THE COLLAPSE

OF

THE DOFASCO NO. 2 ORE BRIDGE

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MARCH 28th 1995

HAMILTON ONTARIO

by

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ABSTRACT

Dofasco started producing steel on their Hamilton bayfront property around 1952. Much of the iron ore and coal required for their blast furnace operations was delivered by ship. Two Ore Bridges, probably fabricated in 1951 and in continuous service since, were used to unload the ships.

On Tuesday March 28th, 1995 at approximately 8:40 am, the No.2 Ore Bridge collapsed.

At the time, the Ore Bridge appeared to be stationary. The operator was also not performing any specified operation. The collapse initiated with the failure of the tie-plate which keeps the Shear Leg from spreading at its base. The tie-plate is a critical member. As the structure contained no reasonable alternative load path for the tension in the tie-plate, the failure of the tie-plate resulted in the collapse of the Ore Bridge.

Initial calculations indicated there was no obvious structural overload that should have precipitated the catastrophic failure of the tie-plate. A metallurgical investigation of the tie-plate material was then initiated.

The metallurgical investigation found the steel in the tie-plate was susceptible to brittle fracture at the approximate air temperature at the time of the collapse. Using a fracture mechanics approach it was concluded the failure of the tie-plate was the consequence of fatigue cracks initiating in corrosion pits on the underside of the plate, along the toe of the reinforcing fillet weld connecting the tie-plate to the rocker block. The fatigue cracks grew and combined until they created a flaw which reached a critical dimension, allowing a brittle fracture to initiate and run rapidly across the width of the plate.

Over the years, the Ore Bridges have seen several alterations which increased the tension load in the tie-plate. In 1968 the apron was extended. In 1975 the trolley payload was increased. In 1990, increases in dead weight on the main span were recognised.

In 1990 however, Dofasco also modified their method of handling iron ore pellets on the bayfront, which required the addition of a hopper into the Shear Leg of the Ore Bridges. The addition of the hopper was critical, as it created unbalanced lateral loadings on the sill truss which were cyclical in nature. The hopper forced the tie-plate to resist these lateral loads by bending horizontally, a loading condition for which it was not originally designed. The cyclic bending stresses resulting from the addition of the hopper led directly to the failure of the tie-plate and the resulting collapse of the No.2 Ore Bridge.

ACKNOWLEDGEMENTS

To deal with the collapse of the No.2 Ore Bridge, Dofasco assembled a Failure Analysis Team consisting of Dofasco personnel who were to conduct an internal investigation of the incident.

Dofasco also retained an outside engineering firm to independently investigate the cause of the collapse. Dr. R.E. Southward, P.Eng. and the author, both of Southward Consultants Limited in Burlington Ontario, carried out the investigation and also involved the services of two metallurgists, Mr. T.R. Wood, P.Eng. of T.R. Wood Consulting and Mr. B.A. Graville, P.Eng. of Graville Associates Inc.

The four month investigation by Southward Consultants Limited culminated with the preparation of a formal report submitted to Dofasco. With Dofasco's permission, the main text and figures of the report are presented here as a project for the M.Eng. degree at McMaster University.

To that end, the author would like to gratefully acknowledge all of the members of the Dofasco Failure Analysis Team for their assistance and cooperation during the investigation. In particular, the following persons: Mr. D.C.F. Wilson, MBA, P.Eng., former General Manager - Environment, Health & Safety; his successor Mr. J.A. Macnamara for granting permission on behalf of Dofasco to allow the investigation report be used as the basis for this project submission; and Mr. K.W. Renshaw, P.Eng. who willingly offered his assistance in the coordination and review of this project.

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NOTE TO READER

The units in this report are Imperial to remain consistent with the original drawings, specifications and calculations which were produced for the No.2 Ore Bridge.

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1. BACKGROUND

Dominion Foundries and Steel Ltd. (Dofasco) started producing steel on their Hamilton bayfront property around 1952. Much of the iron ore and coal required for their blast furnace operations was delivered by ship. As a result, Dofasco contracted Heyl & Patterson (H&P) of Pittsburgh Pennsylvania to design two Ore Bridges, to unload the ships. H&P had been supplying Ore Bridges to North American steel mills for many years. The Dofasco Ore Bridges were designed by H&P during 1950. The detailing and fabrication was contracted out to a Hamilton Ontario firm, Hamilton Bridge Company. Hamilton Bridge was a major fabricator of bridge structures in Canada and was established around 1863. Hamilton Bridge changed their name to Bridge & Tank in 1954 and ceased operations in 1984. Heyl & Patterson remain in existence today.

The Dofasco Ore Bridge presently known as No.2 was apparently fabricated first, probably during late 1950 and early 1951. Ore Bridge No.1 was probably fabricated during 1951. Both Ore Bridges have essentially seen continuous service since they were erected on the Dofasco bayfront, in preparation for the commencement of the steel making operations. When they were first designed, the Ore Bridges had a live load capacity of 12 Tons. In 1961 a third H&P Ore Bridge was added to the Dofasco bayfront which had a capacity of 20 Tons. Ore Bridges No.1 and No.2 were subsequently upgraded by H&P, around 1975, to 17.5 Tons capacity. In 1990 Dofasco changed the method of handling iron ore on their bayfront property, apparently because of the closure of certain mines. Instead of loading ore directly from the Ore Bridges into rail cars on the Hi-Line, to charge the blast furnaces, hoppers were added into the Shear Leg of each of the three Ore Bridges. Rail cars were then filled with ore directly from the new hoppers.

On Tuesday March 28th, 1995 at approximately 8:40 am, the No.2 Ore Bridge collapsed.

2. ORE BRIDGE FUNCTIONS

Figure 1 is an elevation of a typical Dofasco Ore Bridge. This particular drawing depicts the configuration of the three Ore Bridges on the Dofasco bayfront, looking towards the north. (Dofasco, 1995)

Ships, typically laden with iron ore pellets, arrive at the Dofasco No.1 Dock, at the extreme right or east side of the Figure, (A). The Ore Bridge operator rides in a cab (B) next to the bucket (C), both of which are supported from a trolley (D). The operator drives the trolley along the main span (E), onto an apron (F) and out over the ship. This operation is called *"trolleying"*. When the bucket is over the open hatch of the ship it is lowered, a load is picked up, the bucket is raised clear of the ship and the operator trolleys back over the ore pile, to drop the load. This operation is repeated until the hatch is emptied. The operators refer to the ore pile as "*the field*".

When a hatch is empty, the entire Ore Bridge is driven either north or south, by the operator, to line up with another hatch on the ship. This operation is called *"bridging"*. Two or three Ore Bridges may be working on one ship simultaneously. Eventually, through repeated trolleying and bridging of the Ore Bridges, the ship is emptied.

Once the iron ore from the ships is stockpiled, the Ore Bridges are used in the process of charging the pellets into the blast furnaces. The operator picks up a load of pellets in the bucket from the field and trolleys west, to fill up rail cars on the Hi-Line (G). Once the rail cars are filled they travel along the Hi-Line to a blast furnace (H), where their load is dropped through a hopper, into a scale car and then into a skip bucket (I). The loaded skip bucket then travels up an incline (J) and empties the pellets into the top of the blast furnace.

At Dofasco, the Ore Bridges also performed another function.

Figure 2 is identical to Figure 1, except a large coal hopper (K) is now visible on the east side, near the dock. (Dofasco, 1995) The process of turning iron ore pellets into molten iron, then steel, also requires the blast furnace to be charged with coke, which is produced from coal. The Ore Bridges were used to unload coal ships and load the coal into the coal hopper. Rail cars were then driven under the hopper. The coal was dropped from the hopper into the rail cars, then transported to the coking ovens.

The use of the Ore Bridges to handle coal at Dofasco was minimised in the 1980's, as alternative coal handling facilities were introduced.

Figure 3 is an isometric view of the Dofasco No.2 Ore Bridge as it existed at the time of the collapse. The drawing was prepared by Dofasco. (Dofasco, 1995) The view looks towards the north and west. The drawing identifies the various components of the Ore Bridge already discussed, namely the operators cab, bucket, trolley, main span and apron. Other relevant features are also identified.

The support for the main span on the west side, near the blast furnaces is known as the *"Pier Leg"*. The north and south legs of the Pier Leg bear on *"trucks"*, which ride along the dockyard on rails supported by ballast sitting on grade. The two legs of the Pier Leg are tied together at the bottom by a steel framework called a *"sill truss"*.

The support for the east side of the main span, near the dock, is known as the *"Shear Leg"*. The north and south legs of the Shear Leg also bear on trucks, which ride on rails supported by ballast sitting on a pile cap. The two legs of the Shear Leg are also tied together across their base by a steel sill truss.

In 1990 Dofasco modified their method of handling iron ore pellets which included the addition of a hopper into the sill truss of the Shear Legs. The hopper was designed by H&P and added into the No.3 Ore Bridge in March of 1990 and into the No.1 and No.2 Ore Bridges during January 1991.

With this modification, the iron ore pellets were no longer loaded by the Ore Bridges into rail cars on the Hi-Line. Instead, the Ore Bridge operators loaded the pellets into the hoppers on the Shear Leg. The pellets were then dropped from the bottom of that hopper onto a conveyor, which discharged them into a 100 Ton Barrel Car, Figure 4. (Dofasco, 1995) A train containing the filled Barrel Cars then transported the pellets to the Hi-Line.

In Figure 4, the hopper (viewed looking north) is generally identified as (L), the conveyor as (M), the conveyor discharge chute as (N) and the 100 Ton Barrel Car as (O). Additional items of interest on the hopper are the upper dribble shields (P), the conveyor counterweight (Q), the hopper supports (R) and the hopper support upper and lower guide locations, (S) and (T) respectively.

3. ORE BRIDGE OPERATIONS

The basic operation of the Ore Bridge is *"trolleying"*, the movement of the bucket east and west along the main span to unload ships, stockpile iron ore pellets in the field and fill rail cars or hoppers. As well, the entire structure can be *"bridged"* north and south to position the Ore Bridge over a ship, or a new location on the stockpile.

The operator can also rotate the bucket, using the turntable on the trolley, and can either lift and lower the bucket or open and close the bucket. The operator can trolley and bridge at the same time. When he is bridging however, he can not operate the bucket.

When bridging, the Pier Leg is driven by its trucks, independently of the trucks driving the Shear Leg. As a result, the Pier Leg and Shear Leg can become misaligned, causing a *"skew"* of the main span. The inevitability of skewing was recognised by H&P who designed skew limits into the Ore Bridge control system, to prevent the Ore Bridges from being structurally damaged. (Dofasco, 1995)

The first *"Control Limit"* was set by H&P at 15 feet of skew misalignment over the 300' length of the main span between the Pier Leg and the Shear Leg. In fact, Dofasco reduced this working limit to approximately 10 feet. A skew indicator hangs on the Pier Leg, which the operators monitor constantly.

The Control Limit is an electrical limit. If the skew of the main span exceeds the Control Limit the operator is prevented electrically from driving the legs of the Ore Bridge to increase the skew any further. Clamps which hold the Pier and Shear Legs to the rails are also engaged. The operator can then release the rail clamps but can only drive the trucks to bring the main span back within the working skew range.

If for some reason the Control Limit is exceeded, a mechanical *"Power Limit"* is activated when, according to the H&P design, a skew misalignment of 17.5 feet is reached. Dofasco in fact have set this Power Limit at approximately 14 feet. When the Power Limit is exceeded, a gravity operated switch is automatically activated, all power to the Ore Bridge is shut down and the rail clamps are engaged. To correct this situation, the operator must call the maintenance department to re-energise the Ore Bridge.

The H&P drawings indicate that if the Control and Power Limits are both exceeded, steel stop blocks exist at a skew of 20 feet, which physically prevent the main span from becoming misaligned any further. (Dofasco, 1995)

The Ore Bridge operators indicated they monitor the skew of the Ore Bridges constantly and seldom exceed the working limits. Only twice since 1952 did the operators recall any Power Limit being activated.

During normal operations of the Ore Bridges it is not uncommon for the operators to skew their bridges on purpose. When unloading ships for example, the spacing between hatches might be such that two bridges can not sit side by side and trolley directly over a hatch. By driving slightly out of skew however the main span is rotated slightly, swinging the apron over the ship and allowing the bucket to enter the hatch. The Ore Bridges were in fact detailed by H&P to accommodate skewing. The Shear Leg was purposely misaligned vertically such that it leans outward or to the east, 3" at the top when there is no skew, and inward or to the west, 5" at the top at the 20 foot skew limit.

When the Ore Bridges are bridged, warning lights and horns go on and off to indicate they are moving. If two bridges come close together, there is a bumper or paddle on the ends of the trucks, which activates a collision limit stopping the bridges from moving if they come in contact.

The operations of the Ore Bridge can also be shut down automatically by anemometers mounted on the motor/generator (MG) house over the Pier Leg and the hoist room over the Shear Leg, (Figure 3). If either anemometer registers wind speeds exceeding 32 mph (51.5 km per hour), bridging operations are shut down immediately. Both the rail clamps and the truck brakes are engaged automatically. The operation of the trolley is not affected. If the bayfront wind speeds reach 40 mph, the operators are to block the wheels of the Ore Bridges.

Another operation of the Ore Bridges has evolved recently, since the iron ore pellet hoppers were added to the Shear Leg. The train which brings the 100 Ton Barrel Cars to the dockside, to be filled from the hoppers, typically consists of seven to twelve cars. Two Ore Bridges are normally stationed side by side to fill the cars. As one pair of cars is filled, the train indexes the Barrel Cars under the Ore Bridges, to allow another pair of cars to be filled. Sometimes however, the trains are not indexed. The Ore Bridges must then be bridged to align with the empty Barrel Cars.

Typically, the recently added Shear Leg hoppers would not be full of iron ore pellets when the Ore Bridge is bridged. The hoppers could be full however. H&P have apparently advised Dofasco that bridging with full hoppers is acceptable. The practice has not been encouraged by Dofasco however, to minimise wear on the equipment.

4. MODIFICATIONS TO THE NO.2 ORE BRIDGE

Based on discussions with Dofasco personnel and a review of the Dofasco drawings, several modifications have been made to the No.2 Ore Bridge since it was built. The following list includes several of the relevant changes.

- Around 1968, the apron was extended approximately 8 feet.
- In 1974 or 1975 the live load capacity was increased from 12 Tons to 17.5 Tons.
- In 1981 various reinforcing plates were added to the horizontal tie-brace which connects the sill truss to the bottom of the Shear Leg, because of fatigue cracking.
- In 1990 the trolley saw major revisions including a new turntable, new wheels, and new trolley rails, which were increased from 104 to 135 lbs/yard. The operator controls were also changed to a solid state, programmable logic controller, (PLC).
- In 1991 the hinge pins for the apron were modified.
- In 1991 the Shear Leg hopper was also added along with structural modifications to the sill truss and the tie-braces between the sill truss and the bottom of the Shear Leg.
- Single trucks under the Shear Leg were replaced in August, September and December of 1992. The three remaining trucks were replaced in 1993.

5. THE INCIDENT

During January and February 1995, the No.2 Ore Bridge underwent planned maintenance during the normal winter shut-down period. The dockyard shut-down extended generally from the close of Great Lakes shipping in late December to the re-opening of shipping in March. As iron-ore is stockpiled during the summer shipping season, the winter is a slower time for the Ore Bridges, as they are essentially used only to load Barrel Cars.

A two year programme had been initiated by Dofasco which involved sand blasting the No.2 Ore Bridge to bare metal, examining the structure visually for damage, repairing the damage and then re-painting. The programme began in January 1995 at the west or Pier Leg side and proceeded towards the east or Shear Leg side. Only the westerly half of the structure was scheduled to be completed in the first year. Work on the Shear Leg side was scheduled for the 1996 shut-down. Work started at the west side because of the proximity to the blast furnace slag pits and their potentially more corrosive environment.

Additional work was also scheduled in early March 1995 on the Shear Leg hoppers on all three Ore Bridges. Guides for the hoppers had worn since 1991 and there was also a concern that they could become wedged. As a result, new stainless steel guides were installed. As well, the channel supports for the upper guides were replaced. The channels exhibited cracking soon after being installed and were permanently bowed upward. The work was completed on all three Ore Bridges in time for the opening of the 1995 shipping season.

The first ship into Dofasco in 1995 was the Canadian Olympic, which arrived empty on March 22nd to be loaded with slag. All three Ore Bridges worked on loading the ship initially, but No.1 left early for another task. The loaded ship left the Dofasco dock in the late afternoon of Thursday March 23rd. The aprons on all three Ore Bridges were then raised as they returned to loading rail cars.

Operators on the Ore Bridges work twelve hour shifts and can load 50 to 60 of the 100 Ton Barrel Cars in a shift. During the night shift on Monday March 27th 1995, the No. 2 Ore Bridge was working alongside the No.3 Ore Bridge to load Barrel Cars. It was the first shift back for the night shift operator of the No.2 bridge, since the January shut-down. The last Barrel Car was loaded about 3:00 am on Tuesday morning, March 28th. Nothing appeared unusual on the No.2 Ore Bridge, except that for a short time the operator heard a regular clicking sound which seemed to come from the trolley above. He investigated but found nothing abnormal.

The day shift starts at 6:30 am. On Tuesday March 28th, the day shift operator of the No.2 Ore Bridge was apparently late. As a result, the operator of No.3 Ore Bridge (who normally operated No.1 and was being trained on No.3), spent the first part of the shift familiarising himself with the controls. He bridged the No.3 north and practised loading and unloading the bucket and also lowered the apron. At about 8:00 am a train arrived with nine empty Barrel Cars and an additional car to be filled with limestone. By then, the operator of the No.2 had arrived and the No.3 bridge was repositioned alongside No.2 to start loading the Barrel Cars. The No.2 Ore Bridge had apparently not been moved from the position in which it was left by the night shift operator.

Just as they were starting to load the rail cars, the service tractor for the Ore Bridges arrived and both operators lowered their buckets for greasing. The No.3 Ore Bridge was greased first and then started to load a rail car. After that car was loaded, the two Ore Bridges began working together to load the remaining rail cars.

The train operator indexed the rail cars underneath the stationary bridges as they were being filled. After seven cars were filled, the No.3 Ore Bridge was told to leave and bridge north to fill the limestone car, then change his bucket. A ship was expected soon which was to be loaded with coke, which involved the use of a larger bucket. The No.2 Ore Bridge stayed in position and filled the two remaining iron ore pellet cars.

Once all nine of the Barrel Cars were loaded, the train driver left with the full cars. He then told the operator of the No.2 bridge he was going to return with a train of seven more cars to be filled, after which they would take a break. The No.3 Ore Bridge then finished filling the limestone car and started bridging north to have his bucket changed. The time was approximately 8:40 am.

The operators sit in their cabs looking south. The operator of the No.3 Ore Bridge looks directly at the No.2 Ore Bridge. The operator of No.3 was bridging north and advised he was positioning his cab to align with the cab of No.2 when suddenly, with no apparent warning, the east end of the No.2 Ore Bridge collapsed. The operator of the train which had just left looked back, and saw the collapsed Ore Bridge.

The few people who were in the immediate area at the time of the collapse do not recall either seeing or hearing the No.2 Ore Bridge or its trolley moving. The operator was sitting in his cab. There was no apparent reason for him to be moving or performing any specified operation. An eye-witness on the Boiler House roof indicated however that he saw the No.2 bridging to the north just minutes before the incident.

The PLC contains an electrical record of the last operations of the Ore Bridges. At the instant power was lost on No.2, the previous PLC records indicate the trolley was reversing motion. It had been moving slowly towards the west and was probably being *"plugged"*. Plugging is a sequence whereby the operator effectively puts the trolley in reverse, to assist in slowing the trolley and to control the swing of the bucket.

The Shear Leg was not recognised by the PLC as moving, however the controls were set for that Leg to head south, at *"first point"*, the lowest speed setting. The Shear Leg may have been travelling north, slowly, and was also being brought to a stop by engaging first point south, which is apparently common practice amongst the operators. The Pier Leg controls were not set to command it to move, however the PLC registered it moving very slowly northward and decelerating, within milliseconds of the collapse. (Dofasco, 1995)

After filling the rail cars, the operator may have simply moved the No.2 Ore Bridge north over a better spot on the ore pile, or more likely, had started bridging north to change his bucket along with No.3, when he was told another iron ore train was coming to be filled. Apparently it was his practice to sit directly over the Shear Leg with a loaded hopper and a full bucket, if he was waiting to fill rail cars. It was also his practice to position his trolley in the approximate location it was in at the time of the collapse, if he was bridging.

The night shift operator indicated that when he saw the wreckage, the Ore Bridge appeared to be in the same place that he had left it.

6. THE COLLAPSED STRUCTURE

The following photographs, taken by Dofasco, generally record the collapsed structure.

Figure 5 is a view looking south at the No.1 Ore Bridge, which is essentially identical to the No.2 Ore Bridge. The apron is in the up position, which was the situation on No.2 at the time of the collapse. Figure 6 was taken looking north, with the collapsed No.2 Ore Bridge in the foreground and the No.3 Ore Bridge standing behind. The apron on No.3 is down, as it was left by the operator of that bridge when he was familiarising himself with the controls, just prior to the collapse. Barges are already in position recovering parts of the No.2 that fell into the water.

Figure 7 was taken looking towards the west, from across the water. The photograph records the relative positions of the Pier Leg, the main span and the Shear Leg hopper. The Pier Leg and the majority of the main span appeared relatively intact, except for consequential damage, Figure 8.

The east end of the main span impacted the edge of the dock, with the apron over the water, Figure 9. The apron was ripped off by the impact and ended up in the water. The top of the trolley can be seen in the lower right hand corner of the photograph and the top of the hopper in the Shear Leg in the lower left hand corner.

Figure 10 was taken from the west side of the Shear Leg of the No.1 Ore Bridge looking towards the east and is presented for reference purposes. The leg on the left is the north leg and the leg on the right the south leg. This orientation of the Shear Leg will form the basis of the remainder of this report.

Figure 11 is a general view of the condition and location of the north leg of the Shear Leg of the collapsed No.2 Ore Bridge. It is lying flat on the dockside. The leg remains attached to the sill truss underneath the hopper. The top end of the north leg is badly distorted. The main span of the bridge is to the right or south of the hopper. The portion of the Shear Leg which connects the north and south legs directly underneath the main span, *"the haunch"*, is noted by the arrow. It is completely detached and lying upside down between the hopper and the main span.

Figure 12 records the condition and location of the south leg of the Shear Leg, with the main span on the left, or north. The south leg remained partially upright, as can be seen in Figure 7, but was severely damaged and significantly more distorted than the north leg.

7. THE GEOMETRY OF THE COLLAPSE

The general configuration of the collapsed structure was carefully measured. Figure 13 is a sketch of the location of the Ore Bridge wreckage on plan, following the collapse. Because it was suspended by cables, it is believed the bucket would fall vertically during the collapse, which was confirmed by the examination of the bucket and the impression it left in the ground. Based on the final location of the bucket, relative to the Pier Leg, it does not appear the Pier Leg moved along the tracks during the collapse. The overall measurements also suggest the main span was not skewed outside its working range, in the moments leading up to the collapse.

The east end of the Ore Bridge is supported by the Shear Leg. It was apparent from the distorted shape of the collapsed structure that considerable structural trauma had occurred within the Shear Leg. The wreckage around the Shear Leg is detailed in Figure 14.

Figure 15 is a sketch of the Shear Leg, prior to the collapse, with several of its component parts defined. The main span of the bridge is also included in the sketch, with its top and bottom chords and the trolley girders and their support beams defined.

The main span bears on the centre of the Shear Leg, at the top. The weight of the main span is concentrated in a shear girder or *"cross-head"* and is transmitted into the Shear Leg through a 3.25" thick, 9.990" diameter steel bearing pad, which is known as a *"pill"*, puck or aspirin. The pill allows the main span to rotate freely in a horizontal plane (skew) relative to the Shear Leg. There is a functionally similar load transferring arrangement at the top of the Pier Leg.

Wind blowing laterally against the main span, and the inertia of the main span associated with the starting and stopping of bridging, will cause the main span to deflect and twist along its length and rock back and forth in a north/south direction. *"Balance pads"* sit on the top of the Shear Leg, at the north and south corners, to limit this rocking motion.

There is a theoretical clearance of about 1/8" between the underside of the main span and the top of the Shear Leg, at each of the balance pads. The main span actually has a natural twist along its length because of the unbalanced weight of various service equipment and walkways on the span. This imbalance usually results in the north balance pad bearing on the top of the Shear Leg and transferring some of the weight of the main span into the Shear Leg.

8. THE COLLAPSE SEQUENCE

There were some obvious markings on the Shear Leg and main span, following the collapse, which provided clues to the probable sequence of the Ore Bridge collapse.

Figure 16 records the haunch, or top portion of the Shear Leg, which separated completely from the remainder of the Shear Leg during the collapse. The north side is on the left. The top surface of the haunch contained numerous distinct markings, Figure 17. As well, the underside of the main span cross-head contained several unique features, typical examples of which are recorded in Figures 18 and 19.

The various scrape marks, grease imprints, scoring and burring on the underside of the main span cross-head and the top of the Shear Leg were measured and sketched, Figure 20. The markings are shown in their relative positions looking directly down from above the Ore Bridge, as if the markings could be seen through the steel.

As well, very distinctive markings could be seen on the inside of the south leg of the Shear Leg near the bottom, Figures 21 and 22, and on the top of the south equaliser beam, Figures 23 and 24.

A detailed analysis of the markings at the top of the Shear Leg, and of the markings at the bottom inside face of the south leg of the Shear Leg was completed. By combining these analyses the geometry of the Shear Leg during the various sequences of the collapse can be traced.

Figure 25 was taken looking west at the south leg of the Shear Leg of the No.1 Ore Bridge, which was identical to the No.2 Ore Bridge. This area is detailed in Figure 26 which is a sketch of the south leg looking east. For the purpose of discussion, several of the component parts at the base of the Shear Leg are defined in the sketch.

In Figure 27, the first stages of the collapse are depicted, which included the top of the south leg separating from the underside of the main span (1). This action must have been the result of structural damage initiating within the south leg of the Shear Leg. The most obvious location for this damage was at the bottom (2).

The dropping of the south leg away from the cross-head also shifted the weight of the main span onto the north balance pad. The outer flange and tension diagonals in the north leg were then subjected to a significant instantaneous increase in load (3).

A closer examination of the bottom of the south leg indicated the tie-plate had failed near the weld at the rocker block, allowing the tie brace at the bottom of the south leg of the Shear Leg to separate from the leg. Various markings around the bottom of the south leg required the horizontal web member at the bottom of the south leg, which runs between the inside and outside legs, to have pulled apart. As well, the vertical and diagonal braces must have buckled, as illustrated.

The collapse sequence is continued in Figure 28, which depicts the top south side of the Shear Leg dropping away from the main span cross-head sufficiently that the pill is no longer in contact with the main span (4).

The entire top of the Shear Leg was now rotating towards the south, relative to the underside of the cross-head, and the south balance pad ended up outside the south side of the main span. The entire weight of the main span was now bearing directly on the north balance pad, which was also sliding south underneath the cross-head. This action placed unusual bending stresses in the top of the north leg causing further structural damage (5).

The outside flange of the north leg fractured and separated at locations marked "B" and "D". All the tension diagonals also fractured, with minimal initial distortion of the north leg.

Because the south leg was collapsing inward, due to the loss of the tension tie and diagonal members at the bottom, and the main span had not yet begun to rotate appreciably, contact was made between the bottom chord of the main span and the inside of the south leg (6). The contact was quite severe and most likely initiated the buckling of the diagonal members at the top of the south leg.

Figures 29 and 30 record the fractured north leg at locations "B" and "D", while Figures 31 and 32 record a typical failed tension diagonal. Heavy scraping at the location of the impact of the bottom chord of the main span with the inside of the south leg is recorded in Figure 33.

In Figure 34, the main span has begun to drop and rotate towards the south, as the lower portion of the south leg continued to collapse inwards. This behaviour was enhanced by the buckled diagonal members at the top of the south leg and the hinge forming at that point (7). The main span had now also become wedged against the top of the Shear Leg by the keeper rings which originally held the pill, (8). The weight of the main span on the damaged Shear Leg continued to distort the upper portions of the north leg (9) and lower portions of the south leg (10).

