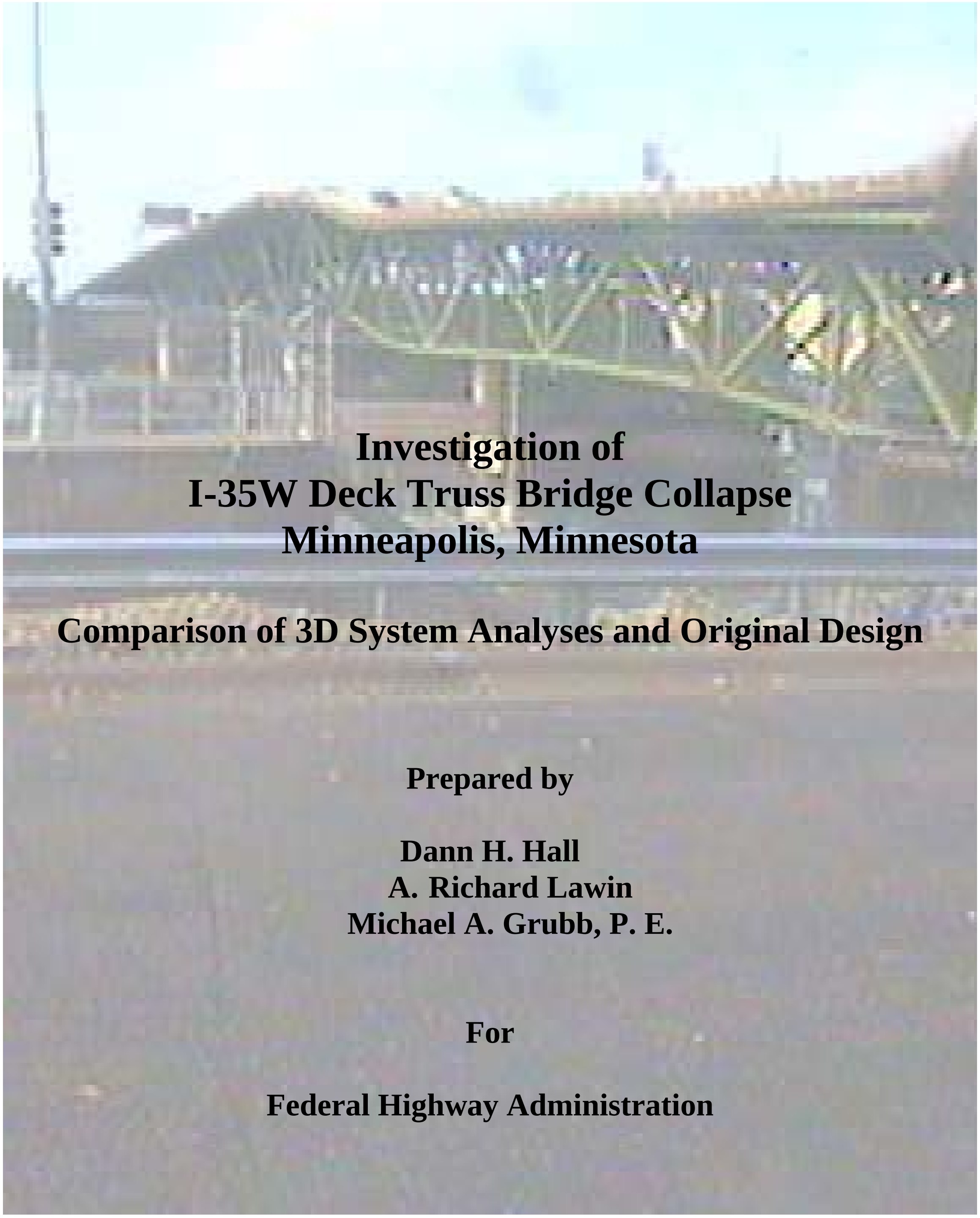

Investigation of I-35W Deck Truss Bridge Collapse Minneapolis, Minnesota

Comparison of 3D System Analyses and Original Design

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Introduction

The bridge carried I-35W over the Mississippi River in Minneapolis, Minnesota. The river generally flows north to south; however, at the crossing, it flows west to east and the bridge was oriented north and south. The bridge was owned by the Minnesota Department of Transportation (MnDOT). The bridge was designed by Sverdrup & Parcel and Associates (S&P), St. Louis, in the mid-1960s and was constructed in 1967. S&P was purchased in 1999 by Jacobs Engineering Group, Inc.

The bridge deck was separated longitudinally; each deck section was approximately 56 feet wide. The northbound and southbound deck units were supported on a common pair of deck trusses. The total length of the bridge, including approach spans at each end of the truss structure, was over 1900 feet. The total length of the truss was approximately 1064 feet. There were four striped traffic lanes on the bridge in each direction plus shoulders in 2007. Traffic on the bridge was reported to be as much as 140,000 vehicles a day. Further detailed descriptions of the as-designed main truss and its components and the approach spans are given later on in this report.

The bridge was modified from its original construction, in 1977 by the addition of a 2-inch-thick concrete overlay that was added to the deck on the main trusses and approach spans. And, in 1998, a new concrete face was added to the inside of the original rails along the exterior edges of the deck and new median barriers replaced the rails on the inside edges of the deck on both the truss spans and approach spans.

The bridge had been the subject of numerous repairs and investigations in addition to routine inspection. URS performed a recent investigation n and prepared a report that was useful in understanding the recent behavior of the bridge. They reported that the main trusses were not expected to be subject to fatigue cracking, although numerous poor welding details were noted. Routine inspections by MnDOT and URS revealed fatigue cracks; some of which were severe in the approach spans and in the cross beams supporting the ends of the approach spans adjacent to the trusses. Some of the cracks had been repaired using various techniques described in their report. The URS report also highlighted what was believed to be inoperable expansion bearings in some locations on the truss. URS estimated that approximately 6 to 7 percent of the traffic using the bridge consisted of heavy trucks.

On August 1, 2007, the deck truss portion of the bridge collapsed suddenly resulting in 13 deaths and 145 injuries. A portion of the bridge deck was under repair at the time. Figure 1 is a photograph taken approximately two hours prior to the collapse (north is to the left).

This report deals with an analysis investigation of the as-designed truss structure utilizing a three-dimensional linear elastic finite element model. Modifications to the truss and conditions at the time of collapse are examined in a subsequent report. This report examines the design gravity loads considered by S&P. The purpose of this report is to confirm the original design of the main members. The majority of this report compares axial member forces due to gravity given on the original Plans, signed in 1965 (hereafter referred to as the Plans), to forces computed from a 3D analysis utilizing the proprietary BSDI 3D System software. Connections and splices are not investigated.

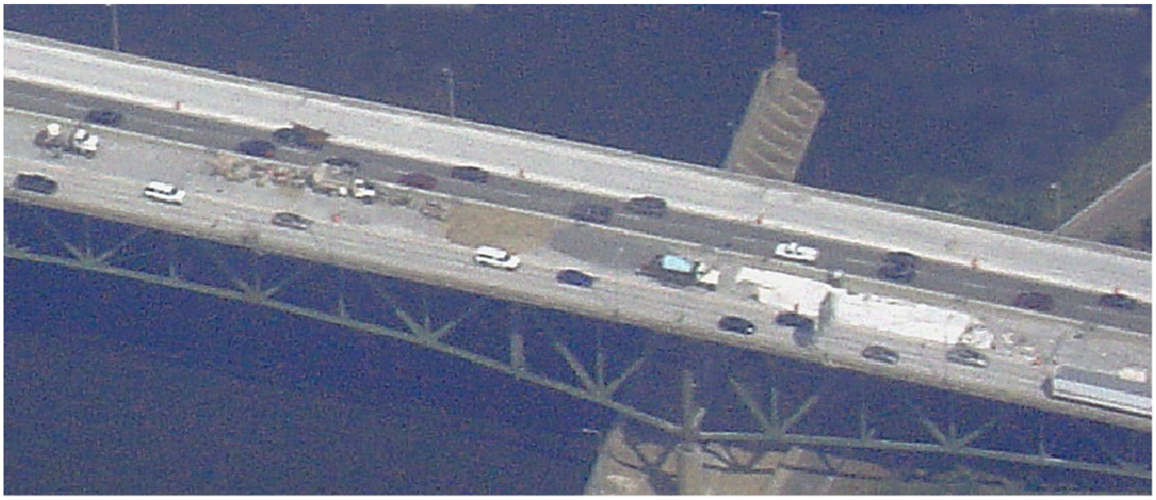


Figure 1 I-35W Truss Approximately Two Hours Prior to Collapse

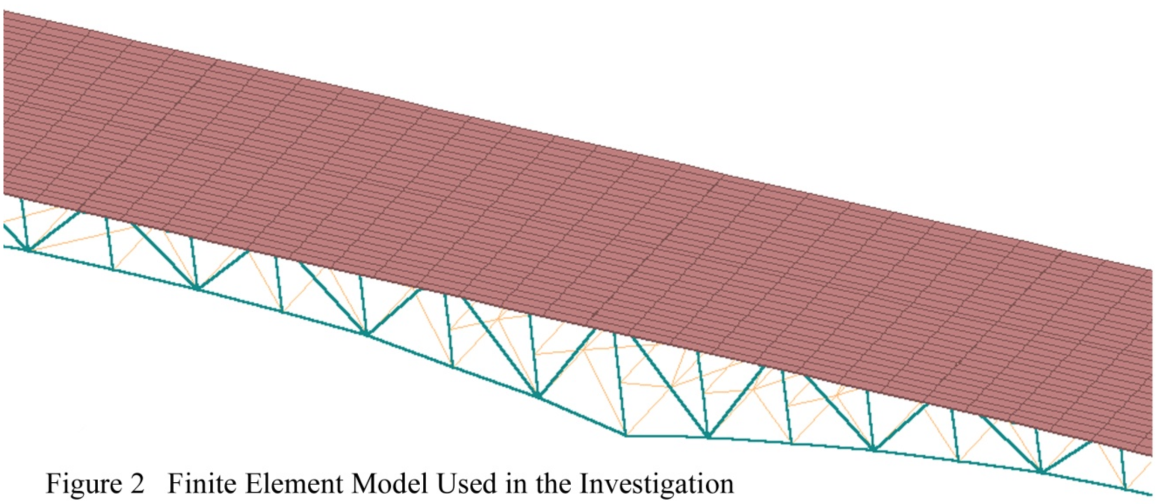


Figure 2 Finite Element Model Used in the Investigation

Figure 2 shows a plot of the three-dimensional (3D) finite element model of the as-designed truss structure developed for this investigation. The approach spans were also analyzed to estimate the load they placed on the truss since they were supported at their termini by the trusses.

Design Specifications

The bridge was designed according to the American Association of State Highway Officials (AASHO) 1961 Edition of the Standard Bridge Specifications and appropriate Interim specifications. The bridge was designed by the service load design method with a factor of safety of approximately 1.82 (i.e. 1/0.55) for both dead and live load. The

AASHO 1961 Edition of the Standard Bridge Specifications did not give welding provisions. Instead, it referred to the *American Welding Society Specifications for Welded Highway and Railway Bridges*. AWS fatigue provisions, in particular, were much more lenient with regard to fatigue for fillet welded details than present-day AASHTO bridge specifications.

Article 1.6.5—*ALTERNATING STRESSES* in the 1961 AASHO Bridge Specifications addressed the case when a member is subjected to net stress reversal due to dead plus live load. The provision called for Group I design stresses in both tension and compression to be taken as the total net stress due to dead plus live load increased by 50 percent of the smaller absolute net stress. The article calls for minimum expected dead load stress to be used in reversal calculations. S&P chose to use 90 of dead load to meet this requirement. The reported member forces (called stresses in Sheet 20 of the Plans) reflected this provision for members subject to stress reversal. For example, the Plans show that upper chord memberU4-U6 had a computed dead load force of 516 kips tension. Live load force for this member was 536 kips tension plus 48 kips of impact or 443 kips compression plus 58 kips of impact. To check for stress reversal, the total compression force equaled: 464 (i.e. 0.9 x 536) – 443 – 58 = -37 kips. Hence, the member was found to be subject to stress reversal. The total tension force equaled: 516 + 536 + 48 = 1100 kips. The smaller absolute value was 37 kips compression. Therefore, 19 kips was added to each of the total stresses to obtain the Group I design forces; 1100 + 19 = 1119 kips tension; -37 – 19 = -56 kips compression. The critical force so computed controlled the member design. The tables of computed member forces from the 3D analysis (discussed later) and comparable Plan forces given in this report do not reflect this provision, which is no longer in effect.

Loads

The AASHO Bridge Design Specifications at that time required the following loads be considered for the superstructure design: dead loads, live loads, wind loads, thermal loads, centrifugal loads, and braking loads. Other loads such as ice and stream loads were likely considered for the substructure design, but were considered outside the scope of this investigation. Wind, thermal and centrifugal loads were not considered in the present investigation, although forces in the main truss members due to wind and centrifugal loads were given in the Plans. According to the tabulation of forces on Sheet 20 none of the members were Group III critical. Therefore wind and centrifugal force are not investigated herein.

The design live load was HS20-44 with an alternative live load designated in the Plans as “PPM 20-4 Section 4C”. This designation refers to the Alternate Military Load consisting of two 24-kip axles spaced at 4 feet. Four lanes of traffic on both the northbound and southbound roadways governed the design of the main trusses. The multiple-presence factor for four lanes or more was 0.75 according to the 1961 AASHO Specifications. The computed actions due to live load were based on applying four lanes of traffic on both decks simultaneously and reducing their effect by 0.75. The impact factors applied to the live load as given in the Plans varied between members. The impact factors were a function of the length of the loaded span and expressed as a percentage of the live load. The percentages were as follows: Cantilever Span 5 - 21%; Spans 6 and 8 - 13%; Span 7 - 9%; Cantilever Span 9 - 17%. The impact percentage applied to the vertical truss members U8-L8 and U8′-L8′, and bearings at Panel Points L8 and L8′ was 11%. The impact percentage applied to the bearings at Panel Points L1 and L1′ was 13%. It appears from the magnitude of impact reported in the Plans that member live load forces computed by loading two spans were based on the average of the impact for the two loaded spans. The impact factor applied to the vertical truss members (other than members U8-L8 and U8′-L8′), floor trusses, and stringers was 30% of live load.

Composition of the Superstructure

Main Trusses

The main structure was comprised of two trusses. Each truss had three spans of approximately 266-456-266 feet. See Figure 3 (north is to the right). The depth of the trusses varied parabolically from 30 feet at each end support to 60 feet at the interior supports and again varied parabolically to 36 feet at the center of the main span. There were twenty-eight 38-foot long panels in each truss measured between end supports along the centerline of the upper chord. The main trusses were built parallel and at the same elevation with respect to each other. The main trusses were straight; they sloped downward from the south end towards the north end, which was slightly more than 7 feet lower than the south end. A portion of the southern end of the roadway on the truss curved to the west with a radius of approximately 1760 feet. The curved portion of the roadway was superelevated. A portion of the northbound roadway widened to the east approximately two feet on the northern end of the truss.

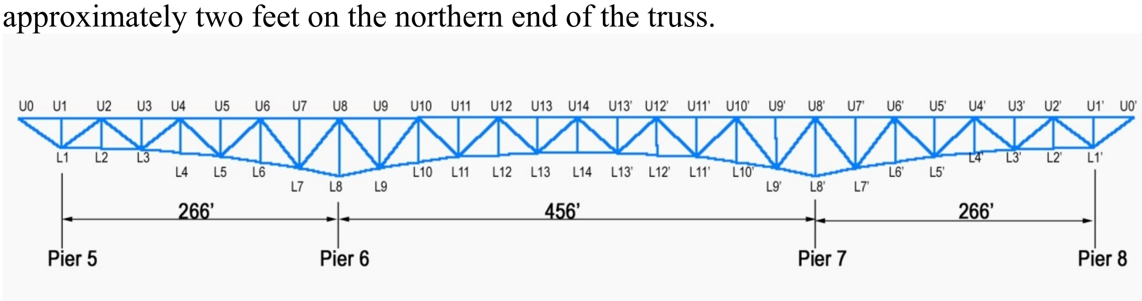


Figure 3 Elevation of Main Truss

Each truss rested on four large bearings. All four pairs of bearings of the main trusses were at right angles to each other in plan. The bearings were supported by four piers; each pier had two round columns. The individual columns for Piers 5 and 8 each rested on footings embedded into rock and were connected at the top with a concrete beam. Columns for Pier 6 rested on a concrete shaft wall that in turn rested on a concrete footing supported by twenty-two round concrete caissons. Columns for Pier 7 rested on a concrete shaft wall supported by two rectangular concrete pedestals, which in turn rested on two larger rectangular concrete seals set on sound rock. The individual columns for Piers 6 and 7 were not connected at the top. Pier 5 was the southernmost support for the trusses. Pier 6 was the southern support of the main span. Pier 7 was the northern support of the main span. The northernmost support for the trusses was Pier 8. See

Figure 3. Bearings at Piers 5, 6, and 8 permitted longitudinal movement; Pier 7 had fixed bearings longitudinally; all bearings restrained lateral movement, but permitted rotations about their transverse axis. Piers are identified in Figure 3.

The upper (U) panel points were numbered from U0 to U14 at the center, and then in descending order from U14 to U0′, with a prime identification applied to each panel point number from the center to the north end. The lower (L) panel points were numbered from L1 at Pier 5 to L14 at the center, and then in descending order from L14 to L1′ at Pier 8.

The main truss chord members were welded box sections nominally 28 inches deep by 21 inches wide. The 21-inch dimension consisted of a 20-inch wide plate with ½ inch on each side of the plate to allow for welding to the web plates. The 20-inch wide plates, called cover plates, were typically 3/8-inch thick. The cover plates were perforated with 10-inch wide hand holes approximately 36 inches long. The top chords had perforations only on the bottom; the other truss box members had perforations on both cover plates. Compression diagonals and verticals on the main trusses were also welded box sections; tension diagonals and verticals were welded I-shapes. The verticals and bottom chord members at Panel Points L8 and L8′ were larger and had a central fifth plate inside the box sections.

Truss chords, verticals and diagonals were riveted to large gusset plates. These gusset plates varied in thickness from one-half inch to one-inch. Some joints had additional plates, called "splice plates", connecting adjoining chords. These plates were of the same thickness as the gusset plates but placed inside the box members. The splice plates usually had two rows of rivets on each side of the joint. The rivets through the splice plates and gusset plates acted in double shear.

Deck and Stringer System

The deck on the truss bridge had a design thickness of 6.5 inches. There were five transverse joints in the deck. Transverse joints were located at Panel Points U4 and U4′ in the side spans, at interior-support locations U8 and U8′, and at the mid-span Panel Point U14. There were rails on the exterior edges of both deck sections and on both edges of the deck at the center of the bridge. The deck was made composite with the stringers with 7/8″ x 4″ stud shear connectors only in the regions near the joints U0, U4, U8, U14, U8′, U4′ and U0′. Each half of the deck was supported on seven stringers spaced at 8'-2", except where the stringer spacing at the north end varied due to the widening. See Figure 4 for typical stringer locations (west is to the left). Stringers were numbered S1 through S14 moving from east to west. The stringers were 27-inch deep hot-rolled wide-flange sections weighing 94 pounds per foot. They were made continuous with bolted splices, except at the deck joints identified above. The stringers were supported by the floor trusses located at each panel point of the main trusses spaced longitudinally at 38 feet. The stringers were braced with 15-inch channel diaphragm sections (not shown in Figure 4) bolted to the webs of the stringers at the floor trusses and at mid-span between floor trusses. At floor trusses without a deck joint, there were four bays between stringers without diaphragms. Between floor trusses, diaphragms were in every bay between the stringers. At deck joints, diaphragms were placed in every bay between the stringers on both sides of the joint.

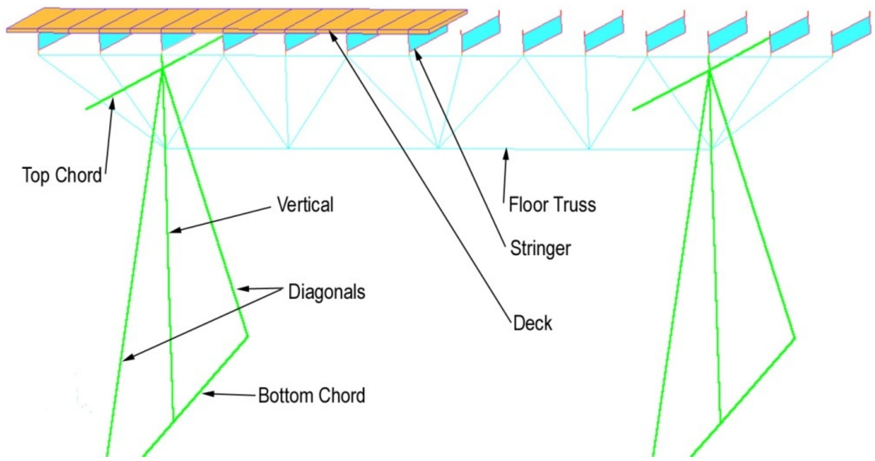


Figure 4 Partial Floor Truss and Stringer System with Southbound Deck

The end panels of the trusses were cantilevered beyond the end supports at each end to receive the stringers of the approach spans at Panel Points U0 and U0′, as shown in Figure 3. The approach-span stringers were attached by bolts or rivets to connection plates that were in turn welded to a cross beam. Each of the cross beams at the north and south ends was supported on the pair of rocker bearings that rested on the east and west main trusses at Panel Points U0′ and U0, respectively. At each end of the trusses, the stringers that spanned the floor trusses and supported the roadway decks were bolted to a floorbeam adjacent to, but separated from, the cross beam for the approach spans. The floorbeams extended beyond the main trusses to the exterior stringers as seen in Figure 5 (west is to the left). The superelevation at the south end is evident in the tapered depth of the floorbeam. At the south end, Panel Point U0, the curvature of the roadway caused the joint and floorbeam to be skewed with respect to the truss. See Figure 6 (north is to the right). The end panels of the main trusses at the north end, U1′-U0′, cantilevered 38 feet beyond the bearings. At the south end, the end panel, U0-U1, of the west truss cantilevered 35.67 feet and the end panel of the east truss cantilevered 40.33 feet.

The stringers were supported on bearings bolted to the transverse floor trusses. All of these bearings were fixed against transverse movement. Bearings at locations without joints resisted uplift. Bearings at joints appear to not have been detailed to resist uplift. The bearings on Stringers 3 and 12, which were located over the main trusses, were free longitudinally, except at one point in each deck section (i.e. at Panel Points U0, U6, U11, U11′, U6′ and U0′) where they were restrained longitudinally. The bearings at the stringers not at transverse deck joints were fixed longitudinally. The bearings at joints on stringers other than 3 and 12 were free longitudinally on one side and fixed on the other side.

