.01 | 0 | 0 | 0 |
| 717 | MinSU5_I_My1115 | 1377 | -40.01 | -4.06 | -0.01 | 0 | 0 | 0 |
| 718 | MinSU5_I_My1115 | 1379 | -12.24 | -4.12 | -0.12 | 0 | 0 | 0 |
| 718 | MinSU5_I_My1115 | 1376 | -12.24 | -4.12 | -0.12 | 0 | 0 | 0 |
| 719 | MinSU5_I_My1115 | 1586 | -13.14 | -4.94 | 0.01 | 0 | 0 | 0 |
| 719 | MinSU5_I_My1115 | 1583 | -13.14 | -4.94 | 0.01 | 0 | 0 | 0 |
| 720 | MinSU5_I_My1115 | 1585 | -9.38 | -5.15 | -0.14 | 0 | 0 | 0 |
| 720 | MinSU5_I_My1115 | 1582 | -9.38 | -5.15 | -0.14 | 0 | 0 | 0 |
| 717 | MinSU6_I_My1115 | 1380 | -32.61 | -4.39 | 0.04 | 0 | 0 | 0 |
| 717 | MinSU6_I_My1115 | 1377 | -32.61 | -4.39 | 0.04 | 0 | 0 | 0 |
| 718 | MinSU6_I_My1115 | 1379 | -24.73 | -4.47 | -0.04 | 0 | 0 | 0 |
| 718 | MinSU6_I_My1115 | 1376 | -24.73 | -4.47 | -0.04 | 0 | 0 | 0 |
| 719 | MinSU6_I_My1115 | 1586 | -9.2 | -5.39 | 0.07 | 0 | 0 | 0 |
| 719 | MinSU6_I_My1115 | 1583 | -9.2 | -5.39 | 0.07 | 0 | 0 | 0 |
| 720 | MinSU6_I_My1115 | 1585 | -15.81 | -5.71 | -0.05 | 0 | 0 | 0 |
| 720 | MinSU6_I_My1115 | 1582 | -15.81 | -5.71 | -0.05 | 0 | 0 | 0 |

Michael Baker INTERNATIONAL

LOADS

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 717 | MinSU7_I_My1115 | 1380 | -35.59 | -4.65 | 0.04 | 0 | 0 | 0 |
| 717 | MinSU7_I_My1115 | 1377 | -35.59 | -4.65 | 0.04 | 0 | 0 | 0 |
| 718 | MinSU7_I_My1115 | 1379 | -23.44 | -4.84 | -0.07 | 0 | 0 | 0 |
| 718 | MinSU7_I_My1115 | 1376 | -23.44 | -4.84 | -0.07 | 0 | 0 | 0 |
| 719 | MinSU7_I_My1115 | 1586 | -11.49 | -5.81 | 0.07 | 0 | 0 | 0 |
| 719 | MinSU7_I_My1115 | 1583 | -11.49 | -5.81 | 0.07 | 0 | 0 | 0 |
| 720 | MinSU7_I_My1115 | 1585 | -16.07 | -6.23 | -0.08 | 0 | 0 | 0 |
| 720 | MinSU7_I_My1115 | 1582 | -16.07 | -6.23 | -0.08 | 0 | 0 | 0 |
| 717 | MinType 3_I_My1115 | 1380 | -40.33 | -3.19 | -0.05 | 0 | 0 | 0 |
| 717 | MinType 3_I_My1115 | 1377 | -40.33 | -3.19 | -0.05 | 0 | 0 | 0 |
| 718 | MinType 3_I_My1115 | 1379 | -1.25 | -3.26 | -0.15 | 0 | 0 | 0 |
| 718 | MinType 3_I_My1115 | 1376 | -1.25 | -3.26 | -0.15 | 0 | 0 | 0 |
| 719 | MinType 3_I_My1115 | 1586 | -13.2 | -3.9 | -0.04 | 0 | 0 | 0 |
| 719 | MinType 3_I_My1115 | 1583 | -13.2 | -3.9 | -0.04 | 0 | 0 | 0 |
| 720 | MinType 3_I_My1115 | 1585 | -4.56 | -4.05 | -0.17 | 0 | 0 | 0 |
| 720 | MinType 3_I_My1115 | 1582 | -4.56 | -4.05 | -0.17 | 0 | 0 | 0 |
| 717 | MinType 3-3_I_My1115 | 1380 | -31.2 | -2.63 | -0.03 | 0 | 0 | 0 |
| 717 | MinType 3-3_I_My1115 | 1377 | -31.2 | -2.63 | -0.03 | 0 | 0 | 0 |
| 718 | MinType 3-3_I_My1115 | 1379 | -0.82 | -2.66 | -0.13 | 0 | 0 | 0 |
| 718 | MinType 3-3_I_My1115 | 1376 | -0.82 | -2.66 | -0.13 | 0 | 0 | 0 |
| 719 | MinType 3-3_I_My1115 | 1586 | -11.21 | -3.25 | -0.02 | 0 | 0 | 0 |
| 719 | MinType 3-3_I_My1115 | 1583 | -11.21 | -3.25 | -0.02 | 0 | 0 | 0 |
| 720 | MinType 3-3_I_My1115 | 1585 | -3.79 | -3.35 | -0.14 | 0 | 0 | 0 |
| 720 | MinType 3-3_I_My1115 | 1582 | -3.79 | -3.35 | -0.14 | 0 | 0 | 0 |
| 717 | MinType 3S2_I_My111ṣ | 1380 | -15.07 | -3.01 | 0.06 | 0 | 0 | 0 |
| 717 | MinType 3S2_I_My111ṣ | 1377 | -15.07 | -3.01 | 0.06 | 0 | 0 | 0 |
| 718 | MinType 3S2_I_My111ṣ | 1379 | -28.77 | -3.12 | 0 | 0 | 0 | 0 |
| 718 | MinType 3S2_I_My111ṣ | 1376 | -28.77 | -3.12 | 0 | 0 | 0 | 0 |
| 719 | MinType 3S2_I_My111ṣ | 1586 | -7.87 | -3.53 | 0.08 | 0 | 0 | 0 |
| 719 | MinType 3S2_I_My111ṣ | 1583 | -7.87 | -3.53 | 0.08 | 0 | 0 | 0 |
| 720 | MinType 3S2_I_My111ṣ | 1585 | -7.73 | -3.77 | -0.01 | 0 | 0 | 0 |
| 720 | MinType 3S2_I_My111ṣ | 1582 | -7.73 | -3.77 | -0.01 | 0 | 0 | 0 |

## LOADS

| Live Load bearing reactions that maximize bending in cantilever cap | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | **Exterior** | | | | **Interior** | | | |
| | **Links 719** | **720** | **Total kip** | **Per Pad kip** | **Links 717** | **718** | **Total kip** | **Per Pad kip** |
| HL-93 | -16.96 | -20.83 | -37.79 | -18.9 | -28.28 | -6.75 | -35.03 | -17.5 |
| 2F1 | -5.75 | -5.45 | -11.2 | -5.6 | -35.96 | -2.05 | -38.01 | -19.0 |
| 3F1 | -9.41 | -8.94 | -18.35 | -9.2 | -15.87 | -31.95 | -47.82 | -23.9 |
| 4F1 | -10.96 | -10.25 | -21.21 | -10.6 | -32.68 | -17.85 | -50.53 | -25.3 |
| 5C1 | -9.98 | -7.14 | -17.12 | -8.6 | -15.53 | -31.83 | -47.36 | -23.7 |
| EV2 | -14.1 | -10.07 | -24.17 | -12.1 | -37.06 | -1.4 | -38.46 | -19.2 |
| EV3 | -24 | -6.75 | -30.75 | -15.4 | -73.74 | -2.04 | -75.78 | -37.9 |
| RPL_60T | -7.26 | -19.4 | -26.66 | -13.3 | -26.38 | -32.88 | -59.26 | -29.6 |
| RPL_65T | -15.54 | -12.05 | -27.59 | -13.8 | -47 | -25.1 | -72.1 | -36.1 |
| SU4 | -13.21 | -7.58 | -20.79 | -10.4 | -40.12 | -10.76 | -50.88 | -25.4 |
| SU5 | -13.14 | -9.38 | -22.52 | -11.3 | -40.01 | -12.24 | -52.25 | -26.1 |
| SU6 | -9.2 | -15.81 | -25.01 | -12.5 | -32.61 | -24.73 | -57.34 | -28.7 |
| SU7 | -11.49 | -16.07 | -27.56 | -13.8 | -35.59 | -23.44 | -59.03 | -29.5 |
| Type3 | -13.2 | -4.56 | -17.76 | -8.9 | -40.33 | -1.25 | -41.58 | -20.8 |
| Type3S2 | -11.21 | -3.79 | -15 | -7.5 | -31.2 | -0.82 | -32.02 | -16.01 |
| Type3-3 | -7.87 | -7.73 | -15.6 | -7.8 | -15.07 | -28.77 | -43.84 | -21.92 |

HL-93 live load reactions that maximize bending in cantilever cap

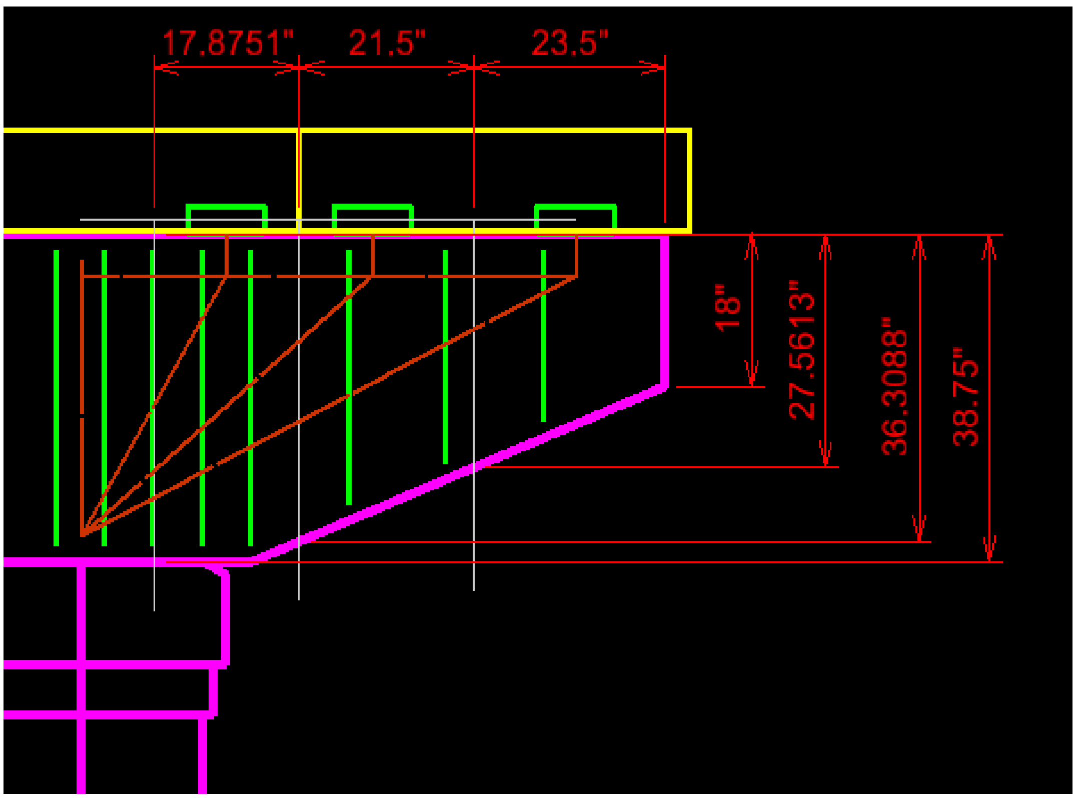
| | | LL+IM | | | | LL+IM | |
|---|---|---|---|---|---|---|---|
| Exterior | 719 | 16.96 | kip | Interior | 717 | 28.28 | kip |
| | 720 | 20.83 | kip | | 718 | 6.75 | kip |
| | | | | | | | |
| Total Exterior | | 37.79 | kip | Total Interior | | 35.03 | kip |
| Exterior/bearing pad | | 18.895 | kip | Interior/bearing pad | | 17.515 | kip |

## PIER CAP SELF WEIGHT

-Additional load from concrete self weight is applied as concentrated loads by tributary area.
-No reduction in width for spalling when calculating weight.

Width 2.50 ft

| Bearing | Trib. Len. ft | Left Depth (ft) ft | Right Depth ft | Avg. Depth ft | Load kip | Total DC kip |
|---|---|---|---|---|---|---|
| 1 | 1.49 | 3.23 | 3.03 | 3.13 | 1.75 | 5.59 |
| 2 | 1.79 | 3.03 | 2.30 | 2.66 | 1.79 | 18.21 |
| 3 | 1.96 | 2.30 | 1.50 | 1.90 | 1.39 | 17.82 |



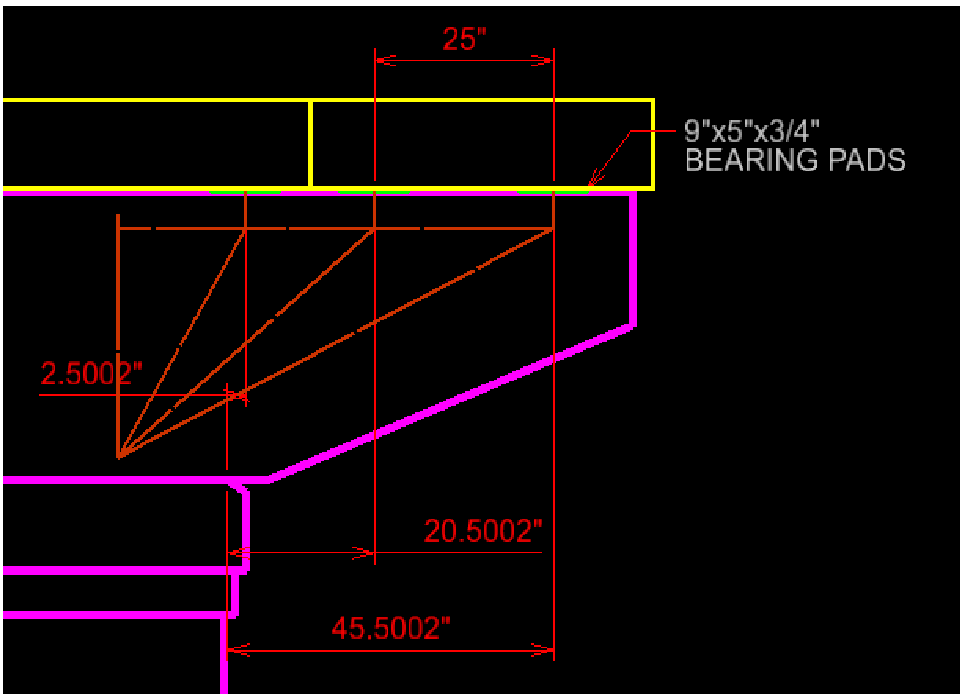
## LOAD COMBINATIONS

-Design loading (HL-93)

-E1 is from the center of a bearing to face of support and E2 is at the first interior beam bearing.
-E2 is from the the most outside bearing to the second most interior bearing.