The development of the hinge near the top of the north leg also forced the north truck to start moving towards the north, pulling the sill truss and hopper. Markings on the rails suggest the east rail clamp was probably engaged at this point, although the west rail clamp may not have been engaged.

The collapse continued to develop, as depicted in Figure 35. The keeper rings on top of the Shear Leg and underneath the cross-head of the main span remain wedged together, (11). The collapsing north leg was also now propelling the north trucks, sill truss and hopper north (12), while the forces on the south trucks were pushing them south (13). The relative movements between the north and south trucks also started to pull the tie members free of the south end (14).

In Figure 36, the main span has now slipped over the top of the keeper rings and the balance pad on the south side has made contact with the underside of the cross-head (15). As well, the sill truss and hopper are tipping towards the water as the falling main span was forcing the top of the Shear Leg towards the east. The bottom chord of the sill truss on the east side actually made contact with the wooden railway ties, as it was moving north (16).

The diagonal brace at the bottom of the south leg straightened as the north trucks, sill truss and hopper continued to separate from the collapsing south leg (17). The inside face at the bottom of the south leg was also hitting the front edges of the south trucks, just before the south leg was driven between the two rails of the track into the wooden ties, (18). The final position of the bottom of the south leg, relative to the south trucks, is recorded in Figure 37, while Figure 38 records the inside of the south leg buried in the wood ties and ballast under the tracks.

Once the inside of the south leg contacted the ground, Figure 39, and the ties which originally held the north and south trucks together had pulled free (19), the north leg was completely released from the south leg, along with the sill truss and hopper. With the bottom of the south leg firmly embedded in the ground, the entire weight of the falling main span also impacted on the top of the south leg, fracturing the tension diagonals in the south leg, (21). The bottom of the sill truss (20) left a record of its northerly movements in the ballast and torn railway ties on the east side of the track, Figure 40.

In Figure 41, the main span is shown continuing its free fall as the haunch section slides under the cross-head and then breaks completely free (22), slipping out through the north side of the main span. The south leg, which had lost the stiffening benefit of its diagonals, had minimal bending capacity and was bent over double, fracturing the inside flange of the leg, (23). Figure 42 is a view of the south leg, after it was removed from the wreckage.

The final configuration of the various parts of the No.2 Ore Bridge Shear Leg and its hopper is shown in Figure 43. The north leg (24) is shown standing, but it actually ended up lying on its side on the dock (see Figure 11). The hopper (25) was also leaning over towards the east and was partially on top of the detached haunch section (26) which landed upside down on the ground. When the main span hit the ground it landed on top of the combination of tension, vertical and diagonal tie members which had separated from the bottom of the south leg (27), bringing the moving north leg, sill truss and hopper to a standstill.

Figure 44 records the upside down haunch. It is believed the haunch landed on the south bottom chord of the main span, damaging the middle tie member or cross beam, as noted in the Figure, then flipped towards the north landing upside down under the hopper. Figure 45 records the main span on top of the tie members.

9. EYE WITNESS ACCOUNTS

The Ore Bridges are located in a remote area of Dofasco. As a result, there were few eye witnesses to the incident. The operator of the No.3 Ore Bridge had just finished loading the limestone car and was preparing to have his bucket changed, to load an export ship with coke. The operator in No.3 was experienced on the No.1 bridge but was being trained on No.3. As a result, there was a regular No. 3 operator in the cab with him. There were also several maintenance people at ground level, sitting in a van waiting to change the bucket. The van was parked in the field near the bucket, north of No.3 and was headed north. An Ore Bridge operator was driving the van.

The men in the van indicated they heard cracking noises and turned to look back towards the No.2 Ore Bridge. The main span appeared intact and the east end was falling between the legs of the Shear Leg. The tip of the main span hit first, crashing into the edge of the dock. The remainder of the east end was flattened as it hit the ground. The Ore Bridge operator driving the van indicated he sensed the trucks under the south leg moved towards the south, which is not inconsistent with the sequence that has been depicted.

The trainee operator in No.3 was apparently bridging north at the time, towards the replacement bucket and advised he was watching the skew indicator as well as the position of the No.2 bridge. He also indicated he was trolleying to align his cab with the cab on No.2. The regular operator in the cab of No.3 indicated he was looking west towards the skew indicator and also north to the van and bucket on the ground and did not see or hear the collapse.

The trainee operator in No. 3 first indicated he saw what appeared to be an explosion of dust on the top east side of the north leg of No.2. Later he advised the explosion was at the bottom chord of the main span, east of the Shear Leg. It is believed, he probably saw the result of the sudden fracture of the outer north leg of the Shear Leg, as depicted in Figures 28, 29 and 30. Alternatively he may have witnessed the results of the bottom chord of the main span impacting the inside of the south leg of the Shear Leg, as depicted in Figure 28.

He then indicated he saw what appeared to be the underside or trolley girder portion of the main span separate from the main span and sag down in the vicinity of the Shear Leg. Upon close examination it was apparent the trolley girder portion did not separate from the main span, until it hit the ground. It is believed the operator probably saw the south trolley girder and bottom chord of the main span appear underneath the main span, as it rotated, as depicted in Figures 34, 35 and 36. Another eye-witness on the Boiler House roof also indicated he saw the main span rotate towards the south.

The trainee operator in No.3 Ore Bridge then recalled the main span simply dropping straight down between the north and south legs, which remained standing upright temporarily, which would compare to Figures 41 and 43. Prior to the main span hitting the ground however he suggested the main span wrapped itself up, over and around the hopper, which clearly did not happen.

Lastly the operator recalled the hopper vibrating and shaking violently and then suddenly shooting out to the north from underneath the main span. It is suspected he saw the cumulative result of the hopper being jostled about, Figure 34; the hopper moving south as the ties straightened, (14 in Figure 35); then the hopper shaking as the ties tightened, (17 in Figure 36); before the ties failed in tension, (19 in Figure 39); which allowed the sill truss and hopper to move rapidly towards the north.

The No. 3 operator is seated such that he can look directly south at the No.2 Ore Bridge. The location of the No.3 Ore Bridge was measured relative to the No.2 following the incident. They were approximately 570 feet apart. The resulting sight lines for the operator in the cab of No. 3 are shown in Figure 46, which suggest it was unlikely he could see the initial damage occurring at the bottom of the south leg of the Shear Leg.

The entire collapse sequence probably only lasted four or five seconds.

10. THE ORIGINAL ORE BRIDGE LOADING CONDITIONS

The H&P drawings provide a summary of the loads and design for the original main span chord members, the main span web members, the Pier Leg and the Shear Leg. Other drawings analyse and design the various other structural framing members throughout the Ore Bridge and the apron. (Dofasco, 1995)

A major load in the structure is its own dead weight. The H&P drawings indicate the dead weight of the main span is 617,000 pounds or 617 kips (kilopounds) per side. The total dead weight of the main span is thus 1,234,000 pounds or 1,234 kips. (Dofasco, 1995) Because this weight is significant, a former Bridge & Tank estimator was retained to calculate the weight in several areas of the span. The estimator found the original estimated weight to be approximately 3% too light, not including any allowance for connections, modifications and electrical and mechanical services.

H&P also advised Dofasco in a 1990 letter the original dead weight was approximately 12.6% too light when the weight of various modifications, connections and electrical feed lines were included. As a result, the originally assumed dead weight of the main span was increased by 10%, to 1,357.4 kips, while maintaining the original weight distribution shown on the H&P drawings. (Dofasco, 1995)

Another significant weight is that of the trolley, a moving load which traverses from one end of the main span to the other.

The H&P drawings indicate the originally assumed dead weight of the trolley was 150,000 pounds, the weight of the bucket 26,000 pounds and the lifted payload 24,000 pounds (12 Tons). The total weight was thus 200,000 pounds or 200 kips. (Dofasco, 1995)

In the previously mentioned 1990 letter to Dofasco, H&P list actual weights recorded for the component parts of the trolley when it was being retrofitted in 1990. The corrected dead weight of the trolley became 192,657 pounds, the bucket 28,800 pounds and with the new payload of 35,000 pounds (17.5 Tons) the total weight was 224,957 pounds. The total weight of the fully loaded trolley was thus assumed to weigh 225 kips. (Dofasco, 1995)

When designing the trolley girders and the cross frames, H&P originally assumed an increased static load to account for the impact associated with the trolley operations of 50% of the total trolley weight, including the lifted loads. They also assumed a 30% impact factor for the fully loaded trolley when designing the members in the main span. H&P in their 1990 calculations actually reduced the impact factor for designing the main span to 25% of the total trolley weight, which is more typical of modern standards. (Dofasco, 1995)

H&P designed the Ore Bridges assuming they would remain in operation with a wind load acting on the structure of five pounds per square foot (psf). (Dofasco, 1995) A 5 psf wind pressure is equivalent to a 43 mph wind speed. The total assumed wind load acting on the main span during operating conditions was thus 46.6 kips, distributed as 24.33 kips at the top of the Shear Leg and 22.27 kips at the top of the Pier Leg. They also assumed the Ore Bridge would be required to safely withstand a maximum wind pressure of 30 psf or 105 mph. The Ore Bridge was not assumed to be lifting a payload under these maximum wind conditions.

Both the above wind loading assumptions appear reasonable.

The horizontal force associated with skewing or misalignment of the Ore Bridge was assumed to be 150.48 kips, which is 20% of the maximum weight acting on the Pier Leg of 752.4 kips. (Dofasco, 1995) This horizontal force is equivalent to the friction which must be overcome to rotate the main span and its trolley horizontally over the Pier leg, which is reasonable. The horizontal force at the top of the Shear Leg required to overcome this friction is 13.98 kips.

H&P also assumed the action of operating the apron created an effective side loading on the Ore Bridge. (Dofasco, 1995)

11. THE ORIGINAL ORE BRIDGE DESIGN

According to the drawings, the No.1 and No.2 Ore Bridges were designed by H&P in the United States in 1950. The drawings did not indicate the grade of steel used in the bridges, but it was assumed to be American Society for Testing and Materials (ASTM) A7, "Standard Specification for Steel for Bridges and Buildings", the common construction grade of steel in use at the time. (ASTM, 1946)

Also, no reference could be found on the drawings as to the design Standard used. The most common Standard in use in the U.S.A. at the time was the American Institute of Steel Construction (AISC) "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings". (AISC, 1946) The fifth edition of that Standard was published in 1946. Alternatively, an early edition of an Association of Iron and Steel Engineers (AISE) Standard, such as No.6, may have been employed.

Regardless of the Standard used, the H&P design drawings indicate the stress allowed in tension members was 18,000 pounds per square inch (psi), which is approximately 55% of the 33,000 psi guaranteed minimum yield stress of A7 material, (33,000 x 0.55 = 18,150). The stress in psi allowed by H&P for main compression members was 15,000 - 1/4 $(\mathbf{L}/\mathbf{r})^2$ where "**L**" is the effective length of the member, "**r**" its least radius of gyration and "**L/r**" the member slenderness ratio.

These values are slightly conservative when compared to the allowable stresses in the 1946 AISC Standard of 0.60 of the yield strength in tension, or 20,000 psi (33,000 x 0.6 = 19,800) and, 17,000 - 0.485 $(\mathbf{L}/\mathbf{r})^2$ in compression.

The H&P drawings provide a complete record of the analysis and design of the hundreds of Ore Bridge structural members. Many potential loadings were considered including various combinations of the following main loads:

- the dead weight of the structure with the apron either up or down;
- the moving 100 Ton load of the trolley including its bucket and 12 Ton payload;
- various impact loads from the loaded trolley and apron;
- skewing loads due to misalignment of the main span;
- wind loads of 5 psf or 30 psf acting against either the end or the side of the bridge.

There is no indication that live loads due to snow and ice were originally considered.

Combined, the No.1 and No.2 Ore Bridges had about 85 years of experience working on the Dofasco bayfront performing the functions for which they were originally intended, without any prior record of major structural problems. Under the circumstances it does not seem reasonable to assume the collapse was initiated because of a gross error in the original design calculations or member selection. Regardless, as part of this investigation the design was checked to the current 9th edition of the AISC specification. (AISC, 1989)

11.1 The Main Span

The main span essentially consists of two vertical trusses 20'-4" apart which are joined together by a horizontal truss at the level of their top chords, creating a three-dimensional space frame. H&P analysed and designed each truss individually as two-dimensional pin jointed frameworks, which was a reasonable approximation. They combined any forces which would act in the horizontal and vertical trusses simultaneously.

A two-dimensional design check was performed for gravity loadings only, acting on the two vertical trusses of the main span, as the horizontal wind forces at the time of the collapse were known to be light. Figure 47 is a plot of the average recorded wind speeds on the morning of March 28th, 1995 at Mt. Hope airport, which is quite remote from the Dofasco bayfront, the Royal Botanical Gardens (RBG) which is closer, and from the Sewage Treatment Plant on Woodward Avenue (STP) which are probably the most relevant readings. Unfortunately the charts from the anemometers on all three Ore Bridges were damaged by the loss of electrical power associated with the collapse. The average wind speed at the time of the collapse from Figure 47 is about 15 km/hr, which translates to a wind pressure of only about 0.3 psf.

As an aside, Figure 48 records the temperatures taken at the same three locations on the morning of the collapse. It can be seen that the temperature through the previous night was quite stable and began to rise slowly around 6:00 am. At the time of the collapse the temperature on the Dofasco bayfront was probably about $4^{o}C$. There was no measurable precipitation at the time of the collapse and no accumulation of snow or ice.

Figure 49 records graphically the results of the design check of the main span based on the commercially available software program RISA-2D, Version 3.03.

The results are a summary of the combined effects of the dead weight of the main span and the trolley as a moving load traversing the length of the main span from one end to the other. The various members are colour coded according to the bar chart, as a ratio of the degree to which the stresses they carry approach those allowed by the present AISC Specification.

A unity ratio less than 1.0 means the member is carrying less stress than it is actually allowed to carry and is therefore oversized for the loading. A unity ratio of 1.0 means the member is carrying its allowable stress exactly. A unity ratio greater than 1.0 means the member is carrying more stress than it is allowed and is therefore undersized for the loading and in need of consideration for reinforcing.

As the stresses in a member in Figure 49 get closer to the allowable, the colour of the member changes from blue through green to pink and is red if the stresses exceed the allowable. At the maximum allowable stress, a unity ratio of 1.0, a member still retains a factor of safety on failure of at least 1.67 in tension and slightly more in compression.

Figure 49a reflects the design check of the main span to the present AISC Specification, as the span was detailed when the Ore Bridge was designed in 1950. The end of the apron and the support conditions at the Pier Leg were modified slightly, for ease in computing. The modifications have no effect on the basic findings. None of the members was found to be overstressed, however in several members the stresses were within one or two percentage points of the allowable. These results compare favourably with the original H&P design as presented on their drawings.

For several members at the west end, the worst loading was the fully loaded trolley against the bumpers west of the Pier Leg, over the Hi-Line. For the majority of members throughout the main span however, the worst loading was the fully loaded trolley against the bumpers on the apron at the extreme east end, over a ship.

The apron was extended in 1968 and the result is recorded in Figure 49b. Because the trolley can now travel further to the east, it increases the effect of the trolley load on the main span. Several of the members now exceed their allowable stresses, but only by 3 or 4 percent. The results are not inconsistent with those of H&P on their drawings, where stresses reaching or slightly exceeding their allowable are tabulated. (Dofasco, 1995)

As a result of the recent increase in the recognised dead weight of the main span and the trolley and the increase around 1975 of the lifted trolley payload to 17.5 Tons, more members become overstressed, even with a reduction of the trolley impact factor from 30% to 25%, Figure 49c. Several members now exhibit stresses beyond the allowable of up to 15%, especially around the Shear Leg, which should have triggered consideration of reinforcing. There was no record however, indicating the members were ever reinforced.

The members in the main span around the Shear Leg are recorded in Figure 50, looking towards the south, following the collapse. Figure 51 was taken along the north side bottom chord of the main span looking east, while Figure 52 records the south side of the main span in the area of the Shear Leg support.

The damage to these members is consistent with the impact of the east end of the main span on the dock, combined with the twisting of the main span towards the south. The twist in the main span was enhanced when the underside of the bottom chord on the north side impacted the bucket, Figure 53.

An examination of the wreckage of the Ore Bridge did not indicate the collapse initiated as a result of a structural overload in any of the members in the main span. Also, several of the eye witnesses indicated the main span hit the ground, intact.

11.2 The Shear Leg

Satisfied that there was no member in the main span which was overloaded to such a degree that a failure would be expected to initiate, the Shear Leg was then analysed.

The original gravity loads on the main span were transferred directly into the Shear Leg and combined in the manner described on the H&P drawings. The impact factor from the trolley was reduced to 15%. As well, the horizontal load on the pill resulting from skewing the main span was combined with the trolley fully loaded, but not including impact, which is reasonable. The maximum results from RISA-2D, for these loadings, are shown in Figure 54. None of the Shear Leg members is overstressed.

The Shear Leg was originally analysed by H&P as a pin-jointed two-dimensional framework. In Figure 54 the tie members connecting the sill truss to the bottom of the north and south legs are shown with small circles at their ends. The circles indicate the ends of those members were assumed to be pinned in the computer program, for structural analysis and design purposes.

The 1968 increase in the length of the apron also did not produce any overstress.

In 1990 the trolley was upgraded which, combined with the 1975 increase in the payload to 17.5 Tons, recognition that the dead weight of the main span was greater than originally assumed, and a 5 psf wind blowing, did produce an overstress, Figure 55. On the leeward side of the Shear Leg, the diagonal "T" members at the top of the leg, directly underneath the haunch, were overloaded by approximately 5%, which is not unlike the findings reported at the time by H&P. (Dofasco, 1995)

Because the Shear Leg is in structural terms a portal frame or tied arch, the tie-brace is a critical member. It carries the tension force which prevents the north and south legs from spreading. Without the tie-brace and the sill truss, the Shear Leg will spread. Considering the 1990 loadings from the main span and trolley, if the tie-brace and the sill truss are removed, virtually every member in the Shear Leg is overstressed to failure, Figure 56.

Although H&P analysed the Shear Leg as a pin-jointed two-dimensional structure, it is in fact a three-dimensional structure. Many of the joints were also detailed and fabricated such that they were continuous, not pinned.

From Figure 25 it is obvious that the tie-brace is not pinned where it changes direction at the intersection of the vertical brace, because of the fixity of the connecting plates. As a result, the tie-brace was modelled as a continuous member at that location which indicated the stresses under the 1990 loadings just about reached the allowable limit, because of local bending stresses in the connection, Figure 57.

12. THE ADDITION OF THE HOPPER

In 1991 the iron ore hopper was added to the Shear Leg of the No.2 Ore Bridge.

In reviewing the H&P calculations it was noted that the original dead weight of the final hopper, including its feed conveyor, was assumed to be 99,600 pounds. (Dofasco, 1995) A detailed weight take-off in the calculations, of the final configuration, indicated the dead weight was actually 149,041 pounds. For analysis purposes, a final weight of 150 kips has been assumed and that the centroid of the entire assembly is 2.36 feet east of the centre of the Shear Leg, Figure 58.

The addition of the hopper is significant as it created loadings in the Shear Leg which were unbalanced laterally, forcing the Shear Leg to function as a three-dimensional space frame.

Because of the imbalance, the hopper can rock back and forth within the Shear Leg. H&P designed a counterweight into the system, in an attempt to minimise the effects of this imbalance, as depicted in Figure 58. The weight of the counterbalance is 23,929 pounds or 24 kips and it is located 15.58 feet to the west of the centreline of the Shear Leg.

The dead weight of the hopper was to be supported on the track rails by two trucks installed within the sill truss framework, Figure 59. A cage was also built above the sill truss for the hopper guides, which transmitted the lateral loads associated with any imbalance back into the sill truss, Figure 60. The sill truss was also reinforced.

The H&P calculations indicate the originally calculated weight of a full load of iron ore pellets in the hopper was 51.6 Tons. In fact, this weight should be closer to 65 Tons, which is the weight indicated on the H&P drawings. (Dofasco, 1995)

Dribble shields were added around the top of the hopper after the contract had been let to fabricate the hoppers. As a result, pellets could in fact be piled higher in the hopper. The operators have indicated they fill the hoppers with 5 buckets, or more, which is at least 5 x 17.5 = 87.5 Tons. For analysis purposes, it has been assumed the hoppers can be filled with 90 Tons (180 kips) of material, Figure 61.

Once the hoppers begin to discharge, the load automatically becomes unbalanced, Figure 62. As well, the pellets can be unbalanced when loaded, Figure 63. The pellets are also known to hang up on the east or discharge side during emptying, Figure 64.

A RISA-2D analysis was then performed on the Shear Leg, with the simplified assumption that it could still be analysed as a two-dimensional framework and that the unbalanced loads from the hopper would be transmitted as vertical loads only into each side of the sill truss. The results are shown in Figure 65, which now indicates the critical tension tie between the sill truss and the bottom of the legs is just overstressed and the diagonal below the haunch is overstressed by about 13%.

The two-dimensional RISA model can not recognise the torsional stiffness of the Shear Leg and the end truck arrangement. As a result, the effects of the unbalanced loads in the hopper can not be properly distributed into the Shear Leg. A much more sophisticated three-dimensional computer model of the Shear Leg was then developed, using the commercially available structural analysis program STAAD-III/ISDS, Version 20.2.

The three-dimensional computer model of the entire Shear Leg is shown in Figure 66a. STAAD-III/ISDS allows the stresses in specific areas of the model to be calculated and viewed in detail. The members framing into the bottom of the south leg were modelled structurally using finite plate elements, as shown in Figure 66b. The actual arrangement of the members at the bottom of the south leg can be seen in Figure 66c, a photograph of the area taken on the No.1 Ore Bridge.

13. DETAILED ANALYSIS OF THE SHEAR LEG

The STAAD-III/ISDS model of the Shear leg enabled a more accurate determination of the levels of stress in the tie-brace members between the sill truss and the base of the north and south legs of the Shear Leg. The first model considered the Shear Leg in its original configuration, prior to the addition of any reinforcing plates, or the hopper.

Dead, trolley and wind loads from the main span were then applied to the model in the various combinations considered by H&P in their original design. Figure 67 records the absolute maximum principal stresses, from the worst loading combination, in the members and plates of the tie-brace at the south leg. The coloured contours in the stress diagram correspond to different levels of stress as defined in the upper left corner of the Figure. The units of stress are kips per square inch (ksi).

As a general observation, it is apparent from the stress plot that the tensile forces which prevent the legs of the Shear Leg from spreading flow from the sill truss through the "T" sections (1), directly into the 1/2" tie-plate (2). The maximum tensile stress in the "T" section is approximately 14 ksi, at (1), because of local bending in the section.

It should be noted in the model that the gussets (3) are not connected to the upstanding leg of the "T" sections (4), as detailed on the original H&P drawings. (Dofasco, 1995) The eccentricity which exists between the tension force in the tie-plate and the geometric centroid of the welds connecting the tie-plate and gussets to the inside of the south leg, results in the gussets being subjected to in-plane bending. There is negligible out-of-plane bending of the gussets.

Because the original Ore Bridge design loads were in the plane of the Shear Leg, the stresses in the tie-brace arrangement are essentially symmetrical about the north/south axis, which can be seen by viewing the tie-brace arrangement from above, Figure 68.

Where the tie-plate is connected to the inside face of the south leg, (5), the tensile stresses are generally uniform across the plate. The stresses are in the order of 12 ksi on the bottom surface of the plate and approximately 8 ksi on the upper surface as shown in the Figure, which indicates there is vertical bending occurring in the tie-plate.

The tensile stresses in the tie-plate are below the value of 18 ksi, allowed by H&P in their original design however, the stress contours indicate several areas of high local stresses.

As the tension forces flow from the east and west "T" sections into the 1/2" plate, the stresses increase from 8 to 10 to 12 to 14 ksi as they approach the sharp inside corners between the "T" section and the plate, (6) in Figure 68. There are also high stress concentrations of approximately 16 ksi at the outer edges of the 1/2" tie-plate (7), where the "T" sections end.

Dofasco maintenance personnel have advised that over the years, fatigue cracking of the 1/2" tie-plate was occurring in the two highly stressed areas identified by the finite element model, which prompted the addition of several reinforcing plates. Figure 69 records plates which were added to the inside corners between the "T" sections and the tie-plate (6), while Figure 70 records a plate which was added to the outer edge of the tie-plate (7). An additional plate was eventually added around 1981 to fill the entire hole between the east and west "T" sections and the 1/2" tie-plate, (8) in Figures 68 and 69.

In 1990 the hopper was added to the Shear Leg. In order to determine the effects of the hopper, the STAAD-III/ISDS computer model was revised to include the hopper, the additional plates which had been added over the years, as well as the sill truss reinforcing and main support members which were added along with the hopper.

The revised STAAD-III/ISDS model also included gusset extension plates which, according to Dofasco, were added at the time the hopper was installed, although there appears to be no record of them on the drawings. (Dofasco, 1995) A gusset extension plate on the No.1 Ore Bridge is noted in Figure 71, while Figure 72 is a close-up of the plate on the north leg of the No.2 Ore Bridge.

The dead, live and wind loadings from the main span were then applied to the revised STAAD-III/ISDS model. The original loads were changed to reflect the apron extension, the increase in payload, the upgraded trolley and the known increase in dead weight of the bridge. Dead, live and wind loads acting on a stationary hopper were also included, as assumed in the H&P hopper calculations. (Dofasco, 1995) Because the hopper was assumed to be stationary, the rail clamps were modelled as being engaged. All these loadings were then considered in various combinations, with appropriate impact factors.

It became apparent from the new computer model that with the addition of the gusset extensions, the flow of stress from the "T" sections into the bottom of the south leg changed. Figure 73 records the resulting stress contours, which again are in the units of 'ksi'. The Figure does not differentiate between tensile or compressive stresses. The high local stresses due to tension and bending in the bottom of the "T" sections, (1), are now exceeded by stresses at the gusset extensions (2) as the tie forces flow through the original gussets in both tension and bending. Local stresses in the corners between the "T" sections and the gusset extensions reach 20 ksi. There is evidence the north leg had cracked previously in this location on the east side, Figure 74, and had been repaired by welding, Figure 75.

The original gussets were connected at right angles to the inside of the north and south legs of the Shear Leg. The sill truss however is several feet wider than the legs of the Shear Leg and the tie-brace narrows from the end of the sill truss to accommodate this change in width, a detail a two-dimensional analysis of the Shear Leg can not recognise. Because of the resulting change in angle, the extension plates which were added to the gussets, are not in the plane of either the "T" or the gusset. The inside faces of the gussets were now found to be subjected generally to tensile stresses in the order of 10 ksi while the outside faces were stressed in compression to 4 ksi, indicating the original gussets were now subjected to out-of-plane bending, which they did not experience originally.

Figure 76 records the flow of stresses from the "T" sections into the tie-plate, viewed from above, when the loads on the Shear Leg are essentially in-plane, similar to the situation which existed before the hopper was added. With the 1981 addition of the solid plate between the "T" sections and the tie-plate, the tensile stresses in the tie-plate have become more uniform across the width of the leg. The stress levels on the lower surface of the tie-plate, where the tie-plate connects to the inside face of the south leg (3), are now in the order of 10 ksi, and 8 ksi for the upper surface, which still indicates the presence of bending in the tie-plate.

Figure 77 records the maximum stresses in the tie-plate assuming a stationary hopper with the rail clamps engaged. They occur when the trolley is at the end of the apron and the hopper, which is unbalanced to the west under its own dead-weight (Figure 58), is unloading ore (Figure 64). A west wind generating 5 psf is also assumed to be blowing. The general stress levels in the tie-plate and "T" sections remain below the tensile stress of 18 ksi allowed in the original H&P design. These results suggest the addition of the hopper, assuming it is stationary, did not greatly influence the magnitude of stresses in the tie-plate. There is a difference in the distribution of the stresses however, when compared to Figure 76, as a result of the tie-plate being forced to respond to the lateral loads introduced into the sill truss, as a result of the addition of the hopper.