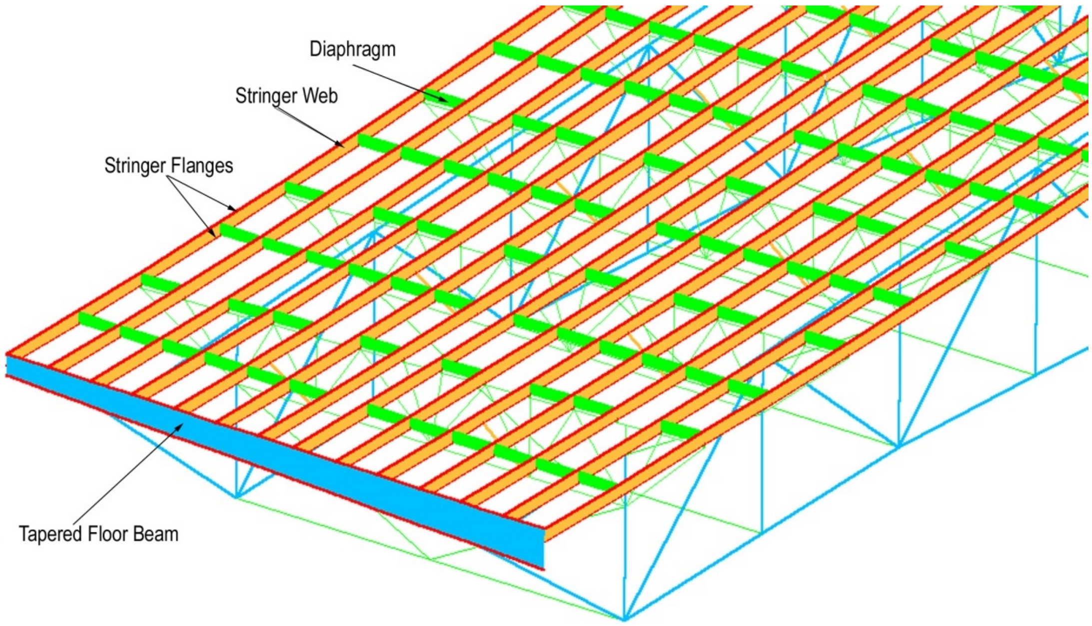


Figure 5 South Portion of Stringer System and Main Trusses

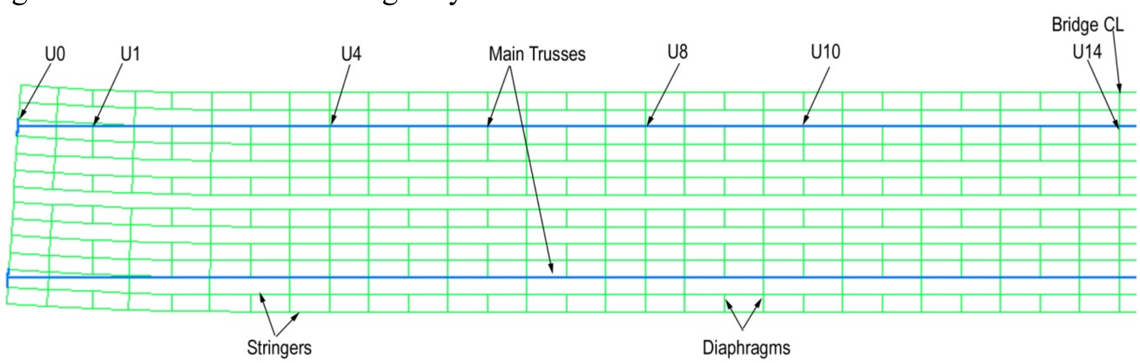


Figure 6 Plan of South Half of Stringer Framing

Figure 7 taken from Sheet 23 of the Plans shows the detail of the typical stringer bearings at locations without deck joints, except at Stringers 3 and 12.

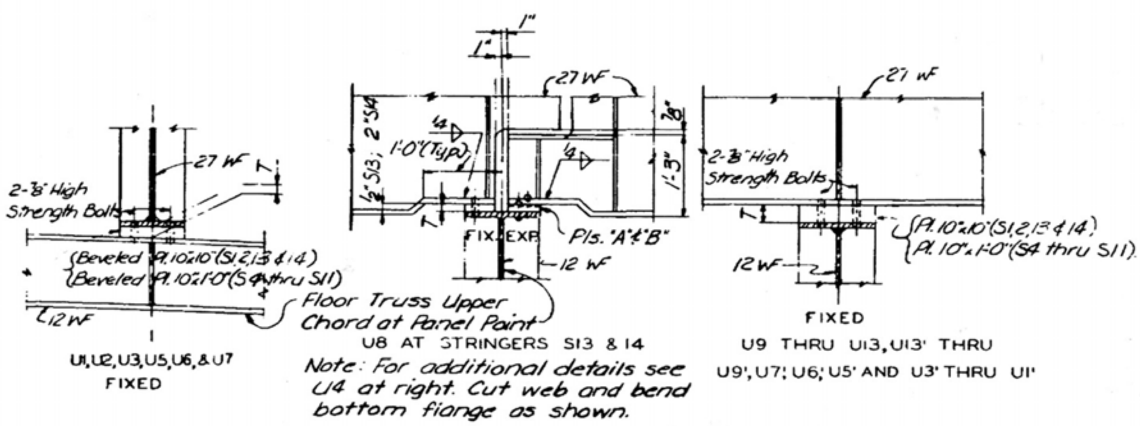


Figure 7 Typical Fixed Stringer Bearings

Figure 8 taken from Sheet 23 of the Plans shows the detail of the stringer bearings at all deck joints (north is to the right at U4, U8 and U14 and to the left at U8′ and U4′ in Figure 8). It appears that the weight of the south side of the deck restrained the north side from uplift. The south side was not so restrained.

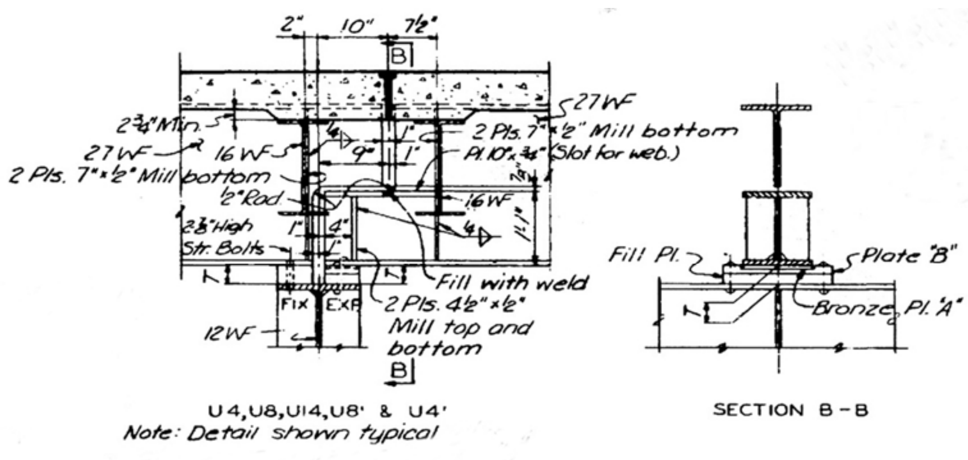


Figure 8 Typical Stringer Bearings at Deck Joints

Floor Trusses

The floor trusses were typically 12 feet deep. They were perpendicular to the main trusses. The bottom chords of all of the floor trusses were horizontal. The top chords of the floor trusses were adjusted to account for the superelevation of the deck on the south end of the main trusses; hence these floor trusses were deeper than 12 feet on the east side. The bottom chords of the floor trusses were attached to the verticals of the main trusses. See Figure 9 (west is to the left). The numbers identify the floor truss members in the corresponding tables in Appendix A.

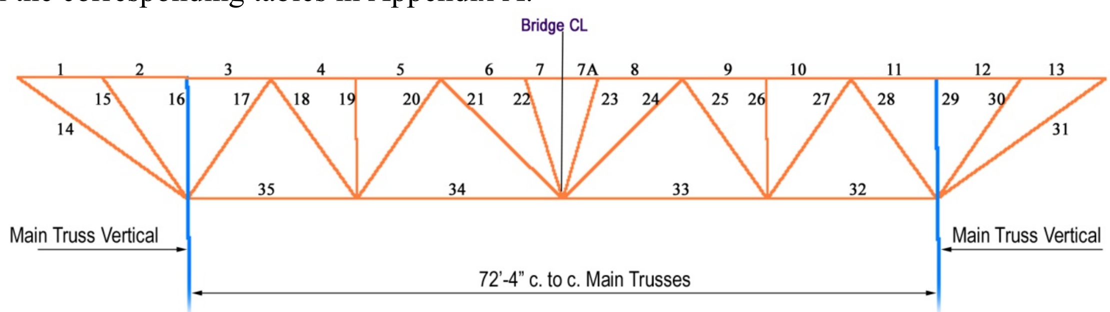


Figure 9 Floor Truss Showing Member Identification

Diagonals 14, 15, 17, 28, 30 and 31 transmitted the load from the floor trusses to the main truss verticals. At even-numbered panel points (see Figure 3), the diagonals of the main trusses were attached to the upper chords of the main trusses. The opposite was the case at the odd-numbered panel points where the main truss diagonals were connected to the bottom chord of the main truss. The effect of this arrangement is examined under floor truss results.

The floor truss chords and diagonals were hot rolled wide-flange sections. Connections of the floor truss diagonals and chords were made by welding the members to common gusset plates. The top chords had bolted splices between Stringers 3 and 4, and between Stringers 11 and 12. The gusset plates at the intersection of the bottom chord of the floor trusses with the diagonals of the floor trusses outside the main trusses were bolted to the main truss verticals.

The top chords of the floor trusses generally rested on top of the main truss upper chords. Superelevation on the southern portion of the main truss required that pedestals be placed on top of the upper chords of the east main truss to support the top chord of the floor truss and the stringer above the main truss.

Lateral Bracing

As shown on Sheet 23 of the Plans, single angle diagonal members connected the bottom chord of each floor truss to the bottom of the bottom flange of Stringers 5 and 10. They can be identified in Figure 10 as the pairs of short members extending to the left from each floor truss. These members acted to stabilize the bottom chord of each floor truss. The heavier lines in Figure 10 identify the main trusses.

A herringbone-type lateral bracing system between floor trusses was located in a plane just below the plane of the top chords of the floor trusses, as described below. See Figure 10 (north is to the right). Bracing members ran from the inside gusset plate at the upper panel points of the main trusses to a point just below the center of the top chord of the adjacent floor truss nearer the center of the main span. A similar lateral bracing system was employed in the plane of the main truss bottom chords. The top and bottom lateral bracing members were welded box sections. Cantilevered lateral bracing members extended from the main trusses to the exterior stringers (S1 and S14)

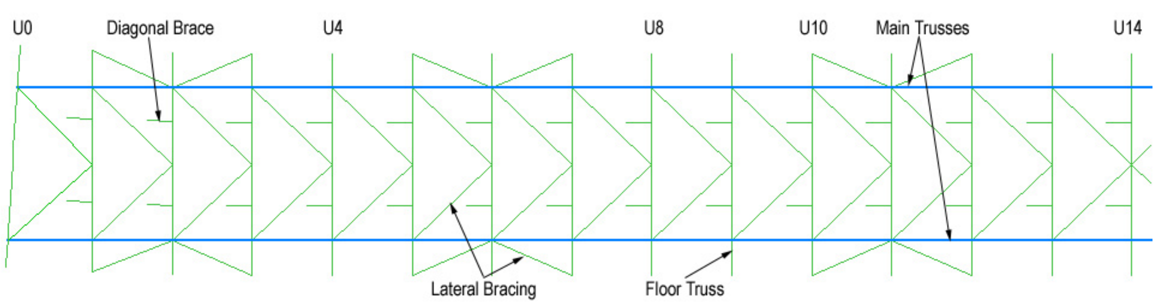


Figure 10 Plan of South Half of Top Lateral Bracing and Floor Trusses

Portal and Sway Bracing

The trusses were braced by portal frames at Panel Points L1 and L1′ and at the interior piers (Panel Points L8 and L8′). See Figure 11. At Panel Points L1 and L1′, the diagonal box members of these frames extended from the bottom chord of the floor trusses to the mid-span of a strut connecting the bottom chords of the main trusses. At Piers 6 and 8, the portal frames had a double “K-frame” configuration with an additional strut mid-height between the bottom chord of the floor truss and the bottom chord of the main trusses. At intermediate panel points not at bearings, the main trusses were braced by sway frames. At Panel Points L6, L6′, L7, L7′, L9, L9′, L10 and L10′, the sway frames consisted of a double “K-frame” configuration as described above. At all other panel points, a single “K-frame” configuration was used. The portal and sway bracing members were welded box sections without hand holes.

There was an inspection walkway under the southbound roadway the full length of the truss. There were transverse walkways at Piers 6 and 7 that were accessed by two ladders. There were navigation lights on the truss at Pier 7 and near Panel L10 that were accessed by walkways and ladders.

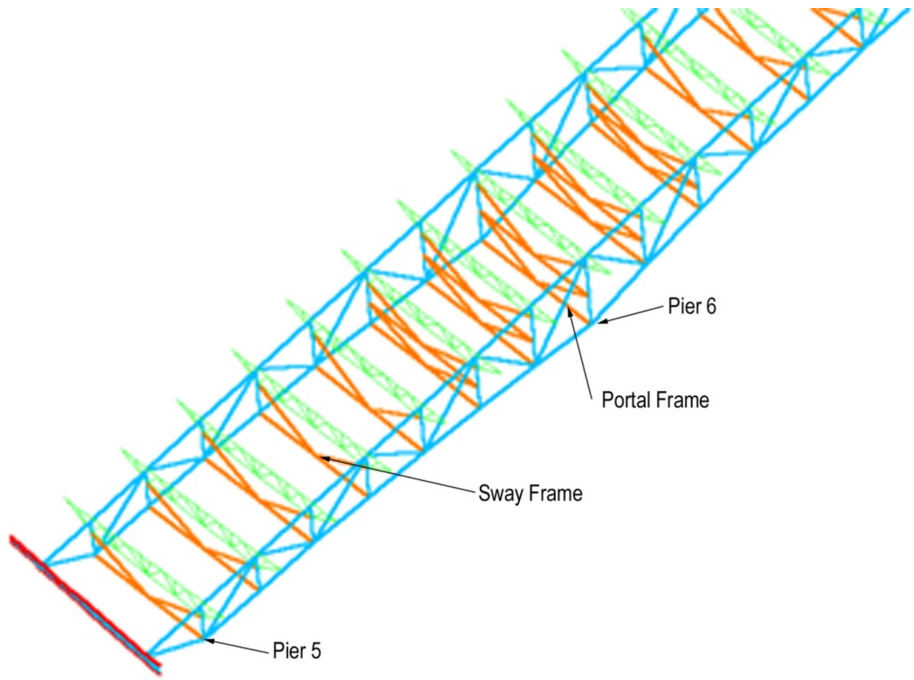


Figure 11 Portal and Sway Bracing

Truss Construction

Riveting, High-strength Bolting and Welding

The bridge was designed and built in a time of transition from riveting to the modern-day era of welding and high-strength bolting. Welding was employed to build box members that a few years earlier would have been built-up from riveted connections of angles and plates. Many of the member connections were riveted, whereas a few years later, they would have been entirely made using high-strength bolts. High-strength bolts were permitted as an option, and were used primarily on the floor trusses. Composite construction had been accepted by AASHO earlier and was employed in the approach spans and over portions of the stringer system near the transverse deck joints, as mentioned previously. Shear connection between the deck and the stringers, where employed, was accomplished with welded studs. The main truss was not designed to be composite with the deck system.

The welded box-shaped members, except the top chords of the main trusses, had welded diaphragms near each end connection; the top chord members had bolted diaphragms. Additional intermediate diaphragms spaced at not more than 15 feet were called out for all box members. There were also welded backup bars inside the box members. Many of the weld details were potentially problematic with respect to fatigue as checked in modern bridge specifications.

The wide-flange shapes in the floor trusses were welded to gusset plates. Unlike the welded diaphragms in the box members, where the welds transferred only shear between elements, the floor truss welds transferred the loads between diagonals and chords.

Camber

Since there were no notes on the Plans regarding the consideration of built-in erection forces, it is not known whether they were considered in the design of the bridge, but it would have been unlikely. It is also not known if the erection introduced any unaccounted forces. If the members were detailed with lengths introduced for camber adjustment based on the dead load forces reported in the Plans, the truss could not have been erected if permanent erection forces had been introduced. The BSDI analysis of the main trusses, discussed later, did not include any erection forces and was found to confirm the assumption that no erection forces were assumed to remain in the erected truss. That is, the design forces used to establish the camber correlated with the 3D analysis results indicating that there were no significant locked-in erection forces introduced into the trusses.

Of course, some main truss members were undoubtedly forced to fit during erection to overcome forces introduced temporarily due to the self-weight of the partially complete trusses. The most interesting force-fit is for the fit of the final members closing the main span. Presumably, the two halves of the truss were built by the cantilever method and closed in the center. The moments at Piers 6 and 7 due to the self-weight of the partially erected trusses would have to have been greater than they were at those locations when the truss was completely erected and made continuous with a positive moment introduced into the center region of the main span. To develop this positive moment, the final members would have to have been installed by jacking the adjacent members of the two truss halves to introduce the force into the members in the mid-span region that would have existed in these members due to the final moment caused by the truss self-weight. The temporary member forces near Piers 6 and 7 would have been reduced as a result of the jacking at mid-span. For example, the compression force in the bottom chord members L8-L9 would have been reduced to the final erected force. If the final members had not been force-fit into place, the truss would have experienced erection stresses that would have to have been taken into account at design. That is the reason that the member forces were given with regard to detailing for camber. If residual erection stresses were planned on, the member forces would have been different and a specific erection scheme would have to have been specified. The distances between bearings under total dead load were called out on Sheet 21 of the plans. These distances would have been worked with the specified dead load member forces by the erector.

Approach Span Loads

As mentioned previously, the stringers of the northbound and southbound approach spans were attached to cross beams that were supported by rocker bearings that rested on the main trusses at Panel Points U0 and U0′. The approach-span structures had separate northbound and southbound units. Both approach-span units on the south end of the truss had 7 stringers, as did the southbound approach-span unit on the north end of the truss. The stringers of these units were spaced at 8′ 2″. The northbound approach-span unit on the north end of the truss had 8 stringers. Five of the stringer spacings in this unit were 8′-2″. The remaining two spacings were approximately 6′-5″. Since the finite element model did not include the approach spans, end reactions from these structures were applied as loads to the cross beams on the trusses, as described in a later section of this report. It was assumed that the approach-span stringers were erected after the truss was erected, but before any deck concrete was placed so that their steel weight was included with the self-weight of the trusses.

In the design, the live load reactions from the approach-span stringers may have been calculated using a wheel-load distribution factor for each stringer. If so, the wheel-load distribution factor specified by AASHO at that time might have been employed as follows: S/5.5, where S equals stringer spacing, or 8.2/5.5 = 1.5 wheels (0.75 lanes/stringer). Hence, the design live load capacity of each seven-stringer approach may have been computed as 7 Stringers * 0.75 = 5.25 lanes. In accordance with the specifications, this number of lanes would have been reduced by multiplying by 0.75 to account for multiple presence of four or more loaded lanes, resulting in 3.94 lanes used in design on each roadway. This approach is somewhat conservative. The total design live load on either the northbound or southbound approach structures may have been computed assuming 3.94 * 2 = 7.88 loaded lanes. This is about 30 percent greater than the design live load on the truss structure, which is computed as: 0.75*8 lanes = 6.00 lanes. The reason for this paradox lies within the AASHO Specifications that traditionally have been based on the design of individual components. Since the live load for individual stringers is usually computed using a wheel load distribution factor based on two lanes with a multiple presence factor of 1.0, two-truss structures designed based on a multiple presence factor of 0.75 tend to provide an economic advantage with respect to live load over multi-girder cross sections.

3-Dimensional Finite Element Model of the Superstructure

The linear elastic finite element model of the truss was constructed with the BSDI 3D System from the deck down to the truss bearings. The model included the deck and stringer system, stringer bearings, floor trusses, floorbeams, main trusses, lateral bracing, sway frames, portal frames, and main truss bearings.

Deck and Stringers

The two deck systems, each composed of seven stringers and a deck, were modeled using the standard 3D System preprocessing software. The widening of the northbound roadway was recognized on the east side at the north end. The horizontal curve and superelevation of both roadways at the south end was included in the model. The five transverse joints in the decks were introduced by providing double nodes in the deck and stringers at those locations.

The decks were modeled with a series of eight-node solid elements, with the thickness of each element representing the structural thickness of the deck. A cross section of the deck was composed of two deck elements between each stringer (each 4′-1″ wide) and one eight-node element for each overhang. Each deck element was 19′-0″ long, or half the distance between panel points on the main truss. The 28-day compressive strength of the deck concrete f'_c was assumed to be 4.0 ksi resulting in an elastic modulus E_c of approximately 3,600 ksi. A Poisson's ratio of 0.16 was assumed for the concrete. The deck was assumed to be fully effective in compression and tension and composite along the entire length of the bridge for the superimposed dead load and live load cases.

The cross section of each stringer was modeled with two isoparametric beam elements representing the top and bottom flanges and a single four-node shell element representing the web. Each element was 19′-0″ long. When analyzing for stresses on a macro level, as was the case in this investigation, the aspect ratio of the individual elements is not critical. For all steel elements in the model, a Young's Modulus E of 29,000 ksi and a Poisson's ratio of 0.3 were assumed.

Full composite action was assumed between the stringers and the hardened concrete deck along the entire length of the stringers in this investigation. Composite action between the stringers and the deck in regions where actual studs were not present most assuredly occurred due to bond. A later analysis of floor truss results and field observations would seem to indicate that the stringers were acting compositely with the floor trusses at most all locations in addition to the regions near the transverse deck joints. The assumption of full composite action likely affected the magnitude of the stresses in the individual stringers and floor trusses to some degree, but did not have a significant effect on the overall results for the main trusses. Beam elements, oriented vertically between each deck and stringer node, assured shear connection between the deck and the stringers in the finite element model.