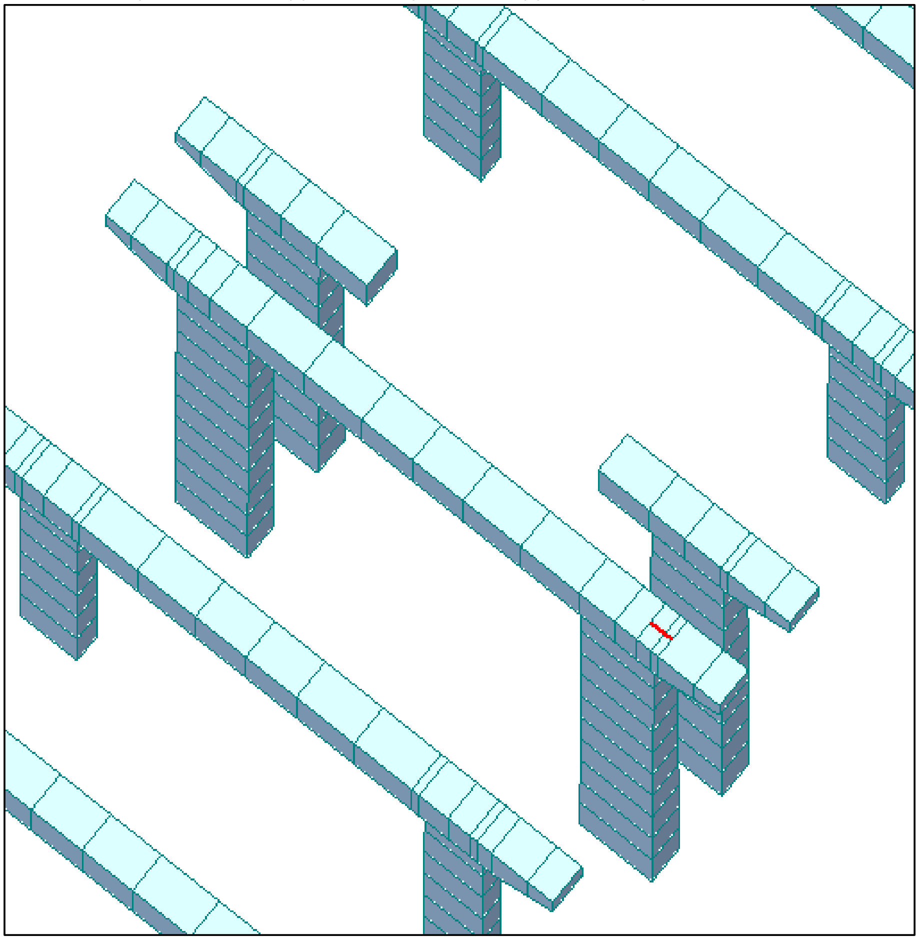
| Bearing | DC kip | DW kip | LL+IM kip | STR 1 kip | E1 in | M1 kip-in | E2 in | M2 kip-in | Location |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 5.59 | 0.58 | 17.52 | 38.50 | 2.50 | 96.26 | | 0.00 | Int. Beam |
| 2 | 18.21 | 0.60 | 18.90 | 56.73 | 20.50 | 1,162.86 | | 0.00 | Ext. Beam 1 |
| 3 | 17.82 | 0.60 | 18.90 | 56.23 | 45.50 | 2,558.59 | 25.00 | 1,405.82 | Ext. Beam 2 |

Total Vu = 151.46 kip $M_{u1}$ = 3,817.71 kip-in $M_{2u}$ = 1,405.82 kip-in
318.14 kip-ft 117.15 kip-ft



## TORSION CHECK BEAM ELEMENT

-Torsion check performed at approximately face of support, erring towards over the column which is conservative



## LOADS

Element 1116_i

| Elem | Load | Stage | Step | Part | Axial (kips) | Shear-y (kips) | Shear-z (kips) | Torsion (ft*kips) | Moment-y (ft*kips) | Moment-z (ft*kips) |
|---|---|---|---|---|---|---|---|---|---|---|
| 1116 | Dead Load | Columns Hinge | 001(last) | I[1375] | -9.32 | -0.09 | -38.71 | -4.35 | -131.09 | -0.32 |
| 1116 | DW | Columns Hinge | 001(last) | I[1375] | -0.25 | 0 | -1.19 | -0.19 | -4.37 | -0.03 |

| Elem | Load | Part | Axial (kips) | Shear-y (kips) | Shear-z (kips) | Torsion (ft*kips) | Moment-y (ft*kips) | Moment-z (ft*kips) |
|---|---|---|---|---|---|---|---|---|
| 1116 | Hl-93(max) | I[1375] | 6.77 | 0.7 | 14.34 | 16.95 | 52.1 | 2.95 |
| 1116 | Hl-93(min) | I[1375] | -15.19 | -0.7 | -39.85 | -22.87 | -136.37 | -2.68 |

Factored Loads

| | | |
|---|---|---|
| Pu = | 11.85 | kip |
| Vu = | -119.91 | kip |
| Mu = | -409.07 | kip-ft |
| Tu = | -45.745 | kip-ft |

Tension is conservative. Compression ignored for Axial forces.

**BEAM-HALF DEPTH**

## DESCRIPTION:

Use B-Beam assumptions to check design level bending and shear in cap at exterior bearing, as a parallel check to the strut and tie model.
Shown to be adequate at the design level so no rating factors were generated.

## DESIGN CHECKS

-Shown to be adequate for design checks so no rating factors were calculated

| Check | | Capacity | Units | | Demand | Units | | | | OK/NG |
|---|---|---|---|---|---|---|---|---|---|---|
| Bending | | 261.38 | kip-ft | ≥ | 117.15 | kip-ft | | | | OK |
| Shear | | 129.29 | kip | ≥ | 112.96 | kip | | | | OK |

## MATERIAL PROPERTIES, SECTION GEOMETRY, AND GENERAL INPUTS:

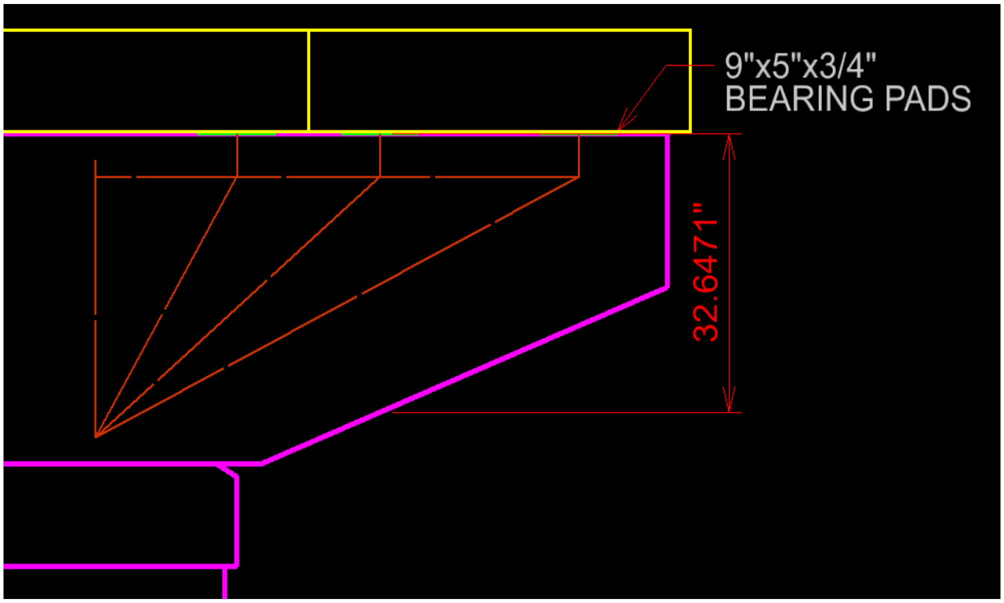
**MATERIAL PROPERTIES:**

| | | | | |
|---|---|---|---|---|
| Concrete compressive strength | $f'c$ = | **4.50** | ksi | |
| Elastic modulus of concrete | $E_c = 1{,}820\sqrt{f'c}$ = | 3861 | ksi | AASHTO C5.4.2.4-1 |
| Stress block factor | $\alpha 1$ = | 0.85 | | AASHTO 5.6.2.2 |
| Effective to total compression depth ratio | $\beta_1$ = | 0.83 | | AASHTO 5.6.2.2 |
| Concrete compressive strain | $\varepsilon_c$ = | **0.003** | | AASHTO 5.7.2.1 |
| Lightweight concrete factor | $\lambda$ = | **1.00** | | AASHTO 5.4.2.8 |
| | | | | |
| Reinforcement yielding | fy = | **60.00** | ksi | |
| Elastic modulus of reinforcement | Es = | **29,000.00** | ksi | |
| Tension limit reinforcement strain | $\varepsilon_{tl}$ = | **0.005** | | AASHTO 5.7.2.1 |
| Compression limit reinforcement strain | $\varepsilon_{tl}$ = | **0.002** | | AASHTO 5.7.2.1 |

**BEAM-HALF DEPTH**

**SECTION GEOMETRY:**

| | | | |
|---|---|---|---|
| Section height | h = | 32.65 | in |
| Section width | b = | 27.00 | in |
| Bottom clear cover | c = | 2.00 | in |
| Side clear cover | $c_s$ = | 2.00 | in |



| | | |
|---|---|---|
| **PROJECT :** *BEL-40-23.37* | | Michael Baker INTERNATIONAL |
| **TASK :** *Rating* | **PROJECT NO :** *195987* | |
| **SUBJECT :** *Pier 5 Cap Rating* | | |
| **CALCULATED BY :** *JCC* **DATE :** *10/3/2023* | **CHECKED BY :** *ETB* | **DATE :** -- |

**BEAM-HALF DEPTH**

**REINFORCEMENT:**

| | | | |
|---|---|---|---|
| Location along cap | | 2.125 | ft |
| Bottom bar size | | **# 8** | |
| Row 1 Bars | | **3** | |
| Row 2 bars (ignored because not fully developed) | | **0** | |
| Row spacing | spa = | **2.00** | in |
| Bars | | 3 | |
| Bar Area | Abar = | 0.79 | $in^2$ |
| Bar Diameter | dbar = | 1 | in |
| Total reinforcement area | As = | 2.37 | $in^2$ |
| Width for spacing | bs = | 27.00 | in |
| Bar Spacing | s = | 11.50 | in |
| Row 1 depth | d1 = h - c - dv - dbar/2 = | 29.52 | in |
| Row 2 depth | d2 = d1 - spa = | 27.52 | in |
| Average depth | d = (d1*n1 + d2*n2) / (n1 + n2) = | 29.52 | in |

**Stirrups**

| | | | |
|---|---|---|---|
| Size | | **# 5** | |
| Area | $A_v$ = | 0.31 | $in^2$ |
| Diameter | dv = | 0.625 | in |
| Spacing at max shear (ignored first stirrup in 6" spacing) | s = | **18.00** | in |
| Legs at max shear | $n_{legs}$ = | **2** | |
| | | | |
| Extreme reinforcement depth | $d = h - c - d_v - d_{bar}/2$ = | 29.5221 | in |

**REDUCTION FACTORS**

| | | | |
|---|---|---|---|
| Flexure - Tension controlled | $\phi_b$ = | **0.90** | AASHTO 5.5.4.2 |
| Flexure - Compression controlled | $\phi_b$ = | **0.75** | AASHTO 5.5.4.2 |
| Compression controlled reinforcement strain | $\varepsilon_{cl}$ = | **0.002** | AASHTO 5.6.2.1 |
| Tension controlled reinforcement strain | $\varepsilon_{tl}$ = | **0.005** | AASHTO 5.6.2.1 |
| Shear | $\phi_v$ = | **0.90** | AASHTO 5.5.4.2 |
| Poor condition factor | $\phi_c$ = | **0.85** | MBE 6A.4.2.3-1 |

Michael Baker INTERNATIONAL

**BEAM-HALF DEPTH**

**DESIGN LOADING**

| | | | |
|---|---|---|---|
| Maximum moment | Mu = | **117.15** | kip-ft |
| Shear | Vu = | **112.96** | kip |
| Moment concurrent with shear | Mu = | **117.15** | kip-ft |
| Axial force concurrent with shear (tension is positive) | Nu = | **0.00** | kip |

## FLEXURE — AASHTO 5.6.3

| | | | | |
|---|---|---|---|---|
| Compression depth | $c = \dfrac{A_s f_y}{\alpha_1 f_c' \beta_1 b} =$ | 1.66 | in | AASHTO 5.6.3.1.2-4 |
| Effective compression depth | $a = \beta_1 c =$ | 1.38 | in | AASHTO 5.6.3.2.3 |
| Flexure capacity | $M_n = \left[A_s f_y \left(d - \dfrac{a}{2}\right)\right] =$ | 4,100.14 | kip-in | AASHTO 5.6.3.2.2-1 |
| | = | 341.68 | kip-ft | |
| Reinforcement strain | $\varepsilon_c \left(\dfrac{d}{c} - 1\right) =$ | 0.050 | | |
| | $\phi = \max\left(0.75, \min\left(0.9, 0.75 + \dfrac{0.15(\varepsilon_t - \varepsilon_{cl})}{\varepsilon_{tl} - \varepsilon_{cl}}\right)\right) =$ | 0.90 | | AASHTO Figure C5.5.4.2-1 |
| | Mr = ϕϕcMn = | 261.38 | | |
| Factored moment | Mu = | 117.15 | kip-ft | OK |

**BEAM-HALF DEPTH**

## SHEAR — AASHTO 5.7.3

| | | | | |
|---|---|---|---|---|
| Shear depth | d - a/2 = | 28.83 | in | |
| | 0.9de = 0.9d = | 26.57 | in | |
| | 0.72h = | 23.50591 | in | |
| | dv = | 28.83 | in | AASHTO 5.7.2.8 |
| Concrete shear width | bv = | 27.00 | in | |
| Lightweight modification factor | λ = | 1.00 | | |
| Concrete compressive strength | f'c = | 4.50 | ksi | |
| Reinforcement strength | fy = | 60.00 | ksi | |
| Shear area | Av = | 0.62 | in^2 | |
| Spacing | s = | 18.00 | in | |

General procedure (AASHTO 5.7.3.4.2)

| | | | | |
|---|---|---|---|---|
| Factored shear | Vu = | 112.96 | kip | |
| Concurrent moment | Mu = | 117.15 | kip-ft | |
| Concurrent axial force | Nu = | 0.00 | kip | |
| Elastic modulus of reinforcement | Es = | 29000 | ksi | |
| Total reinforcement area | As = | 2.37 | $in^2$ | |

$$\varepsilon_s = \frac{\frac{|M_u|}{d_v} + 0.5N_u + |V_u|}{E_s A_s} = 0.002353$$ AASHTO 5.7.3.4.2-4

$$\theta = 29 + 3500\varepsilon_s = 37.24 \text{ deg}$$ AASHTO 5.7.3.4.2-3

$$\beta = \frac{4.8}{1 + 750\varepsilon_s} = 1.74$$ AASHTO 5.7.3.4.2-1

$$V_c = 0.0316\beta\lambda\sqrt{f_c'}b_v d_v = 90.61 \text{ kip}$$ AASHTO 5.7.3.3-3

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = 78.41 \text{ kip}$$

Maximum shear $$V_{nmax} = 0.25 f_c' b_v d_v = 875.82 \text{ kip}$$ AASHTO 5.7.3.3-2

$$V_n = \min(V_{nmax}, V_c + V_s) = 169.01 \text{ kip}$$

Vr = ϕvVn = 129.29 kip

Factored shear Vu = 112.96 kip **OK**

**BEAM-FULL DEPTH**

## DESCRIPTION:

Use B-Beam assumptions to check design level bending and shear in cap at exterior bearing, as a parallel check to the strut and tie model.
Shown to be adequate at the design level so no rating factors were generated.