When considering the results of the computer stress analysis, the finite elements used to model the various plates and members assume the material is free of imperfections and of uniform thickness. The computer analysis has also assumed the connections between the members and plates are 'ideal'. As a result, the analysis does not recognise or take into account stress increases which undoubtedly exist because of local discontinuities and imperfections in the plate material, or discontinuities associated with the welded connections between plates.

14. METALLURGICAL CONSIDERATIONS

The Ore Bridges were probably fabricated from A7 steel, which had a guaranteed minimum tensile strength of 60,000 psi. In spite of this apparent strength, the 1/2" thick by 43-1/2" wide tie-plate, which was originally only intended to carry the tensile force from the base of the Shear Leg through the sill truss, failed catastrophically, Figure 78. Theoretically, the tie-plate alone could carry a minimum tensile load of 1,300,000 pounds, yet it had failed and apparently caused the collapse of the Ore Bridge when loaded to only about 20% of that value. The tie-plate and its associated gussets had literally ripped free of the Shear Leg, Figure 79.

Figure 78 was taken looking south at the remains of the connection on the inside of the south leg. Relevant locations are marked. Figure 79 was taken looking north at the ends of the tie-plate and gussets, which had pulled free from the connection. The locations noted in Figure 78 are similarly identified in Figure 79.

Because neither the initial nor the detailed structural analysis had identified stresses which should have triggered such a massive structural overload, metallurgical investigations of the tie-plate were initiated. Mr. T.R. Wood, P.Eng. of T.R. Wood Consulting and Mr. B.A. Graville, P.Eng. of Graville Associates Inc., conducted the investigations. (Graville, 1995; Wood, 1995)

14.1 The Metallurgical Investigations

Figure 80 is a view looking south at the intersection of the tie-plate and gusset on the underside of the south leg of the Shear Leg, on the east side, (B) in Figure 78. Figure 81 is a similar view of the intersection of the tie-plate and the gusset on the west side, (C) in

Figure 78. Figure 82 illustrates the main features of the fracture faces. (Graville, 1995) (In Figure 82 the upper portion of the Figure is the equivalent to looking at Figure 78, the leg side of the connection. The lower portion of Figure 82 is the mating half of the connection on the tie-brace side, which is equivalent to looking at Figure 79 upside down.)

Chevron or herring-bone patterns on the fracture face indicated the fracture initiated on the east side of the tie-plate, near the intersection of the tie-plate and the gusset, (Figure 80). The fracture ran horizontally out from that intersection, through the tie-plate, ripping the east gusset upwards as it progressed.

As a result of the failure of the tie-plate on the east side, the tensile load the tie-plate carried became concentrated in the west side. A second fracture then initiated near the intersection of the tie-plate and the gusset on that side (Figure 81). The second fracture ran horizontally out from that intersection, through the tie-plate, and also ripped the west gusset upwards as the failure progressed. The two fractures in the horizontal tie-plate overlapped. They intersected between the 10" and 11" marks in Figure 81, just to the east of the west gusset.

Once the fractures intersected, the tie-plate was in two pieces and incapable of carrying any tensile force. The connection linking the bottom of the north and south legs of the Shear Leg was thus effectively lost, allowing the legs to start spreading (as depicted in Figure 56), and the collapse of the No.2 Ore Bridge to begin.

A curious feature of the fracture surface was the minimal distortion or necking down of the tie-plate locally. Ductile failures, normally associated with the structural overload of a steel framework, typically exhibit plastic deformation of the material. Instead, the fracture surface on the tie-plate was generally flat and normal to the surface of the tie-plate which, along with the presence of the chevron patterns, is typical of a brittle failure.

As part of the metallurgical investigation, the toughness of the tie-plate material was then investigated. Charpy V notch (CVN) impact tests were conducted at various temperatures on specimens cut from the plate, as well as K_{Ic} fracture tests. A standard tensile test was also performed. (Graville, 1995; Wood, 1995)

The standard tensile test showed the tie-plate material to behave in a normal ductile manner under slow loading. The yield strength was 43,000 psi, the ultimate strength 65,500 psi and the elongation was 29.6% on a 2" gauge length, all of which meet the requirements for A7 material.

When loaded more rapidly however, at 4°C (the approximate air temperature at the time of the collapse), the CVN impact tests and the K_{Ic} fracture tests indicated the tie-plate material was susceptible to brittle fracture at that temperature. The rolling direction of the plate was also found to be oriented at 90° to the direction of the tensile stresses it carried, which is its least resistant direction for brittle fracture.

Toughness testing was never part of the requirements for A7 material and was not fully understood in 1950, when the No.2 Ore Bridge was designed and fabricated. A chemical analysis of the tie-plate material also indicated it was low in manganese, and relatively high in sulphur, which is a common observation in structural steels sensitive to brittle fracture. (Dofasco, 1995)

Using the results of the toughness tests and a fracture mechanics approach, mathematical relationships were then established between the applied stress and the surface and through thickness crack sizes required to initiate a brittle fracture in the tie-plate material in the south leg of the Shear Leg of the No.2 Ore Bridge. (Graville, 1995)

Initial examinations of the tie-plate fracture in the field, a few days following the collapse, did not identify any visually apparent pre-existing cracks across the fracture surface, although areas of probable lack of weld penetration were obvious. The tie-plate was eventually moved indoors, sectioned and the fracture surface viewed under magnification. (Wood, 1995)

Figure 83 is a close-up of the intersection of the tie-plate and the gusset on the west side. The area in the middle of the fracture surfaces is a region of no weld penetration in the joint. The mating face is recorded in Figure 84, after cleaning. Significant rusting in the area of no weld penetration is obvious as are corrosion pits on the underside of the tie-plate at the toe of the reinforcing fillet weld, noted by the arrows.

A close-up on the corrosion pits indicated they were also the source of crescent shapes containing ratchet marks, which is indicative of fatigue cracking, Figure 85. (Wood, 1995)

The intersection of the tie-plate and gusset on the east side was also cleaned and photographed under oblique lighting, Figure 86. Corrosion pits were again noted underneath the tie-plate, which acted as a source of fatigue cracks. The fatigue cracks were also along the toe of the reinforcing fillet weld underneath the tie-plate. A larger series of concentric curves probably indicative of rapid fatigue growth, marked (B), was also evident. The area marked (A) exhibited characteristics of short brittle fractures or "pop-ins" which had initiated but arrested, prior to the final fracture. (Wood, 1995)

Fatigue cracks were not observed along the fracture surface between the gusset plates.

The metallurgical investigation concluded the failure of the tie-plate probably initiated when the cracks observed on the underside of the tie-plate, near its intersection with the east gusset, had grown sufficiently that they combined, creating a crack through the thickness of the tie-plate which reached a critical dimension. (Graville, 1995)

The through crack which eventually formed appeared to have a length of approximately 4" which, using fracture mechanics methodology, would require a stress level of only 15 ksi to initiate a rapid fracture. (Graville, 1995)

14.2 Fatigue Cracking

Simply stated, fatigue cracks form as a result of repeated or fluctuating tensile stresses in the structure. They start as a very small fissure and grow in size, or propagate, as long as the fluctuations of the tensile stress continue. The initiation of a fatigue crack can be from any imperfection or microcrack in the material, which raises the local stress. Fatigue cracks are typically associated with inclusions or discontinuities within the material or more likely, an imperfection on the surface of the material, where stresses are often highest as a result of bending. (Wood, 1995)

A fundamental concept when dealing with fatigue is stress range, the difference between the maximum and minimum stress to which any particular portion of the structure is subjected. The greater the stress range, the fewer will be the number of cycles of loading required for a fatigue failure. Alternating bending stresses are most damaging as the stresses range from maximum tension to maximum compression during each load cycle.

The fracture surface of a fatigue crack has a characteristic appearance. It generally has a smooth matt finish and frequently exhibits concentric rings spreading out from the point of initiation of the crack. The fracture surface also tends to become less smooth as the rate of propagation of the crack increases. (Wood, 1995)

The tip of a fatigue crack is very sharp. The material just ahead of the tip deforms plastically, however a fatigue failure initiates with very little apparent deformation of the material in the cracked region. Tensile loads cannot be carried across a crack. As a result, the load the cracked area once carried is transferred to the surrounding material. Failure occurs once the surrounding material which remains is eventually overloaded.

Fatigue cracks develop essentially independent of temperature. The eventual failure of the overloaded surrounding material may not however, be independent of temperature.

14.3 Brittle Fracture

Steel structures are generally designed by engineers using classic elastic methods. The commonly measured ductile properties of steel such as yield strength, ultimate strength and elongation form the basis for proportioning the members in the structure. Failures are traditionally expected to be ductile, as a result of an overload of the structure. The overload may be the result of significantly increased loads on the structure, or the reduced ability of a particular member to transmit its load as a result, for example, of corrosion or the presence of fatigue cracks.

With the increased use of welding in the early parts of this century, came the realisation that a greater number of structures were failing in a brittle fashion, under loads normally considered to be safe. Research on this subject began in earnest after the failure of a large number of ships during and immediately following World War II.

Steel subjected to loadings which create a high strain rate exhibit a transition from predominantly ductile failure to predominantly brittle failure, as the temperature decreases. The change from one failure mode to the other may be gradual, but often occurs within a narrow temperature range. A temperature is often selected within this range as a reference point, called the *"transition temperature"*. (Graville, 1995)

Unlike a ductile failure, in which considerable plastic deformation of the steel develops prior to separation, a brittle failure exhibits little or no plastic deformation. A ductile failure surface is typically fibrous as a result of the grains in the material being drawn and distorted during failure, while a brittle fracture leaves a bright faceted surface as a result of cleavage with little or no distortion. (Wood, 1995)

Brittle fractures are rapid, often travelling at the rate of several thousands of feet per second. The fracture faces which remain are generally flat and normal to the surface of the plate. There is little or no indication of distortion, or reduction in thickness of the material.

In order for a brittle fracture to occur, the temperature must be near or below the transition temperature of the material. A pre-existing notch associated with a crack or sharp defect must also exist, which raises the local stress level. As well, the stresses must be tensile. These conditions must coexist in a critical combination.

As knowledge about welding and brittle failure criteria grew, steels were developed which were more weldable and more resistant to brittle fracture. It has now become possible to select steels which have a transition temperature below the expected minimum service temperature of engineered structures commonly subjected to dynamic and cyclic loadings, such as bridges, cranes and ships.

14.4 Welding and Residual Stresses

The metallurgical investigation also revealed other features of the welded connections around the tie-plate on the south leg of the Shear leg, which were disturbing. (Wood, 1995)

Figure 87 is a section of the tie-plate to rocker block weld which shows obvious lack of penetration into the root of the weld. Clearly, no attempt was made at the time the Ore Bridge was fabricated to make this a complete penetration groove weld, although the original drawings suggest it should have been completely penetrated. The reinforcing fillet weld underneath the tie-plate is also clearly visible. The toe of the fillet weld, where the fatigue cracks initiated, is noted. Figure 88, taken at another location along the same weld and viewed from the opposite direction, not only shows lack of penetration but cracks emanating from the root of the welds and gas pockets in the welds.

Figures 89 and 90 were taken looking down on the fillet welds joining the gussets to the inside of the south leg. The welds fractured, no doubt as a result of the collapse, however there were indications of prior cracking at the root of the welds and intermittent fatigue cracks along the length of the welds. (Wood, 1995) Gas pockets (porosity) are clearly visible in the welds in Figure 90.

Figure 91 is a portion of the tie-plate (T), rocker block (R) and inside leg plate (L) which were joined by welding. The weld (B) between the tie-plate and rocker block was sound, however there was the previously observed lack of penetration at the root of the weld. The weld (A) between the rocker block and the inside leg plate however, was cracked the entire length of the weld.

Upon close examination, the crack in Figure 91 appeared to have initiated from the root of the weld and probably failed instantaneously, Figure 92. (Wood, 1995) Combined with the fact that the surfaces of the crack were heavily corroded with a very tenacious rust, it suggested this crack may have occurred while the Shear Leg was still being fabricated, in 1950.

There was evidence throughout the wreckage of the No.2 Ore Bridge of groove welds which lacked complete penetration or fusion through the thickness of the joints. The incomplete penetration groove welds did not appear however to contribute directly to the collapse.

Welding also plays a fundamental role in assessing the susceptibility of a structure to a brittle failure.

A brittle fracture requires a crack or defect to exist, in conjunction with tensile stresses. One of the main consequences of welding is that lack of procedural control can introduce defects or cracks into the structure such as lack of penetration, lack of fusion, slag inclusions, porosity, hydrogen induced cracks, hot cracks etc. Welding also leaves behind residual stresses which are typically tensile in the weld metal and the immediately surrounding plate. (Graville, 1995)

The residual tensile stresses from welding can be at or close to the yield stress of the material. As far as a plate susceptible to brittle fracture is concerned, the source of the tensile stress required to initiate a brittle fracture is not relevant, residual welding stresses and externally applied tensile stresses are equivalent and are cumulative.

Brittle fractures have been known to initiate with no applied stress, but merely from the presence of welding residual stresses.

15. ADDITIONAL LOADING CONSIDERATIONS

The metallurgical investigations of the tie-plate at the bottom of the south leg of the Shear Leg identified the existence of fatigue cracks in corrosion pits at the toe of the reinforcing fillet weld connecting the underside of the tie-plate to the rocker block. These cracks grew until they reached a critical size, initiating the brittle fracture of the tie-plate.

As a result, a more detailed review of the dynamic and cyclical nature of the loadings on the Shear Leg was initiated. These loadings were then applied to the STAAD-III/ISDS model of the Shear Leg to determine their effect on the tie-plate. The dynamic loads were applied as equivalent static loads, which is reasonable, as a preliminary investigation indicated it was unlikely any dynamic amplification of the loads would occur as a result of resonance within the Shear Leg.

15.1 Main Span Trolleying and Bridging Inertia Forces

The Ore Bridge, under normal operating conditions, is subjected to dynamic inertial forces which result when either the trolley or the bridge is brought to a stop, or is taken up to speed. The rate at which the trolley or bridge is stopped or started is termed "*deceleration*" or "*acceleration*", respectively. These inertial forces can be expressed by Newton's Second Law of Motion,

$$\mathbf{F} = \mathbf{M} \times \mathbf{a}$$

where; $\mathbf{F}$ = the inertial force

$\mathbf{M}$ = mass, and

$\mathbf{a}$ = the acceleration or deceleration of the object.

In order to reasonably estimate the acceleration (or deceleration) of the trolley and bridge, their speed and the time or distance to bring them to a stop must be known. Several of the Ore Bridge operators have provided estimates of these parameters.

Regarding *"trolleying"*, the operators have indicated they would normally trolley in the fifth point setting as they travelled back and forth between a ship and the ore pile. Fifth point is the highest setting which corresponds to a maximum trolley speed of 900 ft./min. (Dofasco, 1995) As they approached the location over the ore pile where they wished to empty the bucket, the operators would normally coast the trolley to slow its speed. As the trolley slowed to a stop, the bucket would be opened to drop the ore onto the pile, and the controls would simultaneously be switched to fifth point in the opposite direction, or *"plugged"*, so they could quickly return to fill the bucket.

The operators indicated the time to bring the trolley to a stop from fifth point was in the order of 4 to 6 seconds. To bring the trolley back up to speed in the opposite direction was estimated at approximately 15 seconds.

If the operators were filling the hopper on the Shear Leg, the time to stop the trolley was estimated at between 8 and 10 seconds, as more care was needed to line up the bucket over the hopper. The operators also indicated that the bucket could be emptied over the hopper while the trolley was still in motion, but this depended on the "*timing*" and "*feel*" the individual operators had for the Ore Bridges.

If the trolley is assumed to come to a stop in 5 seconds from a speed of 900 ft./min., the deceleration would be 3.0 ft./sec.^{2}, which gives an inertial force of approximately 9.3% of the mass of the trolley. The trolley including a full bucket weighs 225 kips, and therefore the inertial force associated with stopping the trolley would be about 21.0 kips.

This appears reasonable, as modern design guidelines for crane runways suggest a 20% inertial force to account for the dynamic effects associated with stopping and starting a crane trolley. This inertial force however, would primarily be resisted by the much stiffer Pier Leg, and would have minimal effect on the Shear Leg.

When *"bridging"*, the operators indicated the bridge stops much quicker than the trolley, because of the drag associated with all the motor drives and gear boxes. If the bridge coasted to a stop from its maximum speed of 90 ft./min., the operators estimated it would stop in 3 to 4 feet, which, because of its lower initial speed gives a deceleration of 0.38 to 0.28 ft./sec.^{2}, and a corresponding inertial force in the order of 1% of the mass.

The bridge could also be moving at full speed and if the anemometer recorded a wind speed in excess of 32 mph, the rail clamps would drop automatically. The operators indicated that when this happens, the bridge would come to a stop within a distance of 6 inches to a foot, and result in a noticeable shaking and vibration of the structure.

Stopping the Ore Bridge within 6 inches induces an inertial force of approximately 7.0% of the mass of the bridge and hopper. The weight of the bridge bearing on the Shear Leg is approximately 800 kips, and the trolley 225 kips, creating a 73 kip inertial force. The weight of the hopper is approximately 175 kips, assuming it is being bridged empty, which creates a 13 kip inertial force.

These inertial forces combined produce an instantaneous increase in tensile stress in the tie-plate of approximately 1.5 ksi. This event however, would not be expected to occur often.

15.2 The Hopper Framing

Prior to the addition of the hopper, the sill truss and tie-braces connecting it to the Shear Leg were designed simply as a tension tie. Because the sill truss is lower than the pins at the bottom ends of each of the legs, the sill truss lifts under the large tensile tie forces, as depicted in Figure 93. The original undeformed shape of the Shear leg is shown in black in the Figure and the deflected shape in red, magnified for clarity. With the addition of the hopper however, the loading on the sill truss and in the tie-braces changed significantly.

The hopper is supported vertically by two pairs of columns, which bear on trucks. The trucks ride on the same rails as the Shear Leg. To prevent the hopper from falling over, horizontal guides were installed. Lower guides were installed at the level of the top of the sill truss, and upper guides just below the hopper platform level. The upper guides provided horizontal restraint in both the north/south and east/west directions. The lower guides only provide horizontal restraint in the north/south direction. The horizontal forces from the hopper acting on the upper guides are transmitted by a steel framework or cage into the sill truss.

The original hopper guides were a combination of mild steel and Fabreeka bearing pads, with an approximately 1/16" clearance around all sides. Shortly after installation, these guides apparently wedged or "*seized*", and the guide support channels took on a permanent upward bow. Fatigue cracks were also noted in the support channels in the first year of operation. With the guides wedged there is no vertical slippage between the hopper support columns and the guide support system, which results in more torsional movement of the sill truss and the tie-braces. The original guides were replaced during the January 1995 shutdown with stainless steel rub plates, in an attempt to eliminate wedging.

15.3 Hopper Loading

The loading of the hopper was discussed with several of the operators. When the trolley is over the hopper, and the controls are engaged to open the bucket, the weight of the ore within the bucket tends to force open the bucket on its own. This allows the ore to basically free fall from the bucket into the hopper. The operators estimated it would take about 2 seconds for the ore to leave the bucket.

Assuming the 17.5 Tons of ore in the bucket dropped 15 feet to the hopper, and it took 2 seconds to empty the bucket, the falling ore would create a vertical impact force of 16.9 kips. This force would act at the centre of the hopper if the bucket was centred when it was emptied and would be transmitted directly to the hopper trucks, producing negligible torsion in the sill truss, or tension in the tie-plates.

It is more likely however, that the bucket would be off centre when it was being emptied into the hopper. The falling ore would then impact the sloping sides of the hopper, creating vertical and horizontal impact forces. The falling ore could also be travelling horizontally at the speed of the trolley, creating an additional horizontal impact load.

Assuming the bucket was 5 feet off-centre when the ore was dropped, the resulting in-plane bending stresses in the tie-plate are in the order of 0.6 ksi, tension or compression. The Standard Operating Procedure for the Ore Bridges actually advise that, " the hopper will rock back and forth violently", if it is loaded off centre. (Dofasco, 1995)

Figure 94 records the response of the Shear Leg, viewed from the north or south, to an east/west rocking of the hopper. The original undeformed shape of the Shear Leg is shown in black and the deflected shape in red, magnified for clarity. It is important to recognise that rocking the hopper back and forth causes the legs of the Shear Leg to respond by deflecting out-of-plane and twisting.

Rocking of the hopper causes the upper guides to rotate about the hopper trucks, which results in the top chord of the sill truss and the tie-braces being bent in a horizontal plane. Figure 95 depicts this horizontal bending of the sill truss and tie-plate, viewed from above. Again, the deformed shape, in red, is magnified for clarity and superimposed on the undeformed shape, in purple. Bending the sill truss bends and twists the north and south legs of the Shear Leg because of the stiffness of the diagonal brace and the tie-plate which connect the sill truss to the bottom of the legs.

15.4 Hopper Unloading and Bridging

When the Ore Bridge is not moving and the rail clamps are engaged, the hopper leans against either the east or west guides, depending on whether it is empty, full, or being unloaded. When the hopper is empty, it should lean towards the west, because of the counterweight. If filled unsymmetrically it can lean either way. When the hopper is unloaded, it typically leans to the east. This rocking motion occurs naturally and continuously while the hopper is in use, inducing cyclical loading in the sill truss and as a result, in the legs of the Shear Leg. The resulting in-plane bending stresses in the tie-plate are 1.4 ksi, in either tension or compression, giving a stress range of around 3 ksi, not including the effects of any stress raisers.

A similar rocking motion occurs when the rail clamps are lifted and the Ore Bridges are bridging, because of differences in the elevation of the tracks.

The resulting inertial forces are dependent upon:

- the height of the hopper centre of gravity;
- the mass of the hopper assembly;
- the horizontal stiffness of the Shear Leg, sill truss, and guide support members;
- the degree of track misalignment;
- the bridging speed;

Based on a track height differential of 3/8" over a 6 foot length of track, and a bridging speed of 90 ft./min., an empty hopper will create a lateral inertial force of approximately 10 kips, distributed over the four upper guides. A loaded hopper would create an inertial force in the order of 16 kips. The operators have in fact advised they can sense this rocking motion of the hopper while they are in their cabs. They have suggested the top of the hopper actually moves back and forth in the order of 6" to 12".

Assuming the total movement of the hopper is 6", i.e. 3" in either direction, bending stresses are created at the outer edges of the tie-plate which exceed 8 ksi, Figure 96. The stress contours in Figure 96 record the magnitude of stress but do not differentiate between tension and compression. Therefore, rocking of the hopper back and forth will result in a complete reversal of these stresses creating a stress range in the tie-plate of over 16 ksi, not including the effects of any stress raisers or other loadings.

Combining the rocking motion of the hopper with the loads in the tie-plate from the main span creates a complex state of stress, with the maximum occurring in the vicinity of the intersection of the tie-plate with the gusset where the fatigue cracks existed and the brittle fracture initiated, Figure 97. There is evidence of weld repairs in this region on the top surface of the tie-plate on the east side of the north leg of the No.2 Ore Bridge Shear Leg and on the underside of the tie-plate, along the toe of the reinforcing fillet weld. Rocking the hopper shifts the maximum stresses to the opposite side of the tie-plate, Figure 98. These results should be compared to the relatively uniform stress distribution which existed in the tie-plate prior to the hopper being added, Figure 99.

If the total movement at the top of the hopper is 12" the stresses shown in Figure 96 double, the maximum stress increases to 21 ksi and the stress range doubles to 32 ksi.

Because of the mass of the hopper, skewing the main span within its working limits, independent of any other operation, also increases the stresses in the tie-plate but the increase is relatively small.

It would appear that not all the dynamic effects resulting from the addition of the hopper were considered in the design. The dead weight of the hopper and its live load capacity were also underestimated. In addition the geometric modelling and boundary conditions for the sill truss and tie-brace computer model appear to have been over-simplified. There also did not appear to be any evaluation of the cumulative effects of fatigue.

The design calculations also did not identify the need for the gusset extensions, which were apparently added when the hoppers were installed. With the addition of the gusset extensions the loading on the original gussets, and therefore their welds to the inside face of the south leg, was altered. The dominant effect became out-of-plane bending. The metallurgical investigations found evidence of fatigue cracking in the gusset welds. (Wood, 1995) The cracks were intermittent along the length of the gusset which is consistent with discrete stress raisers and out-of-plane cyclical bending stresses. It does not appear however that the behaviour of the gusset plates played a dominant role in the collapse.

The metallurgical investigations also suggested that the presence of bending stresses in the tie-plate would create high surface tensile stresses in the tie-plate, causing fatigue cracks to readily develop at the toe of the reinforcing fillet weld underneath the tie-plate, joining the tie-plate and the rocker block. (Graville, 1995; Wood, 1995)

The computer modelling has in fact shown that the tie-plate has always deflected to some degree out-of-plane (vertically), as it participated in the frame action required to tie the base of the north and south legs together when supporting the loads from the main span. More recently however, with the addition of the hopper, the tie-plate has been subjected to significantly higher cyclic in-plane bending stresses, as it has attempted to restrain the horizontal deflections of the sill truss during loading and unloading of the hopper and during bridging with the hopper. The fatigue cracks noted in the metallurgical investigation also existed near the outer edges of the tie-plate, which is the most highly stressed region under the hopper loadings.

16. ALTERNATE FAILURE SCENARIOS

To be certain the collapse of the No. 2 Ore Bridge initiated with the failure of the tie-plate at the bottom of the south leg of the Shear Leg, it was important that other possible sources of failure be investigated and eliminated as potential causes. The following is a list of the scenarios investigated.

16.1 External Loads

(a) Excessive wind loads

(b) Impact from a vehicle at the base of the north or south legs of the Shear Leg

(c) Impact of one Ore Bridge against another

(d) Impact of the east end of the Ore Bridge from a ship

(e) Ice and Snow Loading

16.2 Internal Loads

(a) Excessive out-of skew of the Ore Bridge

(b) Impact of the hopper with the bucket

(c) Derailment

(d) Impact of the trolley hitting the bumpers on either end of the main span

(e) Driving the Ore Bridge with the rail clamps engaged

(f) Wedging of the hopper in the guides

16.3 Member Overloads

(a) Structural over-load of the tie-plate at the bottom of the south leg of the Shear Leg

(b) Failure of the Shear Leg diagonals just below the haunch area

(c) Failure of the horizontal member at the bottom of the south leg of the Shear Leg

(d) Failure of the diagonal brace between the Shear Leg and the sill truss

(e) Failure of the trolley girder support beams and hangers

(f) Failure of the main span diagonal members on either side of the cross-head

(g) Failure of the main span bottom chords

(h) Dropping of the apron from the raised position

16.4 Reduction in Member Strength

(a) Excessive corrosion

(b) Fatigue failures

(c) Incomplete penetration groove welds

Having reviewed these various scenarios it was concluded, based on the examination of the wreckage, the eye-witness accounts of the collapse, the computer modelling and the results of the metallurgical investigations, that the failure of the tie-plate at the bottom of the south leg of the Shear Leg must have initiated the collapse of the Dofasco No.2 Ore Bridge, on March 28th, 1995.

17. SUMMARY

Dofasco started producing steel on their Hamilton bayfront property around 1952. Much of the iron ore and coal required for their blast furnace operations was delivered by ship. As a result, Dofasco contracted Heyl & Patterson of Pittsburgh Pennsylvania to design two Ore Bridges, to unload the ships.

The Ore Bridges were probably fabricated in 1951. There was no reference on any of the Ore Bridge drawings to indicate either the Standard used for their design or the material used in their construction. The Ore Bridges have been in continuous service since they were erected on the Dofasco bayfront property.

On Tuesday March 28th, 1995 at approximately 8:40 am, the Shear Leg supporting the east end of the No.2 Ore Bridge failed, causing the collapse of the Ore Bridge. At the time, the Ore Bridge appeared to be stationary. It would seem the operator was simply sitting in his cab as there was no apparent reason for him to be performing any specific operation. There was also no measurable precipitation at the time and the winds were light.