Each stringer was attached to the top chord of each floor truss with a vertical beam element. The length of these beam elements was adjusted as required to account for superelevation and the distance between the center of the floor-truss chord and the bottom of the stringer. The top node of each beam element was assigned proper releases to represent the as-designed fixed or expansion condition at each location. At locations where the Plans showed that the stringers were restrained from longitudinal translation, no releases were provided. All beam elements permitted rotation of the stringers about three axes. All stringer support points were restrained against transverse movement.

The 15-inch deep channel diaphragms between the stringers were modeled with a single plate element 27 inches deep. The input thickness of the plate elements gave properties that represented the actual stiffness of the 15-inch channels. Plate elements were placed along the line of each floor truss and midway between floor trusses. Six intermediate diaphragms were used in each span between floor trusses in each roadway. At the floor trusses without deck joints, only four diaphragms per roadway were used; at transverse floor joints there was a diaphragm in all six bays in each roadway. Diaphragms were used on both sides of the deck joint to support the edges of the deck.

Each deck was bounded by an exterior and interior rail. The stiffness of these rails was not modeled. The exterior rails weighed 538 pounds per foot; 438 pounds per foot was applied on the outside edge of the deck and 100 pounds per foot was applied to the stringer adjacent to the overhang. The rail at the interior of the northbound lane weighed 350 pounds per foot; 240 pounds per foot was applied to the edge of the deck and 110 pounds per foot was applied to the stringer adjacent to the overhang. The rail at the interior of the southbound lane weighed 160 pounds per foot; 125 pounds per foot was applied to the edge of the deck and 35 pounds per foot was applied to the stringer adjacent to the overhang.

Floor Trusses

The top chords of the floor trusses were modeled with a series of beam elements. Their elevation was slightly above the top chords of the main trusses. They were supported in the model by vertically oriented beam elements to properly locate their centroid with respect to the centroid of the 28-inch-deep main truss top chords. The depth of the floor trusses was varied at the south end of the bridge to properly consider the superelevation of the deck system where it caused the top chords of the floor trusses to be even higher than the top chord of the main truss.

Each bottom chord of the floor trusses was modeled with beam elements. The bottom chords were truly level; they framed into the main truss verticals. The sixteen floor truss vertical and diagonal members were rolled beams of various sizes; they were modeled as truss elements having only an axial degree of freedom. The fourteen stringers underneath the two roadways were supported by each floor truss. Uplift, should it occur at any of the stringers, was assumed to be resisted in the model.

Main Trusses

A computer program was written to compute moments of inertia and torsion constants of the main truss welded box- and I-sections. Each truss member between panel points was modeled as a single beam member. Since the elements were "isoparametric", only a single curvature could be represented by an element. In cases where bending moment in the truss was deemed of interest, the member was divided into additional elements to recognize reverse curvature. The downward slope of the trusses from south to north was not recognized in the model; both trusses sloped alike and this slope was not anticipated to have a significant effect on the analysis results.

Structural steel weight was applied to the model based on the self-weight of the individual steel members. The structural steel was given a density of 530 pounds per cubic foot; 8 percent greater than the true density of 490 pounds per cubic foot. This difference accounted for a reasonable estimate based on experience of the weight of the splice plates, diaphragm plates in the box members, rivets, shear connectors, sundry steel, and paint. All bracing members were included in the model so their weight was accounted for directly in the analysis. The inspection walkways and ladders were accounted for by applying additional load to the bottom chords of the floor trusses at appropriate locations.

In addition to the self-weight of the steel members, concentrated loads were applied to the main truss joints to account for the weight of the gusset connections in order to better distribute the true steel weight. The additional dead loads added at panel points are given in Table 1, and were estimated based on take-offs of the gusset plate sizes shown on the Plans. Only loads applied to the southeast joints are given. Equal loads were applied to the remaining joints in a similar manner.

Joint	U1	U2	U3	U4	U5	U6	U7	U8	U9	U10	U11	U12	U13	U14
Weight (kips)	1.0	2.0	2.0	2.0	2.0	2.0	2.0	4.0	2.5	2.0	2.0	2.0	2.0	2.0
Joint	L1	L2	L3	L4	L5	L6	L7	L8	L9	L10	L11	L12	L13	L14
Weight (kips)	1.2	2.2	2.2	2.2	2.5	2.5	2.5	5.0	2.5	2.5	2.5	2.5	2.5	2.5

Table 1 Loads Added at Joints to Reflect Gusset Plate Weight

The stiffness of the expansion dams at the ends of the trusses was not included in the model, but a load of 500 pounds per foot per dam was applied on each floor beam at Panel Points U0 and U0′.

Bearings

Foundation elements, which have both translational and rotational stiffness specified, were used to model the bearings. Boundary conditions in the model were intended to mimic what was assumed in the original design as shown on the Plans. Expansion bearings at Piers 5, 6 and 8 were modeled with foundation elements free in the longitudinal direction of the trusses. All bearing locations were modeled with the foundation elements restrained laterally. Since the Plans specified that the bearings at Pier 7 were fixed longitudinally, the foundation elements at Pier 7 were so fixed. Longitudinal pier flexibility was not recognized in the analysis since only one location was specified to be constrained longitudinally. The three degrees of rotations were unconstrained in all foundation elements. All bearings were modeled as rigid in the vertical direction.

Approach Spans

Since the finite element model did not include the approach spans, it was necessary to determine the end reactions from these structures so they could be applied to both the north and south ends of the trusses. Simple line-girder models of typical single girders were used to estimate the reactions on the end floorbeams at Panel Points U0 and U0′. All stringers on one end were assumed to have the same reaction. In addition to the self weight of the girders, an additional load of 25 lbs/ft for the south approaches and 32 lbs/ft for the north approaches was applied to the steel girder to account for the self-weight of the diaphragms and other steel details. In each case, a railing load of 100 lbs/ft and a wearing surface load of 25 lbs/ft^2 were also assumed applied to each girder in the composite dead load analysis.

The HS20-44 lane load is 640 pounds per foot. The concentrated load portion of the lane load must not be applied to the approach-span stringers since it was considered in loading of the main truss structure. The application of the concentrated live loads to the stringers would have been double-counting. Therefore, the following calculation was used to estimate the approach-span live load reactions on the end cross beams.

The south approach span that rested on the main truss cross beam was 71.6 feet long. Therefore, assume the cross beam receives approximately 0.4 x 71.6 = 28.6 feet of uniform live load as a simple end reaction. The north approach span that rested on the main truss cross beam was 129.6 feet in length. Hence, assume the cross beam in this case receives approximately 0.4 x 129.6 = 51.8 feet of uniform live load as a simple end reaction. Thus:

South approach live load reaction per truss = 0.64 k/ft x 28.6 ft x 3.94 lanes = 72 k.
North approach live load reaction per truss = 0.64 k/ft x 51.8 ft x 3.94 lanes = 131 k.

Analysis Results

Main Trusses

Reactions

Table 2 compares dead and live load reactions from the 3D System analyses of the main truss to those reported on Sheet 20 of the Plans. Dead load results from the 3D System analyses are separated into Stages 1, 6, and 7. Stage 1 is steel load; Stage 6 is wet concrete load; Stage 7 is superimposed dead load. The reactions were not symmetrical due to the lack of symmetry of the alignment of the two decks and the weight of the rails. The unsymmetrical deck also affected the live load responses. The difference in the loads applied from the approach spans also significantly affected the symmetry of the reactions.

The comparison of the reactions from the 3D System and those given in the Plans indicates that the analyses performed at design correlate well with the 3D System analyses with regard to both dead and live load.

Loading	Truss	Pier 5		Pier 6		Pier 7		Pier 8	
		3D	Plans	3D	Plans	3D	Plans	3D	Plans
Stage 1	East	283		1379		1350		411	
	West	285		1365		1337		404	
Stage 6	East	643		2012		1975		820	
	West	648		1990		1945		809	
Stage 7	East	142		331		314		218	
	West	141		332		315		216	
Total DL	East	1068	1098	3722	3660	3639	3589	1449	1446
	West	1074	1098	3687	3660	3597	3589	1429	1446
LL	East	450 +72 522	497	1032	1001	1030	999	442 +131 573	557
	West	424 +72 496	497	1046	1001	1043	999	443 +131 574	557
Impact		65	65	110	110	110	110	72	72
LL+I	East	587	562	1142	1111	1140	1109	645	629
	West	561	562	1156	1111	1153	1109	646	629

Table 2 Comparison of Reactions from 3D Analysis and Plans

Main Truss Members

Analysis results for the main truss members are presented in Tables 3 through 22 (Appendix). A check of the bracing members in the main truss showed insignificant forces due to dead and live load; thus analysis results for these members are not reported. The analyses were performed using the gross areas of the truss members. Member net areas are reported in the tables. *The net area used to compute compression forces is the gross area less the hand holes. The net area used to compute tensile forces is the gross area less the hand holes and the rivet holes.* There are five tables for each of the four quadrants of the trusses; southwest, northwest, southeast, and northeast. Tables 3, 8, 13, and 18 give upper chord results. Tables 4, 9, 14, and 19 give lower chord results; Tables 5, 10, 15, and 20 give diagonal results; Tables 6, 11, 16, and 21 give upper vertical results; Tables 7, 12, 17, and 22 give lower vertical results. “Upper verticals” is the term given to the portion of the vertical members of the main truss above the bottom chord of the floor truss, and “lower verticals” is the term given to the portion of the vertical members below the bottom chord of the floor truss.

The analysis for the steel weight (represented as a Stage 1 load in the 3D System) was performed assuming that the Stage 1 loads were applied at one time. This assumption implied that the steel was fully erected in the no-load position and then released. This assumption also meant that the computed stresses for self-weight of steel did not include erection stresses. Stage 6 in the 3D System is deck weight and was applied to the bare steel section as if the deck was made in a single cast without considering a casting sequence. This is most likely what was assumed in the design; that is, it was likely assumed that the deck was cast wet in a single cast and was not effective until it all hardened. The concrete casting probably included concrete from the approach spans. Although such an assumption may have been made in the design, each section of the deck was probably cast and hardened before the next section was cast. The actual sequence is not known. Two investigations not reported showed that the sequence of deck placement would have had little effect on the final main truss stresses. In the analysis, the deck weight was applied to the top of the stringer nodes as concentrated loads. Superimposed dead load (Stage 7 in the 3D System) was applied to the deck assuming a 3n-composite stiffness of the deck to allow for creep of the concrete. The superimposed dead load consisted of the rail loads applied as described previously.

The sum of the axial forces in the main truss members for the three dead load stages are compared to the dead load forces reported on the Plans. Ratios identified as ′3D DL/Plans′ show generally good correlation between the 3D analyses results and the Plans. The largest ratios are for members with very small forces. The absolute differences in the forces at these points are similar to the differences in these values reported in other members. Other differences occur because of the treatment of the north-end widening, south-end curvature and rail weights in the 3D analyses that appear to have not been treated similarly in the original design.

Live load axial forces were determined by developing a pair of influence surfaces for each truss member, one for each roadway. The live loads, HS20-44, and Alternate Military , were applied to the northbound and southbound influence surfaces. The deck concrete stiffness was computed using n. Each truck or lane live load was applied within a 12-foot-wide lane and positioned within each lane for maximum effect without coming closer than 2 feet to the edge of the lane. Lanes were moved laterally on the deck but were not permitted to cross each other. One truck was applied at the critical longitudinal point in each lane on the influence surface for each roadway. The sum of the responses for the lanes loaded on the two roadways was reduced for multiple presence. For example, when a total of four lanes or more were loaded, a multiple presence factor of 0.75 was applied. The lane load consisted of a uniform load of 640 pounds per foot plus a concentrated load of 26 kips (shear case) in each lane. The uniform load was applied to portions of the influence surface where it caused the maximum or minimum response. The concentrated load or the truck was applied in a similar manner in each loaded lane.

The magnitude of impact applied to the 3D live load results was determined by multiplying the 3D live load response by the ratio of the impact reported in the Plans to the corresponding live load response from the Plans. A flat 30 percent impact was assumed applied to the 3D live load responses from the approaches (indicated as "LL U0-U0′" in the tables) and was added to the 3D impact where appropriate. Both the impact forces reported in the Plans and those applied in the 3D analysis are given in the tables.

The maximum positive (tensile) and negative (compression) live load responses from the 3D analysis were reported for all main truss members. The Plans only showed two live load values where actual stress reversal was found to have occurred based on the algorithm described earlier. Ratios comparing the 3D values for live load and total load on the Plans are given in the aforementioned tables only where the reversed live load values were available on the Plans. The largest ratios, identified as ′3D LL/ Plans LL′, occurred near the ends of the truss. These larger differences are most likely due to the difference in the assumed live load from the approach spans. As explained earlier, the assigned live load from the approach spans in the 3D analysis was based on more loaded lanes than were loaded on the main truss spans. The ratio of the assigned load was [7.88 lanes]/[6 lanes] = 1.31. Other smaller differences where the 3D LL results were smaller than the values given on the Plans may be due to different treatment of the curvature of the deck and the deck widening at the ends. It may have been awkward for the S&P designers to trace the load path where curvature of the deck was present. The most likely explanation for the lower 3D LL forces in the top chord members compared to the design values is that the deck and stringers were assumed attached to the top chord of the trusses in the 3D analysis.

The ratios identified as ′3D Total/Plans′ that are greater than 1.0 indicate that the sum of the dead and live load forces computed by the 3D System was greater than the comparable sum of the forces reported on Sheet 20 of the Plans. All ratios greater than 1.0 are highlighted in red in the tables for easy identification. Some of the stress reversal cases in several of the top chord members, in particular, appear to have larger ratios of computed values to Plan values. The main reason for the large ratios is that the forces are small. Apparently, the present analysis assumptions used on the approach spans differed somewhat from those used by S&P. Those differences do not have a material effect on the overall conclusions from this report.

The Plans give a single live load force of 207 kips (tension or compression) plus 62 kips of impact in all the vertical members, except for Members U8-L8 and U8′-L8′. The Plans give specific dead load forces for each vertical member. The forces in the vertical members reported on Sheet 20 of the Plans give the force in either the upper or lower portion of the members, whichever is greater. The 3D results for vertical members are separated into upper vertical and lower vertical values. At even numbered panel points, the upper verticals intersect the two main-truss diagonals at the upper panel point. The reported dead load force in these upper vertical members is the net sum of the vertical force components in the diagonals at the panel point plus the force from the floor truss supported at that point. In these cases, the reported force in the vertical is reduced by the reaction from the stringer directly above the main truss vertical. The upper vertical members at even numbered panel points transmit the force from the floor truss diagonals to the top of the main truss where main-truss diagonal members receive it. The vertical members below the floor truss in this case have essentially no load. At the odd numbered panel points, the lower vertical members carry the compressive force from the floor truss diagonals and from the stringer directly above the main truss vertical. The upper vertical members at these points carry only the reaction from the stringer directly above them.

Floor Trusses

Three separate analyses were performed to investigate the floor trusses. Floor truss member forces were determined for a typical floor truss at Panel Point U10 where the stringers were continuous over the floor truss and were assumed effective with the deck and the lateral bracing attached to the top chord of the floor truss in the analysis. The second type of analysis was for the floor truss at Panel Point U10 ignoring the interaction effects of the deck, stringers and the lateral bracing. The second type of analysis was believed to be closest to the type assumed by S&P. Since dead load forces were not given in the Plans, it was not possible to check these forces computed by S&P. The third type of analysis performed was for the floor truss at Panel Point U14 where a joint was present in the stringers and deck. This third type of analysis recognized the deck and lateral bracing interaction.

In general, the 3D analyses assuming the deck and bracing interaction was not present showed the design forces in the floor truss members to be reasonably predicted on the Plans, except for two diagonals near the center of the truss where the computed total forces from the 3D analysis were 5 and 7 percent higher than the total forces reported on the Plans. The 3D analysis gave significantly lower bottom chord forces than shown on the Plans. The 3D System analyses showed that bottom chord members 32 and 35 underwent stress reversal. The forces given in the Plans for these members perhaps were increased from the analysis values by 50 percent of the smaller force according to Article 1.6.5. The 3D analysis results with the deck and bracing interaction present never exceeded those reported on the Plans.

The HS20-44 lane and truck and the Alternate Military load were investigated on the floor truss at Panel Point U10. The Alternate Military load did not control. Thirty percent impact was applied to all live load results for the floor trusses, as specified in the Plans.

The interaction between the floor trusses and stringers was found to be complex. Dead load preloads the stringer bearings with a downward (compressive) force. Live load causes irregular loads on the floor trusses. A live load on the northbound deck would have deflected the floor truss, reducing the dead load reaction and potentially causing uplift in some of the bearings of the southbound stringers. Additionally, load applied to the floor trusses outside of the main trusses would have caused a reduction in the dead load reaction in some stringers. The stringers were continuous on some floor trusses, thus they received less load than if they were loaded directly with the live load. Based on the computed force in the floor truss members, it appears that the designer did not take into account the transverse interaction of the deck and stringers with the floor trusses. The Plans called for tie-down devices on some bearings indicating recognition of the potential of the vertical forces between the deck stringers and the floor trusses that could cause uplift.

Inspection reports indicated that the bolts holding the stringer bearings were frequently fractured and/or loose indicating that they may have been highly stressed. Stress in the bolts could occur due to shear and as well as tension as the deck worked with the floor trusses. When the bolts were not effective at a particular location, the support there would cease to function in shear and to resist any uplift; it would resist downward force. Of course the support points were preloaded in compression with dead load. The deck and stringers at the floor trusses at Panel Points U4, U8, U14, U8′, and U4′ had expansion joints. There were generally double stringer bearings at these locations as shown in Figure 7. The interior reaction for a uniform load on a continuous four-span beam with equal spans is 0.928wL, where w = the uniform load and L = the span. This would be the case at supports without joints. The total end reaction for cases with a joint would be the sum of the reactions at the two end supports of adjacent uniformly loaded continuous equal-span units. Thus, the total reaction on the floor truss at an expansion joint is 2 x (0.393wL) = 0.786wL. The deck weight per foot per interior stringer, including the deck haunch, is computed as follows:

$$w = [(6.5/12) \times 8.2 + (2 \times 12)/144] \times 0.150 = 0.69 \text{ k/ft}$$

The average stringer reaction due to the deck weight is 0.69 x 38 = 26.2 k Based on the above, the reactions due to this load at the floor trusses at Panel Points U10 and U14 may be computed as follows:

0.928 x 0.69 x 38 = 24.3 k at floor trusses having continuous stringers such as at U10
0.786 x 0.69 x 38 = 20.6 k at floor trusses having simply supported stringers such as at U14

The coefficient for reactions immediately adjacent to simple supports is approximately 1.15, which adjusts for the lower reaction at simple supports. The stringer reaction due to deck weight adjacent to the simple supports is computed as follows:

1.15 x 0.69 x 38 = 30.1 k at interior floor trusses adjacent to floor trusses such as at U14

The above logic is limited in this case since the floor trusses provided varying stiffnesses under each stringer causing a redistribution of the stringer reactions. The largest stringer reactions were at the main trusses where the support stiffness was the greatest. In cases where the stringers were six-span continuous units (e.g. the stringers supporting the U8-U14 and U14-U8′), the above coefficients are slightly different, but the differences are not considered significant for this illustration.

Fatigue is a more critical concern with regard to the floor trusses since live load produced a greater portion of their design loads and the floor-truss members had welded connections. The floor trusses were fabricated by welding chord and diagonal members to gusset plates. The 1961 AASHO provisions accounted for fatigue according to the provisions of the alternating stress article described earlier. The 1966 AWS addressed fatigue of welded bridge steel. The AWS provisions permitted higher stress levels in fatigue than believed prudent at the present time. By their absence in the Notes, the AWS provisions for fatigue may not have been used in the design of the bridge. According to *AWS Specifications for Welded Highway and Railway Bridges, 1966*, the allowable fatigue stress of base metal connected by a filet weld over a loaded length of less than 100 feet for over 2 million cycles of stress was to be computed as follows:

$$f_a = 7500/[1 - 2R/3]$$

where: R = the ratio of the minimum stress to the maximum stress.

Table 24 gives diagonal forces in the floor truss at U10 with deck and stringers not effective. A check of the permitted fatigue stress based on the above equation is made as follows: In diagonal member 7447, Table 24, the minimum computed stress was zero, giving R=0. Hence, the allowable fatigue stress would have been 7500/[1 – (2/3)0] = 7500 psi. In effect, the member, 7447, would have been overstressed by 6 ksi, or 80 percent.

The above formula is no longer used. Instead, the stress range concept has been adopted. The stress range is computed as the difference between the largest and smallest live load plus impact stress. Fatigue need not be checked if the member undergoes only a net compression. In order to make a comparison with present-day (AASHTO LRFD) fatigue provisions, another set of stress ranges was computed for the factored fatigue design live load given in the AASHTO LRFD Bridge Design Specifications. The stress range specified in the LRFD provisions is due to the passage of a single HS15 truck with a fixed 30-foot rear-axle spacing and 15 percent impact. To check these provisions, stress ranges due to the LRFD fatigue vehicle were computed for the members in the floor trusses at U10 and U14 assuming that the deck and stringers were effective. Computed stress ranges for this load are reported in green in the last column of Tables 25 and 26 for chords and diagonals, respectively in the floor truss at Panel Point U10, and in Tables 28 and 29 for chords, and diagonals, respectively, in the floor truss at Panel Point U14. For cases where the total stress remains in compression throughout the stress cycle, propagation of fatigue cracks is unlikely; these cases are identified in the tables with the stress range in parentheses followed by a “c”. Article C6.6.1.2.1 of the AASHTO LRFD Specifications discusses the issue of stress range in transverse members such as cross-frames; this discussion would also apply to floor trusses. A cycle of stress is often caused by the truck placed in two different transverse positions. As a result, a stress range for these cases is caused by two different trucks and should not be considered a typical fatigue stress cycle. A factor of 0.75 is permitted to be applied to the stress range so computed for this case. However, this factor was conservatively not applied to the reported green stress range values.

The 4th Edition of AASHTO LRFD Specifications give a constant amplitude fatigue threshold value for Category E details of 4.5 ksi. The Specifications suggest that if two times the computed stress range for the fatigue load described above is less than the constant amplitude fatigue threshold, fatigue is unlikely to occur. Hence, when the stress range reported in these tables is less than 2.25 ksi, the propagation of fatigue cracks would not be expected.