## MATERIAL PROPERTIES, SECTION GEOMETRY, AND GENERAL INPUTS:

**MATERIAL PROPERTIES:**

| | | | | |
|---|---|---|---|---|
| Concrete compressive strength | f'c = | **4.50** | ksi | |
| Elastic modulus of concrete | Ec = 1,820 √(f'c) = | 3861 | ksi | AASHTO C5.4.2.4-1 |
| Stress block factor | α1 = | 0.85 | | AASHTO 5.6.2.2 |
| Effective to total compression depth ratio | $\beta_1$ = | 0.83 | | AASHTO 5.6.2.2 |
| Concrete compressive strain | $\varepsilon_c$ = | **0.003** | | AASHTO 5.7.2.1 |
| Lightweight concrete factor | λ = | **1.00** | | AASHTO 5.4.2.8 |
| Reinforcement yielding | fy = | **60.00** | ksi | |
| Elastic modulus of reinforcement | Es = | **29,000.00** | ksi | |
| Tension limit reinforcement strain | $\varepsilon_{tl}$ = | **0.005** | | AASHTO 5.7.2.1 |
| Compression limit reinforcement strain | $\varepsilon_{tl}$ = | **0.002** | | AASHTO 5.7.2.1 |

**SECTION GEOMETRY:**

| | | | |
|---|---|---|---|
| Section height | h = | **38.75** | in |
| Section width | b = | **27.00** | in |
| Bottom clear cover | c = | **2.00** | in |
| Side clear cover | $c_s$ = | **2.00** | in |

**REINFORCEMENT:**

| | | | |
|---|---|---|---|
| Bottom bar size | | **# 8** | |
| Row 1 Bars | | **3** | |
| Row 2 bars | | **2** | |
| Row spacing | spa = | **2.00** | in |
| Bars | | 5 | |
| Bar Area | Abar = | 0.79 | $in^2$ |
| Bar Diameter | dbar = | 1 | in |
| Total reinforcement area | As = | 3.95 | $in^2$ |
| Width for spacing | bs = | 27.00 | in |
| Bar Spacing | s = | 11.50 | in |
| Row 1 depth | d1 = h - c - dv - dbar/2 = | 35.63 | in |
| Row 2 depth | d2 = d1 - spa = | 33.63 | in |
| Average depth | d = (d1*n1 + d2*n2) / (n1 + n2) = | 34.83 | in |

**BEAM-FULL DEPTH**

**Stirrups**

| | | | |
|---|---|---|---|
| Size | | **# 5** | |
| Area | $A_v$ = | 0.31 | $in^2$ |
| Diameter | dv = | 0.625 | in |
| Spacing at max shear (ignored 1 stirrup leg) | s = | **12.00** | in |
| Legs at max shear | $n_{legs}$ = | **2** | |
| Extreme reinforcement depth | $d = h - c - d_v - d_{bar}/2$ = | 35.625 | in |

**REDUCTION FACTORS**

| | | | |
|---|---|---|---|
| Flexure - Tension controlled | $\phi_b$ = | **0.90** | AASHTO 5.5.4.2 |
| Flexure - Compression controlled | $\phi_b$ = | **0.75** | AASHTO 5.5.4.2 |
| Compression controlled reinforcement strain | $\varepsilon_{cl}$ = | **0.002** | AASHTO 5.6.2.1 |
| Tension controlled reinforcement strain | $\varepsilon_{tl}$ = | **0.005** | AASHTO 5.6.2.1 |
| Shear | $\phi_v$ = | **0.90** | AASHTO 5.5.4.2 |
| Poor condition factor | $\phi_c$ = | **0.85** | MBE 6A.4.2.3-1 |

**LOADING**

| | | | |
|---|---|---|---|
| Maximum moment | Mu = | **318.14** | kip-ft |
| Shear | Vu = | **151.46** | kip |
| Moment concurrent with shear | Mu = | **318.14** | kip-ft |
| Axial force concurrent with shear (tension is positive) | Nu = | **0.00** | kip |

**BEAM-FULL DEPTH**

## FLEXURE — AASHTO 5.6.3

| | | | | |
|---|---|---|---|---|
| Compression depth | $c = \dfrac{A_s f_y}{\alpha_1 f_c' \beta_1 b} =$ | 2.76 | in | AASHTO 5.6.3.1.2-4 |
| Effective compression depth | $a = \beta_1 c =$ | 2.29 | in | AASHTO 5.6.3.2.3 |
| Flexure capacity | $M_n = \left[A_s f_y \left(d - \dfrac{a}{2}\right)\right] =$ | 7,981.59 | kip-in | AASHTO 5.6.3.2.2-1 |
| | = | 665.13 | kip-ft | |
| Reinforcement strain | $\varepsilon_c \left(\dfrac{d}{c} - 1\right) =$ | 0.036 | | |
| | $\phi = \max\left(0.75, \min\left(0.9, 0.75 + \dfrac{0.15(\varepsilon_t - \varepsilon_{cl})}{\varepsilon_{tl} - \varepsilon_{cl}}\right)\right) =$ | 0.90 | | AASHTO Figure C5.5.4.2-1 |
| | Mr = ϕϕcMn = | 508.83 | | |
| Factored moment | Mu = | 318.14 | kip-ft | OK |

**BEAM-FULL DEPTH**

## SHEAR — AASHTO 5.7.3

| | | | | |
|---|---|---|---|---|
| Shear depth | d - a/2 = | 33.68 | in | |
| | 0.9de = 0.9d = | 31.34 | in | |
| | 0.72h = | 27.9 | in | |
| | dv = | 33.68 | in | AASHTO 5.7.2.8 |
| Concrete shear width | bv = | 27.00 | in | |
| Lightweight modification factor | λ = | 1.00 | | |
| Concrete compressive strength | f'c = | 4.50 | ksi | |
| Reinforcement strength | fy = | 60.00 | ksi | |
| Shear area | Av = | 0.62 | in^2 | |
| Spacing | s = | 12.00 | in | |

General procedure (AASHTO 5.7.3.4.2)

| | | | | |
|---|---|---|---|---|
| Factored shear | Vu = | 151.46 | kip | |
| Concurrent moment | Mu = | 3,817.71 | kip-in | |
| Concurrent axial force | Nu = | 0.00 | kip | |
| Elastic modulus of reinforcement | Es = | 29000 | ksi | |
| Total reinforcement area | As = | 3.95 | $in^2$ | |

$$\varepsilon_s = \frac{\frac{|M_u|}{d_v} + 0.5N_u + |V_u|}{E_s A_s} = 0.002312$$ AASHTO 5.7.3.4.2-4

$$\theta = 29 + 3500\varepsilon_s = 37.09 \text{ deg}$$ AASHTO 5.7.3.4.2-3

$$\beta = \frac{4.8}{1 + 750\varepsilon_s} = 1.76$$ AASHTO 5.7.3.4.2-1

$$V_c = 0.0316\beta\lambda\sqrt{f_c'}b_v d_v = 107.02 \text{ kip}$$ AASHTO 5.7.3.3-3

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = 138.08 \text{ kip}$$

Maximum shear $$V_{nmax} = 0.25f_c' b_v d_v = 1{,}022.96 \text{ kip}$$ AASHTO 5.7.3.3-2

$$V_n = \min(V_{nmax}, V_c + V_s) = 245.10 \text{ kip}$$

Vr = ϕvϕcVn = 187.50 kip

Factored shear Vu = 151.46 kip **OK**

| | | | |
|---|---|---|---|
| PROJECT : *BEL-40-23.37* | | | Michael Baker INTERNATIONAL |
| TASK : *Rating* | | PROJECT NO : *195987* | |
| SUBJECT : *Pier 5 Cap Rating* | | | |
| CALCULATED BY : *JCC* | DATE : *10/3/2023* | CHECKED BY : *ETB* | DATE : -- |

**TORSION CHECK**

## DESCRIPTION:

Check shear & torsion using combined check from AASHTO 5.7.3.4.2 & 5.7.3.6.
-Checked at the center of support which is conservative.
Shown to be adequate at the design level so no rating factors were generated.

## DESIGN CHECKS

-Shown to be adequate for design checks so no rating factors were calculated

| Check | | Capacity | Units | | Demand | Units | | | | OK/NG |
|---|---|---|---|---|---|---|---|---|---|---|
| Shear | | 183.15 | kip | ≥ | 127.39 | kip | | | | OK |
| Torsion | | 2,619.48 | kip-in | ≥ | 548.94 | kip-in | | | | OK |
| Longitudinal Reinf. | | 560.68 | kip | ≥ | 319.52 | kip | | | | OK |

## MATERIAL PROPERTIES, SECTION GEOMETRY, AND GENERAL INPUTS:

**MATERIAL PROPERTIES:**

| | | | | |
|---|---|---|---|---|
| Concrete compressive strength | f'c = | **4.50** | ksi | |
| Elastic modulus of concrete | Ec = 1,820 √(f'c) = | 3861 | ksi | AASHTO C5.4.2.4-1 |
| Stress block factor | α1 = | 0.85 | | AASHTO 5.6.2.2 |
| Effective to total compression depth ratio | $\beta_1$ = | 0.83 | | AASHTO 5.6.2.2 |
| Concrete compressive strain | $\varepsilon_c$ = | **0.003** | | AASHTO 5.7.2.1 |
| Lightweight concrete factor | λ = | **1.00** | | AASHTO 5.4.2.8 |
| Reinforcement yielding | fy = | **60.00** | ksi | |
| Elastic modulus of reinforcement | Es = | **29,000.00** | ksi | |
| Tension limit reinforcement strain | $\varepsilon_{tl}$ = | **0.005** | | AASHTO 5.7.2.1 |
| Compression limit reinforcement strain | $\varepsilon_{tl}$ = | **0.002** | | AASHTO 5.7.2.1 |

**SECTION GEOMETRY:**

| | | | |
|---|---|---|---|
| Section height | h = | **38.75** | in |
| Section width | b = | **27.00** | in |
| Bottom clear cover | c = | **2.00** | in |
| Side clear cover | $c_s$ = | **2.00** | in |
| Concrete area | Acp = h*b = | 1046.25 | in^2 |
| Length of outside perimeter of the concrete section | pc = 2*(h+b) = | 131.5 | in |

**TORSION CHECK**

**REINFORCEMENT:**

| | | | |
|---|---|---|---|
| Top bar size | | **# 8** | |
| Row 1 Bars | | **3** | |
| Row 2 bars | | **2** | |
| | | | |
| Row spacing | spa = | **2.00** | in |
| Bars | | 5 | |
| Bar Area | Abar = | 0.79 | $in^2$ |
| Bar Diameter | dbar = | 1 | in |
| Total reinforcement area | As = | 3.95 | $in^2$ |
| Width for spacing | bs = | 27.00 | in |
| Bar Spacing | s = | 11.50 | in |
| Row 1 depth | d1 = h - c - dv - dbar/2 = | 35.63 | in |
| Row 2 depth | d2 = d1 - spa = | 33.63 | in |
| Average depth | d = (d1*n1 + d2*n2) / (n1 + n2) = | 34.83 | in |
| | | | |
| Bottom bar size | | **# 11** | |
| Bottom bars | | **4** | |
| Bar Area | Abar = | 1.56 | $in^2$ |
| Bar Diameter | dbar = | 1.41 | in |
| Bottom bar area | Abot = | 6.24 | $in^2$ |
| | | | |
| Side bar size | | **# 6** | |
| Side bars | | **6** | |
| Bar Area | Abar = | 0.44 | $in^2$ |
| Bar Diameter | dbar = | 0.75 | in |
| Side bar area | Aside = | 2.64 | $in^2$ |
| | | | |
| Total longitudinal reinforcement | Al = As + Abot + Aside = | 12.83 | in^2 |

**Stirrups**

| | | | |
|---|---|---|---|
| Size | | **# 5** | |
| Area | $A_v$ = | 0.31 | $in^2$ |
| Diameter | dv = | 0.625 | in |
| Spacing at max shear (ignored 1 stirrup leg) | s = | **12.00** | in |
| Legs at max shear | $n_{legs}$ = | **2** | |
| | | | |
| Extreme reinforcement depth | d = h - c - $d_v$ - $d_{bar}$/2 = | 35.625 | in |
| Area enclosed by stirrups | Aoh = 2*((h - 2c - dv)*(b- - 2c - dv))= | 763.55 | in^2 |
| Area enclosed by shear flow path | Ao = 0.85Aoh = | 649.01 | in^2 |
| -Based on ACI 318-14 22.7.6.1.1. | | | |
| Perimeter of the centerline of the closed transverse torsion reinforcement | | | |
| | ph = 2*((h - 2c - dv)+(b- - 2c - dv))= | 113.00 | in |

**TORSION CHECK**

**REDUCTION FACTORS**

| | | | |
|---|---|---|---|
| Flexure - Tension controlled | $\phi_b$ = | **0.90** | AASHTO 5.5.4.2 |
| Flexure - Compression controlled | $\phi_b$ = | **0.75** | AASHTO 5.5.4.2 |
| Compression controlled reinforcement strain | $\varepsilon_{cl}$ = | **0.002** | AASHTO 5.6.2.1 |
| Tension controlled reinforcement strain | $\varepsilon_{tl}$ = | **0.005** | AASHTO 5.6.2.1 |
| Shear & Torsion | $\phi_v$ = | **0.90** | AASHTO 5.5.4.2 |
| Poor condition factor | $\phi_c$ = | **0.85** | MBE 6A.4.2.3-1 |

**LOADING**

| | | | |
|---|---|---|---|
| Maximum moment | Mu = | **409.07** | kip-ft |
| Shear | Vu = | **119.91** | kip |
| Moment concurrent with shear | Mu = | **409.07** | kip-ft |
| Axial force concurrent with shear (tension is positive) | Nu = | **11.85** | kip |
| Torsion | Tu = | **45.75** | kip-ft |

-Taken from MIDAS model, element 1115i, STR I

TORSION CHECK

## FLEXURE — AASHTO 5.7.2.1-6

-Check

| | | | | |
|---|---|---|---|---|
| Compression depth | $c = \frac{A_s f_y}{\alpha_1 f_c' \beta_1 b} =$ | 2.76 | in | AASHTO 5.6.3.1.2-4 |
| Effective compression depth | $a = \beta_1 c =$ | 2.29 | in | AASHTO 5.6.3.2.3 |
| Flexure capacity | $M_n = \left[A_s f_y\left(d - \frac{a}{2}\right)\right] =$ | 7,981.59 | kip-in | AASHTO 5.6.3.2.2-1 |
| | = | 665.13 | kip-ft | |
| Reinforcement strain | $\varepsilon_c\left(\frac{d}{c} - 1\right) =$ | 0.036 | | |
| | $\phi = \max\left(0.75, \min\left(0.9, 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{cl})}{\varepsilon_{tl} - \varepsilon_{cl}}\right)\right) =$ | 0.90 | | AASHTO Figure C5.5.4.2-1 |
| | Mr = ϕϕcMn = | 508.83 | | |
| Factored moment | Mu = | 409.07 | kip-ft | OK |