In structural terms, the Shear Leg was originally designed as a portal frame or tied arch. The tie which carries the tension force preventing the Shear Legs from spreading is a critical member, without it the Shear Leg would collapse. It was apparent from the geometry of the collapsed structure that the tie-plate had in fact failed.

When first designed, the No.2 Ore Bridge had a live load capacity of 12 Tons. Around 1975 the capacity was increased to 17.5 Tons. In 1968 the apron was extended. In 1990 the trolley was upgraded. All of the above placed increased structural obligations on the tie-plate. In 1990 Dofasco modified their method of handling iron ore pellets on the bayfront which required the addition of a hopper into the sill truss of the Shear Leg.

The addition of the hopper was significant as it created forces which were unbalanced laterally and were cyclic in nature, conditions for which the Shear Leg was not originally designed. The imbalances were created by the dead weight of the hopper, the loading and unloading of the hopper and the bridging of the entire structure.

Initial two-dimensional structural calculations indicated there was no obvious structural overload that should have precipitated the catastrophic failure of the tie-plate. Various other potential causes of the collapse were investigated and eliminated.

A detailed three-dimensional computer model of the Shear Leg was then developed which identified horizontal bending stresses in the tie-plate associated with the cyclic behaviour of the hopper. Combined with the tension force that the tie-plate carried, these additional bending stresses created a complicated state of stress in the tie-plate, which reached its maximum in the general area where the fracture initiated.

A metallurgical investigation then found that the steel in the tie-plate was susceptible to brittle fracture at 4°C, the approximate air temperature at the time of the collapse. Fatigue cracks were also found on the fracture face. The fatigue cracks had initiated in corrosion pits at the toe of a reinforcing fillet weld on the underside of the tie-plate. The fatigue cracks were found at the outer edges of the tie-plate, which is the most highly stressed area from the hopper loadings. (Graville, 1995; Wood, 1995)

Using a fracture mechanics approach it was concluded the failure of the tie-plate was the result of the fatigue cracks growing and combining to create a crack through the thickness of the plate, which reached a critical dimension allowing the tie-plate to suddenly fracture in a brittle manner. (Graville, 1995) The failure was rapid and could be initiated at stresses lower than those allowed when the Ore Bridge was originally designed.

The weld connecting the tie-plate to the rocker block also contained lack of fusion and root cracks which, combined with the inevitable residual tensile stresses associated with welding, made the state of stress in the area where the fracture initiated even more complex.

Welded joints throughout the failed structure exhibited incomplete penetration generally. Cracks also existed in several welds, some of which probably initiated when the Shear Leg was first fabricated. The quality of welding did not however contribute directly to the collapse.

17. CONCLUSIONS

It is the authors opinion that:

1. The Dofasco No.2 Ore Bridge collapsed on March 28th 1995 as a result of a brittle fracture of the tie-plate at the bottom of the south leg of the Shear Leg.

2. The tie-plate is a critical member in the Shear Leg. As there is no reasonable alternative load path for the forces it carries, the failure of the tie-plate would initiate the collapse of the Ore Bridge.

3. There is no evidence to indicate the collapse initiated at any other location.

4. There is no evidence to suggest the operators performed any unusual manoeuvres immediately prior to, or at the time of the collapse, which would cause the failure.

5. The Shear Leg was originally designed as a two-dimensional structure to support the loads on the main span. The tie-plate which failed was originally assumed to carry only a tension load, created by the arching action of the Shear Leg when supporting the main span and its trolley. A more detailed analysis of the tie-plate revealed that it also flexed in a vertical plane, causing higher stresses on its surfaces due to bending.

6. Over the years the loads on the main span were increased gradually as a result of specific modifications which included the extension of the apron, an increase in the lifted load capacity of the bucket and upgrading of the trolley. Each modification created higher stresses in the tie-plate. More importantly, in 1991 a hopper was added into the Shear Leg which created cyclic, lateral loadings on the Shear Leg.

7. The stresses in the tie-plate increased significantly as a result of the addition of the hopper. Horizontal bending stresses were introduced into the tie-plate as a result of the rocking motion of the hopper. The cyclical nature of these bending stresses initiated fatigue cracks in pre-existing corrosion pits on the underside of the tie-plate, at the toe of a reinforcing fillet weld.

8. These fatigue cracks grew and combined and eventually created a flaw through the thickness of the tie-plate.

9. The flaw in the tie-plate of the Dofasco No. 2 Ore Bridge reached a critical size at 8:40 am on March 28, 1995. At that time the combination of temperature, stress and flaw size allowed a rapid brittle fracture of the tie-plate to initiate.

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ASTM (1946), Standard Specifications for Steel For Bridges and Buildings, ASTM Designation: A 7-46; American Society for Testing Materials, Philadelphia, Pennsylvania, U.S.A.

Dofasco (1995), Personal Communication, Dofasco Inc., Hamilton, Ontario

Graville (1995), Personal Communication, Graville Associates Inc., Georgetown, Ontario

Wood (1995), Personal Communication, T.R. Wood Consulting, Burlington, Ontario

APPENDIX A

FIGURES

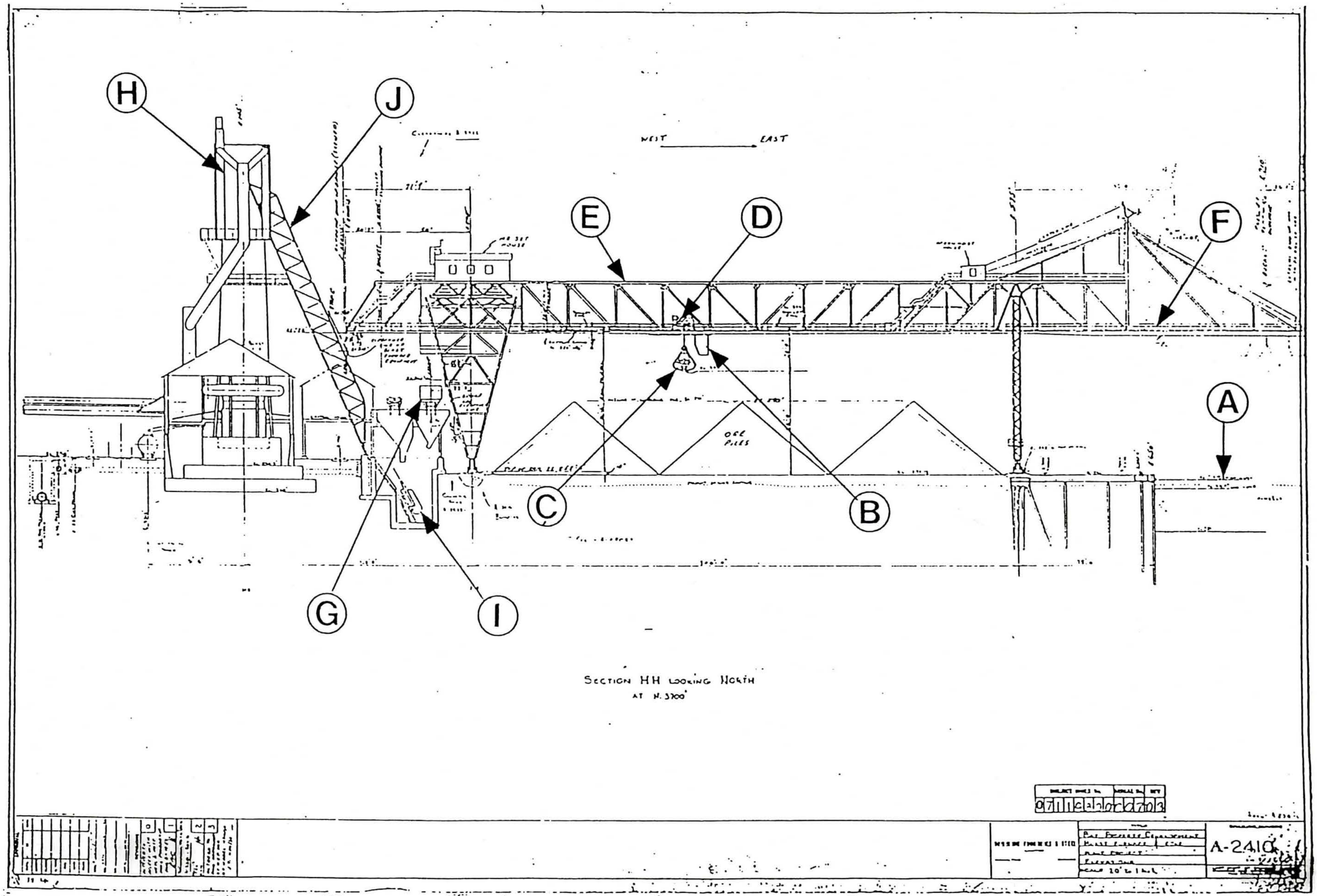


**FIGURE 1: The original Ore Bridge configuration.
(See text for notes) (Dofasco, 1995)**

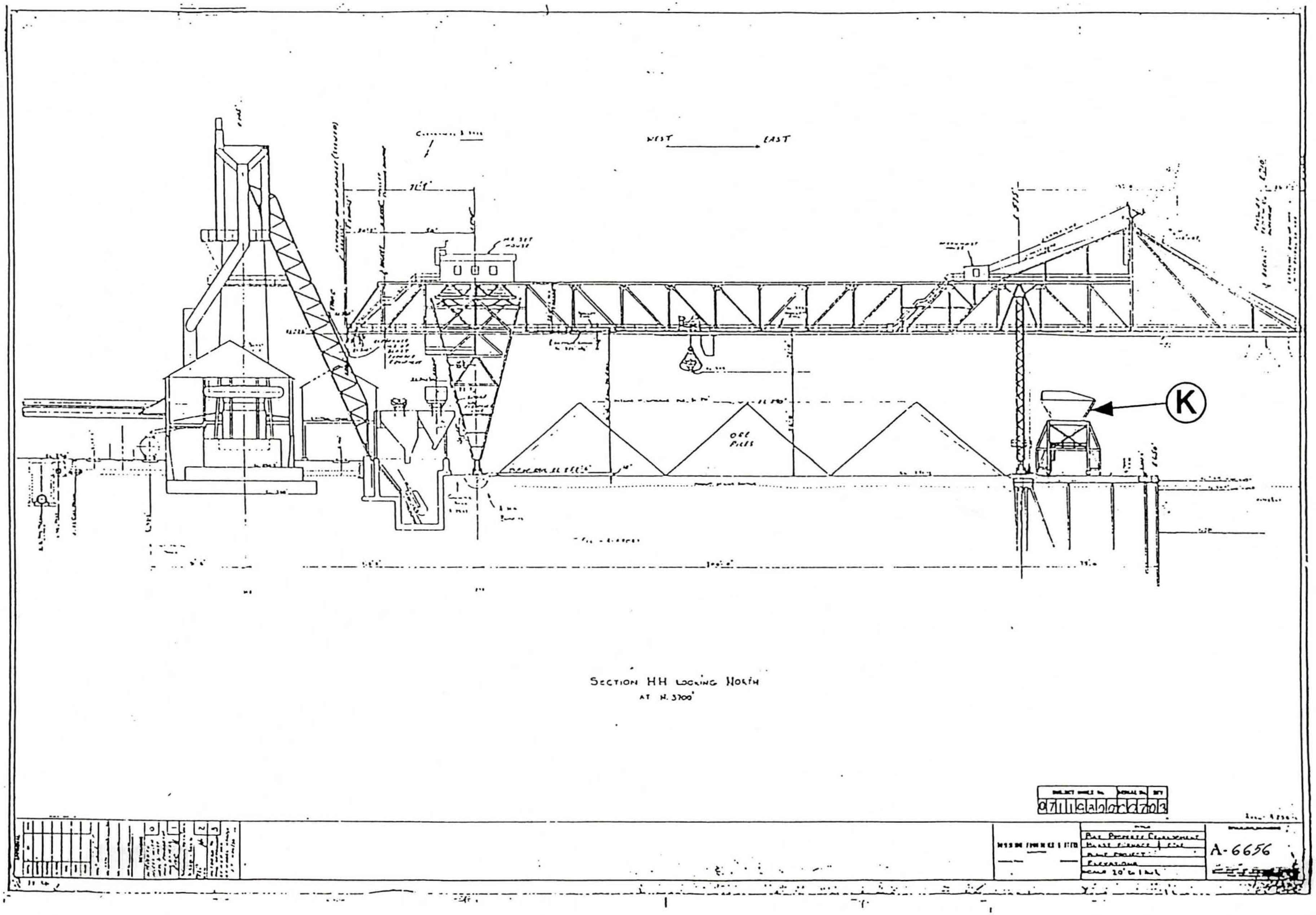


FIGURE 2: The Ore Bridge operations modified for coal handling. (See text for notes) (Dofasco, 1995)

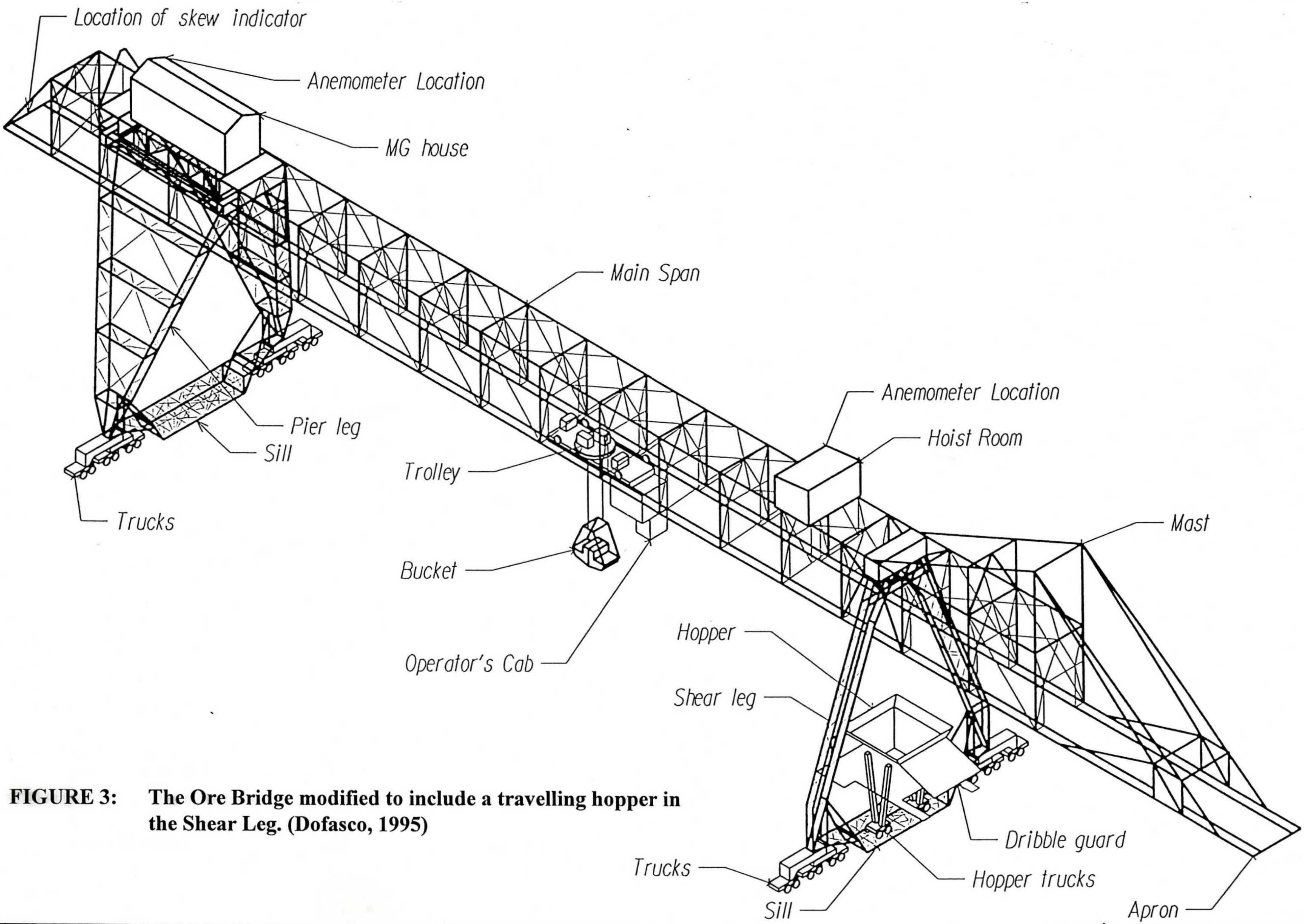


FIGURE 3: The Ore Bridge modified to include a travelling hopper in the Shear Leg. (Dofasco, 1995)

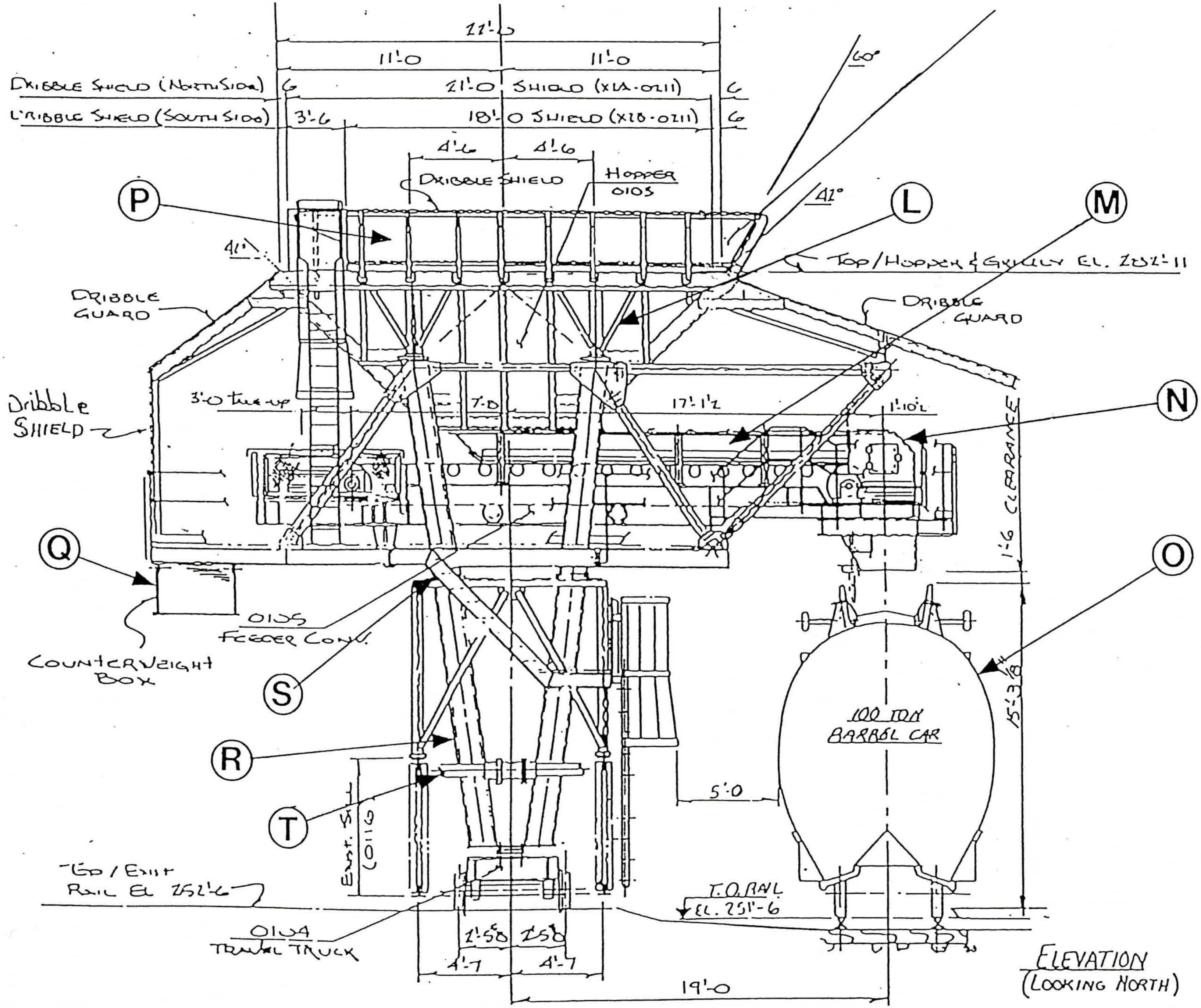


FIGURE 4: **General Arrangement of the hopper (looking north). (See text for notes) (Dofasco, 1995)**

FIGURE 5: View of the No.1 Ore Bridge (looking south).

FIGURE 6: View of the collapsed No.2 Ore Bridge (looking north).

FIGURE 7: **View of the collapsed No.2 Ore Bridge (looking west). (Dofasco, 1995)**

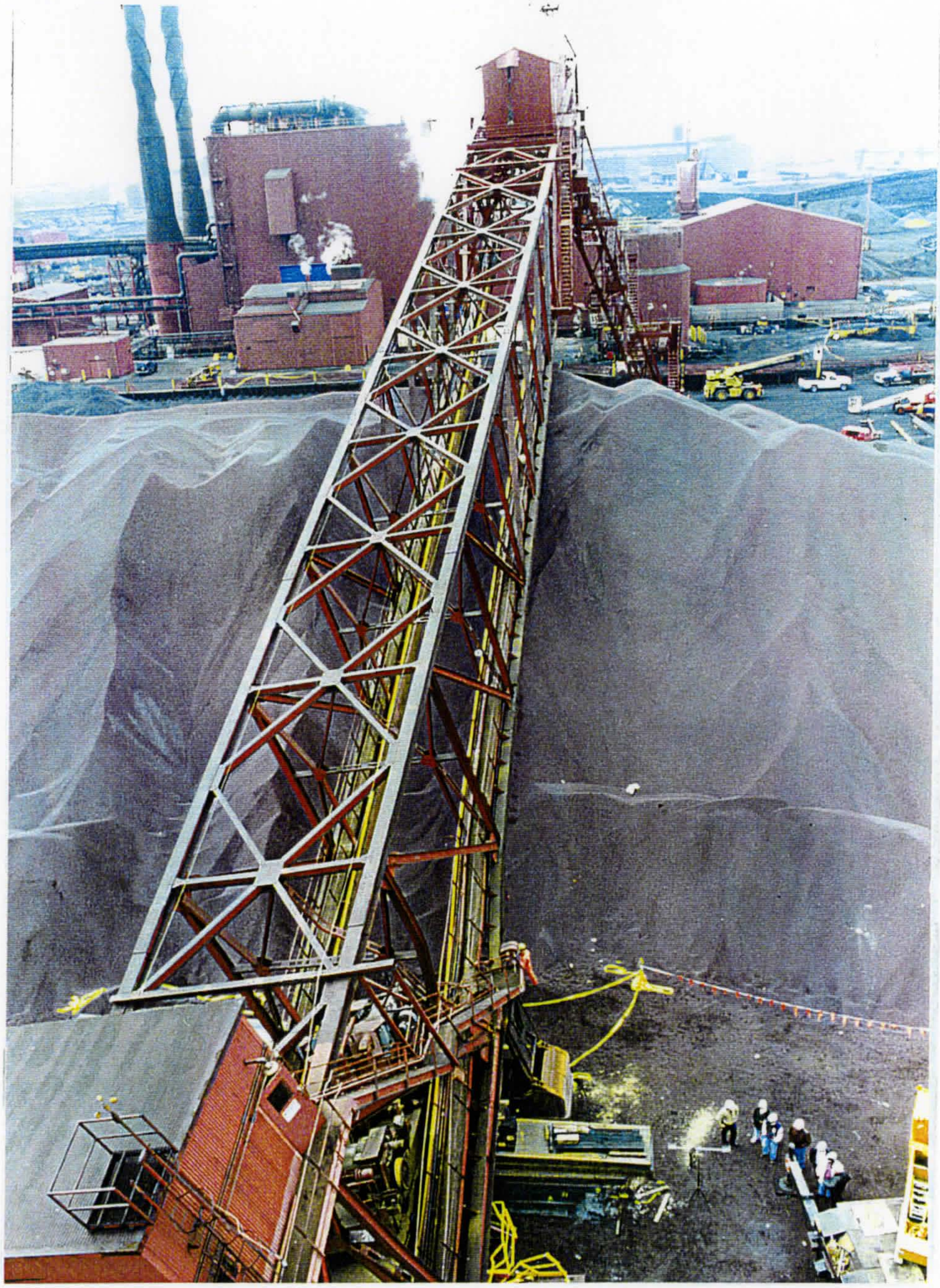


FIGURE 8: **Aerial view of the collapsed No.2 Ore Bridge (looking west). (Dofasco, 1995)**

FIGURE 9: Aerial view of the collapsed No.2 Ore Bridge in the vicinity of the Shear Leg (looking south). (Dofasco, 1995)

FIGURE 10: View of the No.1 Ore Bridge Shear Leg and hopper (looking east).