Investigation of Compositeness of Deck System at Panel Point U10

Two cases of floor truss U10 will be investigated to determine the effect of ignoring the compositeness of the stringers and deck. Panel Point U10 was selected because the stringers and deck were continuous across it. In the first case the deck, stringers and lateral bracing were not considered. The floor truss forces for this case are reported in Tables 23 and 24. That is, loads were applied directly to the bearing points on the floor truss rather than to the stringers or deck. These reported forces are compared to the corresponding values given on Sheet 22 of the Plans. The columns in the tables titled "Member ID" identify the location of the members in the floor truss shown in Figure 9. Note that members 16 and 29 are the upper vertical members of the main trusses. The column titled "Element Number" gives the element number from the finite element model. Since the members of the floor trusses had welded connections, the gross area was used to compute stresses. An exception existed to account for the riveted (bolted) splices in the top chords in members 3 and 11. Although the analyses were made using the gross area of the larger member, the net area for the smaller 12-inch section in these elements is reported and used to compute stresses in Tables 23, 25, and 28. The net area of the smaller section at the splice is computed as follows:

$$15.55 \text{ in}^2 - [1"] \times [4 \times 0.575" + 3 \times 0.345"] = 12.26 \text{ in}^2.$$

Separate results are reported for steel weight; deck weight, and rail weight as well as the total dead load. Live load results including 30 percent impact are given in the next column. The largest live load tensile force is reported on top; the largest live load compression force is reported below in each cell. The sum of dead, plus live, plus impact force is reported in the next column. The next column gives comparable results from the Plans. The next column gives the ratio of the 3D sum to the sum from the Plans. The next column reports the maximum and minimum stress based on the 3D analysis. The last column gives the stress range.

For this case, the computed forces in the top and bottom chord members reported in the Plans were larger than from the 3D analysis. The forces in the diagonal members generally agreed quite well between the Plans and the 3D analyses although the 3D analyses showed diagonal members as much as 7 percent higher than the Plans.

A second analysis of the same floor truss was performed with the stringers, deck and lateral bracing assumed effective. In this case, the stringers, diaphragms, lateral bracing and the hardened deck were made to act compositely with the floor truss. All stringers were laterally restrained at the bearings. Loads were applied to the stringers and deck rather than to the floor truss. Chord and diagonal forces from this second analysis are compared to the forces from the Plans in Tables 25 and 26. In this case, the forces from the 3D analysis were all lower than the forces from the Plans in every case.

The differences between the analysis results for the cases with the deck, stringers and bracing effective and not effective were not large for the noncomposite dead load cases because the only difference between the two cases was the deck was not effective. The stringers and the diaphragms between them effectively stiffened the floor truss; the stringers acted in the longitudinal direction to distribute load away from the floor truss. The differences between the two cases were more significant for the superimposed dead load case, Stage 7, than for Stages 1 and 6 because the deck was effective. In the first case, the rail weight was distributed equally to all stringer reactions. In the second case, the rail loads were applied near the edges of the decks. The assumed composite action of the stringers, bracing, diaphragms and deck along with the different position assumed for the loads caused a reduction in the top chord forces reported in Table 25 compared to those reported in Table 23. In no cases were the total forces from the 3D analysis reported in Tables 25 and 26 assuming the deck and bracing interaction was present greater than the forces given in the Plans. When the bolts were not effective in shear, the behavior of the floor truss would tend to be similar to the case where the deck and lateral bracing interaction was not considered. However the stringers would still distribute some load away from the floor truss.

When live load was placed on the northbound roadway, the stringers of the southbound roadway would tend to have resisted the deflection of the floor truss and vice versa. Potentially, the stringer bearings may have experienced uplift. The stringer bearings were modeled with stiff beam elements; the axial forces in these elements are the reported reactions given in Table 27. It was observed that these reactions were far from equal in spite of nearly equal loads applied to the deck over each stringer. The stringer reactions at the main trusses (Stringers 3 and 12) were much higher than the other reactions. Also, the stringer reaction near the center of the floor truss span received a relatively large tension live load force, as expected. However, at no time did any of the reactions experience a net uplift.

In the Stage 6 analysis the assumption was made that both sides of the concrete deck were cast at the same time. During construction, one side of the deck would have been cast first. This possibly could have caused uplift on the stringers of the other roadway. Once the roadway cast first had hardened and the second roadway was cast, uplift would have been more likely to occur when the deck on the one side was stiffer resisting deflection of the floor truss. That may have been the reason that the bearings at the continuous stringer locations had tie-down devices.

Investigation of Stringer Bearings at Panel Point U14

The deck and stringers are discontinuous at Panel Point U14. The joint in the deck and stringers at Panel Point U14 was recognized in the model by the introduction of double nodes in the stringers and the deck and the use of separate reactions at each end of the stringers. These deck joints had 16WF36 diaphragms in all bays on each side of the joints to support the edges of the deck. S&P employed rather sophisticated bearing constraint arrangements. These bearing restraints were recognized in the model by utilizing beam elements with appropriate longitudinal and lateral restraints. Both of the reaction beam elements on each side of the joint were extremely stiff in flexure, and each had a cross-sectional area of 72 square inches. The double bearing arrangement permitted live load to be applied on each side of the joint independently.

The bearing constraints at the joint at Panel Point U14 were different on the north and south side of the joint. On the south side, the bearings were fixed longitudinally, except over the main trusses where bearings at Stringers 3 and 12 were free longitudinally. On the north side the bearings were all free longitudinally. All bearings on both sides of the joint were fixed transversely. The transverse restraint caused the stringers, diaphragms, and deck to work with the floor truss.

Computed forces in the chords and diagonals of the floor truss at Panel Point U14 for dead and live load are given in Tables 28 and 29, respectively. Both sides of the joint received dead load. The live load was applied to maximize the floor truss actions. The forces in the chords from the 3D analysis were much less than the forces given on the Plans and lower than those computed for the floor truss at Panel Point U10. The 3D forces in the floor truss chords were lower than those given in the Plans or those computed for the floor truss at Panel Point U10. The reason for this is that the diaphragms at Panel Point U14 were continuous, larger and there were two lines of them; one on each side of the joint.

To confirm the effectiveness of the deck and stringer system, an equilibrium check of the forces in the floor truss chord members for Stage 7 loads can be made. Equilibrium can be obtained only when the horizontal shear in the stringer bearing bolts is considered. This effect will be discussed under Stringer Horizontal Reactions below.

The forces in the diagonals from the 3D analysis were closer to those reported on the Plans than were the chord forces. The reason for this is that vertical load was resisted almost entirely in the diagonals, whereas the chord forces were shared between the top chords of the floor truss and the stringer, diaphragm, bracing and deck. There were two diagonals where the computed 3D forces were slightly larger than the comparable forces given on the Plans (4 and 5 percent larger as shown in Table 29). The reason for this is that the loads were redistributed by the deck and stringers to those diagonals.

Stringer Vertical Reactions

Although the individual reactions varied greatly due to the flexibility of the floor trusses, the total reaction at a floor truss should still be close to the total load that the floor truss must support. Comparing the sum of the reactions on the 14 stringers from the 3D analysis to 14 times the classical reaction computed for deck weight earlier for a single girder is a means to perform this check. The sum of the 14 stringer reactions at Panel Point U10 from the analysis is 345 kips. The comparable sum at Panel Point U14 is 321 kips, or 0.93 of that at Panel Point U10. The computed sum from the earlier discussion based on a classical analysis is as follows:

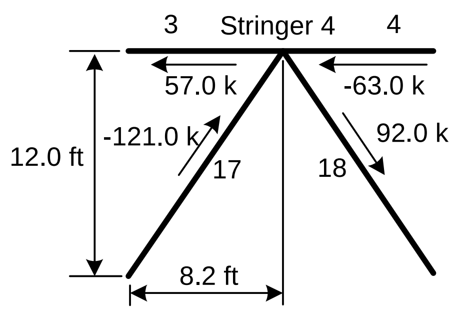
At Panel Point U10: 24.3 kips x 14 stringers = 340.2 kips
At Panel Point U14: 20.6 kips x 14 stringers = 288.4 kips.

The ratio of these values is 0.85. The reason that the total reaction at Panel Point U14 differs more from the classical value is that the elements representing the stringers were 19 feet long and the deck weight was applied as concentrated loads at the nodes, whereas, in fact, the deck load is uniformly distributed over the length. However, it can be concluded that the total stringer reactions are reasonable based on classical methods, even though the distribution of the loads to the individual stringers is complex.

Stringer Horizontal Reactions

Horizontal shear in the bearing bolts in the direction of the floor trusses can be deduced from the chord and diagonal forces in the floor trusses. This shear is created when the stringers and diaphragms work compositely with the floor truss. The small top chord forces given in Table 28 indicated that the stringer system was working with the floor truss. This was true whether or not the bearings permitted longitudinal movement since all were restrained transversely.

Figure 12 shows the forces from the 3D analysis in the top chord and diagonals of the floor truss at Stringer 4 due to the weight of the deck at Panel Point U10 with the stringers assumed not effective and the stringer load applied directly to the floor truss. This condition results in forces in the floor truss close to those given in the Plans. The forces are taken from Tables 23 and 24. Since the stringers were ignored in this analysis, the member forces alone must be in equilibrium without horizontal shear.



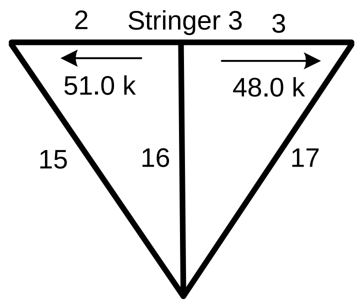
$$\text{Length of diagonal} = \sqrt{(8.2)^2 + (12.0)^2} = 14.53 \text{ ft}$$

$$121.0\left(\frac{8.2}{14.53}\right) + 92.0\left(\frac{8.2}{14.53}\right) = 120.0 \text{ k}$$

$$\text{Horizontal Shear at Stringer 4} = 120.0 \text{ k} - 57.0 \text{ k} - 63.0 \text{ k} = 0.0 \text{ k}$$

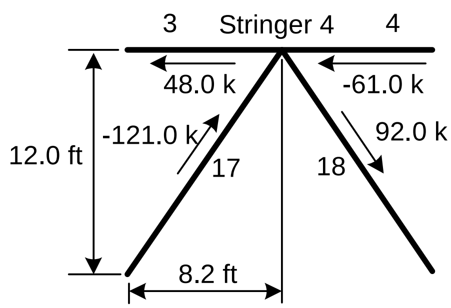
Figure 12 Chord Forces at Stringer 4 at U10 due to Deck Weight with Stringers Not Effective

Figures 13 and 14 show similar forces for the floor truss at Panel Point U10 at Stringers 3 and 4, respectively, assuming the stringers are acting compositely with the floor truss. Forces are taken from Tables 25 and 26. The horizontal shear at Stringers 3 and 4 leads to composite action between the stringers and the floor truss that causes a reduction in the upper chord forces compared to those shown in Figure 12. Figure 6 shows that the diaphragms at the floor trusses are discontinuous at Panel Point U10, so they are not as effective as they are at points where there is a transverse joint and the diaphragms are continuous such as at Panel Point U14.



$$\text{Horizontal Shear at Stringer 3} = 51.0\text{ k} - 48.0\text{ k} = 3.0\text{ k}$$

Figure 13 Chord Forces at Stringer 3 at U10 due to Deck Weight with Stringers Effective

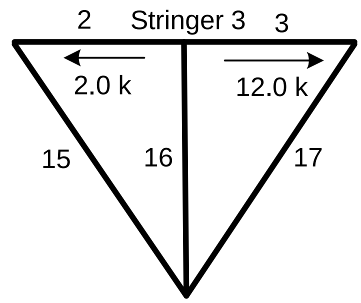


$$121.0\left(\frac{8.2}{14.53}\right) + 92.0\left(\frac{8.2}{14.53}\right) = 120.0\text{ k}$$

$$\text{Horizontal Shear at Stringer 4} = 120.0\text{ k} - 48.0\text{ k} - 61.0\text{ k} = 11.0\text{ k}$$

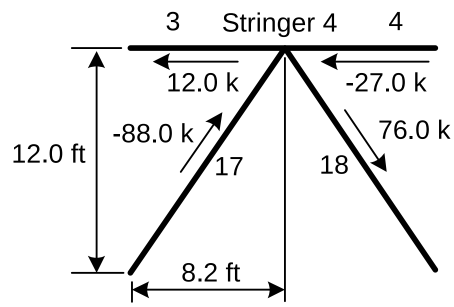
Figure 14 Chord Forces at Stringer 4 at U10 due to Deck Weight with Stringers Effective

Figures 15 and 16 show similar forces for the floor truss at Panel Point U14 at Stringers 3 and 4, respectively. Forces are taken from Tables 28 and 29. These forces were computed assuming that the stringers were effective. Forces are in equilibrium when the horizontal shear force between the floor truss and the stringers is introduced. The shear at Stringers 3 and 4 leads to composite action that causes a further reduction in the upper chord forces compared to those in Figures 13 and 14. Figure 6 shows that the diaphragms at the floor trusses are continuous at Panel Point U14. Hence, the stringer system is much stiffer at Panel Point U14 than at Panel Point U10, creating a larger effective first moment due to the stringers and diaphragms. Therefore, horizontal shears at the bearings are further increased.



$$\text{Horizontal Shear at Stringer } 3 = 12.0 \text{ k} - 2.0 \text{ k} = 10.0 \text{ k}$$

Figure 15 Chord Forces at Stringer 3 at U14 due to Deck Weight with Stringers Effective



$$88.0\left(\frac{8.2}{14.53}\right) + 76.0\left(\frac{8.2}{14.53}\right) = 93.0 \text{ k}$$

$$\text{Horizontal Shear at Stringer } 4 = 93.0 \text{ k} - 12.0 \text{ k} - 27.0 \text{ k} = 54.0 \text{ k}$$

Figure 16 Chord Forces at Stringer 4 at U14 due to Deck Weight with Stringers Effective

Stringer Vertical Reactions at Deck Joints

Tables 30-33 report vertical reactions at the stringers at Panel Point U14 from the 3D analysis. Stringer 1 is on the exterior of the northbound lanes. Table 30 gives reactions on the north side of Panel Point U14 with live load applied on the north side of the joint. Hence, the downward (negative) live load reactions were large. Only reactions due to trucks are reported since fatigue is of interest and uniform live load occurs rarely. It was observed that there was little difference between the maximum reactions due to lane loads and truck loads. The single truck reactions and the critical reactions are given. The “Critical” values are the maximum reactions due to more than one lane of truck loading considering multiple presence. Clearly, a single truck causes much of the total live load reaction. It is of interest to note that the largest reactions were at the main trusses. This occurred because of the relative flexibility of the floor trusses compared to the main truss.

Table 31 gives stringer reactions on the north side of Panel Point U14 with live load applied on the south side of the joint. The dead load reactions are the same as in Table 30 since they are the sum of the reactions on the north and south side of the joint. The live load reactions were upward. The reason for this is that the live load tended to pull the floor truss away from the stringers. At Stringers 2, 4, 11, and 13, the net upward reactions could have potentially exceeded the dead load reactions and put the bearing bolts in tension. Figures 7 and 8 showed the detail of the stringer bearings at the deck joint at Panel Point U14. As mentioned previously, it appears that by the detailing of the stringer bearings, the south side of the deck restrained the north side from uplift at some stringers.

Tables 32 and 33 give stringer reactions on the south side of the joint at Panel Point U14 for live load applied on the south side and on the north side, respectively. Load was transferred to the bearings at the main trusses from adjacent stringers through shear. Table 33 shows the large net uplift forces (shown in red) that could have potentially occurred in the bearings near the main trusses.

Comparison of Field Observations to Predicted Behavior

Inspection reports indicated that the bolts holding the stringer bearings were a continuing maintenance issue. These bolts were found loose or missing and were replaced regularly. A June 2006 Bridge Inspection Report prepared by the Maintenance Operations, Bridge Inspection unit of the MnDOT, Metro Division described the following issues related to stringer-to-floor truss connection bolts that were noted at various inspection intervals:

Panel Point 4 - Bolt replaced at Stringer 10.

Panel Point 8 - One bolt broken at Stringer 4 south floorbeam connection; bearing block rotated at Stringer 2.

Panel Point 11 - Two bolts missing at Stringer 3. Stringer 11 has three bolts replaced.

Panel Point 10' - Stringer 13 has loose bolt.

Panel Point 8' - Bolts replaced with threaded rod at Stringer 4; bolts replaced at Stringer 5. Bolt replaced at Stringer 11.

Panel Point 6' - Stringer 4 connection "working" - SW bolt is loose. Stringer 9 has loose bolt. Stringer 10 has two loose bolts. Stringer 11 has one loose bolt.

Panel Point 3' - Stringer 12 has connection bolts "working".

Panel Point 2' - Stringer 11 has connection bolts "working".

The bridge was striped for four traffic lanes on the northbound and southbound roadways. Assuming 12-foot traffic-lane widths and 2-foot shoulders, it is interesting to note that the majority of these problems were noted on stringers (e.g. Stringers 2, 3, 4, 11, 12, and 13) that were closest to the main trusses and also located directly underneath the two lanes expected to experience the most truck traffic. It is evident that the stringer bearing bolt issue was most pronounced where the horizontal shears in these bolts were largest and not particularly related to uplift. As bolts loosened or fell out, shear would be transferred to other bolts and the floor truss would tend to work as originally assumed in design where the stringers and deck were presumably assumed not effective; i.e., member forces in the floor truss would be increased, particularly those in the top chords. However, as the shear capacity of the bearings was reduced, the likelihood of larger fatigue stresses would also be increased.

Conclusions

These conclusions relate only to the behavior of the as-designed bridge, including behavior due to some changes of the as-designed truss related to in-service conditions.

1. The bridge was designed according to the Working Stress Design method, which provides a factor of safety against first yield or elastic buckling of approximately 1.8 for nominal (unfactored) dead and live loads.
2. Although the main trusses were symmetrical about the longitudinal and transverse axes, the deck system was not symmetrical about either axis. Span 9 in the north approach structure was longer than Span 5 in the south approach structure. The roadway widened to the east at the north end and curved to the west at the south end. The S&P analysis recognized the effect of the difference in approach-span weights on the reported forces in the first few members at the ends of the main trusses. However, they reported the same member forces in-between Panel points U4 and U4′ and in-between Panel Points L5 and L5′. The present analysis treated differences in the framing and in the deck weight as well as the differences in the approach-span reactions on the truss. The largest differences in the two analyses can be attributed to the differences in the assumed live load contribution from the approach spans. The second most significant difference was the S&P use of the same member forces in the central portion of the truss, whereas the 3D analysis results included specific results for each member throughout the truss. The largest observed “overstress” in the main truss was in the Member U0′-L1′ in the northeast quadrant. The computed force in this member from the 3D analysis was found to be 12 percent larger than shown on the Plans. The 3D live load force was 40 percent greater, while the dead load force was approximately the same for both analyses. Slightly smaller “overstresses” were found for members in the vicinity of the south approach spans. The live load forces from the approach spans in the 3D case were based on wheel-load distribution factors. The basis of the S&P analysis for these effects was unknown. A 12 percent “overstress” was not considered worthy of refined investigation. Several “overstresses” in the central portion of the main truss were identified, but considered insignificant.

3. Fatigue was not addressed directly in the 1961 AASHO Bridge Specifications. Article 1.6.5 *ALTERNATING STRESSES* was employed to increase the design force in members subjected to stress reversal. In main truss members in which S&P found stress reversal, the minimum as well as maximum live load forces were given in the Plans. In those cases, a ratio of the computed force from the 3D analysis to the force given in the Plans was included. Due to the small forces in these cases, the ratios appear sometimes large and even with opposite sign at times. In these cases, the reader should look at the magnitude of the force for a more meaningful comparison. The method of reporting main truss member forces herein includes more information than that provided on the Plans, particularly for the main truss verticals where the Plans called out an apparent arbitrary force in most vertical truss members.
4. The present AASHTO LRFD fatigue provisions utilize a live load of a single HS-15 truck with a 30-foot fixed rear-axle spacing plus 15 percent impact. The LRFD fatigue check did not identify any critical main truss members. There were members in the floor trusses that had a computed stress range greater than the 2.25 threshold for the LRFD fatigue vehicle.
5. The floor truss at Panel Point U10 was investigated with and without the deck, bracing and stringers assumed effective with the floor truss. This floor truss supported continuous stringers and deck. The analysis without the deck and bracing interaction was likely the assumption used by S&P. Results for this floor truss were close between the results reported on the Plans and from the 3D analysis assuming no deck and bracing interaction; the largest ratio being 1.07 in a diagonal member. The floor truss at Panel Point U14 was investigated assuming the stringers, bracing and deck were effective with the floor truss. The deck and stringers were discontinuous at this floor truss. No "overstresses" were found in either floor truss when the deck and bracing interaction was assumed present.
6. The horizontal shear in the connections between the stringers and deck was found to be significant when the deck, bracing and stringers were effective with the floor truss. Transverse shear due to deck weight was found to be 54 kips at Stringer 4 in the floor truss at Panel Point U14. This shear was additive to the longitudinal shear. All gravity loads contributed to this shear. It should be noted that this shear was at the end of a stringer at a transverse deck joint where S&P required shear connectors. Most certainly S&P recognized the shear at these locations. Significant transverse shear also was seen to occur at Panel Point U10. The possibility of tension in the bolts connecting the stringers to the floor truss was investigated by determining reactions under the stringers. The possibility of stress reversal was identified at Panel Point U14 where the live load uplift forces could possibly have overcome the dead load. The Plans showed that hold-down devices were incorporated at floor truss locations where the stringers were continuous. A net tie-down force as large as 10 kips was found to be possible at deck joints where tie-downs

were not provided. However, shear seemed to be the more significant force in the bolts attaching the stingers to the floor trusses.

7. This report confirms the S&P analyses for dead and live load design forces. If the bridge were loaded and functioned as designed, no significant overstresses would be expected in the main truss members and floor truss members.