**TORSION CHECK**

## SHEAR — AASHTO 5.7.3

| | | | | |
|---|---|---|---|---|
| Shear depth | d - a/2 = | 33.68 | in | |
| | 0.9de = 0.9d = | 31.34 | in | |
| | 0.72h = | 27.9 | in | |
| Shear depth | dv = | 33.68 | in | AASHTO 5.7.2.8 |
| Concrete shear width | bv = | 27.00 | in | |
| Lightweight modification factor | λ = | 1.00 | | |
| Concrete compressive strength | f'c = | 4.50 | ksi | |
| Reinforcement strength | fy = | 60.00 | ksi | |
| Shear area | Av = | 0.62 | in^2 | |
| Spacing | s = | 12.00 | in | |
| Perimeter of centerline of the closed transverse reinforcement | ph = | 113.00 | in | |
| Area enclosed by the shear flow path | Ao = | 649.01 | in^2 | |

General procedure (AASHTO 5.7.3.4.2)

| | | | | |
|---|---|---|---|---|
| Factored shear | Vu = | 119.91 | kip | |
| Concurrent moment | Mu = | 4,908.78 | kip-in | |
| Concurrent axial force | Nu = | 11.85 | kip | |
| Elastic modulus of reinforcement | Es = | 29000 | ksi | |
| Total reinforcement area | As = | 3.95 | $in^2$ | |
| Torsion | Tu = | 548.94 | kip-in | |

$$V_{eff} = \sqrt{V_u^2 + \left(\frac{0.9 p_h T_u}{2A_o}\right)^2} = \quad 127.39 \text{ kip}$$

$$\varepsilon_s = \frac{\frac{|M_u|}{d_v} + 0.5N_u + |V_u|}{E_s A_s} = \quad 0.002436 \qquad \text{AASHTO 5.7.3.4.2-4}$$

$$\theta = 29 + 3500\varepsilon_s = \quad 37.53 \text{ deg} \qquad \text{AASHTO 5.7.3.4.2-3}$$

$$\beta = \frac{4.8}{1 + 750\varepsilon_s} = \quad 1.70 \qquad \text{AASHTO 5.7.3.4.2-1}$$

$$V_c = 0.0316\beta\lambda\sqrt{f_c'}b_v d_v = \quad 103.49 \text{ kip} \qquad \text{AASHTO 5.7.3.3-3}$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \quad 135.93 \text{ kip}$$

Maximum shear

$$V_{nmax} = 0.25 f_c' b_v d_v = \quad 1{,}022.96 \text{ kip} \qquad \text{AASHTO 5.7.3.3-2}$$

$$V_n = \min(V_{nmax}, V_c + V_s) = \quad 239.41 \text{ kip}$$

## TORSION CHECK

| | | | | |
|---|---|---|---|---|
| | Vr = ϕvϕcVn = | 183.15 | kip | |
| Factored shear | Veff = | 127.39 | kip | OK |

## TORSION CHECK — AASHTO 5.7.3.6.2

| | | | | |
|---|---|---|---|---|
| Area enclosed by the shear flow path | Ao = | 649.01 | in^2 | |
| Area of one leg of closed transverse reinforcement | At = | 0.31 | in^2 | |
| Yield strength | fy = | 60.00 | ksi | |
| Shear angle accounting for torsion | θ = | 37.53 | deg | |
| Stirrup spacing | s = | 12.00 | in | |
| Duct shear strength reduction factor | λduct = | **1.00** | | |
| Nominal shear resistance | $T_n = \dfrac{2A_oA_tf_y\cot\theta}{s}\lambda_{duct} =$ | 2619.476 | kip-in | AASHTO 5.7.3.6.2-1 |
| Factored torsion | Tu = | 548.94 | kip-in | **OK** |

| | | | |
|---|---|---|---|
| PROJECT : *BEL-40-23.37* | | | Michael Baker INTERNATIONAL |
| TASK : *Rating* | | PROJECT NO : *195987* | |
| SUBJECT : *Pier 5 Cap Rating* | | | |
| CALCULATED BY : *JCC* | DATE : *10/3/2023* | CHECKED BY : *ETB* | DATE : -- |

**TORSION CHECK**

## LONGITUDINAL REINFORCEMENT CHECK — AASHTO 5.7.3.6.3

| | | | |
|---|---|---|---|
| Calculate reinforcement | | | |
| Top bar area | Abar = | 0.79 | in^2 |
| Top bars | ntop = | 3 | |
| Flexure demand/capacity ratio | D/C = | 0.80 | |
| Top bar area remaining for longitudinal reinforcement check | Atop = | 0.46 | in^2 |
| | | | |
| Bottom & side bars | | | |
| Bottom bar area | Abot = | 6.24 | in^2 |
| Side bar area | Aside = | 2.64 | in^2 |
| | | | |
| Longitudinal reinforcement check area | As = | 9.34 | in^2 |
| Yield strength | fy = | 60.00 | ksi |
| Absolute value of factored moment | \|Mu\| = | 4908.78 | kip-in |
| Shear depth | dv = | 33.68 | in |
| Combined resistance factor | ϕ = | 0.765 | |
| Factored axial force | Nu = | 11.85 | kip |
| Shear angle of inclination | θ = | 37.53 | deg |
| Factored shear force | Vu = | 119.91 | kip |
| Prestress steel shear resistance | Vp = | **0.00** | kip |
| Reinforcement shear resistance | Vs = | 135.93 | kip |
| Perimeter of closed transverse torsion reinforcement | ph = | 113.00 | in |
| Factored torsion | Tu = | 548.94 | kip-in |
| Area enclosed by shear flow path | Ao = | 649.01 | in^2 |
| | | | |
| Capacity | Asfy = | 560.68 | kip |

Demand

$$\frac{|M_u|}{\phi d_v} + \frac{0.5N_u}{\phi} + \cot(\theta)\sqrt{\left(\left|\frac{V_u}{\phi} - V_p\right| - 0.5V_s\right)^2 + \left(\frac{0.45p_hT_u}{2A_o\phi}\right)^2} = 319.52 \text{ kip}$$

**OK**

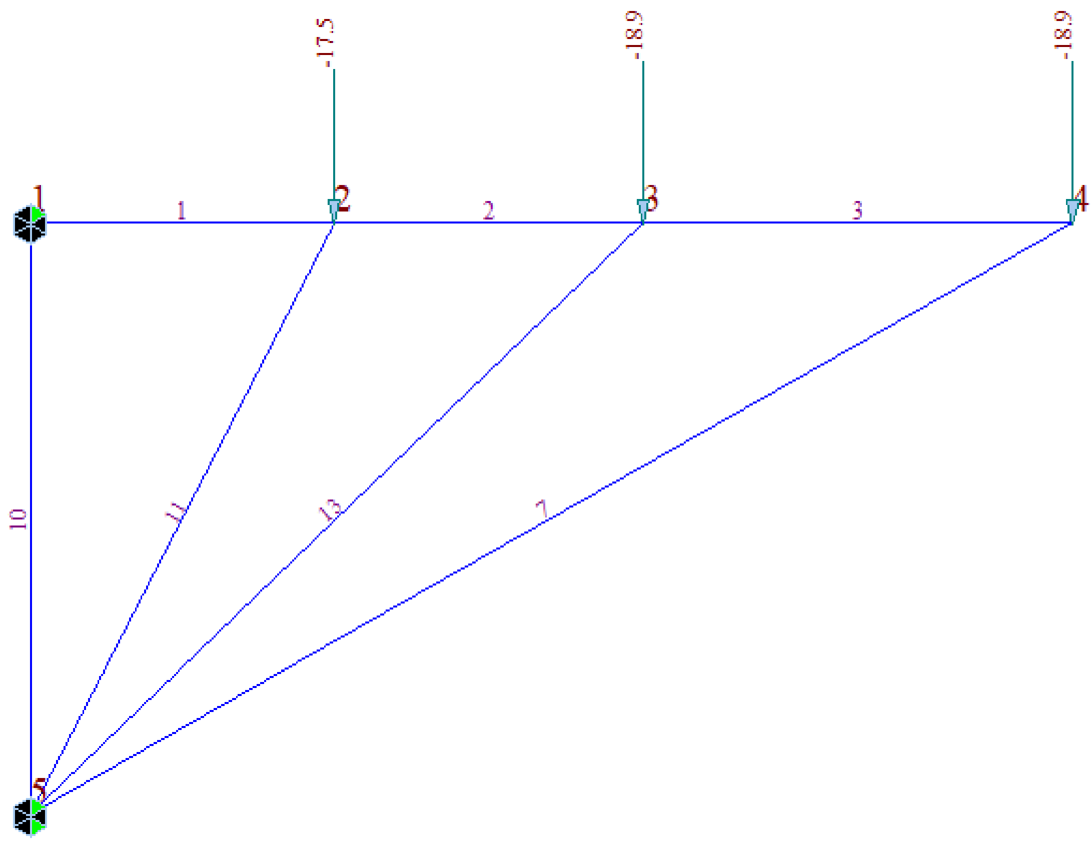
**ELEMENTS 1, 2, 3 (TOP TIES)**

## DESCRIPTION:

Check ties in the strut-and-tie model

## REINFORCEMENT

MODEL:




| | | | |
|---|---|---|---|
| | fy = | 60 ksi | |
| Resistance factor for tension in strut-and-tie models | Phi = | **0.90** | AASHTO 5.5.4.2 |
| Condition factor for poor condition | Phi.c = | **0.85** | MBE |

| Element | As (in^2) | Pn (kip) | Pr (kip) |
|---|---|---|---|
| 1 | 3.95 | 237.00 | 181.31 |
| 2 | 3.55 | 212.77 | 162.77 |
| 3 | 1.42 | 85.06 | 65.07 |

| PROJECT : *BEL-40-23.37* | | | |
|---|---|---|---|
| TASK : *Rating* | | PROJECT NO : *195987* | |
| SUBJECT : *Pier 5 Cap Rating* | | | |
| CALCULATED BY : *JCC* | DATE : *10/3/2023* | CHECKED BY : *ETB* | DATE : -- |

Michael Baker INTERNATIONAL

**ELEMENTS 1, 2, 3 (TOP TIES)**

**MIDAS FORCES**

| Elem | Load | Force-I (kip | Force-J (kips) |
|---|---|---|---|
| 1 | DC | 56.79589 | 56.79589 |
| 2 | DC | 53.72109 | 53.72109 |
| 3 | DC | 33.54721 | 33.54721 |
| 1 | DW | 2.098322 | 2.098322 |
| 2 | DW | 1.779292 | 1.779292 |
| 3 | DW | 1.120123 | 1.120123 |
| 1 | HL-93 | 66.13789 | 66.13789 |
| 2 | HL-93 | 56.50372 | 56.50372 |
| 3 | HL-93 | 35.57097 | 35.57097 |
| 1 | 2F1 | 27.20003 | 27.20003 |
| 2 | 2F1 | 16.74627 | 16.74627 |
| 3 | 2F1 | 10.54233 | 10.54233 |
| 1 | 3F1 | 40.58873 | 40.58873 |
| 2 | 3F1 | 27.43698 | 27.43698 |
| 3 | 3F1 | 17.27249 | 17.27249 |
| 1 | 4F1 | 45.61034 | 45.61034 |
| 2 | 4F1 | 31.71326 | 31.71326 |
| 3 | 4F1 | 19.96455 | 19.96455 |
| 1 | 5C1 | 38.62312 | 38.62312 |
| 2 | 5C1 | 25.59788 | 25.59788 |
| 3 | 5C1 | 16.11471 | 16.11471 |
| 1 | Type3 | 37.9904 | 37.9904 |
| 2 | Type3 | 26.55481 | 26.55481 |
| 3 | Type3 | 16.71713 | 16.71713 |
| 1 | Type3S2 | 31.23439 | 31.23439 |
| 2 | Type3S2 | 22.42805 | 22.42805 |
| 3 | Type3S2 | 14.1192 | 14.1192 |
| 1 | Type3-3 | 35.38232 | 35.38232 |
| 2 | Type3-3 | 23.32517 | 23.32517 |
| 3 | Type3-3 | 14.68397 | 14.68397 |
| 1 | SU4 | 45.07861 | 45.07861 |
| 2 | SU4 | 31.08527 | 31.08527 |
| 3 | SU4 | 19.56921 | 19.56921 |
| 1 | SU5 | 48.0421 | 48.0421 |
| 2 | SU5 | 33.67197 | 33.67197 |
| 3 | SU5 | 21.19762 | 21.19762 |
| 1 | SU6 | 53.16504 | 53.16504 |
| 2 | SU6 | 37.39503 | 37.39503 |
| 3 | SU6 | 23.54141 | 23.54141 |
| 1 | SU7 | 57.4426 | 57.4426 |
| 2 | SU7 | 41.20779 | 41.20779 |
| 3 | SU7 | 25.94167 | 25.94167 |
| 1 | EV2 | 46.71657 | 46.71657 |

| | | | |
|---|---|---|---|
| 2 | EV2 | 36.13906 | 36.13906 |
| 3 | EV2 | 22.75073 | 22.75073 |
| 1 | EV3 | 66.81899 | 66.81899 |
| 2 | EV3 | 45.97749 | 45.97749 |
| 3 | EV3 | 28.94436 | 28.94436 |
| 1 | RPL 60T | 56.16017 | 56.16017 |
| 2 | RPL 60T | 39.86211 | 39.86211 |
| 3 | RPL 60T | 25.09452 | 25.09452 |
| 1 | RPL 65T | 61.08205 | 61.08205 |
| 2 | RPL 65T | 41.25265 | 41.25265 |
| 3 | RPL 65T | 25.96991 | 25.96991 |

## RATINGS

Element 1
Factored resistance 181.305

| | Load | LF | Factored | RF |
|---|---|---|---|---|
| DC | 56.8 | 1.25 | 71.0 | |
| DW | 2.1 | 1.5 | 3.1 | |
| HL-93 INV | 66.1 | 1.75 | 115.7 | 0.93 |
| HL-93 OPR | 66.1 | 1.35 | 89.3 | 1.20 |
| 2F1 | 27.2 | 1.45 | 39.4 | 2.72 |
| 3F1 | 40.6 | 1.45 | 58.9 | 1.82 |
| 4F1 | 45.6 | 1.45 | 66.1 | 1.62 |
| 5C1 | 38.6 | 1.45 | 56.0 | 1.91 |
| Type3 | 38.0 | 1.45 | 55.1 | 1.95 |
| Type3S2 | 31.2 | 1.45 | 45.3 | 2.37 |
| Type3-3 | 35.4 | 1.45 | 51.3 | 2.09 |
| SU4 | 45.1 | 1.45 | 65.4 | 1.64 |
| SU5 | 48.0 | 1.45 | 69.7 | 1.54 |
| SU6 | 53.2 | 1.45 | 77.1 | 1.39 |
| SU7 | 57.4 | 1.45 | 83.3 | 1.29 |
| EV2 | 46.7 | 1.45 | 67.7 | 1.58 |
| EV3 | 66.8 | 1.1 | 73.5 | 1.46 |
| RPL 60T | 56.2 | 1.4 | 78.6 | 1.36 |
| RPL 65T | 61.1 | 1.4 | 85.5 | 1.25 |