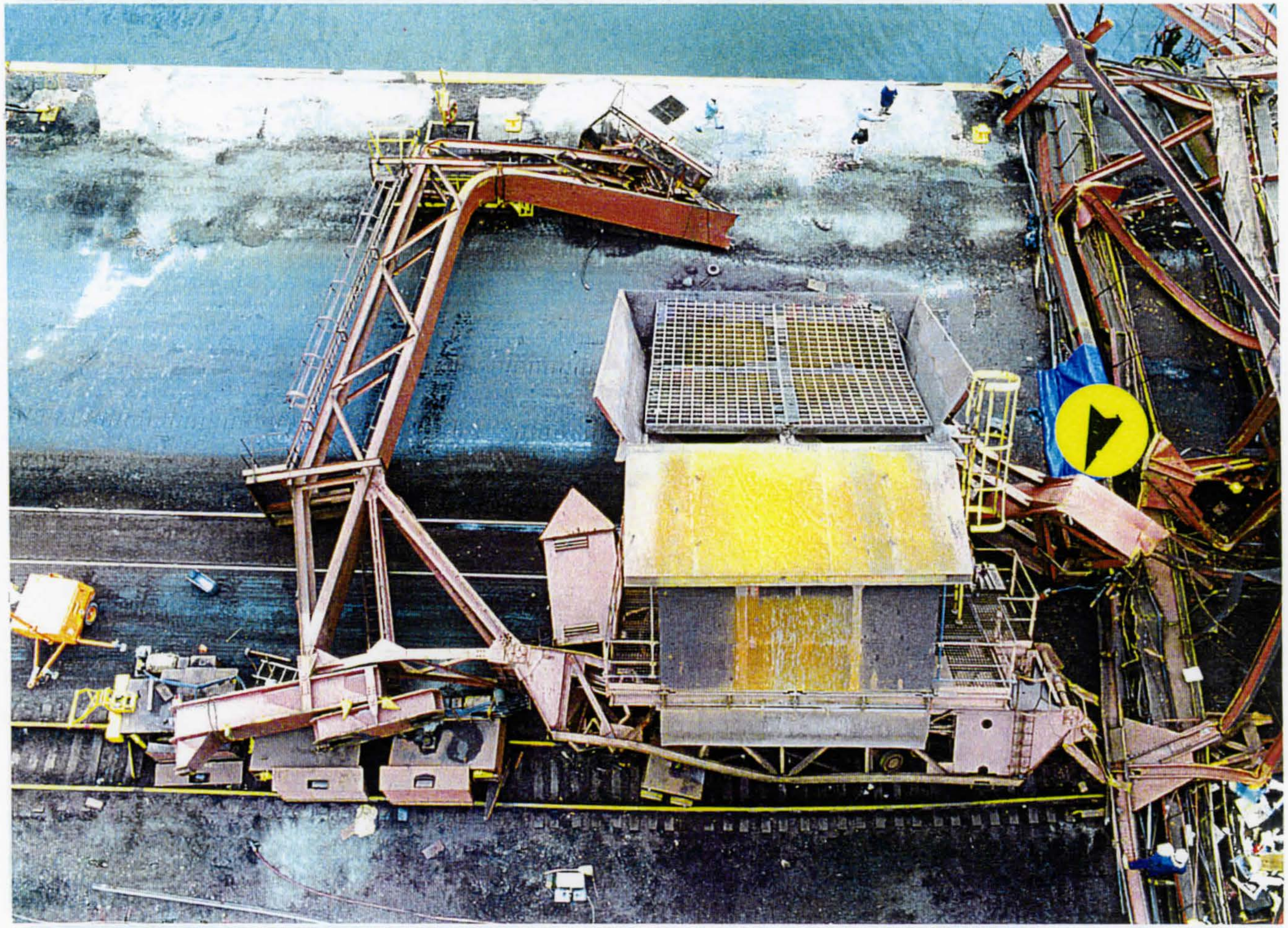


FIGURE 11: Aerial view of North Leg of Shear Leg and hopper (looking east). (Arrow points to haunch section) (Dofasco, 1995)

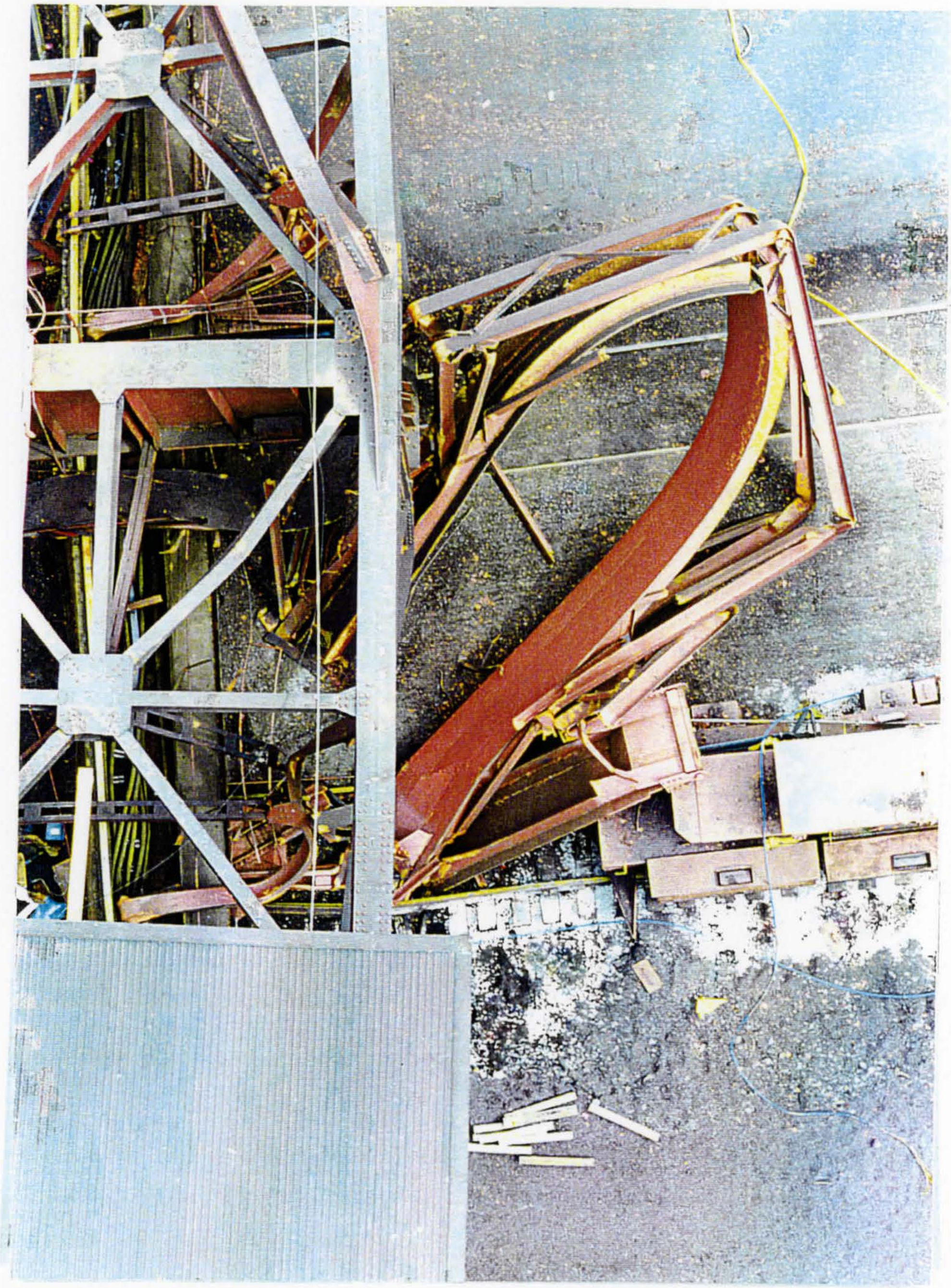


FIGURE 12: Aerial view of South Leg of Shear Leg (looking down). (Dofasco, 1995)

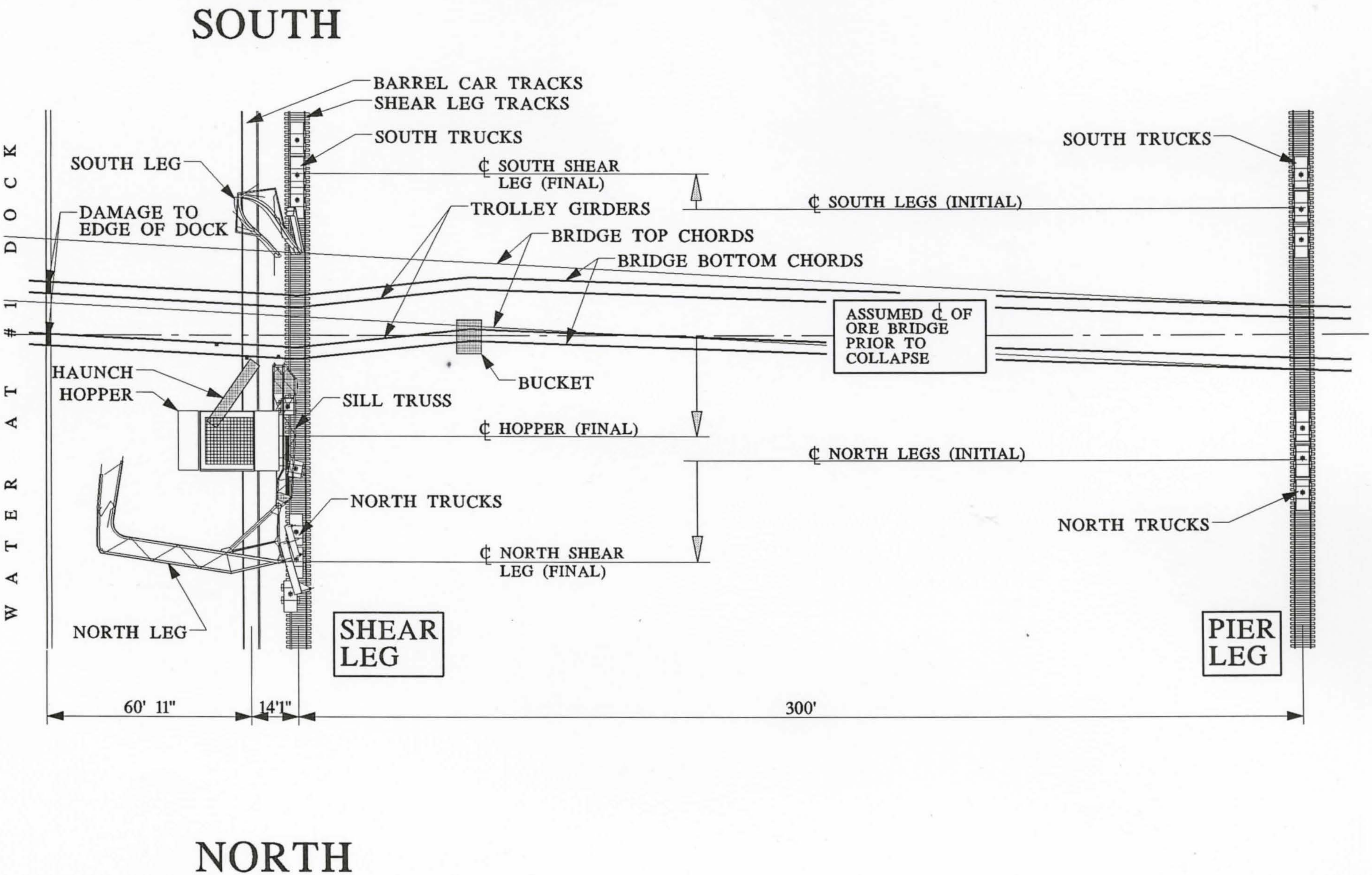


FIGURE 13: Plan view of collapsed No.2 Ore Bridge.

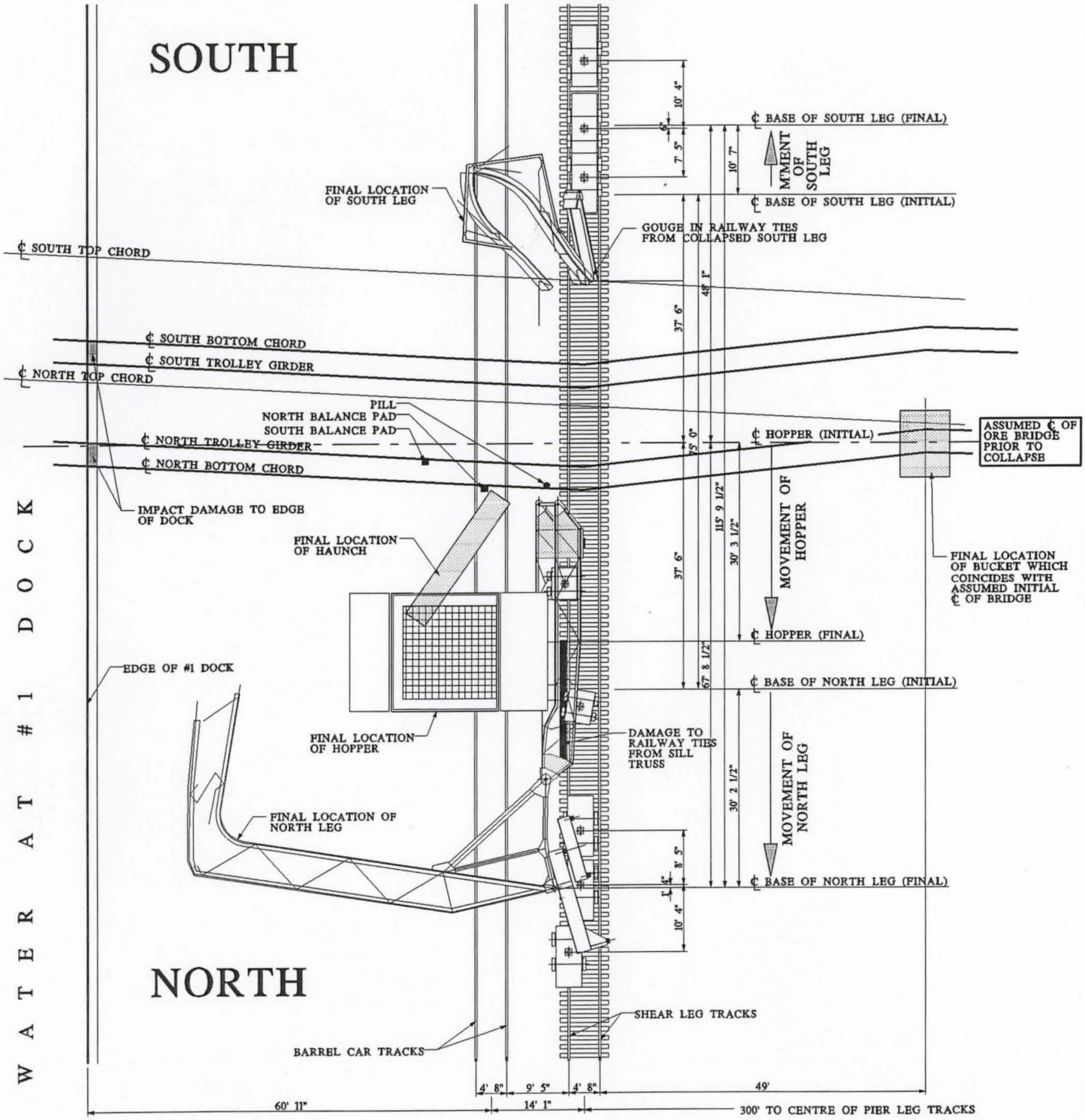


FIGURE 14: Detailed plan view of collapsed No.2 Ore Bridge in vicinity of Shear Leg.

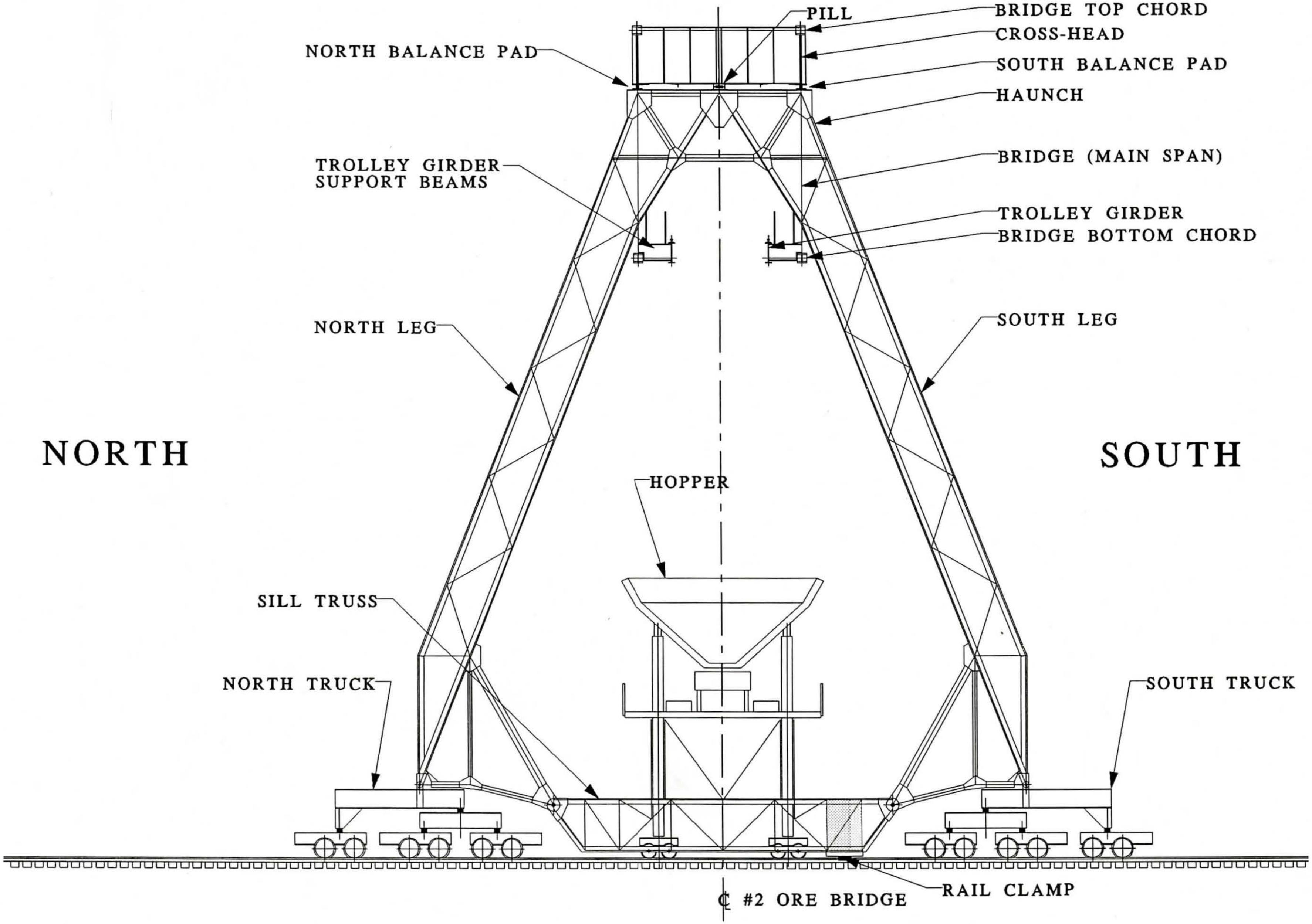


FIGURE 15: No.2 Ore Bridge Shear Leg (looking east).

FIGURE 16: Aerial view of haunch of Shear Leg.

FIGURE 17: View of top of Shear Leg.

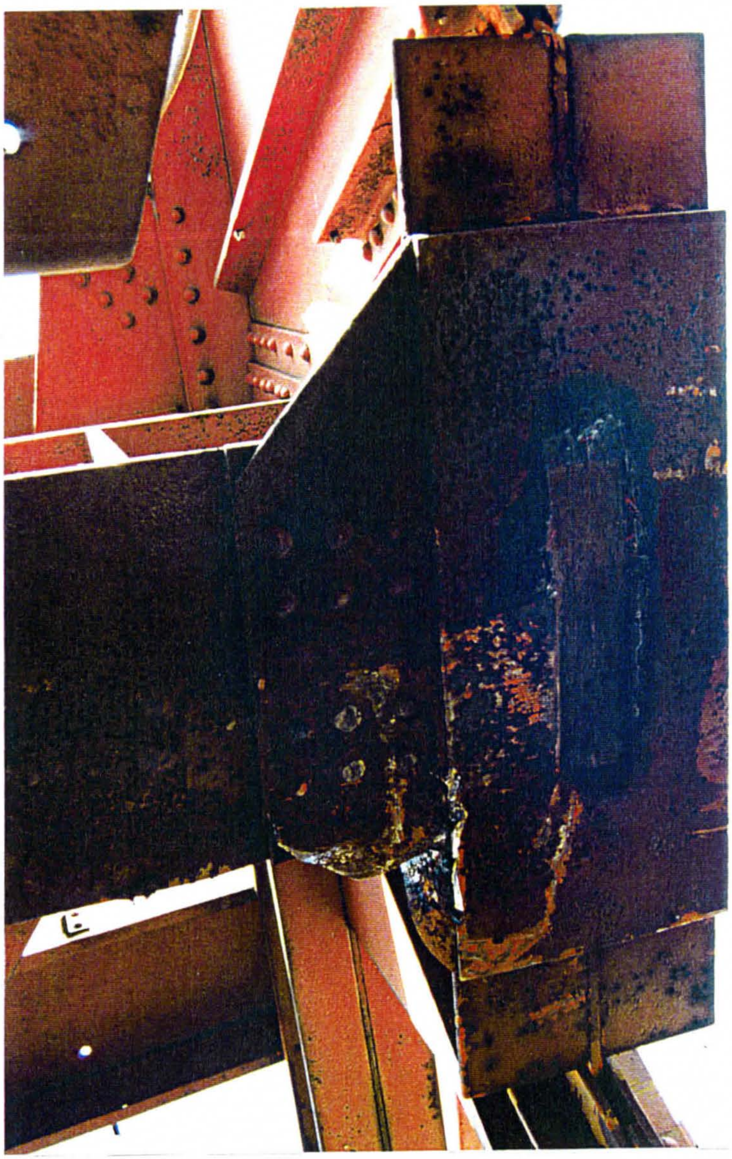
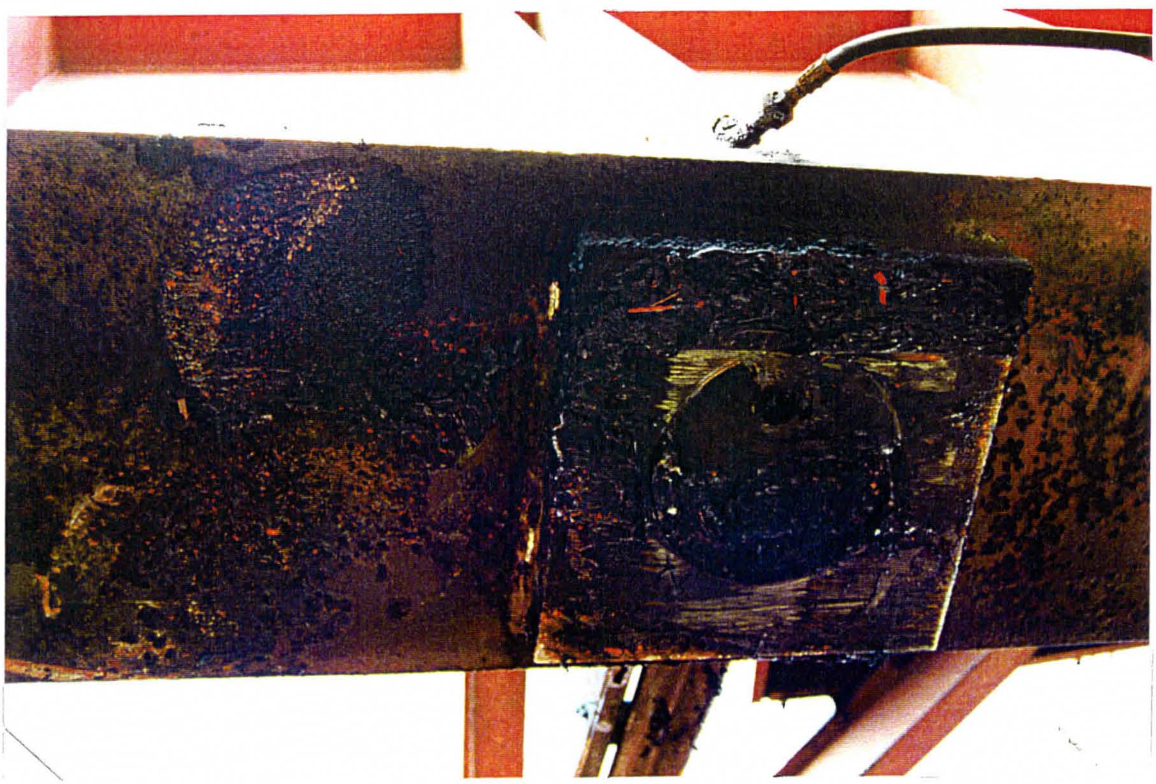


FIGURE 18: View of underside of cross-head at the north edge.

FIGURE 19: View of underside of cross-head at pill.



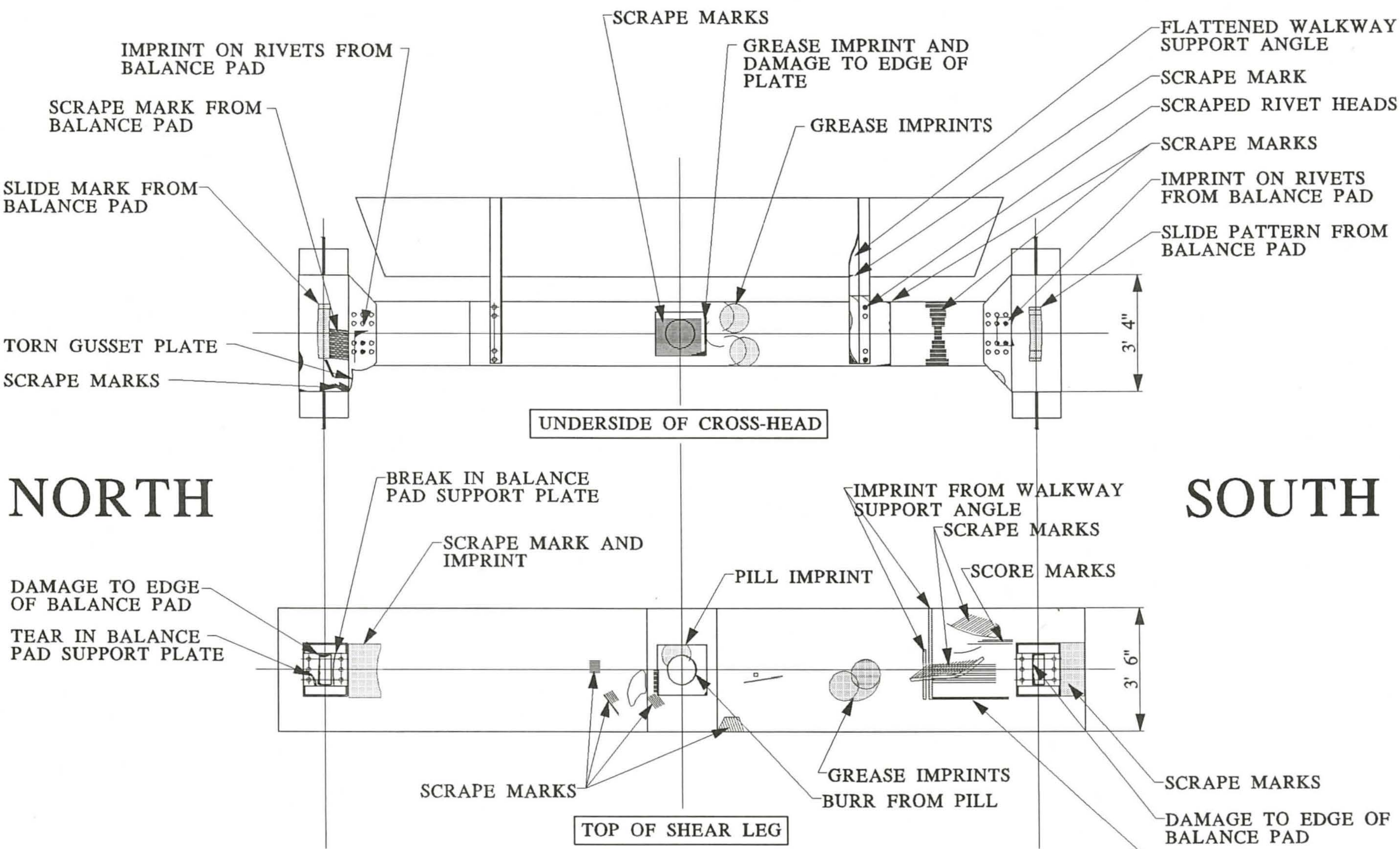


FIGURE 20: **Damage and markings on cross-head and top of Shear Leg looking from above.**

FIGURE 21: Damage and markings on inside face of South Leg of Shear Leg.

FIGURE 22: Damage and markings on inside face of South Leg of Shear Leg.

FIGURE 23: Damage and markings on top of south equaliser beam.

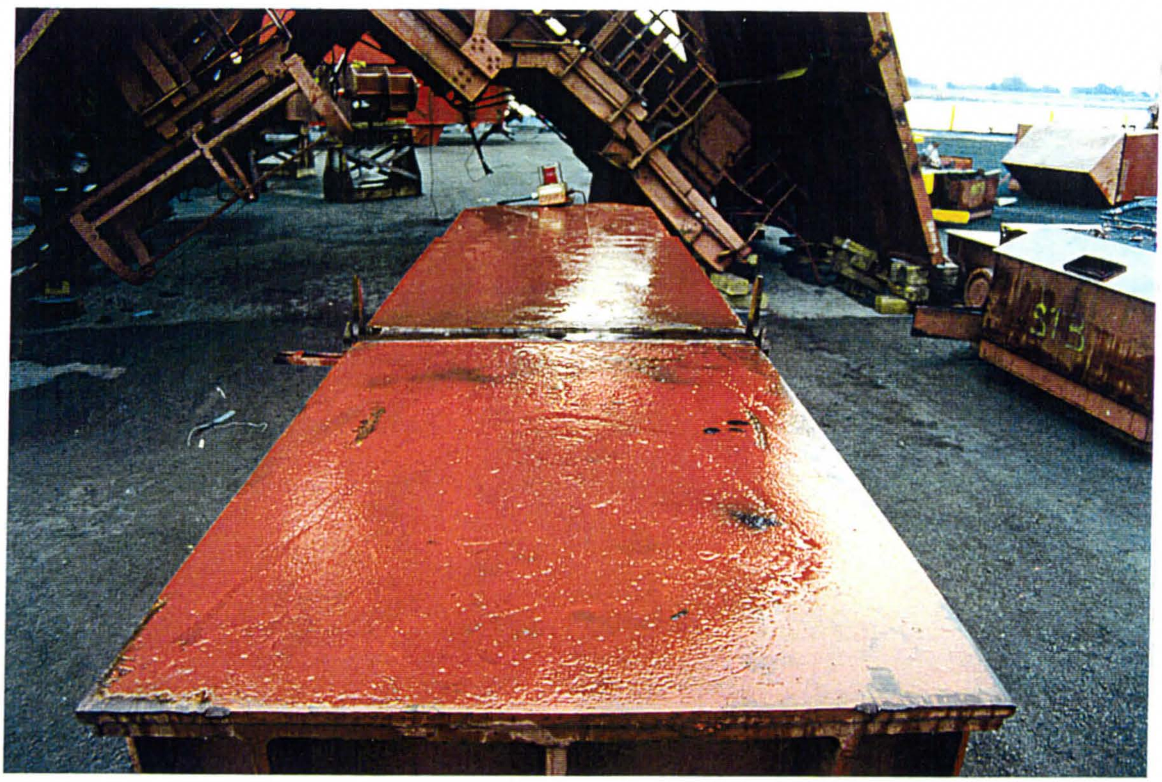


FIGURE 24: Damage and markings on top of south equaliser beam.