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Appendix

Member ID	Element Number	Net Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0-U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL / Plans LL	Ratio 3D Total / Plans	Total 3D Force (Kips)	3D Total Stress (Ksi)
U0-U1	8486	44.4	98	224	69	391	439	0.89			-15		-3			373	8.40
		44.4				391	439		244	51	206	100	73	1.28	1.05	770	17.34
U1-U2	8210	44.4	97	225	69	391	439	0.89			-30		-6			355	8.00
		44.4				391	439		244	51	203	101	73	1.28	1.05	768	17.30
U2-U3	8212	71.0	-51	-158	-3	-212	-226	0.94	-513	-67	-457		-60	0.89	0.90	-729	-10.27
		57.5				-212	-226		437	39	309	68	48	0.89	0.85	213	3.70
U3-U4	8214	71.0	-50	-156	-3	-209	-226	0.92	-513	-67	-455		-59	0.89	0.90	-723	-10.18
		57.5				-209	-226		437	39	309	67	48	0.89	0.86	215	3.74
U4-U5	8216	41.6	194	242	51	487	516	0.94	-443	-58	-381		-50	0.86	3.73	56	1.35
		41.6				487	516		536	48	433	30	48	0.88	0.91	998	23.99
U5-U6	8218	41.6	193	241	51	485	516	0.94	-443	-58	-382		-50	0.86	3.53	53	1.27
		41.6				485	516		536	48	434	29	48	0.88	0.91	996	23.94
U6-U7	8220	91.4	630	987	167	1784	1762	1.01			-120		-13			1651	18.06
		91.4				1784	1762		607	67	568	2	63	0.94	0.99	2417	26.44
U7-U8	8222	91.4	633	993	167	1793	1762	1.02			-112		-12			1669	18.26
		91.4				1793	1762		607	67	567	2	63	0.94	1.00	2425	26.53
U8-U9	8224	82.6	542	883	145	1570	1551	1.01			-86	-10	-11			1463	17.71
		82.6				1570	1551		537	59	488		54	0.91	0.98	2112	25.57
U9-U10	8226	82.6	540	879	145	1564	1551	1.01			-87	-10	-13			1454	17.60
		82.6				1564	1551		537	59	490		54	0.91	0.98	2108	25.52
U10-U11	8228	71.0	-178	-239	-46	-463	-486	0.95	-402	-36	-360	-16	-37	0.94	0.95	-876	-12.34
		71.0				-463	-486				223		20			-220	-3.10
U11-U12	8230	71.0	-178	-241	-45	-464	-486	0.95	-402	-36	-358	-16	-37	0.94	0.95	-875	-12.32
		71.0				-464	-486				230		21			-213	-3.00
U12-U13	8232	135.9	-682	-1033	-181	-1896	-1899	1.00	-817	-74	-753	-22	-75	0.95	0.98	-2746	-20.21
		135.9				-1896	-1899				228		21			-1647	-12.12
U13-U14	8234	135.9	-651	-982	-173	-1806	-1899	0.95	-817	-74	-716	-21	-71	0.91	0.94	-2614	-19.24
		135.9				-1806	-1899				219		20			-1567	-11.53

Table 3 Southwest Upper Chord Forces

Member ID	Element Number	Net Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0-U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL / Plans LL	Ratio 3D Total / Plans	Total 3D Force (Kips)	3D Total Stress (Ksi)
U0-L1	8488	51.3	-129	-294	-89	-512	-560	0.91	-311	-65	-273	-130	-96	1.33	1.08	-1011	-19.73
		51.3				-512	-560			-65	18					-494	-9.63
L1-L2	8262	46.0	32	76	-15	93	80	1.16	-345	-31	-235	-85	-47	0.98	0.92	-274	-5.95
		36.7				93	80		333	43	303		39	0.91	0.95	435	11.86
L2-L3	8264	46.0	33	77	-15	95	80	1.19	-345	-31	-234	-85	-47	0.97	0.91	-271	-5.88
		36.7				95	80		333	43	303		39	0.91	0.96	437	11.91
L3-L4	8266	62.5	-34	24	-12	-22	-18	1.22	-490	-44	-380	-48	-49	0.89	0.90	-499	-7.98
		49.8				-22	-18		572	74	486		63	0.85	0.84	527	10.58
L4-L5	8268	62.5	-35	22	-13	-26	-18	1.44	-490	-44	-381	-48	-49	0.89	0.91	-504	-8.06
		49.8				-26	-18		572	74	487		63	0.85	0.83	524	10.52
L5-L6	8270	83.5	-398	-597	-105	-1100	-1087	1.01	-539	-70	-492	-15	-68	0.94	0.99	-1675	-20.06
		83.5				-1100	-1087				252		33			-815	-9.76
L6-L7	8272	83.5	-398	-597	-105	-1100	-1087	1.01	-539	-70	-491	-15	-68	0.94	0.99	-1674	-20.05
		83.5				-1100	-1087				249		32			-819	-9.81
L7-L8	8274	166.5	-896	-1412	-236	-2544	-2533	1.00	-787	-87	-743		-82	0.94	0.99	-3369	-20.24
		166.5				-2544	-2533				84	7	11			-2442	-14.67
L8-L9	8276	166.5	-899	-1416	-237	-2552	-2543	1.00	-790	-87	-754		-83	0.95	0.99	-3389	-20.35
		166.5				-2552	-2543				82	7	11			-2452	-14.73
L9-L10	8278	62.5	-171	-310	-48	-529	-559	0.95	-324	-36	-310		-34	0.96	0.95	-873	-13.98
		62.5				-529	-559				180	13	24			-312	-4.99
L10-L11	8280	62.5	-170	-309	-48	-527	-559	0.94	-324	-36	-326		-36	1.01	0.97	-889	-14.22
		62.5				-527	-559				199	13	26			-289	-4.62
L11-L12	8282	77.6	480	718	127	1325	1311	1.01			-228		-21			1076	13.87
		77.6				1325	1311		642	58	589	19	59	0.95	0.99	1992	25.67
L12-L13	8284	77.6	482	720	128	1330	1311	1.01			-229		-21			1080	13.92
		77.6				1330	1311		642	58	590	19	59	0.95	0.99	1998	25.75
L13-L14	8286	109.9	719	1085	191	1995	2036	0.98			-214		-19			1762	16.03
		109.9				1995	2036		861	78	780	24	78	0.94	0.97	2877	26.18

Table 4 Southwest Lower Chord Forces

Member ID	Element Number	Net Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Tot (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0-U0' (Kips)	Imp 3D (Kips)	Ratio 3D LL / Plans LL	Ratio 3D Total / Plans	Total 3D Force (Kips)	3D Total Stress (Ksi)
L1-U2	8314	64.0	-165	-384	-69	-618	-662	0.93	-462	-60	-399	-20	-58	0.91	0.92	-1033	-16.14
		64.0				-618	-662				175		23			-439	-6.86
U2-L3	8316	27.7	24	107	24	155	192	0.81	-217	-24	-175		-19	0.81	0.80	-16	-0.58
		25.2				155	192		325	43	240	23	39	0.82	0.82	422	16.77
L3-U4	8318	22.9	111	177	21	309	321	0.96			-141	-26	-24			146	6.38
		22.9				309	321		259	29	218		24	0.84	0.90	531	23.21
U4-L5	8320	55.0	-228	-377	-55	-660	-640	1.03	-331	-36	-308		-33	0.93	0.99	-964	-17.53
		55.0				-660	-640				75	26	8			-555	-10.09
L5-U6	8322	47.6	288	506	78	872	883	0.99			-44	-22	-11			810	17.02
		47.6				872	883		344	38	319		35	0.93	0.97	1195	25.12
U6-L7	8324	91.8	-404	-679	-109	-1192	-1174	1.02	-415	-46	-419		-46	1.01	1.01	-1607	-17.51
		91.8				-1192	-1174				41	21	6			-1126	-12.27
L7-U8	8326	61.6	409	654	106	1169	1216	0.96			-39	-15	-9			1119	18.17
		61.6				1169	1216		388	43	388		43	1.00	0.97	1561	25.34
U8-L9	8328	77.0	562	839	144	1545	1560	0.99			-50		-5			1499	19.47
		77.0				1545	1560		476	52	480	5	2	0.92	0.97	2034	26.42
L9-U10	8330	125.8	-619	-954	-163	-1736	-1680	1.03	-548	-60	-558	-5	-63	1.03	1.03	-2295	-18.25
		125.8				-1736	-1680				54		6			-1678	-13.34
U10-L11	8332	73.4	506	797	135	1438	1432	1.00			-57		-6			1385	18.87
		73.4				1438	1432		489	54	465	5	2	0.87	0.97	1912	26.06
L11-U12	8334	91.8	-429	-679	-117	-1225	-1215	1.01	-459	-51	-442	-4	-50	0.97	1.00	-1667	-18.17
		91.8				-1225	-1215				78		9			-1143	-12.45
U12-L13	8336	47.6	282	442	75	799	834	0.96			-113		-12			690	14.50
		47.6				799	834		380	42	336	4	1	0.81	0.91	1143	24.02
L13-U14	8338	41.8	-86	-124	-24	-234	-214	1.09	-285	-31	-261	-3	-29	0.93	0.99	-494	-11.83
		41.8				-234	-214		213	24	176		20	0.83	-1.66	-54	-1.29

Member ID	Element Number	Net Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0-U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL / Plans LL	Ratio 3D Total / Plans	Total 3D Force (Kips)	3D Total Stress (Kips)
U1-L1	7186	32.5	-19	-34	-4	-57	-323	0.18	-207	-62	-50	-3	-16	0.26	0.21	-126	-3.88
		32.5				-57	-323				2		1			-54	-1.66
U2-L2	7214	18.0	55	133	23	211	266	0.79			-7	-1	-2			201	11.17
		18.0				211	266		207	62	79		24	0.38	0.59	314	17.44
U3-L3	7242	32.5	-19	-28	-1	-48	-320	0.15	-207	-62	-50		-15	0.24	0.19	-113	-3.48
		32.5				-48	-320				6		2			-40	-1.23
U4-L4	7270	18.0	51	98	21	170	234	0.73			-8	-1	-3			158	8.78
		18.0				170	234		207	62	86		26	0.42	0.56	282	15.65
U5-L5	7298	33.8	-18	-30	-2	-50	-318	0.16	-207	-62	-52		-16	0.25	0.20	-118	-3.48
		33.8				-50	-318				6		2			-42	-1.24
U6-L6	7326	18.0	72	141	25	238	275	0.87			-10	-1	-3			224	12.44
		18.0				238	275		207	62	90		27	0.43	0.65	355	19.72
U7-L7	7354	39.3	-23	-26	-1	-50	-333	0.15	-207	-62	-55		-16	0.27	0.20	-121	-3.09
		39.3				-50	-333				5		1			-44	-1.12
U8-L8	7382	179.2	-828	-1230	-197	-2255	-2527	0.89	-714	-79	-637		-70	0.89	0.89	-2962	-16.53
		179.2				-2255	-2527				45	9	8			-2193	-12.24
U9-L9	7410	39.3	-21	-24	-1	-46	-331	0.14	-207	-62	-53		-16	0.26	0.19	-115	-2.93
		39.3				-46	-331				5		1			-40	-1.02
U10-L10	7438	18.0	78	150	27	255	271	0.94			-10		-3			242	13.44
		18.0				255	271		207	62	94		28	0.45	0.70	377	20.95
U11-L11	7466	32.5	-19	-23		-42	-269	0.16	-207	-62	-51		-15	0.25	0.20	-108	-3.33
		32.5				-42	-269				7		2			-33	-1.02
U12-L12	7494	18.0	69	146	26	241	270	0.89			-5		-1			235	13.06
		18.0				241	270		207	62	86		26	0.42	0.65	353	19.60
U13-L13	7522	32.5	-25	-22	1	-46	-330	0.14	-207	-62	-50		-15	0.24	0.19	-111	-3.41
		32.5				-46	-330				6		2			-38	-1.17
U14-L14	7550	18.0	64	109	24	197	244	0.81			-2		-1			194	10.78
		18.0				197	244		207	62	86		26	0.42	0.60	309	17.17

Table 6 Southwest Upper Vertical Forces

Member ID	Element Number	Net Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0-U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL / Plans LL	Ratio 3D Total / Plans	Total 3D Force (Kips)	3D Total Stress (Ksi)
U1-L1	7188	32.5	-76	-216	-41	-333	-323	1.03	-207	-62	-190	-10	-60	0.97	1.00	-593	-18.246
		32.5				-333	-323				29		9			-295	-9.08
U2-L2	7216	22.5	13	-1	0	12	266	0.05			-2		-1			9	
		18.0				12	266		207	62	1		0	0.00	0.02	13	
U3-L3	7244	32.5	-70	-186	-30	-286	-320	0.89	-207	-62	-139		-42	0.67	0.79	-467	-14.369
		32.5				-286	-320				12		4			-270	-8.31
U4-L4	7272	22.5	16	-1	0	15	234	0.06			-3		-1			11	
		18.0				15	234		207	62	1		0	0.00	0.03	16	
U5-L5	7300	33.8	-71	-190	-32	-293	-318	0.92	-207	-62	-145	-1	-44	0.71	0.82	-483	-14.31
		33.8				-293	-318				11		-3			-285	-8.43
U6-L6	7328	22.5	20	2	0	22	275	0.08			-1		0			21	
		18.0				22	275		207	62	1		0	0.00	0.04	23	
U7-L7	7356	39.3	-84	-187	-31	-302	-333	0.91	-207	-62	-152		-46	0.74	0.83	-500	-12.739
		39.3				-302	-333				16		5			-281	-7.15
U8-L8	7384	179.2	-923	-1363	-227	-2513	-2527	0.99	-714	-79	-740		-82	1.04	1.00	-3335	-18.612
		179.2				-2513	-2527				68	9	10			-2426	-13.538
U9-L9	7412	39.3	-82	-187	-32	-301	-331	0.91	-207	-62	-155		-46	0.75	0.84	-502	-12.79
		39.3				-301	-331				15		4			-282	-7.18
U10-L10	7440	22.5	18	2	0	20	271	0.07			-1		0			19	
		18.0				20	271		207	62	0		0	0.00	0.04	20	
U11-L11	7468	32.5	-66	-167	-26	-259	-269	0.96	-207	-62	-146		-44	0.71	0.83	-449	-13.815
		32.5				-259	-269				15		4			-240	-7.38
U12-L12	7496	22.5	20	-1	0	18	270	0.07			-2		-1			15	
		18.0				18	270		207	62	1		0	0.00	0.04	19	
U13-L13	7524	32.5	-75	-181	-29	-285	-330	0.86	-207	-62	-146		-44	0.71	0.79	-475	-14.615
		32.5				-285	-330				9		3			-273	-8.40
U14-L14	7552	22.5	21	-3	-1	17	244	0.07			-4		-1			12	
		18.0				17	244		207	62	0		0	0.00	0.03	17	

Table 7 Southwest Lower Vertical Forces

Member ID	Element Number	Net Area (In2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0&U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL / Plans LL	Ratio 3D Total / Plans	Total 3D Force (Kips)	3DTotal Stress (Ksi)
U0'-U1'	8492	59.1	218	410	150	778	796	0.98			-18		-3			757	12.81
		59.1				778	796		309	53	207	194	94	1.37	1.10	1273	21.54
U1'-U2'	8260	59.1	217	407	150	774	796	0.97			-31		-5			738	12.49
		59.1				774	796		309	53	202	193	93	1.35	1.09	1262	21.35
U2'-U3'	8258	71.0	14	-56	42	0	-31	0.00	-513	-67	-460		-60	0.90	0.85	-520	-7.32
		57.5				0	-31		473	43	310	120	64	0.96	1.02	494	8.59
U3'-U4'	8256	71.0	16	-48	42	10	-31	-0.32	-513	-67	-454		-59	0.88	0.82	-503	-7.08
		57.5				10	-31		473	43	312	120	64	0.96	1.04	506	8.80
U4'-U5'	8254	41.6	214	291	67	572	516	1.11	-443	-58	-363		-48	0.82		161	3.87
		41.6				572	516		536	48	432	45	52	0.91	1.00	1101	26.47
U5'-U6'	8252	41.6	212	287	67	566	516	1.10	-443	-58	-364		-48	0.82		154	3.70
		41.6				566	516		536	48	431	45	52	0.90	0.99	1094	26.30
U6'-U7'	8250	91.4	617	967	158	1742	1762	0.99			-122	-6	-15			1599	17.49
		91.4				1742	1762		607	67	564		62	0.93	0.97	2368	25.91
U7'-U8'	8248	91.4	620	977	160	1757	1762	1.00			-113	-6	-14			1624	17.77
		91.4				1757	1762		607	67	566		62	0.93	0.98	2385	26.09
U8'-U9'	8246	82.6	521	864	134	1519	1551	0.98			-81	-25	-16			1397	16.91
		82.6				1519	1551		537	59	495		54	0.92	0.96	2068	25.04
U9'-U10'	8244	82.6	519	856	133	1508	1551	0.97			-82	-24	-16			1386	16.78
		82.6				1508	1551		537	59	498		55	0.93	0.96	2061	24.95
U10'-U11'	8242	71.0	-192	-253	-53	-498	-486	1.02	-402	-36	-355	-27	-40	0.96	1.00	-920	-12.96
		71.0				-498	-486				230		21			-247	-3.48
U11'-U12'	8240	71.0	-192	-269	-57	-518	-486	1.07	-402	-36	-361	-27	-40	0.98	1.02	-946	-13.32
		71.0				-518	-486				224		20			-274	-3.86
U12'-U13'	8238	135.9	-686	-1041	-185	-1912	-1899	1.01	-817	-74	-751	-26	-76	0.96	0.99	-2765	-20.35
		135.9				-1912	-1899				226		20			-1666	-12.26
U13'-U14	8236	135.9	-656	-993	-177	-1826	-1899	0.96	-817	-74	-719	-26	-73	0.92	0.95	-2644	-19.46
		135.9				-1826	-1899				217		20			-1589	-11.69

Table 8 Northwest Upper Chord Forces

Member ID	Element Number	Net Area (In2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL Plans	LL Plans (Kips)	Impact Plans (Kips)	3D LL (Kips)	LL U0&U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL Plans LL	Ratio 3D Total Plans	3D Total Force (Kips)	3DTotal Stress (Kips)
U0'-L1'	8494	79.0	-279	-524	-193	-996	-1014	0.98	-393	-67	-271	-249	-121	1.39	1.11	-1637	-20.72
		79.0				-996	-1014				22		4			-970	
L1'-L2'	8312	57.8	-59	-66	-78	-203	-190	1.07	-394	-35	-239	-159	-69	1.09	1.08	-670	-11.59
		45.9				-203	-190		333	43	311		40	0.93	0.80	148	3.22
L2'-L3'	8310	57.8	-58	-64	-78	-200	-190	1.05	-394	-35	-239	-158	-69	1.09	1.08	-666	-11.52
		45.9				-200	-190		333	43	311		40	0.93	0.81	151	3.29
L3'-L4;	8308	62.5	-74	-34	-40	-148	-137	1.08	-510	-46	-380	-82	-59	0.94	0.97	-669	-10.70
		49.8				-148	-137		572	74	495		64	0.87	0.81	411	8.25
L4'-L5'	8306	62.5	-74	-35	-40	-149	-137	1.09	-510	-46	-381	-81	-59	0.94	0.97	-670	-10.72
		49.8				-149	-137		572	74	495		64	0.87	0.81	410	8.23
L5'-L6'	8304	83.5	-399	-592	-106	-1097	-1087	1.01	-539	-70	-491	-18	-69	0.95	0.99	-1675	-20.06
		83.5				-1097	-1087				257		33			-807	-9.66
L6'-L7'	8302	83.5	-399	-593	-106	-1098	-1087	1.01	-539	-70	-491	-18	-69	0.95	0.99	-1676	-20.07
		83.5				-1098	-1087				255		33			-810	-9.70
L7'-L8'	8300	166.5	-872	-1365	-220	-2457	-2533	0.97	-787	-87	-736		-61	0.91	0.96	-3254	-19.54
		166.5				-2457	-2533				86	23	16			-2332	-14.01
L8'-L9'	8298	166.5	-874	-1369	-221	-2464	-2543	0.97	-790	-87	-746		-82	0.94	0.96	-3292	-19.77
		166.5				-2464	-2543				84	23	16			-2341	-14.06
L9'-L10'	8296	62.5	-154	-270	-34	-458	-559	0.82	-324	-36	-305		-34	0.94	0.87	-797	-12.75
		62.5				-458	-559				181	26	28			-223	-3.57
L10;-L11'	8294	62.5	-153	-268	-34	-455	-559	0.81	-324	-36	-320		-36	0.99	0.88	-811	-12.98
		62.5				-455	-559				199	26	30			-200	-3.20
L11'-L12'	8292	77.6	491	742	136	1369	1311	1.04			-224		-21			1124	14.48
		77.6				1369	1311		642	58	593	27	62	0.97	1.02	2051	26.43
L12'-L13'	8290	77.6	492	744	136	1372	1311	1.05			-224		-21			1127	14.52
		77.6				1372	1311		642	58	593	27	62	0.97	1.02	2054	26.47
L13'-L14	8288	109.9	719	1085	191	1995	2036	0.98			-214		-19			1762	16.03
		109.9				1995	2036		861	78	780	24	78	0.94	0.97	2877	26.18

Table 9 Northwest Lower Chord Forces

Member ID	Element Number	Net Area (in2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	Total DL (Kips)	DL Plans (Kips)	Ratio 3D DL Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0-U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL PlansLL	Ratio 3D Total Plans	Total 3D Force (Kips)	3D Total Stress (Kips)
L1'-U2'	8364	70.0	-204	-438	-93	-735	-771	0.95	-481	-63	-408	-47	-68	0.96	0.96	-1186	-16.94
		70.0				-735	-771			-63	179		23			-552	-7.89
U2'-L3'	8362	27.1	56	159	46	261	291	0.90		45	-171		-22			94	3.47
		27.1				261	291		343	45	237	48	45	0.85	0.87	550	20.30
L3'-U4'	8360	27.9	77	120	-3	194	220	0.88	-200	-26	-144	-53	-35	1.03	6.33	1	0.04
		22.9				194	220		258	29	218		25	0.85	0.86	416	18.18
U4'-L5'	8358	55.0	-197	-323	-32	-552	-640	0.86	-331	-36	-306		-33	0.92	0.88	-854	-15.53
		55.0				-552	-640			-36	76	51	24			-421	-7.65
L5'-U6'	8356	57.7	263	466	62	791	883	0.90		38	-45	-41	-17			709	12.29
		47.6				791	883		344	38	319		35	0.93	0.91	1114	23.41
U6'-L7'	8354	91.8	-381	-639	-92	-1112	-1174	0.95	-415	-46	-415		-46	1.00	0.96	-1523	-16.60
		91.8				-1112	-1174			-46	40	39	16			-1029	-11.22
L7'-U8'	8352	61.6	392	625	94	1111	1216	0.91		43	-40	-29	-13			1046	16.98
		61.6				1111	1216		388	43	388		43	1.00	0.94	1503	24.40
U8'-L9'	8350	77.0	558	840	144	1542	1560	0.99		52	-51		-6			1495	19.42
		77.0				1542	1560		476	52	481	4	54	1.02	1.00	2031	26.38
L9'-U10'	8348	125.8	-610	-943	-159	-1712	-1680	1.02	-548	-60	-555	-3	-62	1.02	1.02	-2266	-18.02
		125.8				-1712	-1680			-60	57		6			-1651	-13.13
U10'-L11'	8346	73.4	503	790	132	1425	1432	1.00		54	-57		-6			1372	18.70
		73.4				1425	1432		489	54	465	3	52	0.96	0.98	1897	25.86
L11'-U12'	8344	91.8	-423	-665	-112	-1200	-1215	0.99	-459	-51	-442		-49	0.96	0.98	-1638	-17.85
		91.8				-1200	-1215			-51	80	1	9			-1115	-12.15
U12'-L13'	8342	47.6	275	425	70	770	834	0.92		42	-115	-1	-13			658	13.83
		47.6				770	834		380	42	335		37	0.88	0.91	1109	23.31
L13'-U14	8340	41.8	-79	-108	-18	-205	-214	0.96	-285	-31	-260		-28	0.91	0.93	-461	-11.04
		41.8				-205	-214		213	24	179	3	21	0.86	-0.09	-19	-0.46