Element 2
Factored resistance 162.77 kip

| | Load | LF | Factored | RF |
|---|---|---|---|---|
| DC | 53.7 | 1.25 | 67.2 | |
| DW | 1.8 | 1.5 | 2.7 | |
| HL-93 INV | 56.5 | 1.75 | 98.9 | 0.94 |
| HL-93 OPR | 56.5 | 1.35 | 76.3 | 1.22 |
| 2F1 | 16.7 | 1.45 | 24.3 | 3.83 |
| 3F1 | 27.4 | 1.45 | 39.8 | 2.34 |
| 4F1 | 31.7 | 1.45 | 46.0 | 2.02 |
| 5C1 | 25.6 | 1.45 | 37.1 | 2.50 |
| Type3 | 26.6 | 1.45 | 38.5 | 2.41 |
| Type3S2 | 22.4 | 1.45 | 32.5 | 2.86 |
| Type3-3 | 23.3 | 1.45 | 33.8 | 2.75 |
| SU4 | 31.1 | 1.45 | 45.1 | 2.06 |
| SU5 | 33.7 | 1.45 | 48.8 | 1.90 |
| SU6 | 37.4 | 1.45 | 54.2 | 1.71 |
| SU7 | 41.2 | 1.45 | 59.8 | 1.56 |
| EV2 | 36.1 | 1.45 | 52.4 | 1.77 |
| EV3 | 46.0 | 1.1 | 50.6 | 1.84 |
| RPL 60T | 39.9 | 1.4 | 55.8 | 1.67 |

| | | | | |
|---|---|---|---|---|
| RPL 65T | 41.3 | 1.4 | 57.8 | 1.61 |

| | | | | |
|---|---|---|---|---|
| Element | | 3 | | |
| Factored resistance | | 65.07 | kip | |
| | Load | LF | Factored | RF |
| DC | 33.5 | 1.25 | 41.9 | |
| DW | 1.1 | 1.5 | 1.7 | |
| HL-93 INV | 35.6 | 1.75 | 62.2 | 2.21 |
| HL-93 OPR | 35.6 | 1.35 | 48.0 | 2.87 |
| 2F1 | 10.5 | 1.45 | 15.3 | 9.01 |
| 3F1 | 17.3 | 1.45 | 25.0 | 5.50 |
| 4F1 | 20.0 | 1.45 | 28.9 | 4.76 |
| 5C1 | 16.1 | 1.45 | 23.4 | 5.89 |
| Type3 | 16.7 | 1.45 | 24.2 | 5.68 |
| Type3S2 | 14.1 | 1.45 | 20.5 | 6.73 |
| Type3-3 | 14.7 | 1.45 | 21.3 | 6.47 |
| SU4 | 19.6 | 1.45 | 28.4 | 4.85 |
| SU5 | 21.2 | 1.45 | 30.7 | 4.48 |
| SU6 | 23.5 | 1.45 | 34.1 | 4.03 |
| SU7 | 25.9 | 1.45 | 37.6 | 3.66 |
| EV2 | 22.8 | 1.45 | 33.0 | 4.17 |
| EV3 | 28.9 | 1.1 | 31.8 | 4.32 |
| RPL 60T | 25.1 | 1.4 | 35.1 | 3.92 |
| RPL 65T | 26.0 | 1.4 | 36.4 | 3.79 |

| | |
|---|---|
| Governing | |
| HL-93 INV | 0.93 |
| HL-93 OPR | 1.20 |
| 2F1 | 2.72 |
| 3F1 | 1.82 |
| 4F1 | 1.62 |
| 5C1 | 1.91 |
| Type3 | 1.95 |
| Type3S2 | 2.37 |
| Type3-3 | 2.09 |
| SU4 | 1.64 |
| SU5 | 1.54 |
| SU6 | 1.39 |
| SU7 | 1.29 |
| EV2 | 1.58 |
| EV3 | 1.46 |
| RPL 60T | 1.36 |
| RPL 65T | 1.25 |

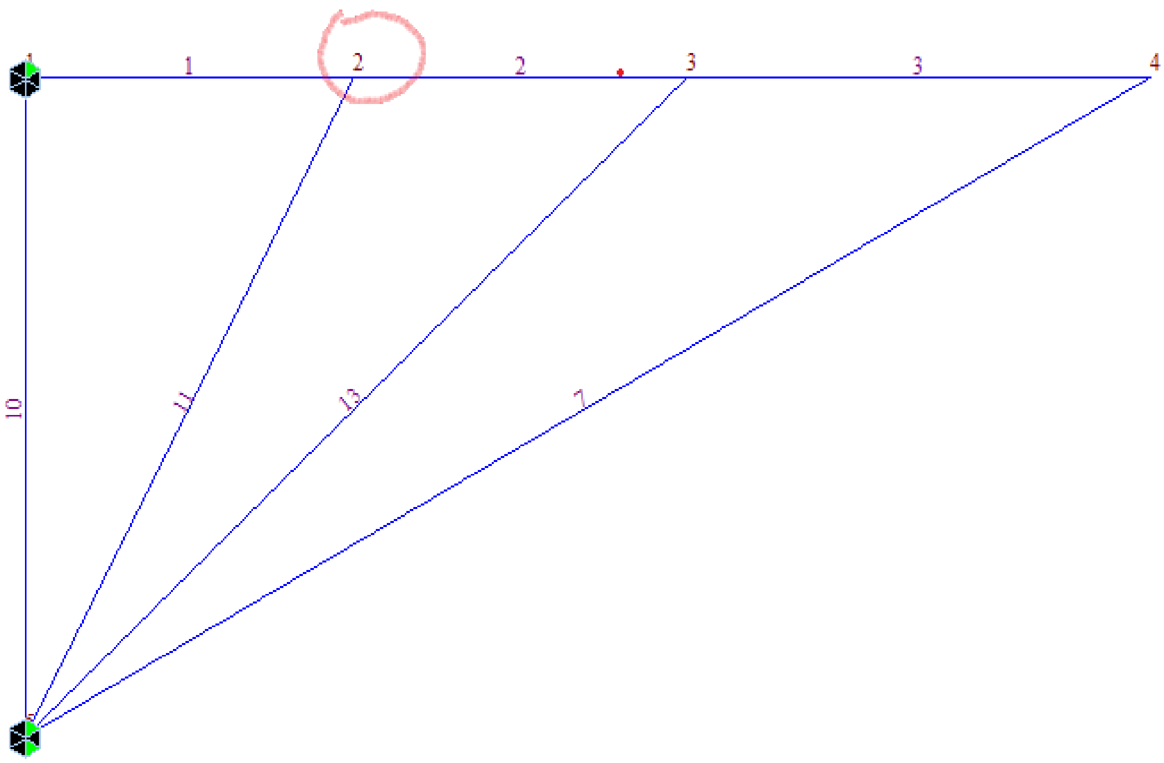
| | | | |
|---|---|---|---|
| PROJECT : *BEL-40-23.37* | | | Michael Baker INTERNATIONAL |
| TASK : *Rating* | | PROJECT NO : *195987* | |
| SUBJECT : *Pier 5 Cap Rating* | | | |
| CALCULATED BY : *JCC* | DATE : *10/3/2023* | CHECKED BY : *ETB* | DATE : -- |

**NODE 2 (CTT)**

## DESCRIPTION:

CTT check at Node 2.
Ties checked separately.

## GEOMETRY



**NODE 2 (CTT)**

## CAPACITY

| | | | | |
|---|---|---|---|---|
| **RESISTANCE FACTORS** | | | | |
| Compression in strut-and-tie models | Phi.c1 = | **0.70** | | AASHTO 5.5.4.2 |
| Poor condition factor | Phi.c2 = | **0.85** | | MBE 6A.4.2.3-1 |
| **MATERIALS** | | | | |
| Compressive strength of concrete | f'c = | 4.50 | ksi | |
| Concrete efficiency factor | v = | 0.45 | | |
| Confinement modification factor | m = | 1.00 | | |
| Node face compressive stress | fcu = mvf'c = | 2.03 | ksi | |
| **GEOMETRY** | | | | |
| Bearing face length | lb = | 9 | in | |
| Back face height | ha = 2*Reinforcement Centroid | 9.95 | in | |
| Angle to horizontal tie | θ = | **61.45** | deg | |
| Strut-Node Interface | s = ha*cos(θ) + lb*sin(θ) = | 12.66 | in | |
| Concrete width | bw = | 27.00 | in | |
| **STRUT CAPACITY** | | | | |
| Node face concrete area | Acn = s*bw = | 341.84 | in^2 | |
| Nominal Resistance | Pn = fcu*Acn = | 692.22 | kip | |
| Factored resistance | Pr = Phi.c1*Phi.c2*Pn = | 411.87 | kip | |
| **BEARING CAPACITY** | | | | |
| Area | A1 = | 45 | in^2 | |
| Factored resistance | Pr = Phi.c1*Phi.c2*A1*fcu = | 54.22 | kip | |

| PROJECT : *BEL-40-23.37* | | | |
|---|---|---|---|
| TASK : *Rating* | | PROJECT NO : *195987* | |
| SUBJECT : *Pier 5 Cap Rating* | | | |
| CALCULATED BY : *JCC* | DATE : *10/3/2023* | CHECKED BY : *ETB* | DATE : -- |

Michael Baker INTERNATIONAL

**NODE 2 (CTT)**

## LOADS

Vertical Forces:

| Node | Load Case | FX (kips) | FY (kips) | FZ (kips) | MX (in*kips | MY (in*kips | MZ (in*kips | Group |
|---|---|---|---|---|---|---|---|---|
| 2 | DC | 0 | 0 | -5.59 | 0 | 0 | 0 | Default |
| 2 | DW | 0 | 0 | -0.58 | 0 | 0 | 0 | Default |
| 2 | HL-93 | 0 | 0 | -17.52 | 0 | 0 | 0 | Default |
| 2 | 2F1 | 0 | 0 | -19 | 0 | 0 | 0 | Default |
| 2 | 3F1 | 0 | 0 | -23.91 | 0 | 0 | 0 | Default |
| 2 | 4F1 | 0 | 0 | -25.27 | 0 | 0 | 0 | Default |
| 2 | 5C1 | 0 | 0 | -23.68 | 0 | 0 | 0 | Default |
| 2 | Type3 | 0 | 0 | -20.79 | 0 | 0 | 0 | Default |
| 2 | Type3S2 | 0 | 0 | -16.01 | 0 | 0 | 0 | Default |
| 2 | Type3-3 | 0 | 0 | -21.92 | 0 | 0 | 0 | Default |
| 2 | SU4 | 0 | 0 | -25.44 | 0 | 0 | 0 | Default |
| 2 | SU5 | 0 | 0 | -26.13 | 0 | 0 | 0 | Default |
| 2 | SU6 | 0 | 0 | -28.67 | 0 | 0 | 0 | Default |
| 2 | SU7 | 0 | 0 | -29.52 | 0 | 0 | 0 | Default |
| 2 | EV2 | 0 | 0 | -19.23 | 0 | 0 | 0 | Default |
| 2 | EV3 | 0 | 0 | -37.89 | 0 | 0 | 0 | Default |
| 2 | RPL 60T | 0 | 0 | -29.63 | 0 | 0 | 0 | Default |
| 2 | RPL 65T | 0 | 0 | -36.05 | 0 | 0 | 0 | Default |

Element outputs

| Elem | Load | Force-I (kip | Force-J (kips) |
|---|---|---|---|
| 11 | DC | -6.379848 | -6.379848 |
| 11 | DW | -0.661952 | -0.661952 |
| 11 | HL-93 | -19.98981 | -19.98981 |
| 11 | 2F1 | -21.69034 | -21.69034 |
| 11 | 3F1 | -27.2884 | -27.2884 |
| 11 | 4F1 | -28.83486 | -28.83486 |
| 11 | 5C1 | -27.0259 | -27.0259 |
| 11 | Type3 | -23.72756 | -23.72756 |
| 11 | Type3S2 | -18.27216 | -18.27216 |
| 11 | Type3-3 | -25.01722 | -25.01722 |
| 11 | SU4 | -29.03459 | -29.03459 |
| 11 | SU5 | -29.81637 | -29.81637 |
| 11 | SU6 | -32.72097 | -32.72097 |
| 11 | SU7 | -33.68537 | -33.68537 |
| 11 | EV2 | -21.94713 | -21.94713 |
| 11 | EV3 | -43.24373 | -43.24373 |
| 11 | RPL 60T | -33.81662 | -33.81662 |
| 11 | RPL 65T | -41.14374 | -41.14374 |

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NODE 2 (CTT)

## STRUT CHECK

| | | Case | Load | L.F. | Factored | RF |
|---|---|---|---|---|---|---|
| Element | 11 | | | | | |
| Factored Resistance | | DC | 6.38 | 1.25 | 7.97 | |
| Pr = | 411.87 kip | DW | 0.66 | 1.5 | 0.99 | |
| | | HL-93 INV | 19.99 | 1.75 | 34.98 | 11.52 |
| | | HL-93 OPR | 19.99 | 1.35 | 26.99 | 14.93 |
| | | 2F1 | 21.69 | 1.45 | 31.45 | 12.81 |
| | | 3F1 | 27.29 | 1.45 | 39.57 | 10.18 |
| | | 4F1 | 28.83 | 1.45 | 41.81 | 9.64 |
| | | 5C1 | 27.03 | 1.45 | 39.19 | 10.28 |
| | | Type3 | 23.73 | 1.45 | 34.40 | 11.71 |
| | | Type3S2 | 18.27 | 1.45 | 26.49 | 15.21 |
| | | Type3-3 | 25.02 | 1.45 | 36.27 | 11.11 |
| | | SU4 | 29.03 | 1.45 | 42.10 | 9.57 |
| | | SU5 | 29.82 | 1.45 | 43.23 | 9.32 |
| | | SU6 | 32.72 | 1.45 | 47.45 | 8.49 |
| | | SU7 | 33.69 | 1.45 | 48.84 | 8.25 |
| | | EV2 | 21.95 | 1.45 | 31.82 | 12.66 |
| | | EV3 | 43.24 | 1.1 | 47.57 | 8.47 |
| | | RPL 60T | 33.82 | 1.4 | 47.34 | 8.51 |
| | | RPL 65T | 41.14 | 1.4 | 57.60 | 6.99 |

## BEARING

| | | Case | Load | L.F. | Factored | RF |
|---|---|---|---|---|---|---|
| Node | 2 | | | | | |
| Factored Resistance | | DC | 2.80 | 1.25 | 3.49 | |
| Pr = | 54.22 kip | DW | 0.29 | 1.5 | 0.44 | |
| | | HL-93 INV | 8.76 | 1.75 | 15.33 | 3.28 |
| -Use half of applied force to check | | HL-93 OPR | 8.76 | 1.35 | 11.83 | 4.25 |
| only under one beam end. | | 2F1 | 9.50 | 1.45 | 13.78 | 3.65 |
| | | 3F1 | 11.96 | 1.45 | 17.33 | 2.90 |
| | | 4F1 | 12.64 | 1.45 | 18.32 | 2.75 |
| | | 5C1 | 11.84 | 1.45 | 17.17 | 2.93 |
| | | Type3 | 10.40 | 1.45 | 15.07 | 3.34 |
| | | Type3S2 | 8.01 | 1.45 | 11.61 | 4.33 |
| | | Type3-3 | 10.96 | 1.45 | 15.89 | 3.16 |
| | | SU4 | 12.72 | 1.45 | 18.44 | 2.73 |
| | | SU5 | 13.07 | 1.45 | 18.94 | 2.65 |
| | | SU6 | 14.34 | 1.45 | 20.79 | 2.42 |
| | | SU7 | 14.76 | 1.45 | 21.40 | 2.35 |
| | | EV2 | 9.62 | 1.45 | 13.94 | 3.61 |
| | | EV3 | 18.95 | 1.1 | 20.84 | 2.41 |
| | | RPL 60T | 14.82 | 1.4 | 20.74 | 2.42 |
| | | RPL 65T | 18.03 | 1.4 | 25.24 | 1.99 |