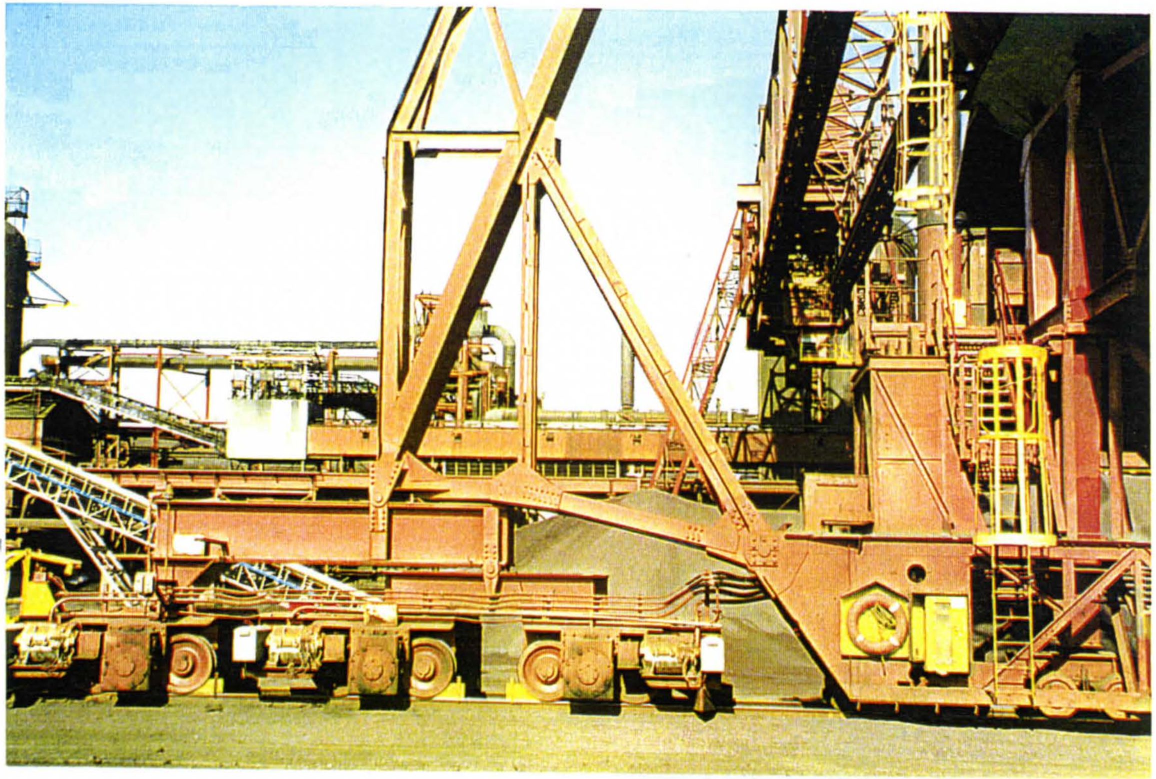


FIGURE 25: View of base of South Leg of No.1 Ore Bridge Shear Leg.

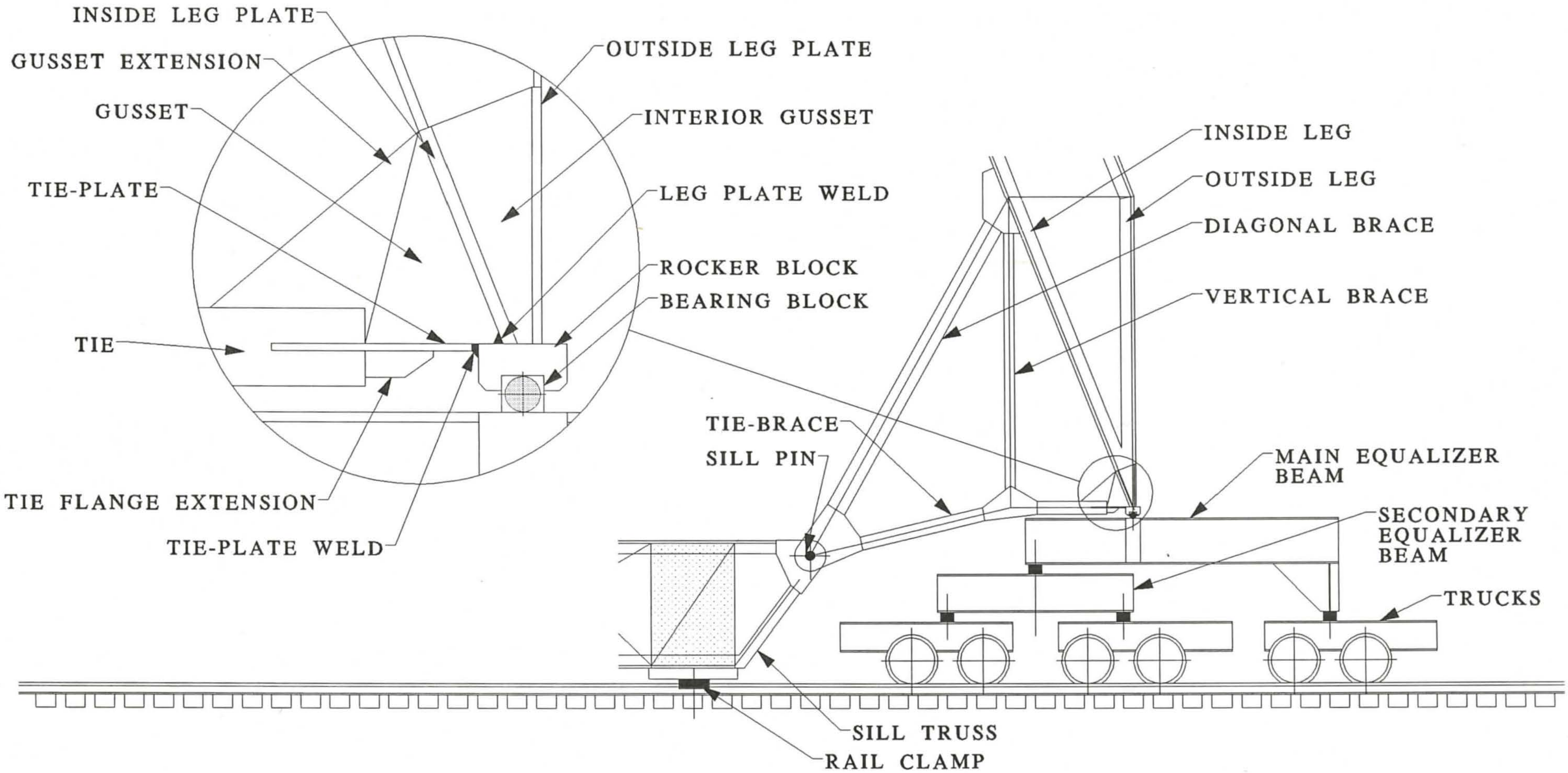


FIGURE 26: **Detail view of base of South Leg of No.2 Ore Bridge Shear Leg (looking east).**

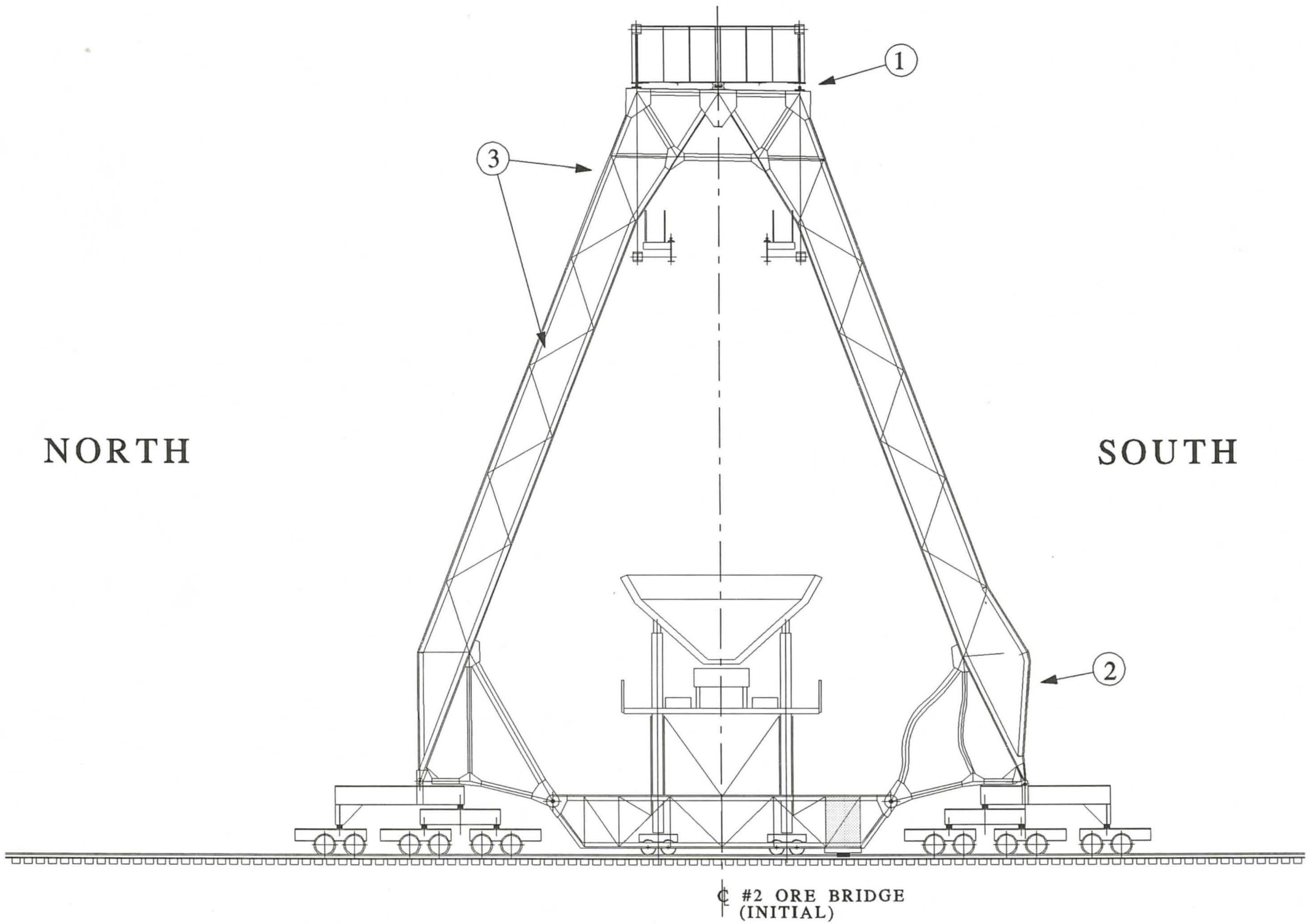


FIGURE 27: Collapse of Shear Leg of No.2 Ore Bridge (looking east). Sequence 1 (See text for notes)

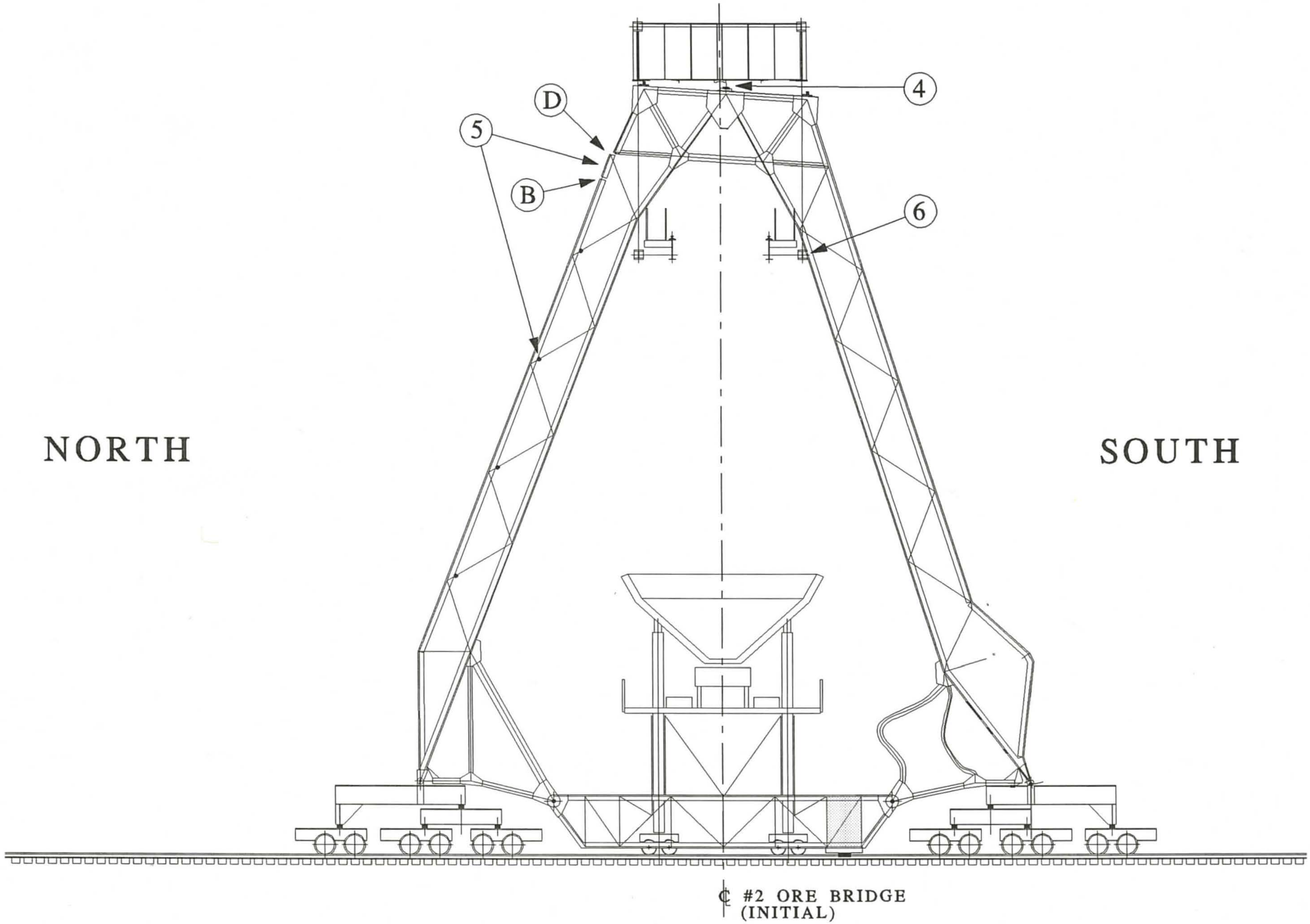


FIGURE 28: Collapse of Shear Leg of No.2 Ore Bridge (looking east). Sequence 2 (See text for notes)

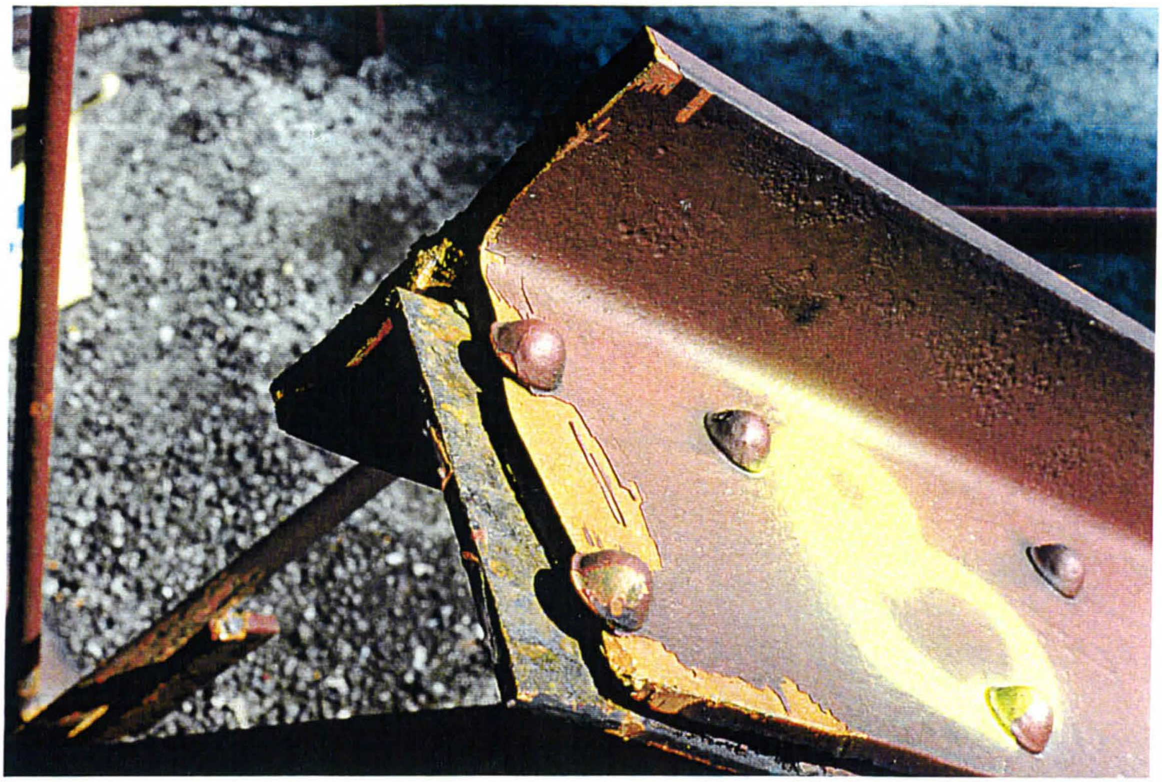


FIGURE 29: Failed connection "B". (See Figure 28)

FIGURE 30: Failed connection "D". (See Figure 28)

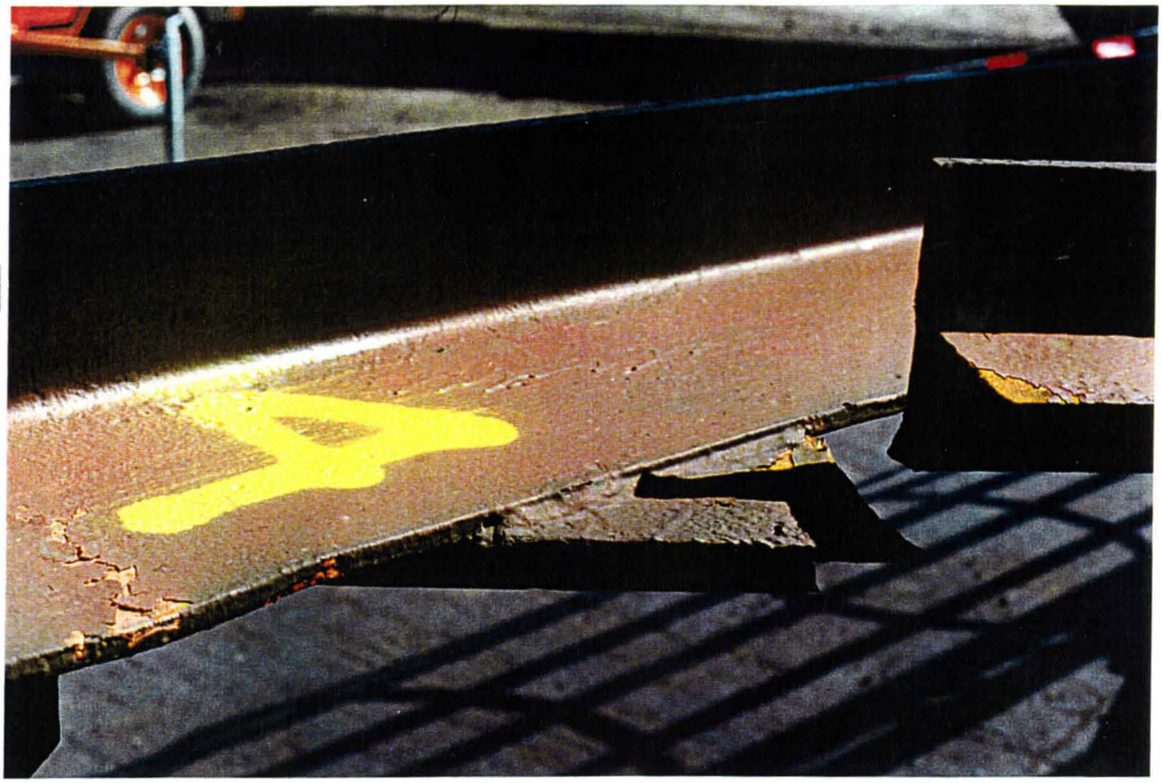


FIGURE 31: Failed tension diagonal in North Leg of Shear Leg.

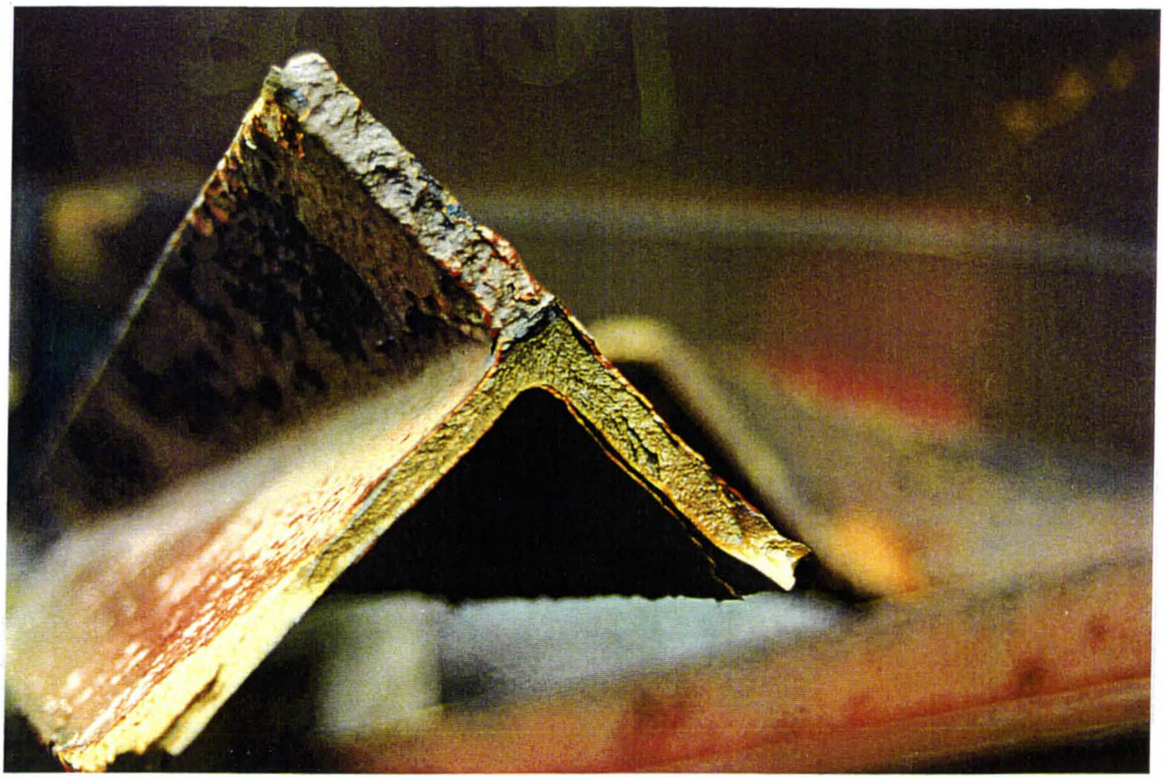


FIGURE 32: Closeup view of typical fracture face of failed tension diagonals in North Leg of Shear Leg.

FIGURE 33: Impact damage to inside face of South Leg of Shear Leg near the haunch section.