Table 10 Northwest Diagonal Forces

Member ID	Element Number	Net Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0-U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL / PlanLL	Ratio 3D Total / Plans	Total 3D Force (Kips)	3D Total Stress (Kips)
U1'-L1'	7914	32.5	-23	-36	-5	-64	-323	0.20	-207	-62	-61	-5	-20	0.32	0.25	-150	-4.62
		32.5				-64	-323				3		1			-60	-1.85
U2'-L2'	7886	18	58	136	23	217	266	0.82			-9	-1	-3			204	11.33
		18				217	266		207	62	82		25	0.40	0.61	324	18.00
U3'-L3'	7858	32.5	-19	-25	-1	-45	-320	0.14	-207	-62	-48		-14	0.23	0.18	-107	-3.29
		32.5				-45	-320				7		2			-36	-1.11
U4'-L4'	7830	18	50	85	18	153	234	0.65			-8	-2	-3			140	7.78
		18				153	234		207	62	71		21	0.34	0.49	245	13.61
U5'-L5'	7802	33.8	-18	-29	-1	-48	-318	0.15	-207	-62	-50	-1	-15	0.25	0.19	-114	-3.38
		33.8				-48	-318				9		3			-36	-1.07
U6'-L6'	7774	18	74	142	24	240	275	0.87			-12	-1	-4			223	12.39
		18				240	275		207	62	93		28	0.45	0.66	361	20.06
U7'-L7'	7746	34.3	-23	-26	-1	-50	-333	0.15	-207	-62	-54		-16	0.26	0.20	-120	-3.50
		34.3				-50	-333				5		1			-44	-1.28
U8'-L8'	7718	179.2	-812	-1220	-193	-2225	-2527	0.88	-714	-79	-651		-72	0.91	0.89	-2948	-16.45
		179.2				-2225	-2527				45	20	11			-2149	-11.99
U9'-L9'	7690	39.3	-22	-25	-1	-48	-331	0.15	-207	-62	-53		-16	0.26	0.20	-117	-2.98
		39.3				-48	-331				6		2			-40	-1.02
U10'-L10'	7662	18	73	147	26	246	271	0.91			-12		-4			230	12.78
		18				246	271		207	62	93		28	0.45	0.68	367	20.39
U11'-L11'	7634	32.5	-19	-23	0	-42	-269	0.16	-207	-62	-51		-15	0.25	0.20	-108	-3.32
		32.5				-42	-269				7		2			-33	-1.02
U12'-L12'	7606	18	69	147	26	242	270	0.90			-5		-1			236	13.11
		18				242	270		207	62	86		26	0.42	0.66	354	19.67
U13'-L13'	7578	32.5	-25	-22	1	-46	-330	0.14	-207	-62	-49		-15	0.24	0.18	-110	-3.38
		32.5				-46	-330				7		2			-37	-1.14

Table 11 Northwest Upper Vertical Forces

Member ID	Element Number	Net Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0-U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL / PlanLL	Ratio 3D Tot / Plans	Total 3D Force (Kips)	3D Total Stress (Kips)
U1'-L1'	7916	32.5	-79	-212	-41	-332	-323	1.03	-207	-62	-184	-11	-58	0.94	0.99	-585	-18
		32.5				-332	-323				29		9			-294	-9.0462
U2'-L2'	7888	18	15	-1	0	14	266	0.05			-2	0	-1			11	0.61
		18				14	266		207	62	1		0	0.00	0.03	15	0.83
U3'-L3'	7860	32.5	-69	-185	-30	-284	-320	0.89	-207	-62	-139	0	-42	0.67	0.79	-465	-14.308
		32.5				-284	-320				13		4			-267	-8.2154
U4'-L4'	7832	18	16	-1	0	15	234	0.06			-3	0	-1			11	0.61
		18				15	234		207	62	1		0	0.00	0.03	16	0.89
U5'-L5'	7804	27	-72	-194	-34	-300	-318	0.94	-207	-62	-143	-2	-43	0.70	0.83	-488	-18.074
		27				-300	-318				12		4			-284	-10.519
U6'-L6'	7776	18	20	1	0	21	275	0.08			-1		0			20	1.11
		18				21	275		207	62	1		0	0.00	0.04	22	1.22
U7'-L7'	7748	39.3	-84	-188	-32	-304	-333	0.91	-207	-62	-153		-46	0.74	0.84	-503	-12.815
		39.3				-304	-333				16		5			-283	-7.21
U8'-L8'	7720	179.2	-907	-1339	-217	-2463	-2527	0.97	-714	-79	-740		-82	1.04	0.99	-3285	-18.332
		179.2				-2463	-2527				68	20	14			-2361	-13.175
U9'-L9'	7692	39.3	-83	-190	-33	-306	-331	0.92	-207	-62	-154		-46	0.74	0.84	-506	-12.892
		39.3				-306	-331				14		4			-288	-7.34
U10'-L10'	7664	18	18	2	0	20	271	0.07			-1		0			19	1.06
		18				20	271		207	62			0	0.00	0.04	20	1.11
U11'-L11'	7636	32.5	-66	-168	-27	-261	-269	0.97	-207	-62	-146		-44	0.71	0.84	-451	-13.877
		32.5				-261	-269				15		4			-242	-7.45
U12'-L12'	7608	18	20	-2	0	18	270	0.07			-2		-1			15	0.83
		18				18	270		207	62	1		0	0.00	0.04	19	1.06
U13'-L13'	7580	32.5	-74	-180	-29	-283	-330	0.86	-207	-62	-146		-44	0.71	0.79	-473	-14.554
		32.5				-283	-330				10		3			-270	-8.31

Member ID	Element Number	Net Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0-U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL / Plans LL	Ratio 3D Total / Plans	Total 3D Force (Kips)	3D Total Stress (Kips)
U0-U1	8485	44.4	103	250	78	431	439	0.98			-19		-4			408	9.19
		44.4				431	439		244	51	197	112	75	1.30	1.11	815	18.36
U1-U2	8209	44.4	102	246	77	425	439	0.97			-30		-6			389	8.76
		44.4				425	439		244	51	191	110	73	1.27	1.09	799	18.00
U2-U3	8211	71.0	-52	-150	-1	-203	-226	0.90	-513	-67	-455		-59	0.89	0.89	-717	-10.10
		57.5				-203	-226		437	39	308	70	48	0.89	0.89	223	3.88
U3-U4	8213	71.0	-50	-147	0	-197	-226	0.87	-513	-67	-453		-59	0.88	0.88	-709	-9.99
		57.5				-197	-226		437	39	310	71	49	0.90	0.93	233	4.05
U4-U5	8215	41.6	194	244	51	489	516	0.95	-443	-58	-380		-50	0.86	3.93	59	1.42
		41.6				489	516		536	48	433	30	48	0.88	0.91	1000	24.04
U5-U6	8217	41.6	193	243	51	487	516	0.94	-443	-58	-379		-50	0.86	3.87	58	1.39
		41.6				487	516		536	48	432	29	47	0.87	0.90	995	23.92
U6-U7	8219	91.4	634	993	166	1793	1762	1.02			-120		-13			1660	18.16
		91.4				1793	1762		607	67	564	2	63	0.93	0.99	2422	26.50
U7-U8	8221	91.4	636	998	167	1801	1762	1.02			-112		-12			1677	18.35
		91.4				1801	1762		607	67	563	2	63	0.93	1.00	2429	26.58
U8-U9	8223	82.6	546	888	145	1579	1551	1.02			-85	-10	-12			1472	17.82
		82.6				1579	1551		537	59	484		53	0.90	0.99	2116	25.62
U9-U10	8225	82.6	545	886	145	1576	1551	1.02			-86	-10	-12			1468	17.77
		82.6				1576	1551		537	59	486		53	0.90	0.99	2115	25.61
U10-U11	8227	71.0	-179	-242	-45	-466	-486	0.96	-402	-36	-357	-16	-37	0.94	0.95	-876	-12.34
		71.0				-466	-486				223		20			-223	-3.14
U11-U12	8229	71.0	-178	-242	-45	-465	-486	0.96	-402	-36	-357	-16	-37	0.94	0.95	-875	-12.32
		71.0				-465	-486				230		21			-214	-3.01
U12-U13	8231	135.9	-685	-1040	-180	-1905	-1899	1.00	-817	-74	-747	-22	-74	0.95	0.98	-2748	-20.22
		135.9				-1905	-1899				229		21			-1655	-12.18
U13-U14	8233	135.9	-654	-988	-172	-1814	-1899	0.96	-817	-74	-710	-21	-71	0.90	0.94	-2616	-19.25
		135.9				-1814	-1899				221		20			-1573	-11.57

Table 13 Southeast Upper Chord Forces

Member ID	Element Number	Net Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0-U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL / Plans LL	Ratio 3D Total / Plans	Total 3D Force (Kips)	3D Total Stress (Kips)
U0-L1	8487	51.3	-128	-313	-98	-539	-560	0.96	-311	-65	-252	-142	-95	1.30	1.10	-1028	-20.06
		51.3				-539	-560				22		-5			-522	-10.18
L1-L2	8261	46.0	31	58	-21	68	80	0.85	-345	-31	-234	-93	-49	1.00	1.04	-308	-6.70
		36.7				68	80		333	43	301		39	0.90	0.89	408	11.12
L2-L3	8263	46.0	31	60	-21	70	80	0.88	-345	-31	-233	-93	-49	1.00	1.03	-305	-6.63
		36.7				70	80		333	43	303		39	0.91	0.90	412	11.23
L3-L4	8265	62.5	-33	20	-13	-26	-18	1.44	-490	-44	-379	-50	-49	0.90	0.91	-504	-8.06
		49.8				-26	-18		572	74	484		63	0.85	0.83	521	10.46
L4-L5	8267	62.5	-34	18	-14	-30	-18	1.67	-490	-44	-379	-49	-49	0.89	0.92	-507	-8.11
		49.8				-30	-18		572	74	484		63	0.85	0.82	517	10.38
L5-L6	8269	83.5	-399	-599	-105	-1103	-1087	1.01	-539	-70	-490	-14	-68	0.94	0.99	-1675	-20.06
		83.5				-1103	-1087				252		33			-818	-9.80
L6-L7	8271	83.5	-399	-600	-105	-1104	-1087	1.02	-539	-70	-488	-15	-68	0.94	0.99	-1675	-20.06
		83.5				-1104	-1087				249		32			-823	-9.86
L7-L8	8273	166.5	-903	-1421	-235	-2559	-2533	1.01	-787	-87	-736		-81	0.93	0.99	-3376	-20.28
		166.5				-2559	-2533				84	8	12			-2455	-14.74
L8-L9	8275	166.5	-906	-1427	-236	-2569	-2543	1.01	-790	-87	-743		-82	0.94	0.99	-3394	-20.38
		166.5				-2569	-2543				84	8	12			-2465	-14.80
L9-L10	8277	62.5	-173	-311	-47	-531	-559	0.95	-324	-36	-308		-34	0.95	0.95	-873	-13.97
		62.5				-531	-559				179	13	24			-315	-5.04
L10-L11	8279	62.5	-173	-311	-47	-531	-559	0.95	-324	-36	-324		-36	1.00	0.97	-891	-14.26
		62.5				-531	-559				199	13	26			-293	-4.69
L11-L12	8281	77.6	482	723	128	1333	1311	1.02			-227		-21			1085	13.98
		77.6				1333	1311		642	58	587	20	59	0.95	0.99	1999	25.76
L12-L13	8283	77.6	483	724	128	1335	1311	1.02			-228		-21			1086	13.99
		77.6				1335	1311		642	58	589	20	59	0.95	1.00	2003	25.81
L13-L14	8285	109.9	722	1092	191	2005	2036	0.98			-214		-19			1772	16.12
		109.9				2005	2036		861	78	776	24	77	0.93	0.97	2882	26.22

Table 14 Southeast Lower Chord Forces

Member ID	Element Number	Net Area (In^2)	Stagw 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0-U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL / Plans LL	Ratio 3D Total / Plans	Total 3D Force (Kips)	3D Total Stress (Kips)
L1-U2	8313	64.0	-170	-390	-72	-632	-662	0.95	-462	-60	-394	-25	-59	0.92	0.94	-1047	-16.36
		64.0				-632	-662				177		23			-451	-7.05
U2-L3	8315	25.2	27	119	28	174	192	0.91	-217	-24	-175		-19	0.80	0.41	3	0.12
		25.2				174	192		325	43	239	27	40	0.83	0.86	444	17.64
L3-U4	8317	22.9	111	171	18	300	321	0.93			-140	-29	-24			135	5.90
		22.9				300	321		259	29	217		24	0.84	0.89	521	22.77
U4-L5	8319	55.0	-229	-373	-53	-655	-640	1.02	-331	-36	-305		-33	0.92	0.99	-956	-17.38
		55.0				-655	-640				74	28	16			-549	-9.98
L5-U6	8321	47.6	291	506	76	873	883	0.99			-43	-22	-11			812	17.07
		47.6				873	883		344	38	315		35	0.92	0.97	1192	25.05
U6-L7	8323	91.8	-409	-683	-107	-1199	-1174	1.02	-415	-46	-412		-46	0.99	1.01	-1607	-17.51
		91.8				-1199	-1174				40	21	11			-1134	-12.36
L7-U8	8325	61.6	414	660	105	1179	1216	0.97			-39	-16	-9			1128	18.31
		61.6				1179	1216		388	43	383		42	0.99	0.97	1566	25.42
U8-L9	8327	77.0	567	849	143	1559	1560	1.00			-49		-5			1514	19.66
		77.0				1559	1560		476	52	473	4	53	1.00	1.00	2040	26.49
L9-U10	8329	125.8	-624	-964	-163	-1751	-1680	1.04	-548	-60	-551	-5	-62	1.02	1.04	-2303	-18.31
		125.8				-1751	-1680				56		6			-1691	-13.45
U10-L11	8331	73.4	511	804	135	1450	1432	1.01			-57		-6			1397	19.04
		73.4				1450	1432		489	54	459	5	52	0.95	1.00	1918	26.15
L11-U12	8333	91.8	-433	-685	-117	-1235	-1215	1.02	-459	-51	-436	-4	-50	0.96	1.00	-1671	-18.21
		91.8				-1235	-1215				79		9			-1152	-12.56
U12-L13	8335	47.6	285	445	75	805	834	0.97			-112		-12			697	14.65
		47.6				805	834		380	42	332	4	38	0.89	0.94	1145	24.06
L13-U14	8337	41.8	-87	-125	-23	-235	-214	1.10	-285	-31	-258	-3	-29	0.92	0.99	-492	-11.78
		41.8				-235	-214		213	24	174		20	0.82	-1.78	-57	-1.37

Table 15 Southeast Diagonal Forces

Member ID	Element Number	Net Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio <u>3D DL</u> Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0-U0' (Kips)	Impact 3D (Kips)	Ratio <u>3D LL</u> Plans LL	Ratio <u>3D Total</u> Plans	Total 3D Force (Kips)	3D Total Stress (Kips)
U1-L1	7182	32.5	-20	-37	-4	-61	-323	0.19	-207	-62	-52	-3	-16	0.26	0.22	-132	-4.06
		32.5				-61	-323				3		1			-57	-1.75
U2-L2	7210	22.5	56	131	22	209	266	0.79			-8	-1	-3			197	8.76
		18.0				209	266		207	62	78		23	0.38	0.58	310	17.22
U3-L3	7238	32.5	-19	-26	0	-45	-320	0.14	-207	-62	-48		-14	0.23	0.18	-107	-3.29
		32.5				-45	-320				6		2			-37	-1.14
U4-L4	7266	22.5	52	101	22	175	234	0.75			-8	-1	-3			163	7.24
		18.0				175	234		207	62	89		27	0.43	0.58	291	16.17
U5-L5	7294	33.8	-18	-29	-1	-48	-318	0.15	-207	-62	-51		-15	0.25	0.19	-114	-3.38
		33.8				-48	-318				5		1			-42	-1.24
U6-L6	7322	22.5	73	144	25	242	275	0.88			-10	-1	-3			228	10.13
		18.0				242	275		207	62	90		27	0.43	0.66	359	19.94
U7-L7	7350	39.3	-23	-26	0	-49	-333	0.15	-207	-62	-54		-16	0.26	0.20	-119	-3.03
		39.3				-49	-333				5		1			-43	-1.10
U8-L8	7378	179.2	-836	-1242	-196	-2274	-2527	0.90	-714	-79	-629		-70	0.88	0.90	-2973	-16.59
		179.2				-2274	-2527				43	9	7			-2215	-12.36
U9-L9	7406	39.3	-21	-24	-1	-46	-331	0.14	-207	-62	-53		-16	0.26	0.19	-115	-2.93
		39.3				-46	-331				5		1			-40	-1.02
U10-L10	7434	22.5	78	153	27	258	271	0.95			-10		-3			245	10.89
		18.0				258	271		207	62	93		28	0.45	0.70	379	21.06
U11-L11	7462	32.5	-19	-23	1	-41	-269	0.15	-207	-62	-51		-15	0.25	0.20	-107	-3.29
		32.5				-41	-269				7		2			-32	-0.98
U12-L12	7490	22.5	70	149	26	245	270	0.91			-5		-1			239	10.62
		18.0				245	270		207	62	85		25	0.41	0.66	355	19.72
U13-L13	7518	32.5	-25	-22	2	-45	-330	0.14	-207	-62	-49		-15	0.24	0.18	-109	-3.35
		32.5				-45	-330				7		2			-36	-1.11
U14-L14	7546	22.5	65	110	23	198	244	0.81			-2		-1			195	8.67
		18.0				198	244		207	62	84		25	0.41	0.60		0.00

Table 16 Southeast Upper Vertical Forces

Member ID	Element Number	Net Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans	LL 3D (Kips)	LL U0-U0'	Impact 3D	Ratio 3D LL / Plans LL	Ratio 3D Total / Plans	Total 3D Force (Kips)	3D Total Stress (Kips)
U1-L1	7184	32.5	-76	-210	-39	-325	-323	1.01	-207	-62	-182	-8	-57	0.92	0.97	-572	-17.60
		32.5				-325	-323				30		9			-286	-8.80
U2-L2	7212	22.5	13	-1	0	12	266	0.05			-2		-1			9	0.40
		18.0				12	266		207	62	1		0	0.00	0.02	13	0.72
U3-L3	7240	32.5	-72	-189	-31	-292	-320	0.91	-207	-62	-139		-42	0.67	0.80	-473	-14.55
		32.5				-292	-320				12		4			-276	-8.49
U4-L4	7268	22.5	16	-1	0	15	234	0.06			-3		-1			11	0.49
		18.0				15	234		207	62	1		0	0.00	0.03	16	0.89
U5-L5	7296	33.8	-72	-193	-32	-297	-318	0.93	-207	-62	-144	-2	-44	0.71	0.83	-487	-14.42
		33.8				-297	-318				11		3			-283	-8.39
U6-L6	7324	22.5	20	2	0	22	275	0.08			-1		0			21	0.92
		18.0				22	275		207	62	1		0	0.00	0.04	23	1.28
U7-L7	7352	39.3	-86	-190	-31	-307	-333	0.92	-207	-62	-151		-45	0.73	0.84	-503	-12.82
		39.3				-307	-333				16		5			-286	-7.29
U8-L8	7380	179	-934	-1380	-226	-2540	-2527	1.01	-714	-79	-730		-81	1.02	1.01	-3351	-18.70
		179				-2540	-2527				66	10	10			-2454	-13.69
U9-L9	7408	39.3	-84	-190	-32	-306	-331	0.92	-207	-62	-153		-46	0.74	0.84	-505	-12.87
		39.3				-306	-331				15		4			-287	-7.31
U10-L10	7436	22.5	18	2	0	20	271	0.07			-1		0			19	0.84
		18.0				20	271		207	62			0	0.00	0.04	20	1.11
U11-L11	7464	32.5	-67	-169	-26	-262	-269	0.97	-207	-62	-144		-43	0.70	0.83	-449	-13.82
		32.5				-262	-269				15		4			-243	-7.48
U12-L12	7492	22.5	20	-1	0	18	270	0.07			-2		-1			15	0.68
		18.0				16	270		207	62	1		0	0.00	0.03	17	0.94
U13-L13	7520	32.5	-76	-183	-29	-260	-330	0.79	-207	-62	-144		-43	0.70	0.75	-447	-13.75
		32.5				-260	-330				9		3			-248	-7.63
U14-L14	7548	22.5	21	-3	-1	17	244	0.07			-4		-1			12	0.53
		18.0					244		207	62			0	0.00			0.00