**NODE 2 (CTT)**

## GOVERNING RATING FACTOR

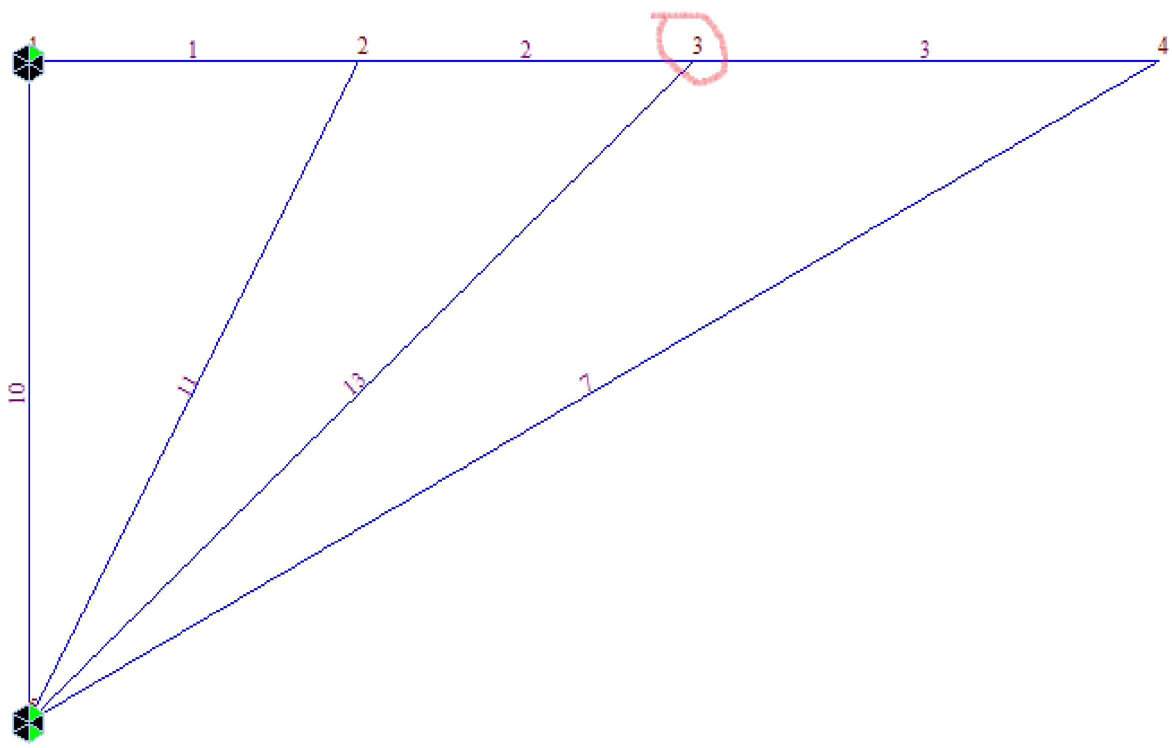
| | |
|---|---|
| HL-93 INV | 3.28 |
| HL-93 OPR | 4.25 |
| 2F1 | 3.65 |
| 3F1 | 2.90 |
| 4F1 | 2.75 |
| 5C1 | 2.93 |
| Type3 | 3.34 |
| Type3S2 | 4.33 |
| Type3-3 | 3.16 |
| SU4 | 2.73 |
| SU5 | 2.65 |
| SU6 | 2.42 |
| SU7 | 2.35 |
| EV2 | 3.61 |
| EV3 | 2.41 |
| RPL 60T | 2.42 |
| RPL 65T | 1.99 |

NODE 3 (CTT)

## DESCRIPTION:

CTT check at Node 3.
Ties checked separately.

## GEOMETRY



**NODE 3 (CTT)**

## CAPACITY

**RESISTANCE FACTORS**

| | | | | |
|---|---|---|---|---|
| Compression in strut-and-tie models | Phi.c1 = | **0.70** | | AASHTO 5.5.4.2 |
| Poor condition factor | Phi.c2 = | **0.85** | | MBE 6A.4.2.3-1 |

**MATERIALS**

| | | | |
|---|---|---|---|
| Compressive strength of concrete | f'c = | 4.50 | ksi |
| Concrete efficiency factor | v = | 0.45 | |
| Confinement modification factor | m = | 1.00 | |
| Node face compressive stress | fcu = mvf'c = | 2.03 | ksi |

**GEOMETRY**

| | | | |
|---|---|---|---|
| Bearing face length | lb = | **9.00** | in |
| Back face height | ha = 2*Reinforcement Centroid | 9.95 | in |
| Angle to horizontal tie | θ = | **42.39** | deg |
| Strut-Node Interface | s = ha*cos(θ) + lb*sin(θ) = | 13.42 | in |
| Concrete width | bw = | 27.00 | in |

**STRUT CAPACITY**

| | | | |
|---|---|---|---|
| Node face concrete area | Acn = s*bw = | 362.24 | in^2 |
| Nominal Resistance | Pn = fcu*Acn = | 733.54 | kip |
| Factored resistance | Pr = Phi.c1*Phi.c2*Pn = | 436.46 | kip |

**BEARING CAPACITY**

| | | | |
|---|---|---|---|
| Area | A1 = | 45 | in^2 |
| Factored resistance | Pr = Phi.c1*Phi.c2*A1*fcu = | 54.22 | kip |

NODE 3 (CTT)

## LOADS

Vertical Forces:

| Node | Load Case | FX (kips) | FY (kips) | FZ (kips) | MX (in*kips | MY (in*kips | MZ (in*kips | Group |
|---|---|---|---|---|---|---|---|---|
| 3 | DC | 0 | 0 | -18.21 | 0 | 0 | 0 | Default |
| 3 | DW | 0 | 0 | -0.59 | 0 | 0 | 0 | Default |
| 3 | HL-93 | 0 | 0 | -18.89 | 0 | 0 | 0 | Default |
| 3 | 2F1 | 0 | 0 | -5.6 | 0 | 0 | 0 | Default |
| 3 | 3F1 | 0 | 0 | -9.18 | 0 | 0 | 0 | Default |
| 3 | 4F1 | 0 | 0 | -10.61 | 0 | 0 | 0 | Default |
| 3 | 5C1 | 0 | 0 | -8.56 | 0 | 0 | 0 | Default |
| 3 | Type3 | 0 | 0 | -8.88 | 0 | 0 | 0 | Default |
| 3 | Type3S2 | 0 | 0 | -7.5 | 0 | 0 | 0 | Default |
| 3 | Type3-3 | 0 | 0 | -7.8 | 0 | 0 | 0 | Default |
| 3 | SU4 | 0 | 0 | -10.39 | 0 | 0 | 0 | Default |
| 3 | SU5 | 0 | 0 | -11.26 | 0 | 0 | 0 | Default |
| 3 | SU6 | 0 | 0 | -12.51 | 0 | 0 | 0 | Default |
| 3 | SU7 | 0 | 0 | -13.78 | 0 | 0 | 0 | Default |
| 3 | EV2 | 0 | 0 | -12.09 | 0 | 0 | 0 | Default |
| 3 | EV3 | 0 | 0 | -15.38 | 0 | 0 | 0 | Default |
| 3 | RPL 60T | 0 | 0 | -13.33 | 0 | 0 | 0 | Default |
| 3 | RPL 65T | 0 | 0 | -13.79 | 0 | 0 | 0 | Default |

Element outputs

| Elem | Load | Force-I (kip | Force-J (kips) |
|---|---|---|---|
| 13 | DC | -27.17701 | -27.17701 |
| 13 | DW | -0.887991 | -0.887991 |
| 13 | HL-93 | -28.19931 | -28.19931 |
| 13 | 2F1 | -8.357563 | -8.357563 |
| 13 | 3F1 | -13.69297 | -13.69297 |
| 13 | 4F1 | -15.82714 | -15.82714 |
| 13 | 5C1 | -12.77513 | -12.77513 |
| 13 | Type3 | -13.25271 | -13.25271 |
| 13 | Type3S2 | -11.19317 | -11.19317 |
| 13 | Type3-3 | -11.64089 | -11.64089 |
| 13 | SU4 | -15.51373 | -15.51373 |
| 13 | SU5 | -16.80467 | -16.80467 |
| 13 | SU6 | -18.66274 | -18.66274 |
| 13 | SU7 | -20.56558 | -20.56558 |
| 13 | EV2 | -18.03592 | -18.03592 |
| 13 | EV3 | -22.94599 | -22.94599 |
| 13 | RPL 60T | -19.89399 | -19.89399 |
| 13 | RPL 65T | -20.58796 | -20.58796 |

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NODE 3 (CTT)

## STRUT CHECK

Element 13
Factored Resistance
Pr = 436.46 kip

| Case | Load | L.F. | Factored | RF |
|---|---|---|---|---|
| DC | 27.18 | 1.25 | 33.97 | |
| DW | 0.89 | 1.5 | 1.33 | |
| HL-93 INV | 28.20 | 1.75 | 49.35 | 8.13 |
| HL-93 OPR | 28.20 | 1.35 | 38.07 | 10.54 |
| 2F1 | 8.36 | 1.45 | 12.12 | 33.10 |
| 3F1 | 13.69 | 1.45 | 19.85 | 20.20 |
| 4F1 | 15.83 | 1.45 | 22.95 | 17.48 |
| 5C1 | 12.78 | 1.45 | 18.52 | 21.66 |
| Type3 | 13.25 | 1.45 | 19.22 | 20.88 |
| Type3S2 | 11.19 | 1.45 | 16.23 | 24.72 |
| Type3-3 | 11.64 | 1.45 | 16.88 | 23.77 |
| SU4 | 15.51 | 1.45 | 22.49 | 17.83 |
| SU5 | 16.80 | 1.45 | 24.37 | 16.46 |
| SU6 | 18.66 | 1.45 | 27.06 | 14.82 |
| SU7 | 20.57 | 1.45 | 29.82 | 13.45 |
| EV2 | 18.04 | 1.45 | 26.15 | 15.34 |
| EV3 | 22.95 | 1.1 | 25.24 | 15.89 |
| RPL 60T | 19.89 | 1.4 | 27.85 | 14.40 |
| RPL 65T | 20.59 | 1.4 | 28.82 | 13.92 |

## BEARING

Node 3
Factored Resistance
Pr = 54.22 kip

-Use half of applied force to check only under one beam end.

| Case | Load | L.F. | Factored | RF |
|---|---|---|---|---|
| DC | 9.11 | 1.25 | 11.38 | |
| DW | 0.30 | 1.5 | 0.44 | |
| HL-93 INV | 9.45 | 1.75 | 16.53 | 2.56 |
| HL-93 OPR | 9.45 | 1.35 | 12.75 | 3.32 |
| 2F1 | 2.80 | 1.45 | 4.06 | 10.44 |
| 3F1 | 4.59 | 1.45 | 6.66 | 6.37 |
| 4F1 | 5.31 | 1.45 | 7.69 | 5.51 |
| 5C1 | 4.28 | 1.45 | 6.21 | 6.83 |
| Type3 | 4.44 | 1.45 | 6.44 | 6.59 |
| Type3S2 | 3.75 | 1.45 | 5.44 | 7.80 |
| Type3-3 | 3.90 | 1.45 | 5.66 | 7.50 |
| SU4 | 5.20 | 1.45 | 7.53 | 5.63 |
| SU5 | 5.63 | 1.45 | 8.16 | 5.19 |
| SU6 | 6.26 | 1.45 | 9.07 | 4.67 |
| SU7 | 6.89 | 1.45 | 9.99 | 4.24 |
| EV2 | 6.05 | 1.45 | 8.77 | 4.84 |
| EV3 | 7.69 | 1.1 | 8.46 | 5.01 |
| RPL 60T | 6.67 | 1.4 | 9.33 | 4.54 |
| RPL 65T | 6.90 | 1.4 | 9.65 | 4.39 |

NODE 3 (CTT)

## GOVERNING RATING FACTOR

| | |
|---|---|
| HL-93 INV | 2.56 |
| HL-93 OPR | 3.32 |
| 2F1 | 10.44 |
| 3F1 | 6.37 |
| 4F1 | 5.51 |
| 5C1 | 6.83 |
| Type3 | 6.59 |
| Type3S2 | 7.80 |
| Type3-3 | 7.50 |
| SU4 | 5.63 |
| SU5 | 5.19 |
| SU6 | 4.67 |
| SU7 | 4.24 |
| EV2 | 4.84 |
| EV3 | 5.01 |
| RPL 60T | 4.54 |
| RPL 65T | 4.39 |

| | | | |
|---|---|---|---|
| PROJECT : *BEL-40-23.37* | | | |
| TASK : *Rating* | | PROJECT NO : *195987* | |
| SUBJECT : *Pier 5 Cap Rating* | | | |
| CALCULATED BY : *JCC* | DATE : *10/3/2023* | CHECKED BY : *ETB* | DATE : *--* |

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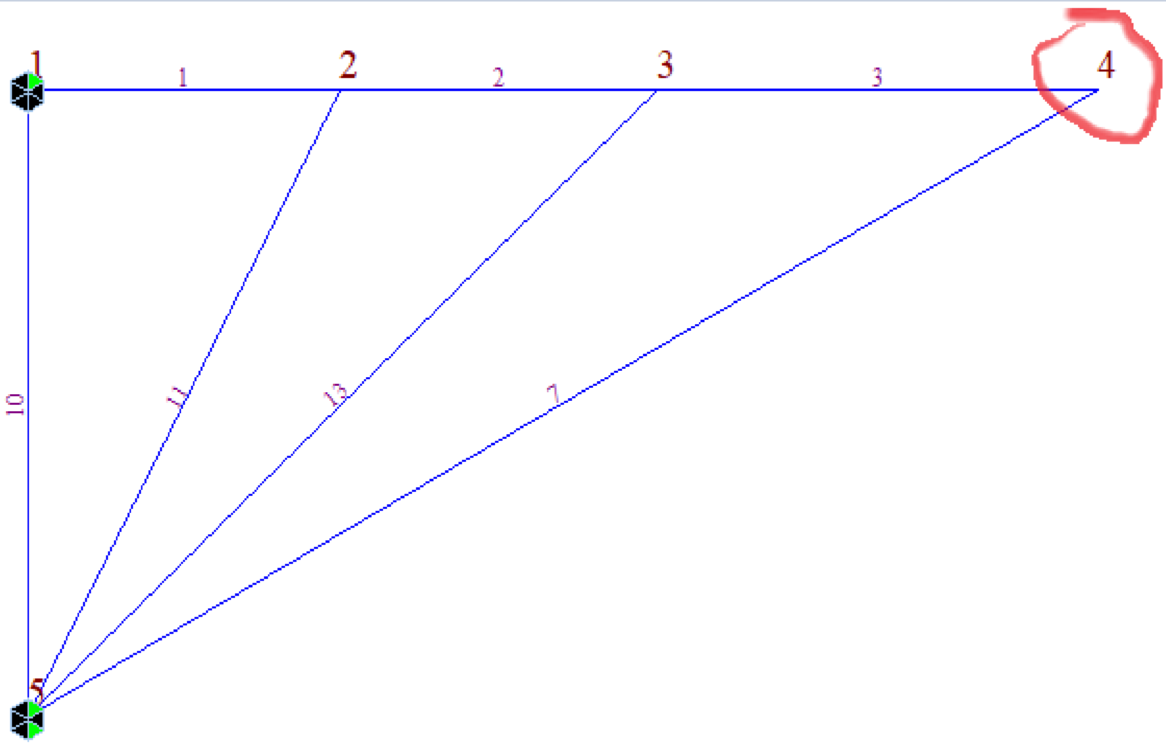
**NODE 4 (CCT)**

## DESCRIPTION:

CCT Node at Node 4
Ties checked separately.