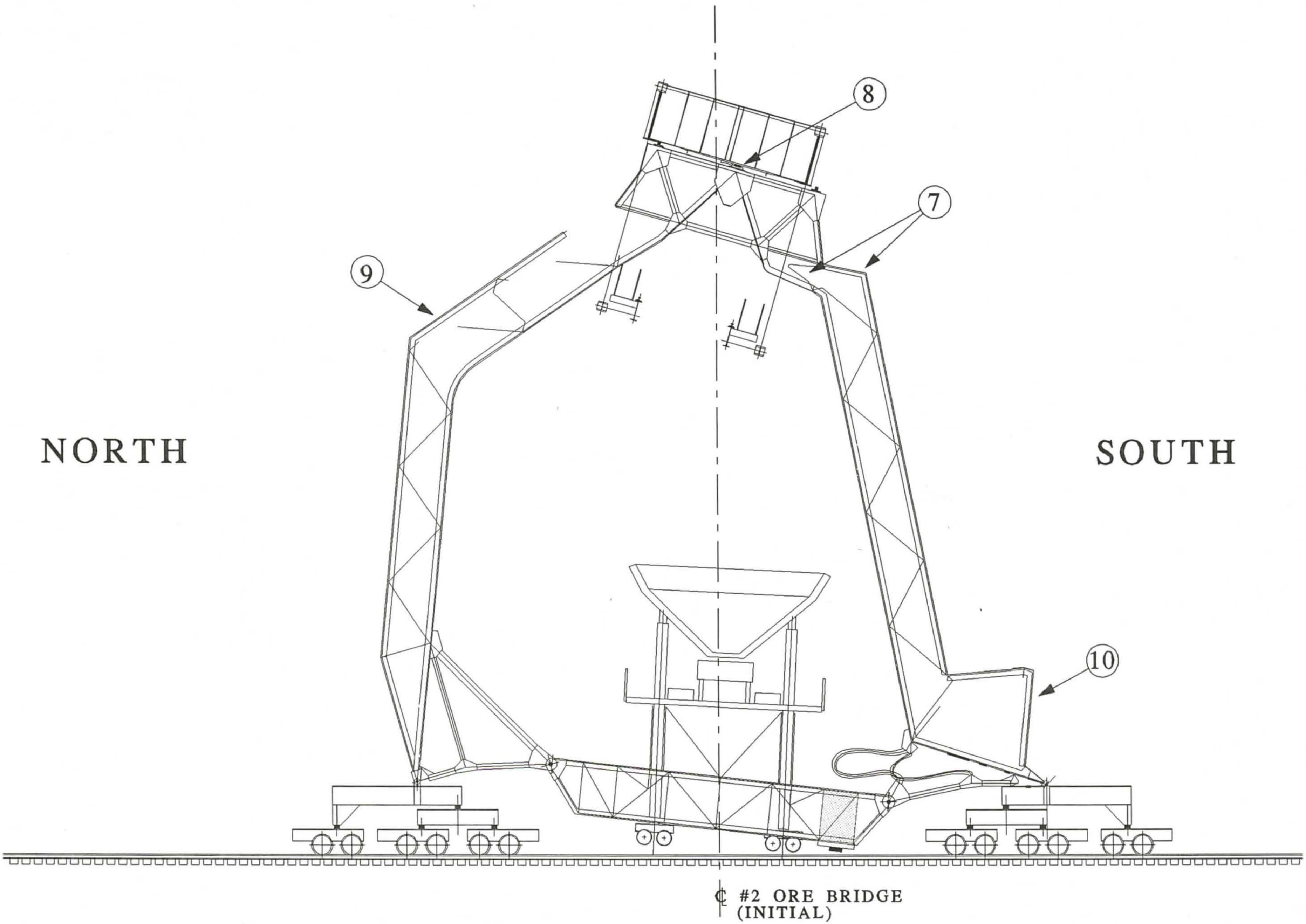


FIGURE 34: Collapse of Shear Leg of No.2 Ore Bridge (looking east). Sequence 3 (See text for notes)

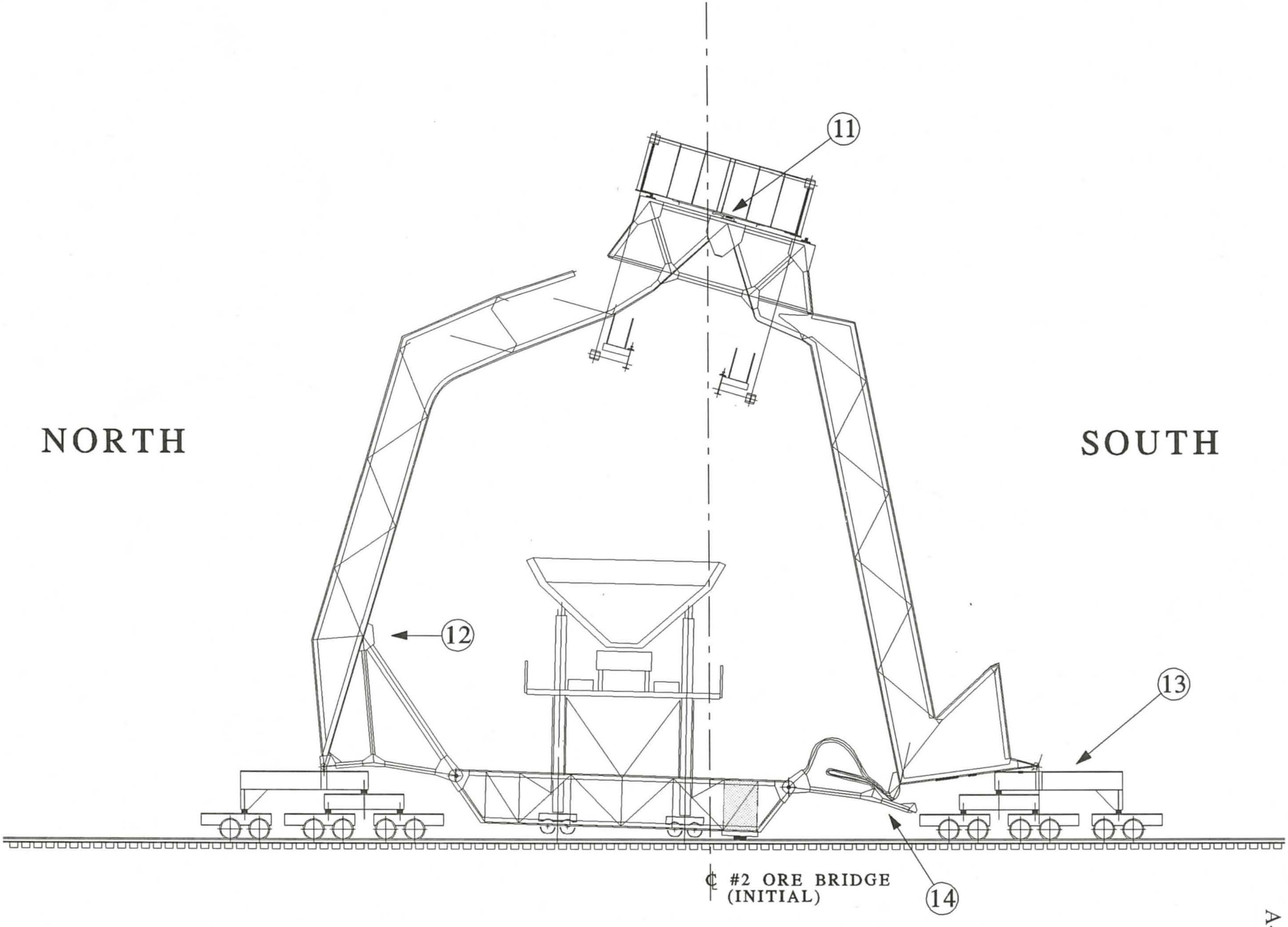


FIGURE 35: Collapse of Shear Leg of No.2 Ore Bridge (looking east). Sequence 4 (See text for notes)

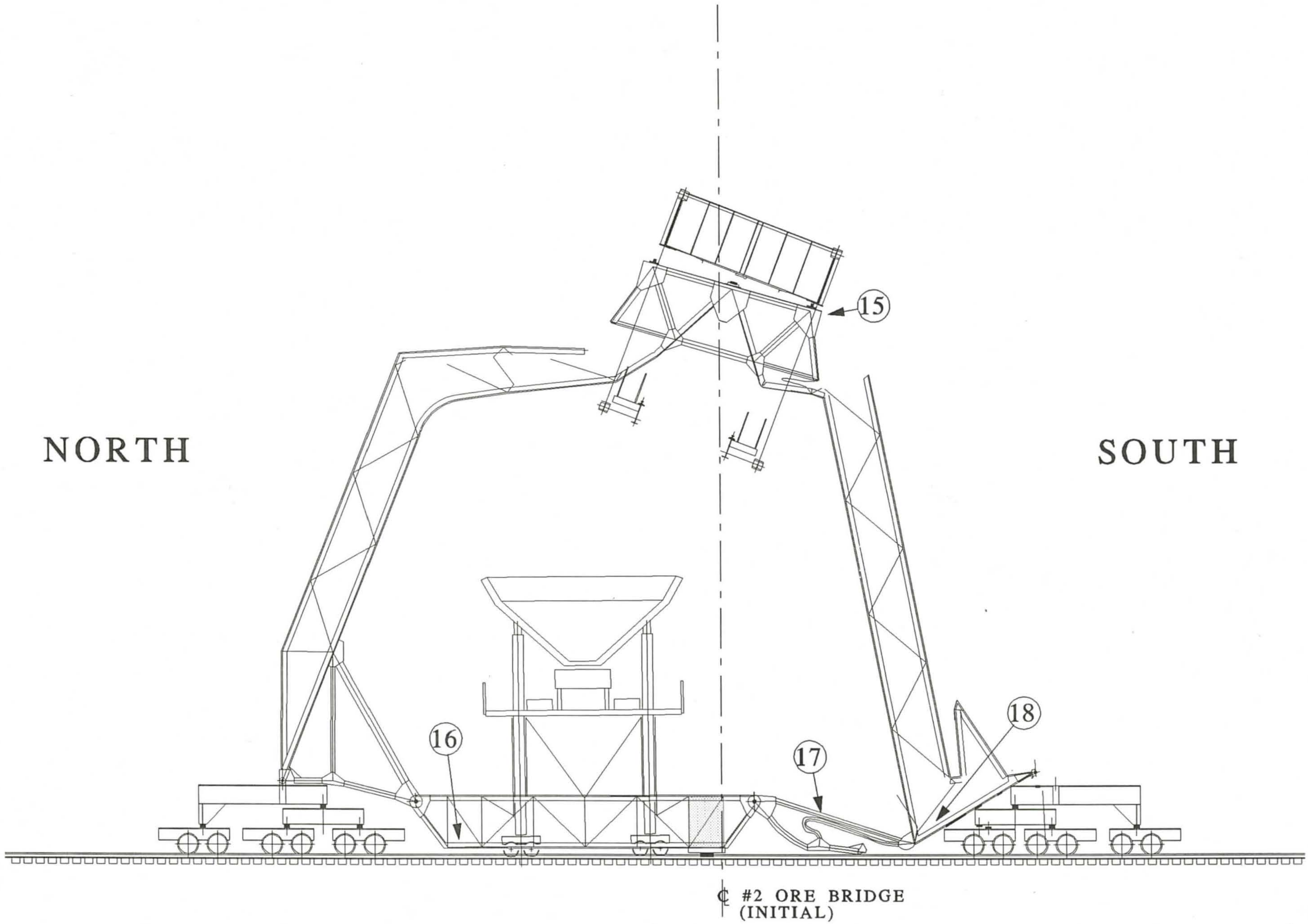


FIGURE 36: Collapse of Shear Leg of No.2 Ore Bridge (looking east). Sequence 5 (See text for notes)

FIGURE 37: Collapsed lower portion of South Leg of Shear Leg partially resting on south drive truck (looking east).

FIGURE 38: South Leg of Shear Leg where it was driven into the railway ties and ballast.

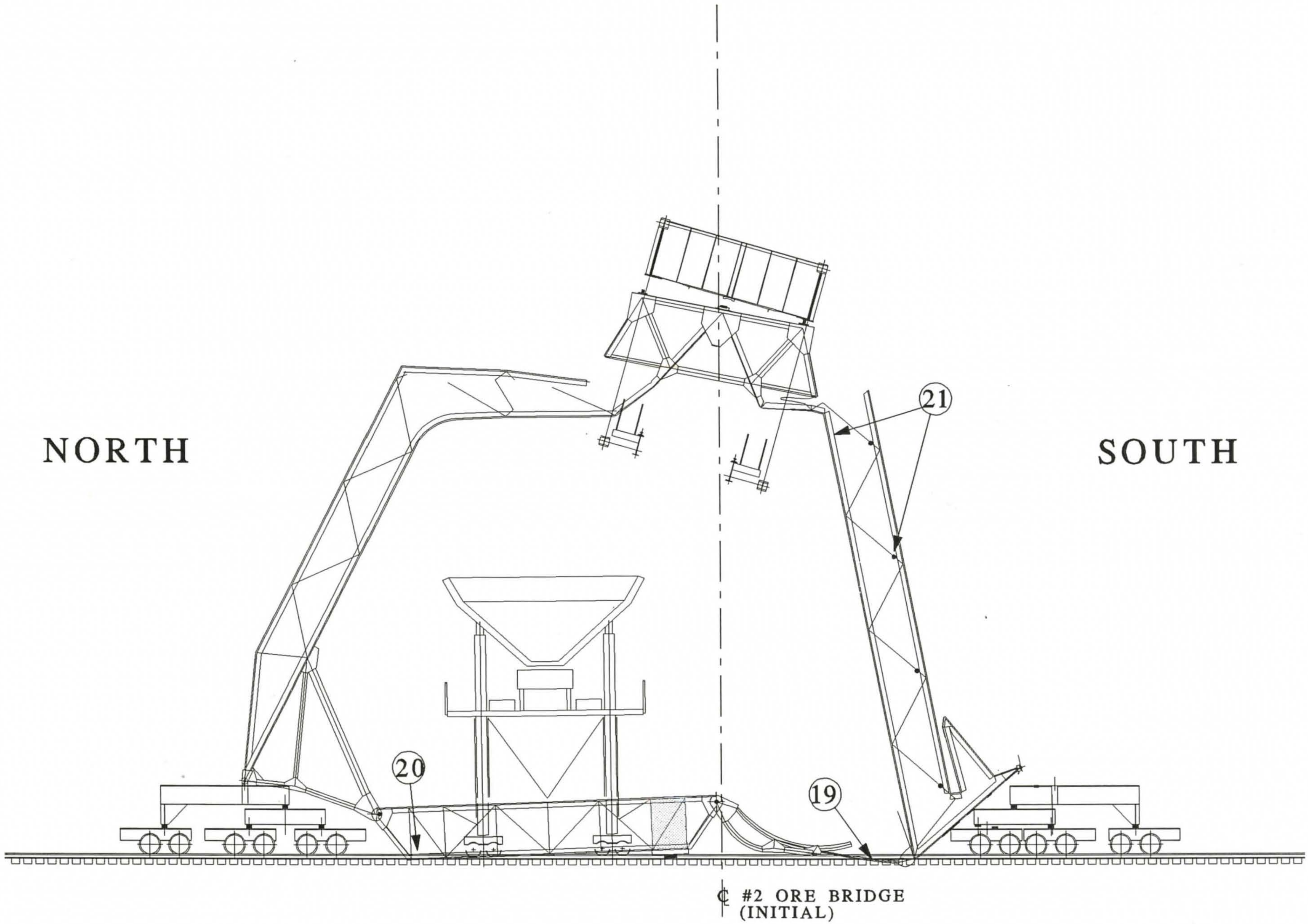


FIGURE 39: Collapse of Shear Leg of No.2 Ore Bridge (looking east).
Sequence 6 (See text for notes)

FIGURE 40: View of damage to railway ties and ballast from the northerly movement of the sill truss and hopper.

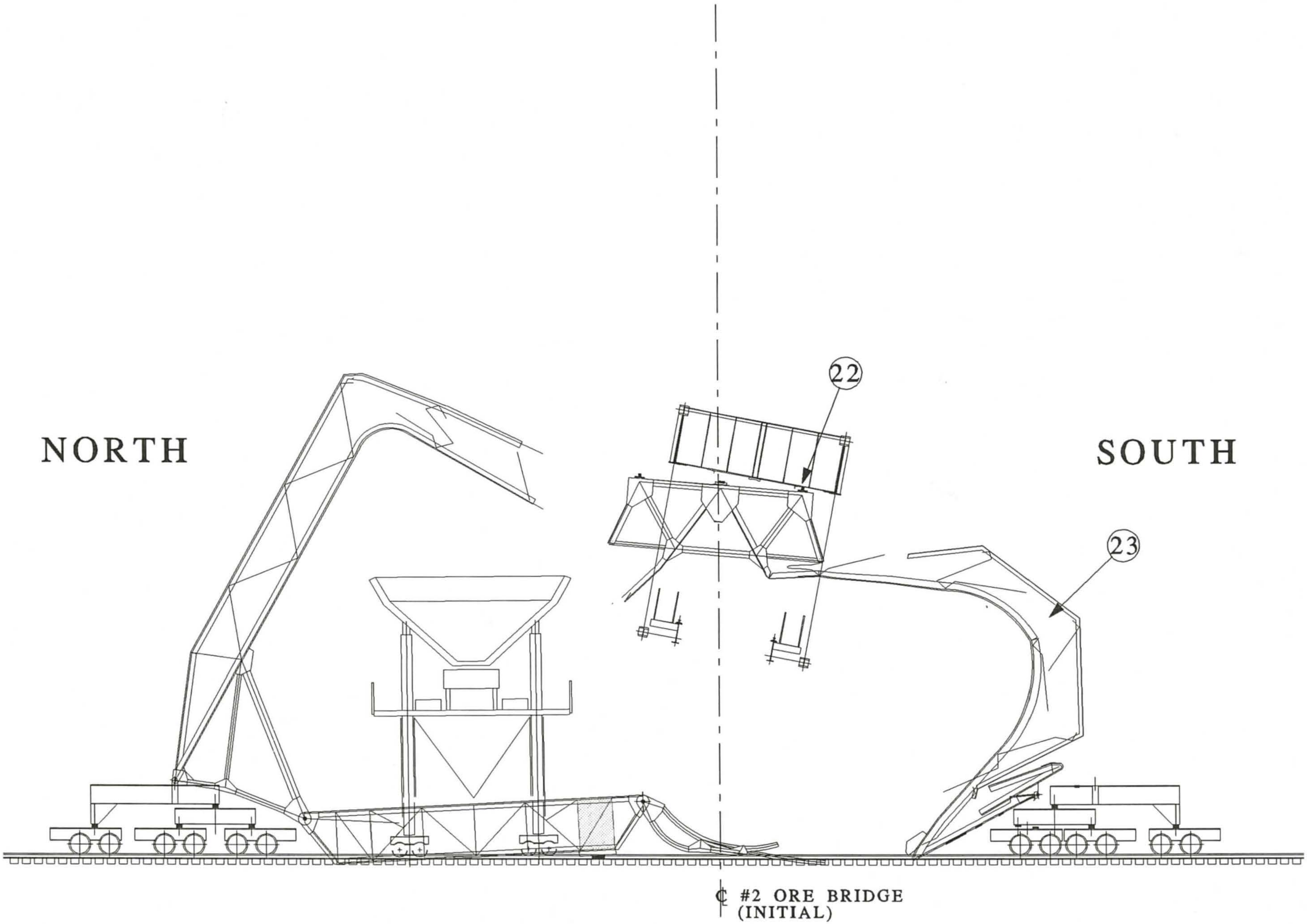


FIGURE 41: Collapse of Shear Leg of No.2 Ore Bridge (looking east). Sequence 7 (See text for notes)

FIGURE 42: Aerial view of collapsed South Leg of Shear Leg.

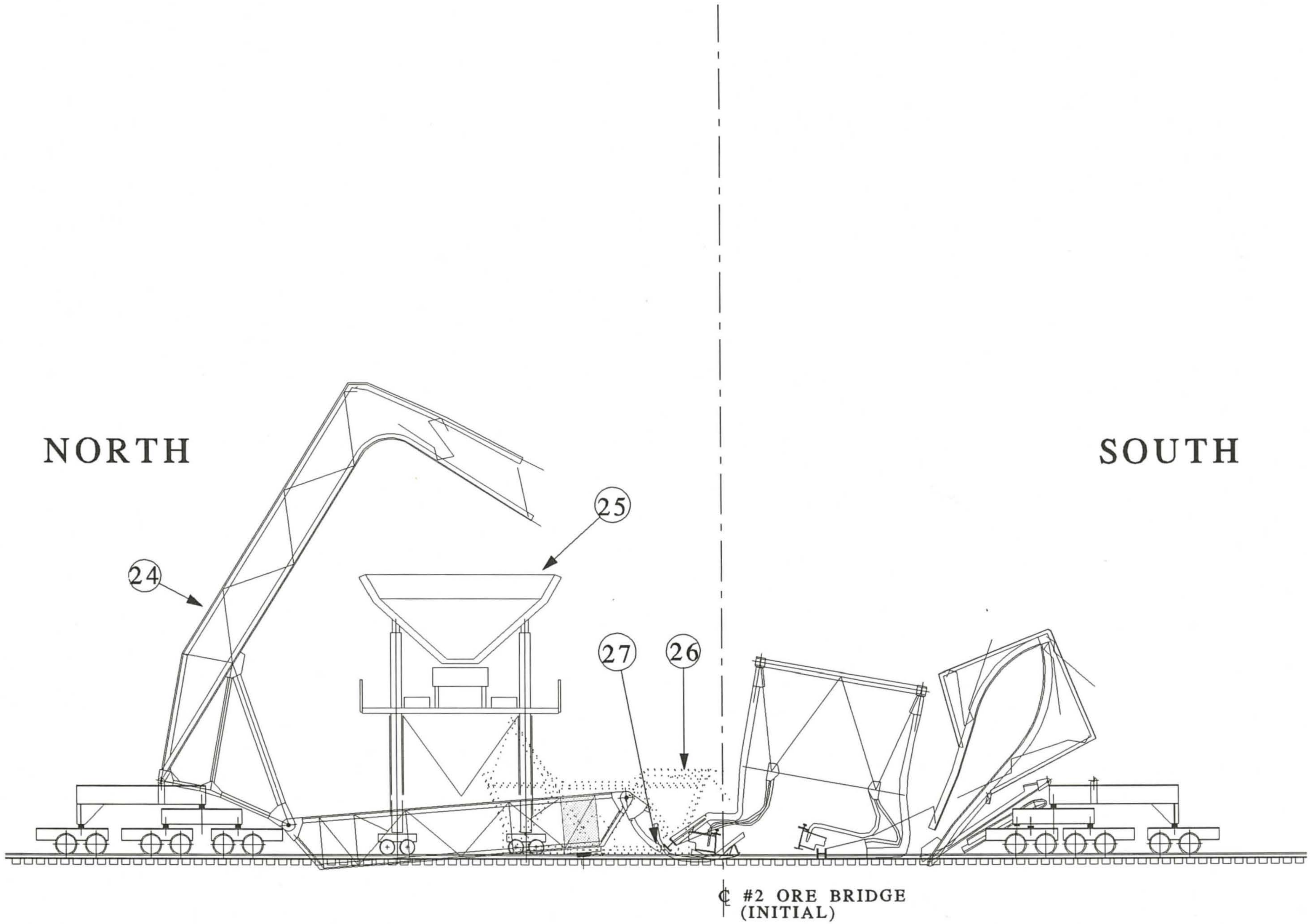


FIGURE 43: **Collapse of Shear Leg of No.2 Ore Bridge (looking east). Sequence 8 (See text for notes)**

FIGURE 44: Final location of haunch section (Arrow notes damage to cross beam).

FIGURE 45: Final location of tie-brace underneath the north bottom chord of the main span.

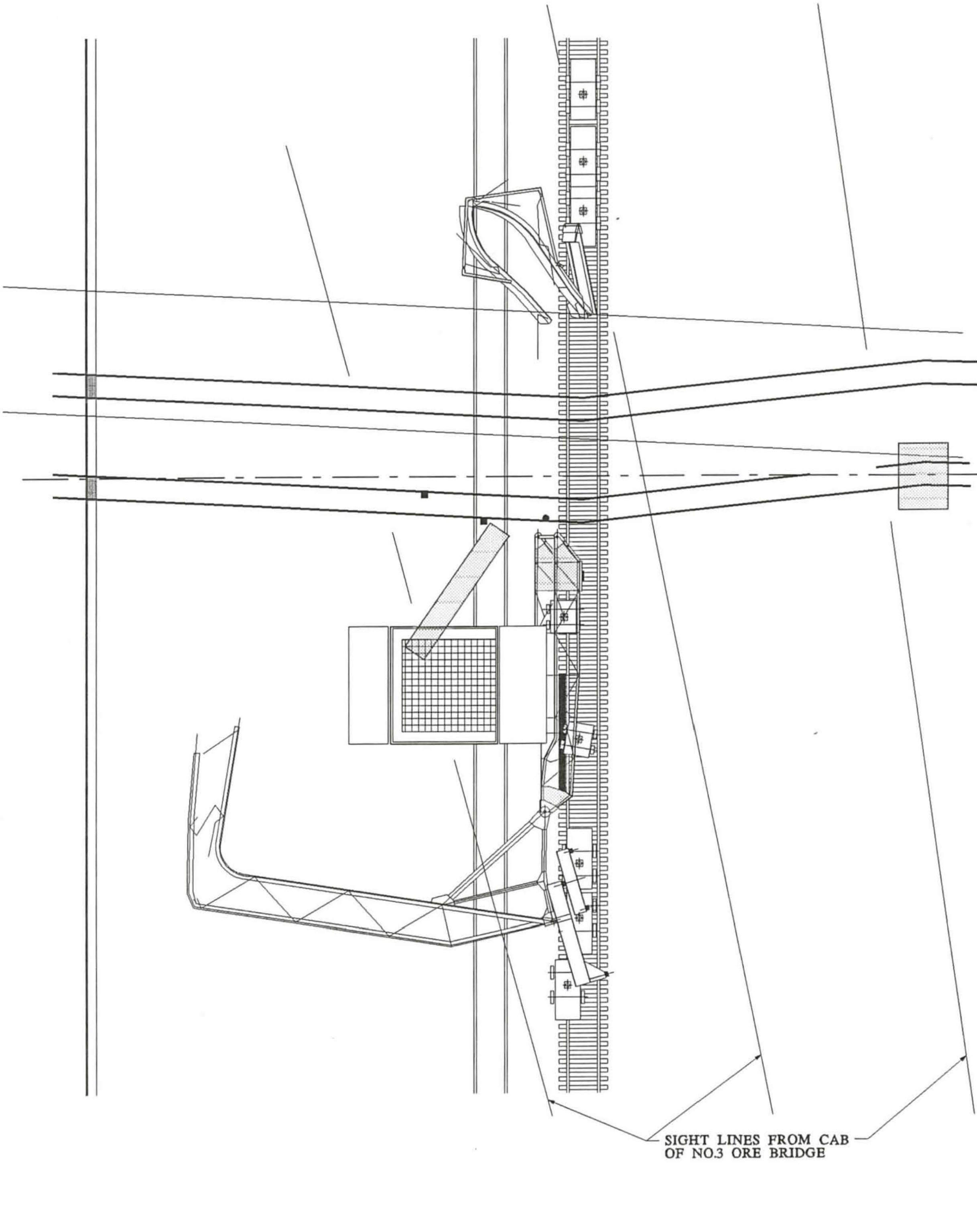


FIGURE 46: Sight lines from the cab of the No.3 Ore Bridge.

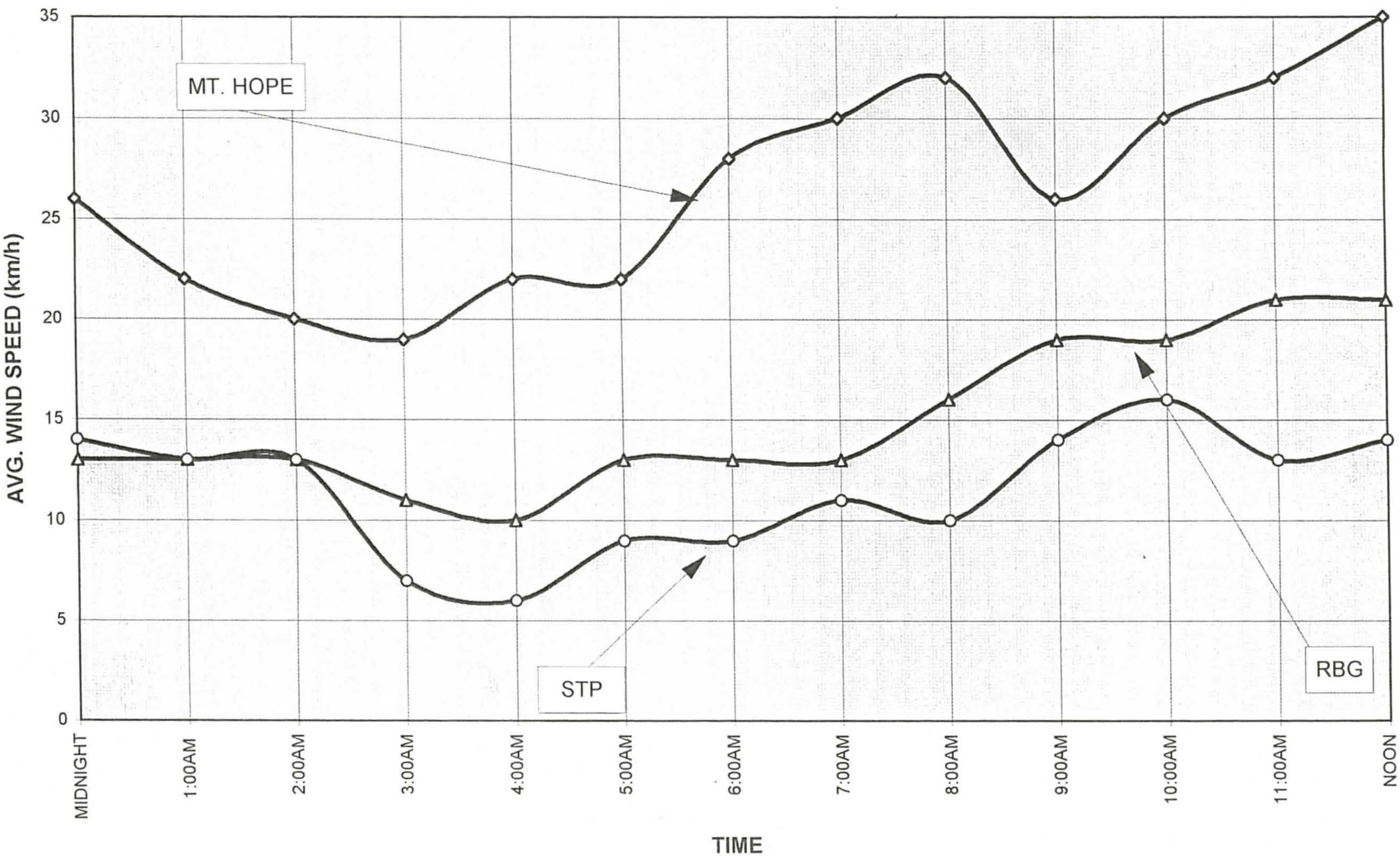


FIGURE 47: Average wind speeds (km/hr) at the time of the collapse as measured at the Royal Botanical Gardens (RBG), Woodward Avenue Sewage Treatment Plant (STP) and the Mount Hope Airport (MT. HOPE).