Table 17 Southeast Lower Vertical Forces

Member ID	Element Number	Net Area (in2)	Stage 1 (kips)	Stage 6 (kips)	Stage 7 (kips)	3D DL Total (kips	DL Plans (kips)	Ratio 3D DL Plans	LL Plans (kips)	Imapct Plans (kips)	LL 3D (kips)	LL U0&U0' (kips)	Impact 3D (kips)	Ratio 3D LL Plans LL	Ratio 3D Total Plans	Total 3D Force (kips)	3D Total Stress (ksi)
U0'-U1'	8491	59.1	221	412	151	784	796	0.98			-17		-3			764	12.93
		59.1				784	796		309	53	212	194	95	1.38	0.90	1285	21.74
U1'-U2'	8259	59.1	220	410	150	780	796	0.98			-31		-5			744	12.59
		59.1				780	796		309	53	206	193	93	1.36	0.89	1272	21.52
U2'-U3'	8257	71.0	13	-58	42	-3	-31	0.10	-513	-67	-456		-60	0.89	0.84	-519	-7.31
		57.5				-3	-31		473	43	310	120	64	0.96	0.69	491	8.54
U3'-U4'	8255	71.0	15	-49	42	8	-31	-0.26	-513	-67	-450		-59	0.88	0.81	-501	-7.06
		57.5				8	-31		473	43	312	120	64	0.96	0.72	504	8.77
U4'-U5'	8253	41.6	213	283	66	562	516	1.09	-443	-58	-369		-48	0.83	2.64	145	3.49
		41.6				562	516		536	48	432	45	52	0.91	0.94	1091	26.23
U5'-U6'	8251	41.6	211	279	66	556	516	1.08	-443	-58	-369		-46	0.83	2.56	141	3.39
		41.6				556	516		536	48	431	45	52	0.90	0.94	1084	26.06
U6'-U7'	8249	91.4	621	978	159	1758	1762	1.00			-121	-6	-15			1616	17.68
		91.4				1758	1762		607	67	563		62	0.93	0.98	2383	26.07
U7'-U8'	8247	91.4	624	988	161	1773	1762	1.01			-112	-6	-14			1641	17.95
		91.4				1773	1762		607	67	564		62	0.93	0.99	2399	26.25
U8'-U9'	8245	82.6	523	864	132	1519	1551	0.98			-82	-25	-17			1395	16.89
		82.6				1519	1551		537	59	487		54	0.91	0.96	2060	24.94
U9'-U10'	8243	82.6	522	859	132	1513	1551	0.98			-83	-24	-16			1390	16.83
		82.6				1513	1551		537	59	490		54	0.91	0.96	2057	24.90
U10'-U11	8241	71.0	-194	-261	-54	-509	-486	1.05	-402	-36	-353	-27	-40	0.96	0.97	-929	-13.08
		71.0				-509	-486				225		20			-264	-3.72
U11'-U12	8239	71.0	-193	-267	-56	-516	-486	1.06	-402	-36	-358	-28	-40	0.97	0.98	-942	-13.27
		71.0				-516	-486				224		20			-272	-3.83
U12'-U13	8237	135.9	-690	-1047	-184	-1921	-1899	1.01	-817	-74	-746	-27	-76	0.95	0.98	-2770	-20.39
		135.9				-1921	-1899				228		21			-1672	-12.30
U13'-U14	8235	135.9	-659	-998	-176	-1833	-1899	0.97	-817	-74	-715	-26	-73	0.91	0.94	-2647	-19.48
		135.9				-1833	-1899				218		20			-1595	-11.74

Table 18 Northeast Upper Chord Forces

Member ID	Element Number	Net Area (In2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	3D LL	LL U0-U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL / Plans LL	Ratio 3D Total / Plans	3D Total Force (Kips)	3DTotal Stress (Kips)
U0'-L1'	8493	79.0	-283	-528	-193	-1004	-1014	0.99	-393	-67	-275	-249	-122	1.40	1.12	-1650	-20.89
		79.0				-1004	-1014				20		3			-981	
L1'-L2'	8311	57.8	-58	-65	-78	-201	-190	1.06	-394	-35	-240	-158	-69	1.09	1.08	-668	-11.56
		45.9				-201	-190		333	43	308		40	0.93	0.79	147	3.20
L2'-L3'	8309	57.8	-58	-63	-78	-199	-190	1.05	-394	-35	-239	-158	-69	1.09	1.07	-665	-11.51
		45.9				-199	-190		333	43	310		40	0.93	0.81	151	3.29
L3'-L4;	8307	62.5	-73	-35	-40	-148	-137	1.08	-510	-46	-379	-82	-59	0.94	0.96	-668	-10.69
		49.8				-148	-137		572	74	488		63	0.85	0.79	403	8.09
L4'-L5'	8305	62.5	-74	-36	-40	-150	-137	1.09	-510	-46	-380	-81	-59	0.94	0.97	-670	-10.72
		49.8				-150	-137		572	74	489		63	0.85	0.79	402	8.07
L5'-L6'	8303	83.5	-401	-599	-106	-1106	-1087	1.02	-539	-70	-489	-18	-69	0.95	0.99	-1682	-20.14
		83.5				-1106	-1087				254		33			-819	-9.81
L6'-L7'	8301	83.5	-401	-599	-105	-1105	-1087	1.02	-539	-70	-488	-17	-68	0.94	0.99	-1678	-20.10
		83.5				-1105	-1087				252		33			-820	-9.82
L7'-L8'	8299	166.5	-878	-1382	-220	-2480	-2533	0.98	-787	-87	-730		-81	0.93	0.97	-3291	-19.77
		166.5				-2480	-2533				86	23	16			-2355	-14.14
L8'-L9'	8297	166.5	-882	-1388	-221	-2491	-2543	0.98	-790	-87	-739		-81	0.94	0.97	-3311	-19.89
		166.5				-2491	-2543				84	23	16			-2368	-14.22
L9'-L10'	8295	62.5	-155	-276	-34	-465	-559	0.83	-324	-36	-303		-34	0.94	0.87	-802	-12.83
		62.5				-465	-559				180	26	28			-231	-3.70
L10-L11'	8293	62.5	-155	-276	-34	-465	-559	0.83	-324	-36	-318		-35	0.98	0.89	-818	-13.09
		62.5				-465	-559				199	26	30			-210	-3.36
L11'-L12'	8291	77.6	493	745	136	1374	1311	1.05			-224		-20			1130	14.56
		77.6				1374	1311		642	58	589	27	61	0.97	1.02	2051	26.43
L12'-L13'	8289	77.6	493	746	136	1375	1311	1.05			-223		-20			1132	14.59
		77.6				1375	1311		642	58	591	27	61	0.97	1.02	2054	26.47
L13'-L14	8287	109.9	722	1092	191	2005	2036	0.98			-214		-19			1772	16.12
		109.9				2005	2036		861	78	776	24	77	0.93	0.97	2882	26.22

Table 19 Northeast Lower Chord Forces

Member ID	Element Number	Net Area (In2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0&U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL / PlansLL	Ratio 3D Total / Plans	Total 3D Force (Kips)	3D Total Stress (Kips)
L1'-U2'	8363	70.0	-208	-444	-93	-745	-771	0.97	-481	-63	-405	-47	-67	0.95	0.96	-1193	-17.04
		70.0				-745	-771				178		23			-563	-8.04
U2'-L3'	8361	27.1	58	160	46	264	291	0.91			-171		-22			97	3.58
		27.1				264	291		343	45	235	48	46	0.85	0.87	551	20.33
L3'-U4'	8359	22.9	78	123	-3	198	220	0.90	-200	-26	-141	-53	-34	1.01	5.00	8	0.35
		22.9				198	220		258	29	217		24	0.84	0.87	419	18.31
U4'-L5'	8357	55.0	-199	-329	-33	-561	-640	0.88	-331	-36	-304		-33	0.92	0.89	-861	-15.65
		55.0				-561	-640				76	51	24			-430	-7.82
L5'-U6'	8355	47.6	265	471	60	796	883	0.90			-44	-41	-17			715	15.03
		47.6				796	883		344	38	315		35	0.92	0.91	1115	23.43
U6'-L7'	8353	91.8	-385	-649	-92	-1126	-1174	0.96	-415	-46	-411		-46	0.99	0.97	-1533	-16.71
		91.8				-1126	-1174				41	39	16			-1042	-11.36
L7'-U8'	8351	61.6	396	634	94	1124	1216	0.92			-40	-29	-13			1059	17.19
		61.6				1124	1216		388	43	383		42	0.99	0.94	1511	24.53
U8'-L9'	8349	77.0	564	850	143	1557	1560	1.00			-51		-6			1510	19.61
		77.0				1557	1560		476	52	474	4	53	1.01	1.00	2039	26.48
L9'-U10'	8347	125.8	-616	-956	-160	-1732	-1680	1.03	-548	-60	-550	-2	-61	1.01	1.02	-2280	-18.13
		125.8				-1732	-1680				56		6			-1672	-13.30
U10'-L11'	8345	73.4	507	798	132	1437	1432	1.00			-57		-6			1384	18.87
		73.4				1437	1432		489	54	460	3	52	0.95	0.99	1904	25.95
L11'-U12'	8343	91.8	-427	-673	-112	-1212	-1215	1.00	-459	-51	-437		-45	0.95	0.98	-1645	-17.93
		91.8				-1212	-1215				79	1	9			-1128	-12.29
U12'-L13'	8341	47.6	277	430	70	777	834	0.93			-114	-1	-13			666	14.00
		47.6				777	834		380	42	331		37	0.87	0.91	1112	23.37
L13'-U14	8339	41.8	-80	-110	-18	-208	-214	0.97	-285	-31	-257		-28	0.90	0.93	-461	-11.04
		41.8				-208	-214		213	24	177	3	21	0.85	-0.30	-24	-0.57

Table 20 Northeast Diagonal Forces

Member ID	Element Number	Net Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL Plans	LL Plans (Kips)	Impact Plans	LL 3D (Kips)	LL U0&U0'	Impact 3D	Ratio 3D LL PlanLL	Ratio 3D Total Plans	Total 3D Force (Kips)	3D Total Stress (Kips)
U1'-L1'	7910	32.5	-23	-35	-4	-62	-323	0.19	-207	-62	-59	-5	-19	0.31	0.24	-145	-4.46
		32.5				-62	-323	0.19			3		1			-58	-1.78
U2'-L2'	7882	18	59	140	24	223	266	0.84			-9	-1	-3			210	11.67
		18				223	266	0.84	207	62	82		25	0.40	0.62	330	18.33
U3'-L3'	7854	32.5	-19	-25	0	-44	-320	0.14	-207	-62	-48		-14	0.23	0.18	-106	-3.26
		32.5				-44	-320	0.14			8		2			-34	-1.05
U4'-L4'	7826	18	52	99	23	174	234	0.74			-9	-2	-3			160	8.89
		18				174	234	0.74	207	62	86		26	0.42	0.57	286	15.89
U5'-L5'	7798	33.8	-18	-28	0	-46	-318	0.14	-207	-62	-51	-1	-16	0.25	0.19	-114	-3.38
		33.8				-46	-318	0.14			7		2			-37	-1.10
U6'-L6'	7770	18	74	146	25	245	275	0.89			-11	-1	-4			229	12.72
		18				245	275	0.89	207	62	91		27	0.44	0.67	363	20.17
U7'-L7'	7742	39.3	-23	-25	-1	-49	-333	0.15	-207	-62	-53		-16	0.26	0.20	-118	-3.01
		39.3				-49	-333	0.15			5		1			-43	-1.10
U8'-L8'	7714	179	-819	-1226	-186	-2231	-2527	0.88	-714	-79	-631		-70	0.88	0.88	-2932	-16.36
		179				-2231	-2527	0.88			47	20	11			-2153	-12.01
U9'-L9'	7686	39.3	-21	-24	0	-45	-331	0.14	-207	-62	-53		-16	0.26	0.19	-114	-2.90
		39.4				-45	-331	0.14			5		1			-39	-0.99
U10'-L10'	7658	18	75	152	27	254	271	0.94			-10		-3			241	13.39
		18				254	271	0.94	207	62	92		28	0.45	0.69	374	20.78
U11'-L11'	7630	32.5	-19	-23	1	-41	-269	0.15	-207	-62	-51		-15	0.25	0.20	-107	-3.29
		32.5				-41	-269	0.15			7		2			-32	-0.98
U12'-L12'	7602	18	70	150	26	246	270	0.91			-5		-1			240	13.33
		18				246	270	0.91	207	62	85		25	0.41	0.66	356	19.78
U13'-L13'	7574	32.5	-25	-22	1	-46	-330	0.14	-207	-62	-49		-15	0.24	0.18	-110	-3.38
		32.5				-46	-330	0.14			7		2			-37	-1.14

Table 21 Northeast Upper Vertical Forces

Member ID	Element Number	Net Area (In^2)	Stage Stg 1 (Kips)	Stage Stg 6 (Kips)	Stage Stg 7 (Kips)	3D DL Total (Kips)	DL Plans (Kips)	Ratio 3D DL / Plans	LL Plans (Kips)	Impact Plans (Kips)	LL 3D (Kips)	LL U0&U0' (Kips)	Impact 3D (Kips)	Ratio 3D LL / PlanLL	Ratio 3D Total / Plans	3D Total Force (Kips)	3D Total Stress (Kips)
U1'-L1'	7912	32.5	-82	-217	-43	-342	-323	1.06	-207	-62	-185	-12	-59	0.95	1.01	-598	-18.4
		32.5				-342	-323				27		8			-307	-9.45
U2'-L2'	7884	18	15	-1	0	14	266	0.05			-2		-1			11	0.61
		18				14	266		207	62	1		0	0.00	0.03	15	0.83
U3'-L3'	7856	32.5	-70	-187	-30	-287	-320	0.90	-207	-62	-137		-41	0.66	0.79	-465	-14.31
		32.5				-287	-320				13		4			-270	-8.31
U4'-L4'	7828	18	16	-1	0	15	234	0.06			-3		-1			11	0.61
		18				15	234		207	62	1		0	0.00	0.03	16	0.89
U5'-L5'	7800	33.8	-72	-195	-33	-300	-318	0.94	-207	-62	-141	-2	-43	0.69	0.83	-486	-14.40
		33.8				-300	-318				11		3			-286	-8.47
U6'-L6'	7772	18	20	1	0	21	275	0.08			-1		0			20	1.11
		18				21	275		207	62	1		0	0.00	0.04	22	1.22
U7'-L7'	7744	39.3	-86	-191	-32	-309	-333	0.93	-207	-62	-151		-46	0.73	0.84	-506	-12.89
		39.3				-309	-333				16		5			-288	-7.34
U8'-L8'	7716	179	-916	-1360	-217	-2493	-2527	0.99	-714	-79	-730		-81	1.02	1.00	-3304	-18.44
		179				-2493	-2527				68	20	14			-2391	-13.34
U9'-L9'	7688	39.3	-84	-191	-32	-307	-331	0.93	-207	-62	-152		-46	0.74	0.84	-505	-12.87
		39.3				-307	-331				15		4			-288	-7.34
U10'-L10'	7660	18	18	2	0	20	271	0.07			-1		0			19	1.06
		18				20	271		207	62	0		0	0.00	0.04	20	1.11
U11'-L11'	7632	32.5	-67	-169	-27	-263	-269	0.98	-207	-62	-144		-43	0.70	0.84	-450	-13.85
		32.5				-263	-269				15		4			-244	-7.51
U12'-L12'	7604	18	20	-1	0	19	270	0.07			-2		-1			16	0.89
		18				19	270		207	62	1		0	0.00	0.04	20	1.11
U13'-L13'	7576	32.5	-75	-182	-29	-286	-330	0.87	-207	-62	-144		-43	0.70	0.79	-473	-14.55
		32.5				-286	-330				9		3			-274	-8.43

Table 22 Northeast Lower Vertical Forces

Member ID	Element Number	Gross Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	Total DL (Kips)	3D LL + I (Kips)	3D Total (Kips)	Plans (Kips)	Ratio 3D/Plans	3D Stress (Ksi)	3D Range (Ksi)
1	6936	15.6	7	38	6	51	56	107	127	0.84	6.86	3.59
		15.6				51	0	51	127		3.27	
2	6935	15.6	9	55	8	72	84	156	193	0.81	10.00	5.38
		15.6				72	0	72	193		4.62	
3	6934*	12.3	10	57	8	75	53	128	193	0.66	10.44	4.65
		12.3				75	-4	71	193		5.79	
4	6933	19.1	-20	-63	-11	-94	73	-21	-282		-1.10	12.82
		19.1				-94	-172	-266	-282	0.94	-13.92	
5	6932	19.1	-20	-63	-11	-94	73	-21	-282		-1.10	12.82
		19.1				-94	-172	-266	-282	0.94	-13.92	
6	6931	31.2	-35	-121	-21	-177	75	-102	-465		-3.27	10.87
		31.2				-177	-264	-441	-465	0.95	-14.14	
7	6929	31.2	-34	-113	-20	-167	75	-92	-440		-2.95	10.55
		31.2				-167	-254	-421	-440	0.96	-13.50	
8	6928	31.2	-35	-119	-21	-175	75	-100	-465		-3.21	10.93
		31.2				-175	-266	-441	-465	0.95	-14.14	
9	6927	19.1	-20	-62	-11	-93	74	-19	-282		-0.99	12.93
		19.1				-93	-173	-266	-282	0.94	-13.92	
10	6926	19.1	-20	-62	-11	-93	74	-19	-282		-0.99	12.93
		19.1				-93	-173	-266	-282	0.94	-13.92	
11	6925*	12.3	10	56	8	74	53	127	193	0.66	10.36	5.30
		12.3				74	-12	62	193		5.06	
12	6924	15.6	9	54	8	71	84	155	193	0.80	9.94	5.58
		15.6				71	-3	68	193		4.36	
13	6923	15.6	7	37	6	50	56	106	127	0.83	6.79	3.59
		15.6				50	0	50	127		3.21	
32	7432	19.1	7	12	2	21	97	118	207	0.57	6.17	9.00
		19.1				21	-75	-54	207		-2.83	
33	7431	25.0	29	98	17	144	223	367	472	0.78	14.69	11.93
		25.0				144	-75	69	472		2.76	
34	7430	25.0	29	97	17	143	225	368	472	0.78	14.73	12.00
		24.9				143	-75	68	472		2.73	
35	7429	19.1	7	12	2	21	97	118	207	0.57	6.17	9.05
		19.1				21	-76	-55	207		-2.88	

* Net area used due to splice.

Table 23 Chord Forces in Floor Truss at U10 with Deck and Stringers not Effective

Member ID	Element Number	Gross Area (In²)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	Total DL (Kips)	3D LL + I (Kips)	3D Total (Kips)	Plans (Kips)	Ratio 3D/Plans	3D Stress (Ksi)	3D Range (Ksi)
14	7456	15.9	-8	-47	-7	-62	0	-62	-205	0.30	-3.90	4.60
		15.9				-62	-73	-135	-205	0.66	-8.50	
15	7455	10.3	-5	-30	-5	-40	0	-40	-116	0.34	-3.88	6.41
		10.3				-40	-66	-106	-116	0.91	-10.29	
16	7434	22.5	39	150	25	214	197	411	450	0.91	18.27	8.76
		22.5				214	0	214	450	0.48	9.51	
17	7454	27.1	-30	-121	-20	-171	13	-158	-369	0.43	-5.84	7.98
		27.1				-171	-203	-374	-369	1.01	-13.82	
18	7453	15.9	24	92	15	131	153	284	283	1.00	17.88	10.89
		15.9				131	-20	111	283	0.39	6.99	
19	7452	10.3	-5	-25	-4	-34	1	-33	-96	0.34	-3.20	5.53
		10.3				-34	-56	-90	-96	0.94	-8.74	
20	7451	15.9	-15	-62	-11	-88	29	-59	-201	0.29	-3.72	9.63
		15.9				-88	-124	-212	-201	1.05	-13.35	
21	7450	10.3	9	32	6	47	94	141	132	1.07	13.69	13.40
		10.3				47	-44	3	132		0.29	
22	7449	10.3	-5	-27	-4	-36	4	-32	-117	0.27	-3.11	6.99
		10.3				-36	-68	-104	-117	0.89	-10.10	
23	7448	10.3	-5	-21	-4	-30	4	-26	-117	0.22	-2.52	6.99
		10.3				-30	-68	-98	-117	0.84	-9.51	
24	7447	10.3	9	32	6	47	94	141	132	1.07	13.69	13.40
		10.3				47	-44	3	132		0.29	
25	7446	15.9	-15	-61	-11	-87	29	-58	-201	0.29	-3.65	9.63
		15.9				-87	-124	-211	-201	1.05	-13.29	
26	7445	10.3	-5	-25	-4	-34	1	-33	-96	0.34	-3.20	5.53
		10.3				-34	-56	-90	-96	0.94	-8.74	
27	7444	15.9	24	91	15	130	153	283	283	1.00	17.82	10.89
		15.9				130	-20	110	283	0.39	6.93	
28	7443	27.1	-30	-120	-20	-170	13	-157	-369	0.43	-5.80	7.98
		27.1				-170	-203	-373	-369	1.01	-13.78	
29	7438	22.5	39	150	25	214	197	411	450	0.91	18.27	8.76
		22.5				214	0	214	450	0.48	9.51	
30	7442	10.3	-5	-30	-5	-40	0	-40	-116	0.34	-3.88	6.41
		10.3				-40	-66	-106	-116	0.91	-10.29	
31	7441	15.9	-8	-46	-7	-61	0	-61	-205	0.30	-3.84	4.60
		15.9				-61	-73	-134	-205	0.65	-8.44	

Table 24 Diagonal Forces in Floor Truss at U10 with Deck and Stringers not Effective

Member ID	Element Number	Gross Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	Total DL (Kips)	3D LL + I (Kips)	3D Total (Kips)	Plans (Kips)	Ratio 3D/PLANS	3D Stress (Ksi)	3D Range (Ksi)
1	6936	15.6	8	34	7	49	10	59	127	0.46	3.78	1.15
		15.6				49	-8	41	127	0.32	2.63	**0.49**
2	6935	15.6	11	51	-8	54	16	70	193	0.36	4.49	2.18
		15.6				54	-18	36	193	0.19	2.31	**0.76**
3	6934*	12.3	12	48	-3	57	39	96	193	0.50	7.83	4.89
		12.3				57	-21	36	193	0.19	2.94	**0.57**
4	6933	19.1	-25	-61	-4	-90	17	-73	-282	0.26	-3.82	3.19
		19.1				-90	-44	-134	-282	0.48	-7.01	**(0.54)c**
5	6932	19.1	-29	-69	0	-98	31	-67	-282	0.24	-3.51	3.87
		19.1				-98	-43	-141	-282	0.50	-7.38	**(0.85)c**
6	6931	31.2	-50	-122	-3	-175	51	-124	-465	0.27	-3.98	6.03
		31.2				-175	-137	-312	-465	0.67	-10.00	**(0.97)c**
7	6929	31.2	-49	-119	5	-163	68	-95	-440	0.22	-3.05	7.47
		31.2				-163	-165	-328	-440	0.75	-10.52	**(1.15)c**
8	6928	31.2	-49	-119	-5	-173	49	-124	-465	0.27	-3.98	6.00
		31.2				-173	-138	-311	-465	0.67	-9.97	**(1.07)c**
9	6927	19.1	-30	-64	-4	-98	30	-68	-282	0.24	-3.56	3.98
		19.1				-98	-46	-144	-282	0.51	-7.54	**(0.91)c**
10	6926	19.1	-26	-58	-6	-90	16	-74	-282	0.26	-3.87	3.30
		19.1				-90	-47	-137	-282	0.49	-7.17	**(0.65)c**
11	6925*	12.3	13	52	-4	61	38	99	193	0.51	8.08	4.81
		12.3				61	-21	40	193	0.21	3.26	**0.58**
12	6924	15.6	11	54	-8	57	16	73	193	0.38	4.68	2.18
		15.6				57	-18	39	193	0.20	2.50	**0.78**
13	6923	15.6	8	36	7	51	10	61	127	0.48	3.91	1.15
		15.6				51	-8	43	127	0.34	2.76	**0.49**
32	7432	19.1	4	16	-20	0	66	66	207	0.32	3.45	6.23
		19.1				0	-53	-53	207		-2.77	**2.45**
33	7431	25.0	35	105	-8	132	156	288	472	0.61	11.53	8.49
		25.0				132	-56	76	472	0.16	3.04	**2.53**
34	7430	25.0	36	102	-7	131	156	287	472	0.61	11.49	8.49
		25.0				131	-56	75	472	0.16	3.00	**2.53**
35	7429	19.1	5	13	-20	-2	66	64	207	0.31	3.35	6.23
		19.1				-2	-53	-55	207		-2.88	**2.45**

* Net area used due to splice.