## GEOMETRY



**NODE 4 (CCT)**

## CAPACITY

**RESISTANCE FACTORS**

| | | | | |
|---|---|---|---|---|
| Compression in strut-and-tie models | Phi.c1 = | **0.70** | | AASHTO 5.5.4.2 |
| Poor condition factor | Phi.c2 = | **0.85** | | MBE 6A.4.2.3-1 |

**MATERIALS**

| | | | |
|---|---|---|---|
| Compressive strength of concrete | f'c = | 4.50 | ksi |
| Concrete efficiency factor | v = | 0.45 | |
| Confinement modification factor | m = | 1.00 | |
| Node face compressive stress | fcu = mvf'c = | 2.03 | ksi |

**GEOMETRY**

| | | | |
|---|---|---|---|
| Bearing face length | lb = | 9 | in |
| Back face height | ha = 2*Reinforcement Centroid | 9.95 | in |
| Angle to horizontal tie | θ = | **28.24** | deg |
| Strut-Node Interface | s = ha*cos(θ) + lb*sin(θ) = | 13.02 | in |
| Concrete width | bw = | 27.00 | in |

**STRUT CAPACITY**

| | | | |
|---|---|---|---|
| Node face concrete area | Acn = s*bw = | 351.65 | in^2 |
| Nominal Resistance | Pn = fcu*Acn = | 712.10 | kip |
| Factored resistance | Pr = Phi.c1*Phi.c2*Pn = | 423.70 | kip |

**BEARING CAPACITY**

| | | | |
|---|---|---|---|
| Area | A1 = | 45 | in^2 |
| Factored resistance | Pr = Phi.c1*Phi.c2*A1*fcu = | 54.22 | kip |

NODE 4 (CCT)

## LOADS

Vertical Forces:

| Node | Load Case | FX (kips) | FY (kips) | FZ (kips) | MX (in*kips | MY (in*kips | MZ (in*kips | Group |
|---|---|---|---|---|---|---|---|---|
| 4 | DC | 0 | 0 | -17.82 | 0 | 0 | 0 | Default |
| 4 | DW | 0 | 0 | -0.59 | 0 | 0 | 0 | Default |
| 4 | HL-93 | 0 | 0 | -18.89 | 0 | 0 | 0 | Default |
| 4 | 2F1 | 0 | 0 | -5.6 | 0 | 0 | 0 | Default |
| 4 | 3F1 | 0 | 0 | -9.18 | 0 | 0 | 0 | Default |
| 4 | 4F1 | 0 | 0 | -10.61 | 0 | 0 | 0 | Default |
| 4 | 5C1 | 0 | 0 | -8.56 | 0 | 0 | 0 | Default |
| 4 | Type3 | 0 | 0 | -8.88 | 0 | 0 | 0 | Default |
| 4 | Type3S2 | 0 | 0 | -7.5 | 0 | 0 | 0 | Default |
| 4 | Type3-3 | 0 | 0 | -7.8 | 0 | 0 | 0 | Default |
| 4 | SU4 | 0 | 0 | -10.39 | 0 | 0 | 0 | Default |
| 4 | SU5 | 0 | 0 | -11.26 | 0 | 0 | 0 | Default |
| 4 | SU6 | 0 | 0 | -12.51 | 0 | 0 | 0 | Default |
| 4 | SU7 | 0 | 0 | -13.78 | 0 | 0 | 0 | Default |
| 4 | EV2 | 0 | 0 | -12.09 | 0 | 0 | 0 | Default |
| 4 | EV3 | 0 | 0 | -15.38 | 0 | 0 | 0 | Default |
| 4 | RPL 60T | 0 | 0 | -13.33 | 0 | 0 | 0 | Default |
| 4 | RPL 65T | 0 | 0 | -13.79 | 0 | 0 | 0 | Default |

| PROJECT : BEL-40-23.37 | | | Michael Baker INTERNATIONAL |
|---|---|---|---|
| TASK : Rating | | PROJECT NO : 195987 | |
| SUBJECT : Pier 5 Cap Rating | | | |
| CALCULATED BY : JCC | DATE : 10/3/2023 | CHECKED BY : ETB | DATE : -- |

**NODE 4 (CCT)**

Element outputs

| Elem | Load | Force-I (kip | Force-J (kips) |
|---|---|---|---|
| 3 | DC | 33.54721 | 33.54721 |
| 7 | DC | -37.98642 | -37.98642 |
| 3 | DW | 1.120123 | 1.120123 |
| 7 | DW | -1.268346 | -1.268346 |
| 3 | HL-93 | 35.57097 | 35.57097 |
| 7 | HL-93 | -40.27797 | -40.27797 |
| 3 | 2F1 | 10.54233 | 10.54233 |
| 7 | 2F1 | -11.93737 | -11.93737 |
| 3 | 3F1 | 17.27249 | 17.27249 |
| 7 | 3F1 | -19.5581 | -19.5581 |
| 3 | 4F1 | 19.96455 | 19.96455 |
| 7 | 4F1 | -22.6064 | -22.6064 |
| 3 | 5C1 | 16.11471 | 16.11471 |
| 7 | 5C1 | -18.24712 | -18.24712 |
| 3 | Type3 | 16.71713 | 16.71713 |
| 7 | Type3 | -18.92926 | -18.92926 |
| 3 | Type3S2 | 14.1192 | 14.1192 |
| 7 | Type3S2 | -15.98755 | -15.98755 |
| 3 | Type3-3 | 14.68397 | 14.68397 |
| 7 | Type3-3 | -16.62705 | -16.62705 |
| 3 | SU4 | 19.56921 | 19.56921 |
| 7 | SU4 | -22.15874 | -22.15874 |
| 3 | SU5 | 21.19762 | 21.19762 |
| 7 | SU5 | -24.00264 | -24.00264 |
| 3 | SU6 | 23.54141 | 23.54141 |
| 7 | SU6 | -26.65657 | -26.65657 |
| 3 | SU7 | 25.94167 | 25.94167 |
| 7 | SU7 | -29.37446 | -29.37446 |
| 3 | EV2 | 22.75073 | 22.75073 |
| 7 | EV2 | -25.76127 | -25.76127 |
| 3 | EV3 | 28.94436 | 28.94436 |
| 7 | EV3 | -32.77448 | -32.77448 |
| 3 | RPL 60T | 25.09452 | 25.09452 |
| 7 | RPL 60T | -28.4152 | -28.4152 |
| 3 | RPL 65T | 25.96991 | 25.96991 |
| 7 | RPL 65T | -29.40643 | -29.40643 |

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NODE 4 (CCT)

## STRUT CHECK

| | | Case | Load | L.F. | Factored | RF |
|---|---|---|---|---|---|---|
| Element | 7 | | | | | |
| Factored Resistance | | DC | 37.99 | 1.25 | 47.48 | |
| Pr = | 423.70 kip | DW | 1.27 | 1.5 | 1.90 | |
| | | HL-93 INV | 40.28 | 1.75 | 70.49 | 5.31 |
| | | HL-93 OPR | 40.28 | 1.35 | 54.38 | 6.88 |
| | | 2F1 | 11.94 | 1.45 | 17.31 | 21.63 |
| | | 3F1 | 19.56 | 1.45 | 28.36 | 13.20 |
| | | 4F1 | 22.61 | 1.45 | 32.78 | 11.42 |
| | | 5C1 | 18.25 | 1.45 | 26.46 | 14.15 |
| | | Type3 | 18.93 | 1.45 | 27.45 | 13.64 |
| | | Type3S2 | 15.99 | 1.45 | 23.18 | 16.15 |
| | | Type3-3 | 16.63 | 1.45 | 24.11 | 15.53 |
| | | SU4 | 22.16 | 1.45 | 32.13 | 11.65 |
| | | SU5 | 24.00 | 1.45 | 34.80 | 10.75 |
| | | SU6 | 26.66 | 1.45 | 38.65 | 9.68 |
| | | SU7 | 29.37 | 1.45 | 42.59 | 8.79 |
| | | EV2 | 25.76 | 1.45 | 37.35 | 10.02 |
| | | EV3 | 32.77 | 1.1 | 36.05 | 10.38 |
| | | RPL 60T | 28.42 | 1.4 | 39.78 | 9.41 |
| | | RPL 65T | 29.41 | 1.4 | 41.17 | 9.09 |

## BEARING

| | | Case | Load | L.F. | Factored | RF |
|---|---|---|---|---|---|---|
| Node | 4 | | | | | |
| Factored Resistance | | DC | 8.91 | 1.25 | 11.14 | |
| Pr = | 54.22 kip | DW | 0.30 | 1.5 | 0.44 | |
| | | HL-93 INV | 9.45 | 1.75 | 16.53 | 2.58 |
| -Use half of applied force to check | | HL-93 OPR | 9.45 | 1.35 | 12.75 | 3.34 |
| only under one beam end. | | 2F1 | 2.80 | 1.45 | 4.06 | 10.50 |
| | | 3F1 | 4.59 | 1.45 | 6.66 | 6.41 |
| | | 4F1 | 5.31 | 1.45 | 7.69 | 5.54 |
| | | 5C1 | 4.28 | 1.45 | 6.21 | 6.87 |
| | | Type3 | 4.44 | 1.45 | 6.44 | 6.62 |
| | | Type3S2 | 3.75 | 1.45 | 5.44 | 7.84 |
| | | Type3-3 | 3.90 | 1.45 | 5.66 | 7.54 |
| | | SU4 | 5.20 | 1.45 | 7.53 | 5.66 |
| | | SU5 | 5.63 | 1.45 | 8.16 | 5.22 |
| | | SU6 | 6.26 | 1.45 | 9.07 | 4.70 |
| | | SU7 | 6.89 | 1.45 | 9.99 | 4.27 |
| | | EV2 | 6.05 | 1.45 | 8.77 | 4.86 |
| | | EV3 | 7.69 | 1.1 | 8.46 | 5.04 |
| | | RPL 60T | 6.67 | 1.4 | 9.33 | 4.57 |
| | | RPL 65T | 6.90 | 1.4 | 9.65 | 4.42 |

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NODE 4 (CCT)

## GOVERNING RATING FACTOR

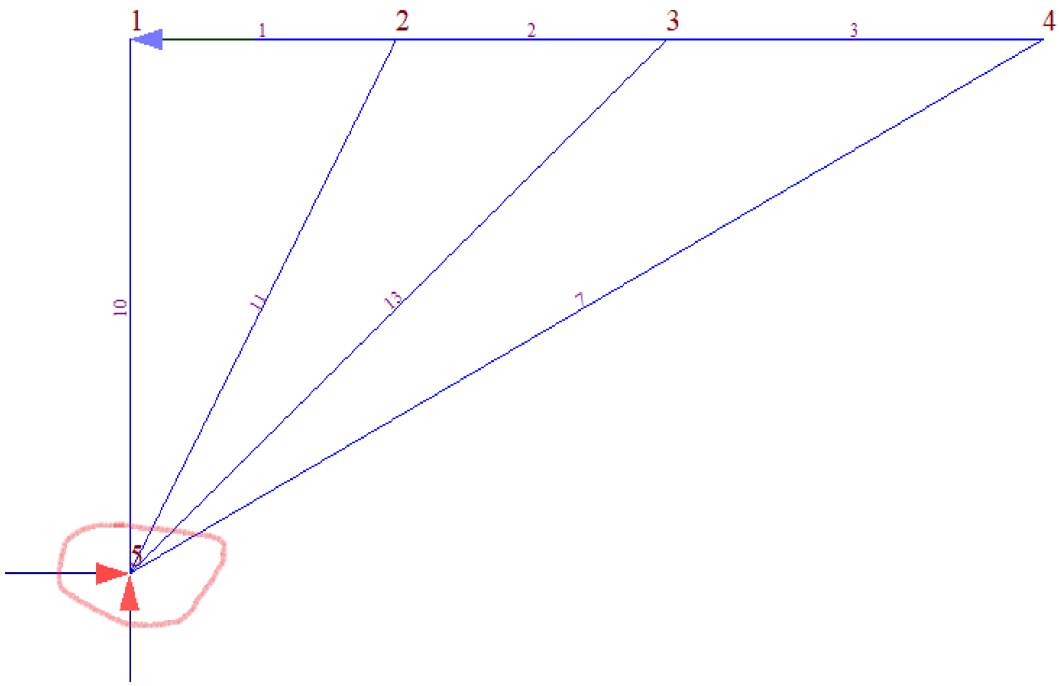
| | |
|---|---|
| HL-93 INV | 2.58 |
| HL-93 OPR | 3.34 |
| 2F1 | 10.50 |
| 3F1 | 6.41 |
| 4F1 | 5.54 |
| 5C1 | 6.87 |
| Type3 | 6.62 |
| Type3S2 | 7.84 |
| Type3-3 | 7.54 |
| SU4 | 5.66 |
| SU5 | 5.22 |
| SU6 | 4.70 |
| SU7 | 4.27 |
| EV2 | 4.86 |
| EV3 | 5.04 |
| RPL 60T | 4.57 |
| RPL 65T | 4.42 |

**NODE 5 (CCC)**

## DESCRIPTION:

CCC Node at Node 5

## GEOMETRY




-Elements 7,11 & 13 are both struts and a resultant force is calculated from them.
-This requires the angle between strut/horizontal, strut angle, strut face area, and capacity dependent on the loading.

| | | | |
|---|---|---|---|
| PROJECT : *BEL-40-23.37* | | | Michael Baker INTERNATIONAL |
| TASK : *Rating* | | PROJECT NO : *195987* | |
| SUBJECT : *Pier 5 Cap Rating* | | | |
| CALCULATED BY : *JCC* | DATE : *10/3/2023* | CHECKED BY : *ETB* | DATE : -- |

**NODE 5 (CCC)**

## CAPACITY

**RESISTANCE FACTORS**

| | | | |
|---|---|---|---|
| Compression in strut-and-tie models | Phi.c1 = | **0.70** | AASHTO 5.5.4.2 |
| Poor condition factor | Phi.c2 = | **0.85** | MBE 6A.4.2.3-1 |

**MATERIALS**

| | | | |
|---|---|---|---|
| Compressive strength of concrete | f'c = | 4.50 | ksi |
| Concrete efficiency factor | v = | 0.45 | |
| Confinement modification factor | m = | 1.00 | |
| Node face compressive stress | fcu = mvf'c = | 2.03 | ksi |

**GEOMETRY**

Depth of back face is determined using conventional flexure calculations per AASHTO 5.8.2.5.2.

| | | | |
|---|---|---|---|
| Bearing face length | lb = | **30.50** | in |
| -Taken as half the column width supporting the cap | | | |
| Back face height | ha = a = | 2.29 | in |
| -Taken as compression depth calculated in the full depth bending check | | | |
| Concrete width | bw = | 27.00 | in |

**BEARING CAPACITY**

| | | | |
|---|---|---|---|
| Area | A1 = lb * bw = | 823.5 | in^2 |
| Factored resistance | Pr = Phi.c1*Phi.c2*A1*fcu = | 992.21 | kip |

| PROJECT : *BEL-40-23.37* | | |
|---|---|---|
| TASK : *Rating* | PROJECT NO : *195987* | Michael Baker INTERNATIONAL |
| SUBJECT : *Pier 5 Cap Rating* | | |
| CALCULATED BY : *JCC* DATE : *10/3/2023* | CHECKED BY : *ETB* | DATE : *--* |

**NODE 5 (CCC)**

## LOADS

Element outputs

| Elem | Load | Force-I (kip | Force-J (kips) |
|---|---|---|---|
| 7 | DC | -37.98642 | -37.98642 |
| 11 | DC | -6.379848 | -6.379848 |
| 13 | DC | -27.17701 | -27.17701 |
| 7 | DW | -1.268346 | -1.268346 |
| 11 | DW | -0.661952 | -0.661952 |
| 13 | DW | -0.887991 | -0.887991 |
| 7 | HL-93 | -40.27797 | -40.27797 |
| 11 | HL-93 | -19.98981 | -19.98981 |
| 13 | HL-93 | -28.19931 | -28.19931 |
| 7 | 2F1 | -11.93737 | -11.93737 |
| 11 | 2F1 | -21.69034 | -21.69034 |
| 13 | 2F1 | -8.357563 | -8.357563 |
| 7 | 3F1 | -19.5581 | -19.5581 |
| 11 | 3F1 | -27.2884 | -27.2884 |
| 13 | 3F1 | -13.69297 | -13.69297 |
| 7 | 4F1 | -22.6064 | -22.6064 |
| 11 | 4F1 | -28.83486 | -28.83486 |
| 13 | 4F1 | -15.82714 | -15.82714 |
| 7 | 5C1 | -18.24712 | -18.24712 |
| 11 | 5C1 | -27.0259 | -27.0259 |
| 13 | 5C1 | -12.77513 | -12.77513 |
| 7 | Type3 | -18.92926 | -18.92926 |
| 11 | Type3 | -23.72756 | -23.72756 |
| 13 | Type3 | -13.25271 | -13.25271 |
| 7 | Type3S2 | -15.98755 | -15.98755 |
| 11 | Type3S2 | -18.27216 | -18.27216 |
| 13 | Type3S2 | -11.19317 | -11.19317 |
| 7 | Type3-3 | -16.62705 | -16.62705 |
| 11 | Type3-3 | -25.01722 | -25.01722 |
| 13 | Type3-3 | -11.64089 | -11.64089 |
| 7 | SU4 | -22.15874 | -22.15874 |
| 11 | SU4 | -29.03459 | -29.03459 |
| 13 | SU4 | -15.51373 | -15.51373 |
| 7 | SU5 | -24.00264 | -24.00264 |
| 11 | SU5 | -29.81637 | -29.81637 |
| 13 | SU5 | -16.80467 | -16.80467 |
| 7 | SU6 | -26.65657 | -26.65657 |
| 11 | SU6 | -32.72097 | -32.72097 |
| 13 | SU6 | -18.66274 | -18.66274 |
| 7 | SU7 | -29.37446 | -29.37446 |
| 11 | SU7 | -33.68537 | -33.68537 |
| 13 | SU7 | -20.56558 | -20.56558 |

**NODE 5 (CCC)**

| | | | |
|---|---|---|---|
| 7 | EV2 | -25.76127 | -25.76127 |
| 11 | EV2 | -21.94713 | -21.94713 |
| 13 | EV2 | -18.03592 | -18.03592 |
| 7 | EV3 | -32.77448 | -32.77448 |
| 11 | EV3 | -43.24373 | -43.24373 |
| 13 | EV3 | -22.94599 | -22.94599 |
| 7 | RPL 60T | -28.4152 | -28.4152 |
| 11 | RPL 60T | -33.81662 | -33.81662 |
| 13 | RPL 60T | -19.89399 | -19.89399 |
| 7 | RPL 65T | -29.40643 | -29.40643 |
| 11 | RPL 65T | -41.14374 | -41.14374 |
| 13 | RPL 65T | -20.58796 | -20.58796 |

NODE 5 (CCC)

## STRUT CHECK

Element 7 angle to horizontal 28.24 deg

Element 7

| Case | Load | L.F. | Factored | Horz. | Vert. |
|---|---|---|---|---|---|
| DC | 37.99 | 1.25 | 47.48 | 41.83 | 22.47 |
| DW | 1.27 | 1.5 | 1.90 | 1.68 | 0.90 |
| HL-93 INV | 40.28 | 1.75 | 70.49 | 62.10 | 33.35 |
| HL-93 OPR | 40.28 | 1.35 | 54.38 | 47.90 | 25.73 |
| 2F1 | 11.94 | 1.45 | 17.31 | 15.25 | 8.19 |
| 3F1 | 19.56 | 1.45 | 28.36 | 24.98 | 13.42 |
| 4F1 | 22.61 | 1.45 | 32.78 | 28.88 | 15.51 |
| 5C1 | 18.25 | 1.45 | 26.46 | 23.31 | 12.52 |
| Type3 | 18.93 | 1.45 | 27.45 | 24.18 | 12.99 |
| Type3S2 | 15.99 | 1.45 | 23.18 | 20.42 | 10.97 |
| Type3-3 | 16.63 | 1.45 | 24.11 | 21.24 | 11.41 |
| SU4 | 22.16 | 1.45 | 32.13 | 28.31 | 15.20 |
| SU5 | 24.00 | 1.45 | 34.80 | 30.66 | 16.47 |
| SU6 | 26.66 | 1.45 | 38.65 | 34.05 | 18.29 |
| SU7 | 29.37 | 1.45 | 42.59 | 37.52 | 20.15 |
| EV2 | 25.76 | 1.45 | 37.35 | 32.91 | 17.68 |
| EV3 | 32.77 | 1.1 | 36.05 | 31.76 | 17.06 |
| RPL 60T | 28.42 | 1.4 | 39.78 | 35.05 | 18.82 |
| RPL 65T | 29.41 | 1.4 | 41.17 | 36.27 | 19.48 |

Element 11 angle to horizontal 61.45 deg

Element 11

| Case | Load | L.F. | Factored | Horz. | Vert. |
|---|---|---|---|---|---|
| DC | 6.38 | 1.25 | 7.97 | 3.81 | 7.01 |
| DW | 0.66 | 1.5 | 0.99 | 0.47 | 0.87 |
| HL-93 INV | 19.99 | 1.75 | 34.98 | 16.72 | 30.73 |
| HL-93 OPR | 19.99 | 1.35 | 26.99 | 12.90 | 23.71 |
| 2F1 | 21.69 | 1.45 | 31.45 | 15.03 | 27.63 |
| 3F1 | 27.29 | 1.45 | 39.57 | 18.91 | 34.76 |
| 4F1 | 28.83 | 1.45 | 41.81 | 19.98 | 36.73 |
| 5C1 | 27.03 | 1.45 | 39.19 | 18.73 | 34.42 |
| Type3 | 23.73 | 1.45 | 34.40 | 16.44 | 30.22 |
| Type3S2 | 18.27 | 1.45 | 26.49 | 12.66 | 23.27 |
| Type3-3 | 25.02 | 1.45 | 36.27 | 17.33 | 31.87 |
| SU4 | 29.03 | 1.45 | 42.10 | 20.12 | 36.98 |
| SU5 | 29.82 | 1.45 | 43.23 | 20.66 | 37.98 |
| SU6 | 32.72 | 1.45 | 47.45 | 22.67 | 41.68 |
| SU7 | 33.69 | 1.45 | 48.84 | 23.34 | 42.91 |
| EV2 | 21.95 | 1.45 | 31.82 | 15.21 | 27.95 |

**NODE 5 (CCC)**

| | | | | | |
|---|---|---|---|---|---|
| EV3 | 43.24 | 1.1 | 47.57 | 22.73 | 41.79 |
| RPL 60T | 33.82 | 1.4 | 47.34 | 22.62 | 41.59 |
| RPL 65T | 41.14 | 1.4 | 57.60 | 27.53 | 50.60 |

Element 13 angle to horizontal 42.39 deg

Element 13

| Case | Load | L.F. | Factored | Horz. | Vert. |
|---|---|---|---|---|---|
| DC | 27.18 | 1.25 | 33.97 | 25.09 | 22.90 |
| DW | 0.89 | 1.5 | 1.33 | 0.98 | 0.90 |
| HL-93 INV | 28.20 | 1.75 | 49.35 | 36.45 | 33.27 |
| HL-93 OPR | 28.20 | 1.35 | 38.07 | 28.12 | 25.66 |
| 2F1 | 8.36 | 1.45 | 12.12 | 8.95 | 8.17 |
| 3F1 | 13.69 | 1.45 | 19.85 | 14.66 | 13.38 |
| 4F1 | 15.83 | 1.45 | 22.95 | 16.95 | 15.47 |
| 5C1 | 12.78 | 1.45 | 18.52 | 13.68 | 12.49 |
| Type3 | 13.25 | 1.45 | 19.22 | 14.19 | 12.95 |
| Type3S2 | 11.19 | 1.45 | 16.23 | 11.99 | 10.94 |
| Type3-3 | 11.64 | 1.45 | 16.88 | 12.47 | 11.38 |
| SU4 | 15.51 | 1.45 | 22.49 | 16.61 | 15.16 |
| SU5 | 16.80 | 1.45 | 24.37 | 18.00 | 16.43 |
| SU6 | 18.66 | 1.45 | 27.06 | 19.99 | 18.24 |
| SU7 | 20.57 | 1.45 | 29.82 | 22.03 | 20.10 |
| EV2 | 18.04 | 1.45 | 26.15 | 19.32 | 17.63 |
| EV3 | 22.95 | 1.1 | 25.24 | 18.64 | 17.02 |
| RPL 60T | 19.89 | 1.4 | 27.85 | 20.57 | 18.78 |
| RPL 65T | 20.59 | 1.4 | 28.82 | 21.29 | 19.43 |

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**NODE 5 (CCC)**

-Capacity calculated using resultant total force

ha = 2.29 in

lb = 30.50 in

bw = 27.00 in

fcu = 2.03 ksi

s = ha*cos(θ) + lb*sin(θ)

Acn = s*bw

Phi.c1 = 0.70

Phi.c2 = 0.85

Total factored loads used to determine capacity.

| Case | Horz. | Vert. | Resultant | Angle | s | Acn | Pn | Pr |
|---|---|---|---|---|---|---|---|---|
| HL-93 INV | 189.13 | 152.40 | 242.89 | 38.86 | 20.92 | 564.94 | 1,144.00 | 680.68 |
| HL-93 OPR | 162.78 | 130.14 | 208.41 | 38.64 | 20.84 | 562.63 | 1,139.33 | 677.90 |
| 2F1 | 113.10 | 99.03 | 150.33 | 41.21 | 21.82 | 589.12 | 1,192.97 | 709.82 |
| 3F1 | 132.42 | 116.61 | 176.45 | 41.37 | 21.88 | 590.73 | 1,196.22 | 711.75 |
| 4F1 | 139.67 | 122.75 | 185.95 | 41.31 | 21.86 | 590.17 | 1,195.10 | 711.08 |
| 5C1 | 129.58 | 114.48 | 172.91 | 41.46 | 21.91 | 591.65 | 1,198.09 | 712.86 |
| Type3 | 128.68 | 111.21 | 170.08 | 40.83 | 21.68 | 585.35 | 1,185.33 | 705.27 |
| Type3S2 | 118.94 | 100.23 | 155.54 | 40.12 | 21.41 | 578.05 | 1,170.55 | 696.47 |
| Type3-3 | 124.91 | 109.70 | 166.24 | 41.29 | 21.85 | 589.96 | 1,194.68 | 710.83 |
| SU4 | 138.91 | 122.40 | 185.14 | 41.38 | 21.89 | 590.91 | 1,196.60 | 711.98 |
| SU5 | 143.19 | 125.92 | 190.68 | 41.33 | 21.86 | 590.35 | 1,195.46 | 711.30 |
| SU6 | 150.58 | 133.26 | 201.07 | 41.51 | 21.93 | 592.15 | 1,199.10 | 713.47 |
| SU7 | 156.76 | 138.21 | 208.98 | 41.40 | 21.89 | 591.09 | 1,196.95 | 712.19 |
| EV2 | 141.30 | 118.31 | 184.29 | 39.94 | 21.34 | 576.17 | 1,166.74 | 694.21 |
| EV3 | 147.00 | 130.91 | 196.84 | 41.69 | 22.00 | 593.93 | 1,202.71 | 715.61 |
| RPL 60T | 152.11 | 134.23 | 202.87 | 41.43 | 21.90 | 591.35 | 1,197.48 | 712.50 |
| RPL 65T | 158.95 | 144.56 | 214.85 | 42.28 | 22.22 | 599.90 | 1,214.80 | 722.81 |

Michael Baker INTERNATIONAL

## NODE 5 (CCC)

Calculate Rating Factors

| Case | Horz. | Vert. | Resultant | RF |
|---|---|---|---|---|
| DC | 70.73 | 52.37 | 88.01 | |
| DW | 3.13 | 2.67 | 4.12 | |
| HL-93 INV | 115.26 | 97.35 | 150.87 | 3.90 |
| HL-93 OPR | 88.92 | 75.10 | 116.39 | 5.03 |
| 2F1 | 39.23 | 43.99 | 58.94 | 10.48 |
| 3F1 | 58.56 | 61.56 | 84.96 | 7.29 |
| 4F1 | 65.81 | 67.71 | 94.42 | 6.56 |
| 5C1 | 55.72 | 59.43 | 81.46 | 7.62 |
| Type3 | 54.81 | 56.16 | 78.48 | 7.81 |
| Type3S2 | 45.07 | 45.18 | 63.82 | 9.47 |
| Type3-3 | 51.04 | 54.65 | 74.78 | 8.27 |
| SU4 | 65.04 | 67.35 | 93.63 | 6.62 |
| SU5 | 69.32 | 70.87 | 99.14 | 6.25 |
| SU6 | 76.71 | 78.21 | 109.55 | 5.67 |
| SU7 | 82.89 | 83.16 | 117.42 | 5.28 |
| EV2 | 67.43 | 63.26 | 92.46 | 6.51 |
| EV3 | 73.13 | 75.86 | 105.37 | 5.92 |
| RPL 60T | 78.24 | 79.19 | 111.32 | 5.57 |
| RPL 65T | 85.08 | 89.51 | 123.50 | 5.11 |

## BEARING & BACK FACE

No Section loss at connection between column and cap. No rating factors calculated.

## GOVERNING RATING FACTOR

| | |
|---|---|
| HL-93 INV | 3.90 |
| HL-93 OPR | 5.03 |
| 2F1 | 10.48 |
| 3F1 | 7.29 |
| 4F1 | 6.56 |
| 5C1 | 7.62 |
| Type3 | 7.81 |
| Type3S2 | 9.47 |
| Type3-3 | 8.27 |
| SU4 | 6.62 |
| SU5 | 6.25 |
| SU6 | 5.67 |
| SU7 | 5.28 |
| EV2 | 6.51 |
| EV3 | 5.92 |
| RPL 60T | 5.57 |
| RPL 65T | 5.11 |