Green numbers give stress range due to one HS15 truck plus 15% impact.

Table 25 Chord Forces in Floor Truss at U10 wigh Deck and Stringers Effective

Member ID	Element Number	Gross Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	Total DL (Kips)	3D LL + I (Kips)	3D Total (Kips)	Plans (Kips)	Ratio 3D/Plans	3D Stress (Ksi)	3D Range (Ksi)
14	7456	15.9	-10	-44	-27	-81	4	-77	-205	0.38	-4.85	3.02
		15.9				-81	-44	-125	-205	0.61	-7.87	(1.79)c
15	7455	10.3	-5	-27	-5	-37	9	-28	-116	0.24	-2.72	7.57
		10.3				-37	-47	-84	-116	0.72	-10.29	(2.76)c
16	7434	22.5	78	153	27	258	121	379	540	0.70	16.84	5.96
		22.5				258	-13	245	540	0.45	10.89	
17	7454	27.1	-38	-121	-8	-167	12	-155	-369	0.42	-5.73	4.66
		27.1				-167	-114	-281	-369	0.76	-10.38	(1.37)c
18	7453	15.9	33	92	10	135	96	231	283	0.82	14.55	6.93
		15.9				135	-14	121	283	0.43	7.62	2.06
19	7452	10.3	-5	-22	2	-25	9	-16	-96	0.17	-1.55	3.50
		10.3				-25	-27	-52	-96	0.54	-5.05	(1.67)c
20	7451	15.9	-22	-66	-12	-100	21	-79	-201	0.39	-4.97	6.99
		15.9				-100	-90	-190	-201	0.95	-11.96	(2.28)c
21	7450	10.3	15	31	8	54	70	124	132	0.94	12.04	9.42
		10.3				54	-27	27	132	0.20	2.62	2.85
22	7449	10.3	-7	-27	-2	-36	20	-16	-117	0.14	-1.55	7.38
		10.3				-36	-56	-92	-117	0.79	-8.93	(3.01)c
23	7448	10.3	-7	-20	-10	-37	20	-17	-117	0.15	-1.65	7.96
		10.3				-37	-62	-99	-117	0.85	-9.61	(3.51)c
24	7447	10.3	13	32	9	54	74	128	132	0.97	12.43	9.81
		10.3				54	-27	27	132	0.20	2.62	2.68
25	7446	15.9	-21	-66	-12	-99	20	-79	-201	0.39	-4.97	6.93
		15.9				-99	-90	-189	-201	0.94	-11.90	(2.34)c
26	7445	10.3	-5	-22	2	-25	9	-16	-96	0.17	-1.55	3.50
		10.3				-25	-27	-52	-96	0.54	-5.05	(1.67)c
27	7444	15.9	35	92	10	137	96	233	283	0.82	14.67	6.86
		15.9				137	-13	124	283	0.44	7.81	2.06
28	7443	27.1	-40	-121	-8	-169	12	-157	-369	0.43	-5.80	4.62
		27.1				-169	-113	-282	-369	0.76	-10.42	(1.37)c
29	7438	22.5	77	150	27	254	122	376	540	0.70	16.71	6.00
		22.5				254	-13	241	540	0.45	10.71	
30	7442	10.3	-5	-28	-5	-38	9	-29	-116	0.25	-2.82	5.44
		10.3				-38	-47	-85	-116	0.73	-8.25	(2.76)c
31	7441	15.9	-10	-47	-27	-84	4	-80	-205	0.39	-5.04	3.02
		15.9				-84	-44	-128	-205	0.62	-8.06	(1.79)c

Green Numbers give stress range due to one HS15 truck plus 15% impact.

Table 26 Diagonal Forces in Floor Truss at U10 with Deck and Stringers Effective

Stringer	Beam Element	Gross Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	3D LL + I (Kips)	3D Total (Kips)	Range (Kips)
1	2128	72.0	-5	-28	-18	-51	3	-48	31
		72.0				-51	-28	-79	
2	2129	72.0	-4	-24	-3	-31	9	-22	46
		72.0				-31	-37	-68	
3	2130	72.0	-7	-31	-5	-43	4	-39	103
		72.0				-43	-99	-142	
4	2131	72.0	-3	-26	2	-27	12	-15	52
		72.0				-27	-40	-67	
5	2132	72.0	-4	-23	2	-25	8	-17	38
		72.0				-25	-30	-55	
6	2133	72.0	-5	-30	-3	-38	7	-31	44
		72.0				-38	-37	-75	
7	2134	72.0	-4	-20	-11	-35	18	-17	79
		72.0				-35	-61	-96	
8	5524	72.0	-4	-28	-3	-35	19	-16	73
		72.0				-35	-54	-89	
9	5525	72.0	-5	-30	-4	-39	7	-32	44
		72.0				-39	-37	-76	
10	5526	72.0	-5	-23	2	-26	8	-18	38
		72.0				-26	-30	-56	
11	5527	72.0	-3	-26	1	-28	11	-17	51
		72.0				-28	-40	-68	
12	5528	72.0	-7	-31	-5	-43	4	-39	104
		72.0				-43	-100	-143	
13	5529	72.0	-4	-23	-3	-30	8	-22	45
		72.0				-30	-37	-67	
14	5530	72.0	-5	-26	-18	-49	3	-46	31
		72.0				-49	-28	-77	

Table 27 Stringer Reactions at U10 wigh Deck and Stringers Effective

Member ID	Element Number	Gross Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	Total DL (Kips)	3D LL + I (Kips)	3D Total (Kips)	Plans (Kips)	Ratio 3D/Plans	3D Stress (Ksi)	3D Range (Ksi)
1	6992	15.6	6	21	3	30	19	49	127	0.39	3.14	1.41
		15.6				30	-3	27	127	0.21	1.73	**0.72**
2	6991	15.6	-4	2	-6	-8	7	-1	193	-0.01	-0.06	1.79
		15.6				-8	-21	-29	193		-1.86	**0.62**
3	6990*	12.3	1	12	-2	11	11	22	193	0.11	1.79	2.20
		12.3				11	-16	-5	193	-0.03	-0.41	**0.93**
4	6989	19.1	-12	-27	-2	-41	16	-25	-282	0.09	-1.31	2.83
		19.1				-41	-38	-79	-282	0.28	-4.13	**(0.84)c**
5	6988	19.1	-12	-22	1	-33	35	2	-282	-0.01	0.10	3.66
		19.1				-33	-35	-68	-282	0.24	-3.56	**0.89**
6	6987	31.2	-43	-89	0	-132	66	-66	-465	0.14	-2.12	6.48
		31.2				-132	-136	-268	-465	0.58	-8.59	**(1.17)c**
7	6985	31.2	-47	-103	9	-141	99	-42	-440	0.10	-1.35	8.85
		31.2				-141	-177	-318	-440	0.72	-10.20	**(1.46)c**
8	6984	31.2	-42	-87	-4	-133	66	-67	-465	0.14	-2.15	6.57
		31.2				-133	-139	-272	-465	0.58	-8.72	**(1.22)c**
9	6983	19.1	-13	-20	-2	-35	34	-1	-282	0.00	-0.05	3.72
		19.1				-35	-37	-72	-282	0.26	-3.77	**(0.99)c**
10	6982	19.1	-12	-25	-4	-41	16	-25	-282	0.09	-1.31	2.83
		19.1				-41	-38	-79	-282	0.28	-4.13	**(0.90)c**
11	6981*	12.3	1	13	-3	11	11	22	193	0.11	1.79	2.20
		12.3				11	-16	-5	193	-0.03	-0.41	**0.94**
12	6980	15.6	-4	3	-6	-7	7	0	193	0.00	0.00	1.79
		15.6				-7	-21	-28	193		-1.79	**0.63**
13	6979	15.6	6	22	3	31	19	50	127	0.39	3.21	1.41
		15.6				31	-3	28	127	0.22	1.79	**0.72**
32	7544	19.1	-1	11	-18	-8	71	63	207	0.30	3.30	6.59
		19.1				-8	-55	-63	207		-3.30	**2.5**
33	7543	25.0	30	87	-7	110	173	283	472	0.60	11.33	9.25
		25.0				110	-58	52	472	0.11	2.08	**2.78**
34	7542	25.0	31	89	-6	114	174	288	472	0.61	11.53	9.28
		24.9				114	-58	56	472	0.12	2.25	**2.78**
35	7541	19.1	1	13	-17	-3	70	67	207	0.32	3.51	6.54
		19.1				-3	-55	-58	207		-3.04	**2.5**

*Net area used due to splice   **Green numbers** give stress range due to one HS15 truck plus 15% Impact

Table 28 Chord Forces in Floor Truss at U14

Member ID	Element Number	Gross Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	Total DL (Kips)	3D LL + I (Kips)	3D Total (Kips)	Plans (Kips)	Ratio 3D/Plans	3D Stress (Ksi)	3D Range (Ksi)
14	7568	15.9	-10	-36	-22	-68	1	-67	-205		-4.22	3.27
		15.9				-68	-51	-119	-205	0.58	-7.49	**(2.06)c**
15	7567	10.3	1	-13	-6	-18	16	-2	-116		-0.19	6.41
		10.3				-18	-38	-106	-116	0.91	-10.29	**(2.43)c**
17	7566	27.1	-29	-88	-6	-123	9	-114	-369	0.31	-4.21	4.80
		27.1				-123	-121	-244	-369	0.66	-9.02	**(1.37)c**
18	7565	15.9	30	76	9	115	105	220	283	0.78	13.85	7.37
		15.9				115	-12	103	283	0.36	6.49	**2.28**
19	7564	10.3	-3	-13	1	-15	8	-7	-96		-0.68	3.20
		10.3				-15	-25	-40	-96	0.42	-3.88	**(1.42)c**
20	7563	15.9	-23	-59	-10	-92	18	-74	-201	0.37	-4.66	7.18
		15.9				-92	-96	-188	-201	0.94	-11.84	**(2.50)c**
21	7562	10.3	18	35	6	59	78	137	132	1.04	13.30	10.39
		10.3				59	-29	30	132	0.23	2.91	**3.35**
22	7561	10.3	-11	-29	-1	-41	18	-23	-117	0.20	-2.23	7.28
		10.3				-41	-57	-98	-117	0.84	-9.51	**(3.52)c**
23	7560	10.3	-11	-23	-7	-41	18	-23	-117	0.20	-2.23	8.06
		10.3				-41	-65	-106	-117	0.91	-10.29	**(3.94)c**
24	7559	10.3	17	35	6	58	81	139	132	1.05	13.50	10.78
		10.3				58	-30	28	132	0.21	2.72	**3.27**
25	7558	15.9	-22	-59	-12	-93	18	-75	-201	0.37	-4.72	7.18
		15.9				-93	-96	-189	-201	0.94	-11.90	**(2.55)c**
26	7557	10.3	-2	-14	2	-14	9	-5	-96		-0.49	3.20
		10.3				-14	-24	-38	-96	0.40	-3.69	**(1.51)c**
27	7556	15.9	32	76	9	117	105	222	283	0.78	13.98	7.37
		15.9				117	-12	105	283	0.37	6.61	**2.28**
28	7555	27.1	-30	-88	-6	-124	9	-115	-369	0.31	-4.25	4.51
		27.1				-124	-113	-237	-369	0.64	-8.76	**(1.37)c**
30	7554	10.3	1	-13	-6	-18	16	-2	-116		-0.19	5.24
		10.3				-18	-38	-56	-116	0.48	-5.44	**(2.34)c**
31	7553	15.9	-10	-38	-22	-70	1	-69	-205	0.34	-4.35	3.34
		15.9				-70	-52	-122	-205	0.60	-7.68	**(2.06)c**

Green numbers give stress range due to one HS 15 truck plus 15% impact.

Table 29 Diagonal Forces in Floor Truss at U14

Stringer	Beam Element	Gross Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	3D LL+ I (Kips)	3D Total (Kips)	Live load Truck Only
1	3309	72.0	-2.6	-11.9	-7.3	-21.8	-36.2	-58.0	1 Truck
	8741	72.0				-21.8	-37.3	-59.1	Critical
2	3310	72.0	0.9	-6	-1.8	-6.9	-37.1	-44.0	1 Truck
	8742	72.0				-6.9	-44.2	-51.1	Critical
3	3311	72.0	-9	-25.1	-2	-36.1	-50.2	-86.3	1 Truck
	8743	72.0				-36.1	-70.8	-106.9	Critical
4	3312	72.0	1.2	-6.2	1.2	-3.8	-37.8	-41.6	1 Truck
	8744	72.0				-3.8	-47.2	-51.0	Critical
5	3313	72.0	-1.4	-7.9	0.2	-9.1	-35.7	-44.8	1 Truck
	8745	72.0				-9.1	-45.3	-54.4	Critical
6	3314	72.0	-2.4	-11.8	-2	-16.2	-38	-54.2	1 Truck
	8746	72.0				-16.2	-47.7	-63.9	Critical
7	3315	72.0	-2.7	-10.7	-4	-17.4	-51.8	-69.2	1 Truck
	8747	72.0				-17.4	-56.1	-73.5	Critical
8	6705	72.0	-2.6	-13.8	-1.1	-17.5	-43.4	-60.9	1 Truck
	8748	72.0				-17.5	-47.4	-64.9	Critical
9	6706	72.0	-2.5	-12.1	-1.4	-16.0	-38	-54.0	1 Truck
	8749	72.0				-16.0	-47.5	-63.5	Critical
10	6707	72.0	-1.4	-7.8	-0.1	-9.3	-35.7	-45.0	1 Truck
	8750	72.0				-9.3	-45.3	-54.6	Critical
11	6708	72.0	1	-6.5	1	-4.5	-38.2	-42.7	1 Truck
	8751	72.0				-4.5	-47.1	-51.6	Critical
12	6709	72.0	-8.7	-24.7	-2	-35.4	-50.2	-85.6	1 Truck
	8752	72.0				-35.4	-70.8	-106.2	Critical
13	6710	72.0	0.7	-6	-1.9	-7.2	-37.1	-44.3	1 Truck
	8753	72.0				-7.2	-44.2	-51.4	Critical
14	6711	72.0	-2.6	-11.1	-7.4	-21.1	-36	-57.1	1 Truck
	8754	72.0				-21.1	-37.2	-58.3	Critical

Table 30 Stringer Reactions North Side at U14 with Live Load on North Side

Stringer	Beam Element	Gross Area (In^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	*3D LL+ I* (Kips)	3D total (Kips)	Live load Truck Only
1	3309	72.0	-2.6	-11.9	-7.3	-21.8	4.6	-17.2	1 Truck
	8741	72.0				-21.8	6.7	-15.1	Critical
2	3310	72.0	0.9	-6	-1.8	-6.9	9.5	2.6	1 Truck
	8742	72.0				-6.9	13.8	6.9	Critical
3	3311	72.0	-9	-25.1	-2	-36.1	0.9	-35.2	1 Truck
	8743	72.0				-36.1	1.2	-34.9	Critical
4	3312	72.0	1.2	-6.2	1.2	-3.8	7.6	3.8	1 Truck
	8744	72.0				-3.8	14.1	10.3	Critical
5	3313	72.0	-1.4	-7.9	0.2	-9.1	8.6	-0.5	1 Truck
	8745	72.0				-9.1	9	-0.1	Critical
6	3314	72.0	-2.4	-11.8	-2	-16.2	6.9	-9.3	1 Truck
	8746	72.0				-16.2	7.7	-8.5	Critical
7	3315	72.0	-2.7	-10.7	-4	-17.4	3	-14.4	1 Truck
	8747	72.0				-17.4	6.1	-11.3	Critical
8	6705	72.0	-2.6	-13.8	-1.1	-17.5	3	-14.5	1 Truck
	8748	72.0				-17.5	5.9	-11.6	Critical
9	6706	72.0	-2.5	-12.1	-1.4	-16.0	6.9	-9.1	1 Truck
	8749	72.0				-16.0	7.8	-8.2	Critical
10	6707	72.0	-1.4	-7.8	-0.1	-9.3	8.6	-0.7	1 Truck
	8750	72.0				-9.3	9	-0.3	Critical
11	6708	72.0	1	-6.5	1	-4.5	7.6	3.1	1 Truck
	8751	72.0				-4.5	14.3	9.8	Critical
12	6709	72.0	-8.7	-24.7	-2	-35.4	0.9	-34.5	1 Truck
	8752	72.0				-35.4	1.2	-34.2	Critical
13	6710	72.0	0.7	-6	-1.9	-7.2	9.5	2.3	1 Truck
	8753	72.0				-7.2	13.7	6.5	Critical
14	6711	72.0	-2.6	-11.1	-7.4	-21.1	4.6	-16.5	1 Truck
	8754	72.0				-21.1	6.7	-14.4	Critical

Table 31 Stringer Reactions North Side at U14 with Live Load on South Side

Stringer	Beam Element	Gross Area (in^2)	Stage 1 (Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	3D LL+I (Kips)	3D Total (Kips)	Live load Truck Only
1	3316	72.0	-2.5	-12	-7.2	-21.7	-33.6	-55.3	1 Truck
	8727	72.0				-21.7	-35.2	-56.9	Critical
2	3317	72.0	0.7	-5	-2.1	-6.4	-31.9	-38.3	1 Truck
	8728	72.0				-6.4	-38.5	-44.9	Critical
3	3318	72.0	-8.8	-26.8	-2.5	-38.1	-52.3	-90.4	1 Truck
	8729	72.0				-38.1	-85.9	-124.0	Critical
4	3319	72.0	1	-4.7	1	-2.7	-35.2	-37.9	1 Truck
	8730	72.0				-2.7	-44.8	-47.5	Critical
5	3320	72.0	-1.3	-8.8	1.8	-8.3	-28.2	-36.5	1 Truck
	8731	72.0				-8.3	-35.4	-43.7	Critical
6	3321	72.0	-2.3	-10.6	-2.6	-15.5	-34.6	-50.1	1 Truck
	8732	72.0				-15.5	-44.5	-60.0	Critical
7	3322	72.0	-2.7	-11.1	-4.3	-18.1	-52.5	-70.6	1 Truck
	8733	72.0				-18.1	-61	-79.1	Critical
8	6712	72.0	-2.6	-14.3	-0.8	-17.7	-43.7	-61.4	1 Truck
	8734	72.0				-17.7	-51.2	-68.9	Critical
9	6713	72.0	-2.3	-10.7	-2.1	-15.1	-34.6	-49.7	1 Truck
	8735	72.0				-15.1	-44.4	-59.5	Critical
10	6714	72.0	-1.5	-8.7	1	-9.2	-28.2	-37.4	1 Truck
	8736	72.0				-9.2	-35.4	-44.6	Critical
11	6715	72.0	1.1	-4.6	0.9	-2.6	-35.2	-37.8	1 Truck
	8737	72.0				-2.6	-44.8	-47.4	Critical
12	6716	72.0	-8.8	-27	-2.6	-38.4	-52.3	-90.7	1 Truck
	8738	72.0				-38.4	-86.5	-124.9	Critical
13	6717	72.0	0.7	-4.6	-2.1	-6.0	-31.9	-37.9	1 Truck
	8739	72.0				-6.0	-38.5	-44.5	Critical
14	6718	72.0	-2.5	-11.3	-7.1	-20.9	-33.5	-54.4	1 Truck
	8740	72.0				-20.9	-35.1	-56.0	Critical

Table 32 Stringer Reactions south Side at U14 with Live Load on South Side

Stringer	Beam Element	Gross Area (in^2)	Stage 1 Kips)	Stage 6 (Kips)	Stage 7 (Kips)	3D DL Total (Kips)	3D LL+I (Kips)	3D Total (Kips)	Live load Truck Only
1	3316	72.0	-2.5	-12	-7.2	-21.7	7.1	-14.6	1 Truck
	8727	72.0				-21.7	8.7	-13.0	Critical
2	3317	72.0	0.7	-5	-2.1	-6.4	16.4	10.0	1 Truck
	8728	72.0				-6.4	18.1	11.7	Critical
3	3318	72.0	-8.8	-26.8	-2.5	-38.1	1.2	-36.9	1 Truck
	8729	72.0				-38.1	1.5	-36.6	Critical
4	3319	72.0	1	-4.7	1	-2.7	10.8	8.1	1 Truck
	8730	72.0				-2.7	16.1	13.4	Critical
5	3320	72.0	-1.3	-8.8	1.8	-8.3	16.1	7.8	1 Truck
	8731	72.0				-8.3	19.7	11.4	Critical
6	3321	72.0	-2.3	-10.6	-2.6	-15.5	11.4	-4.1	1 Truck
	8732	72.0				-15.5	11.7	-3.8	Critical
7	3322	72.0	-2.7	-11.1	-4.3	-18.1	4	-14.1	1 Truck
	8733	72.0				-18.1	7.5	-10.6	Critical
8	6712	72.0	-2.6	-14.3	-0.8	-17.7	4	-13.7	1 Truck
	8734	72.0				-17.7	7.7	-10.0	Critical
9	6713	72.0	-2.3	-10.7	-2.1	-15.1	11.3	-3.8	1 Truck
	8735	72.0				-15.1	11.5	-3.6	Critical
10	6714	72.0	-1.5	-8.7	1	-9.2	16.1	6.9	1 Truck
	8736	72.0				-9.2	19.7	10.5	Critical
11	6715	72.0	1.1	-4.6	0.9	-2.6	10.8	8.2	1 Truck
	8737	72.0				-2.6	16.3	13.7	Critical
12	6716	72.0	-8.8	-27	-2.6	-38.4	1.2	-37.2	1 Truck
	8738	72.0				-38.4	1.4	-37.0	Critical
13	6717	72.0	0.7	-4.6	-2.1	-6.0	16.4	10.4	1 Truck
	8739	72.0				-6.0	17.9	11.9	Critical
14	6718	72.0	-2.5	-11.3	-7.1	-20.9	7	-13.9	1 Truck
	8740	72.0				-20.9	8.6	-12.3	Critical

Table 33 Stringer Reactions South Side at U14 with Live Load on North Side