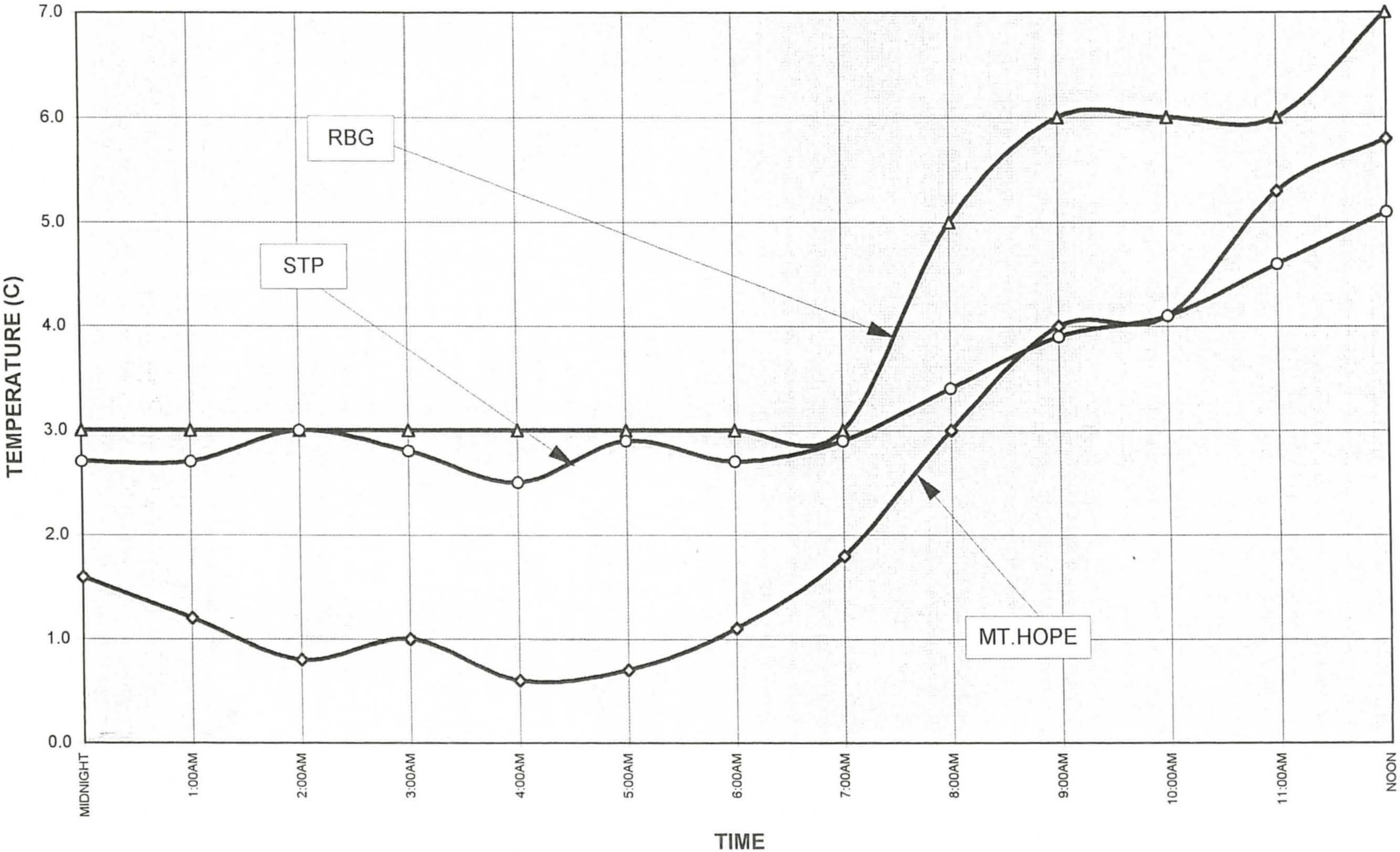


FIGURE 48: **Average temperatures (^{0}C) at the time of the collapse as measured at the Royal Botanical Gardens (RBG), Woodward Avenue Sewage Treatment Plant (STP) and the Mount Hope Airport (MT. HOPE).**

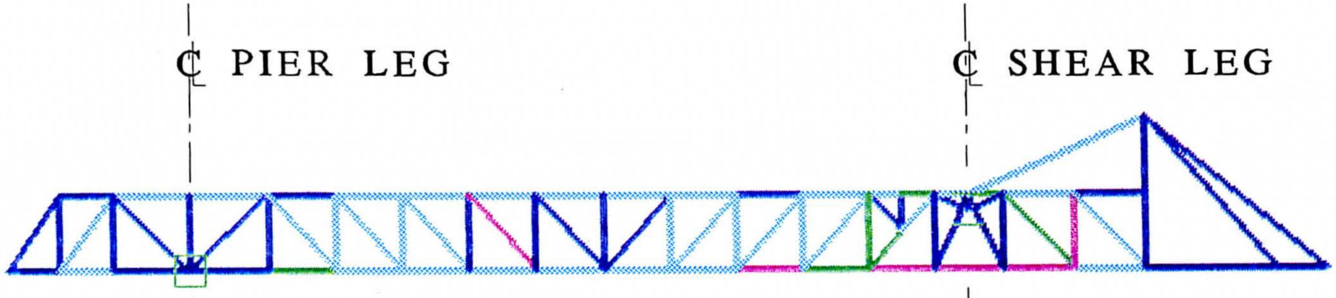


Fig. 49a No.2 Ore Bridge - Original Design (1950)

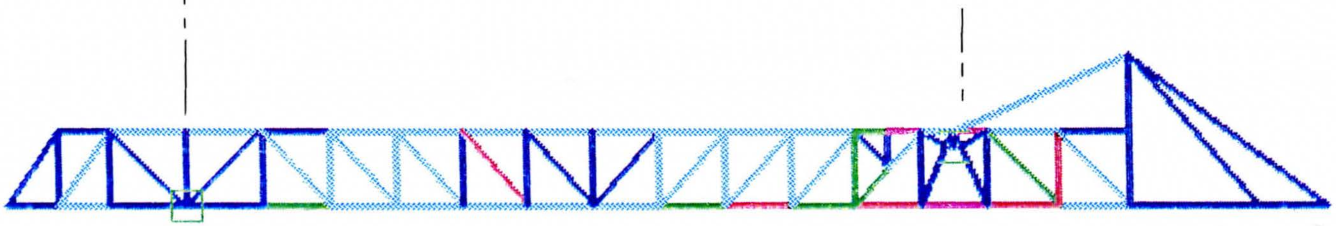


Fig. 49b No.2 Ore Bridge - Apron Extended (1968)

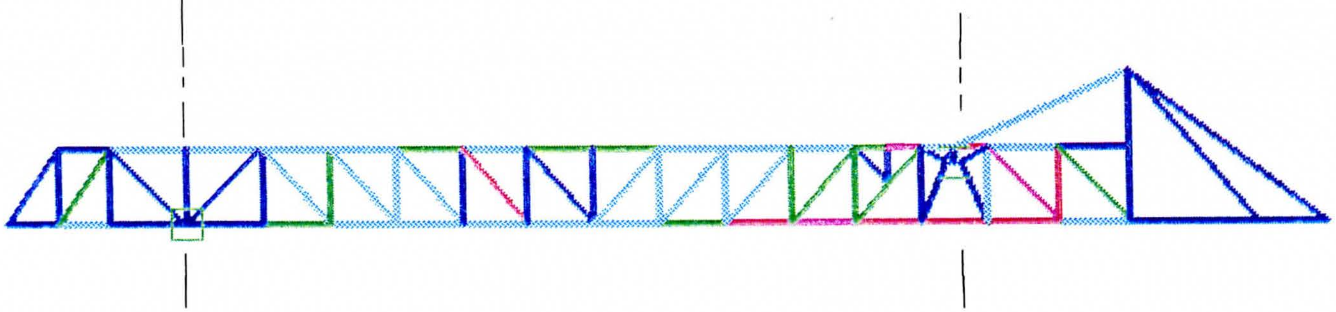
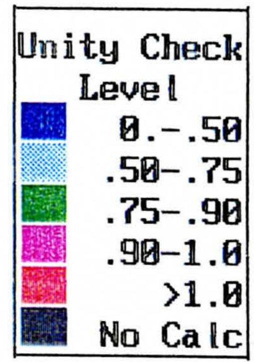


Fig. 49c No.2 Ore Bridge - Final Configuration (1995)



LEGEND

FIGURE 49: Results of a two-dimensional structural design check of the Main Span of the No.2 Ore Bridge.

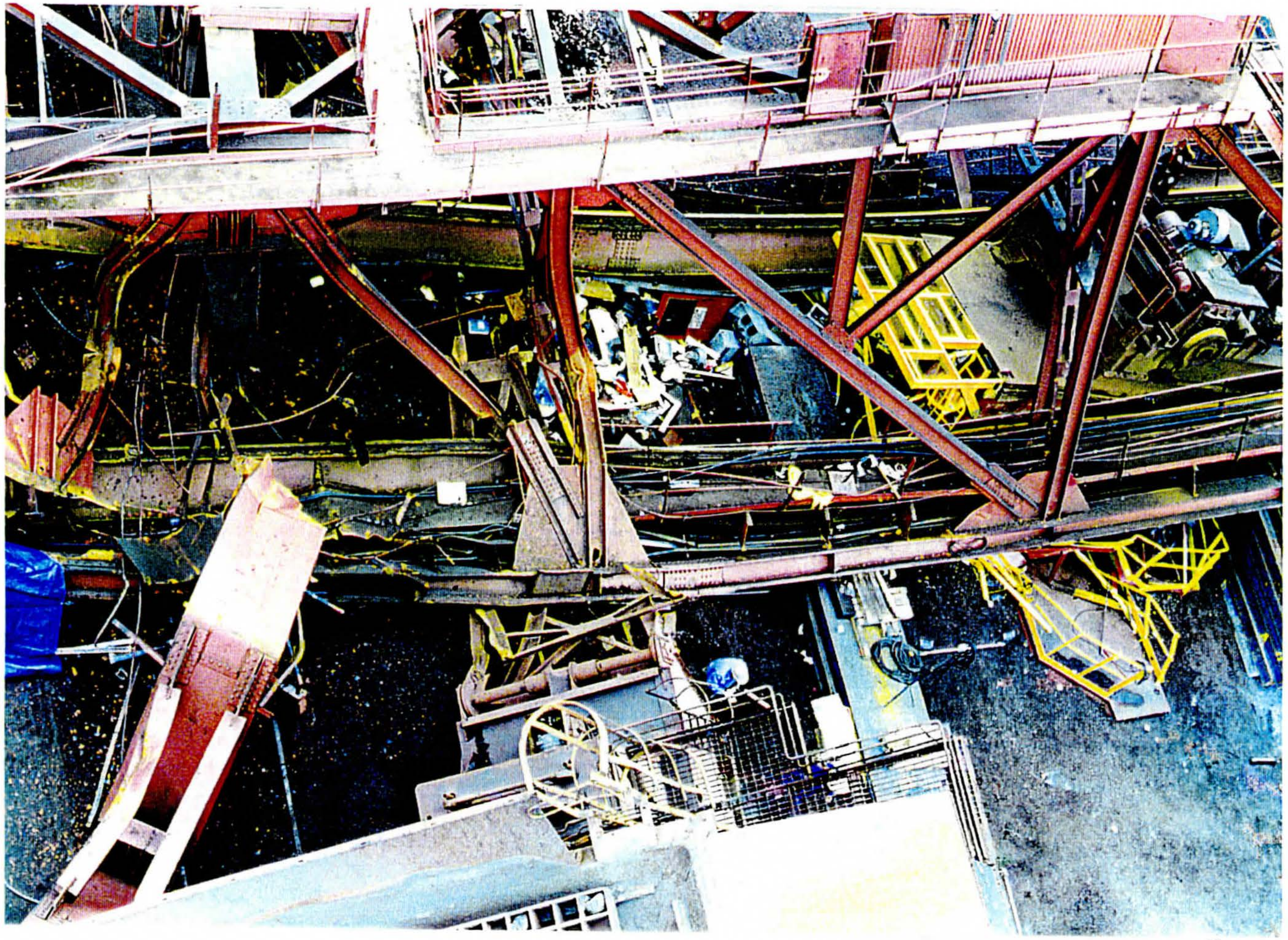


FIGURE 50: Aerial view of main span members in the vicinity of the Shear Leg (looking south). (Dofasco, 1995)

FIGURE 51: **View of the north bottom chord of the Main Span where it impacted the ground during the collapse. (Dofasco, 1995)**

FIGURE 52: View of the south side of the Main Span in the vicinity of the Shear Leg (looking north). (Dofasco, 1995)

FIGURE 53: View of the north side of the Main Span where it impacted the bucket (looking south). (Dofasco, 1995)

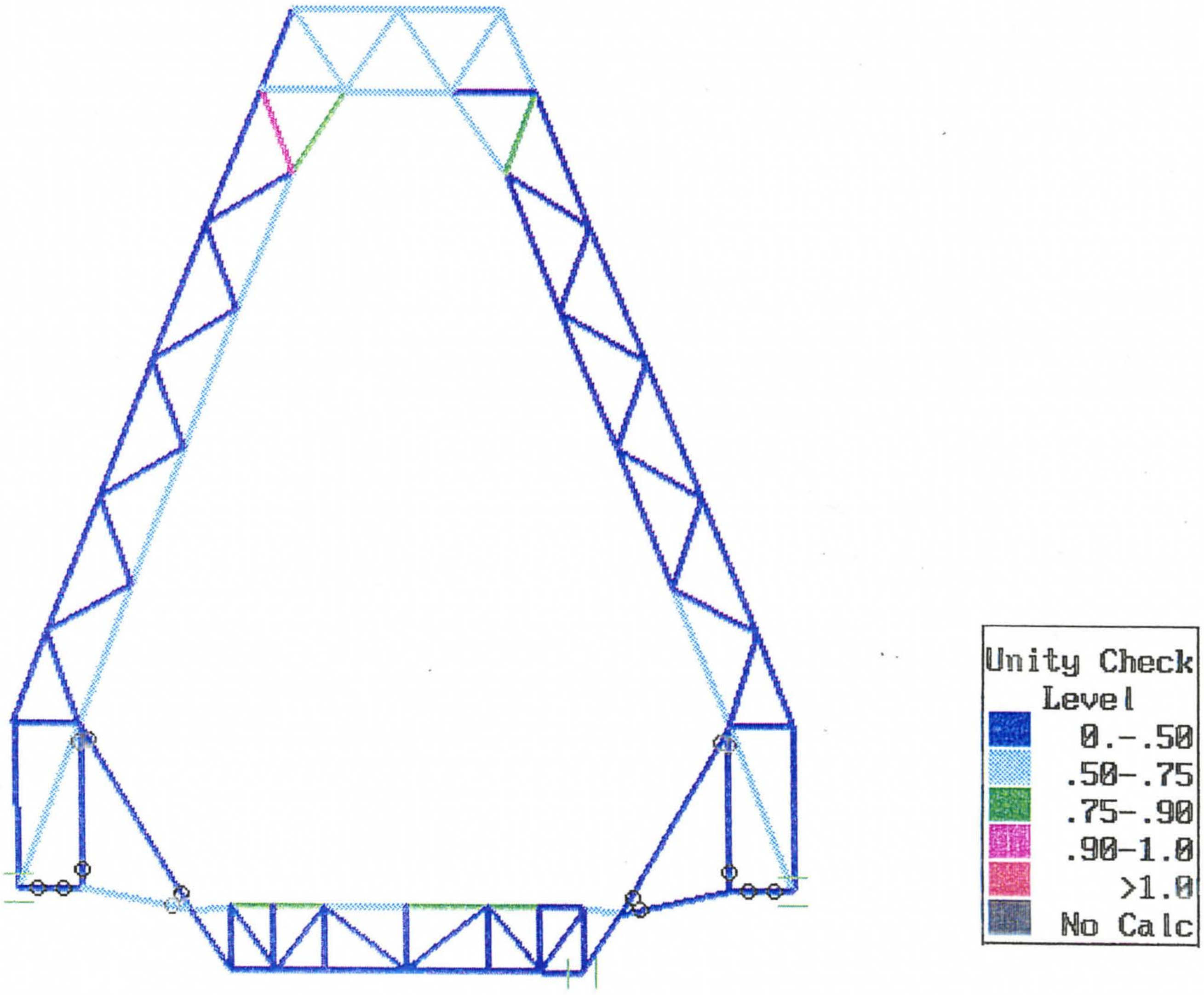


FIGURE 54: Results of a two-dimensional structural design check of the Shear Leg of the No.2 Ore Bridge as originally designed (1952).

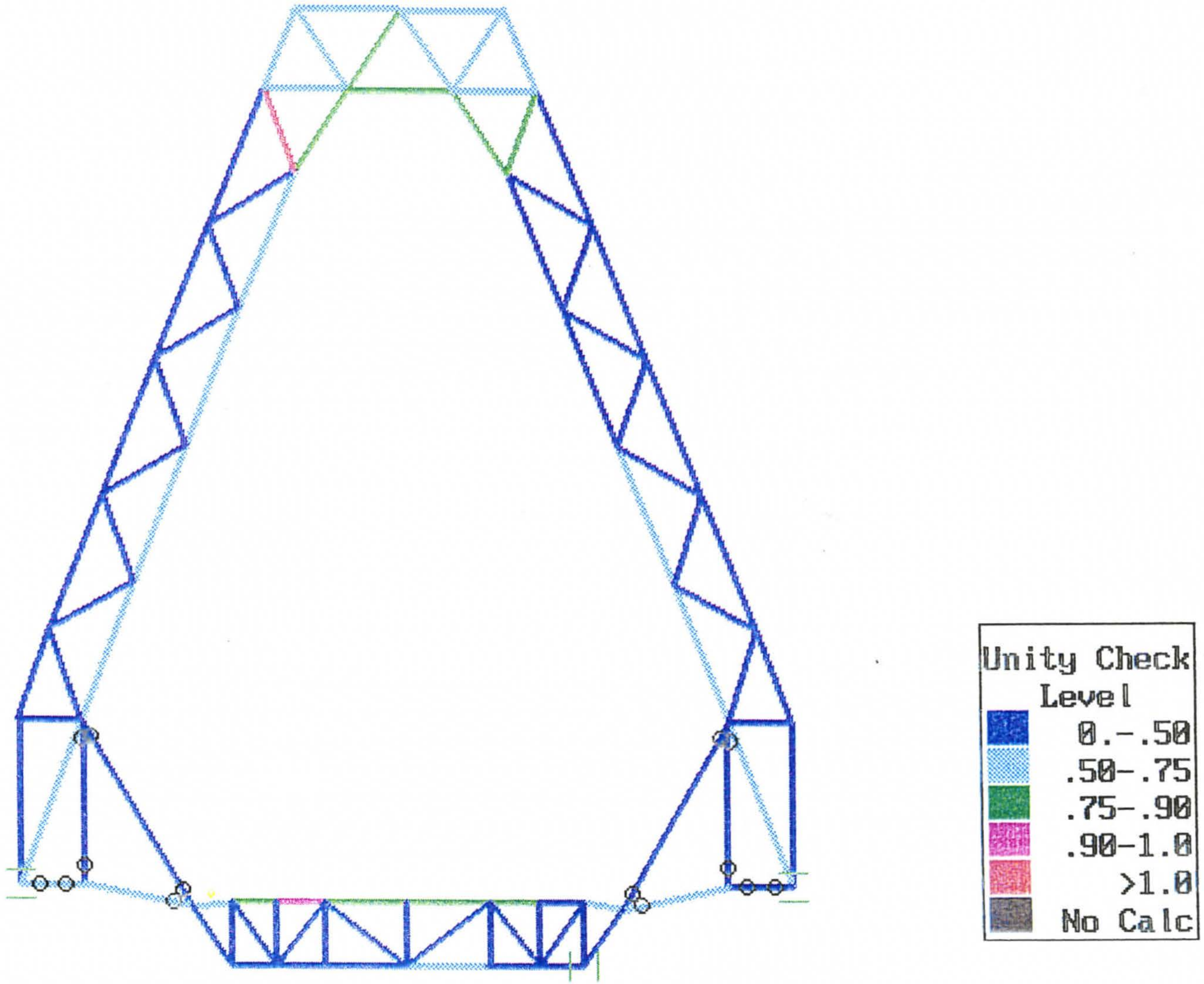


FIGURE 55: Results of a two-dimensional structural design check of the Shear Leg of the No.2 Ore Bridge with the payload and dead-weights increased (1990).

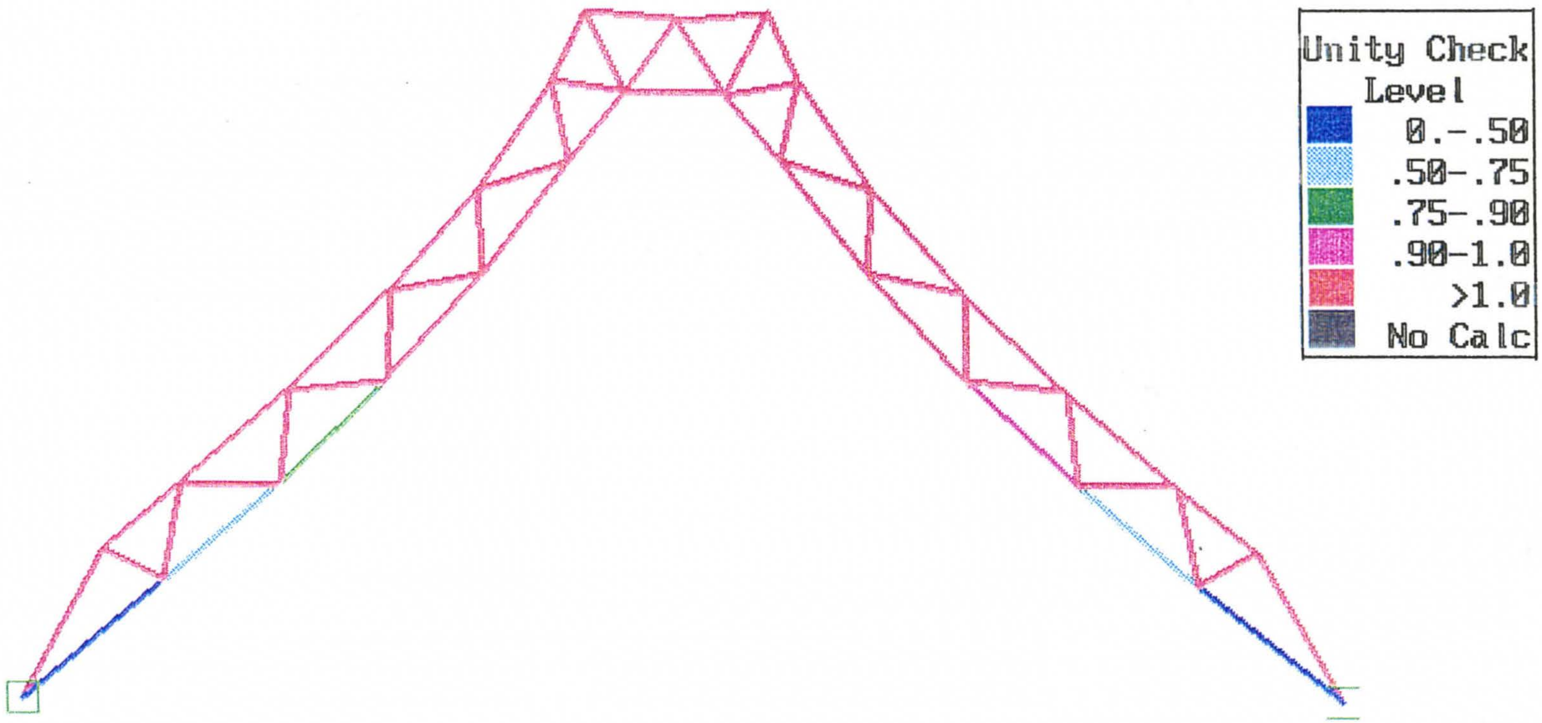


FIGURE 56: Results of a two-dimensional structural design check of the Shear Leg of the No.2 Ore Bridge with the Tension Tie removed.

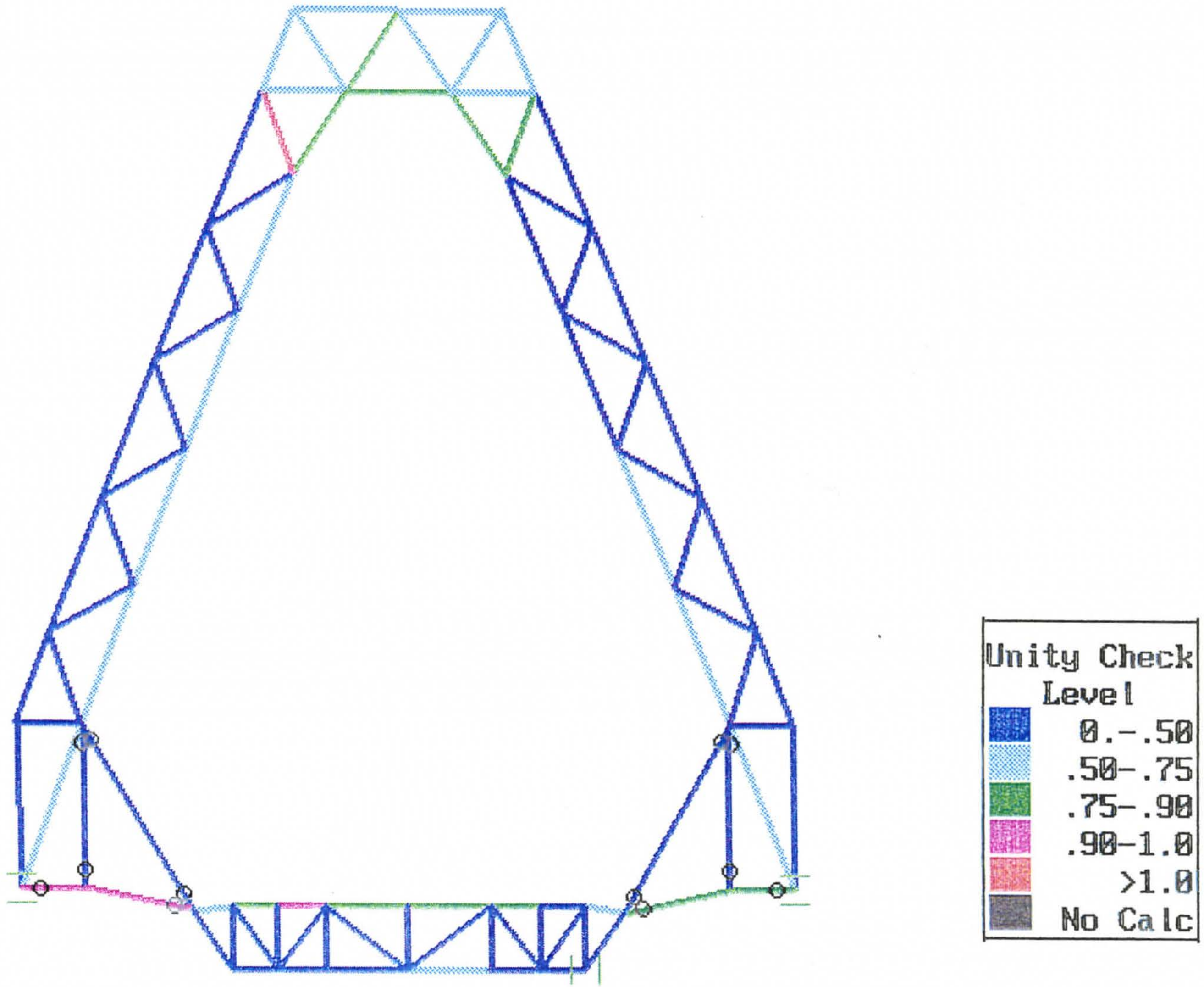


FIGURE 57: **Results of a two-dimensional structural design check of the Shear Leg of the No.2 Ore Bridge with a continuous Tension Tie (1990).**

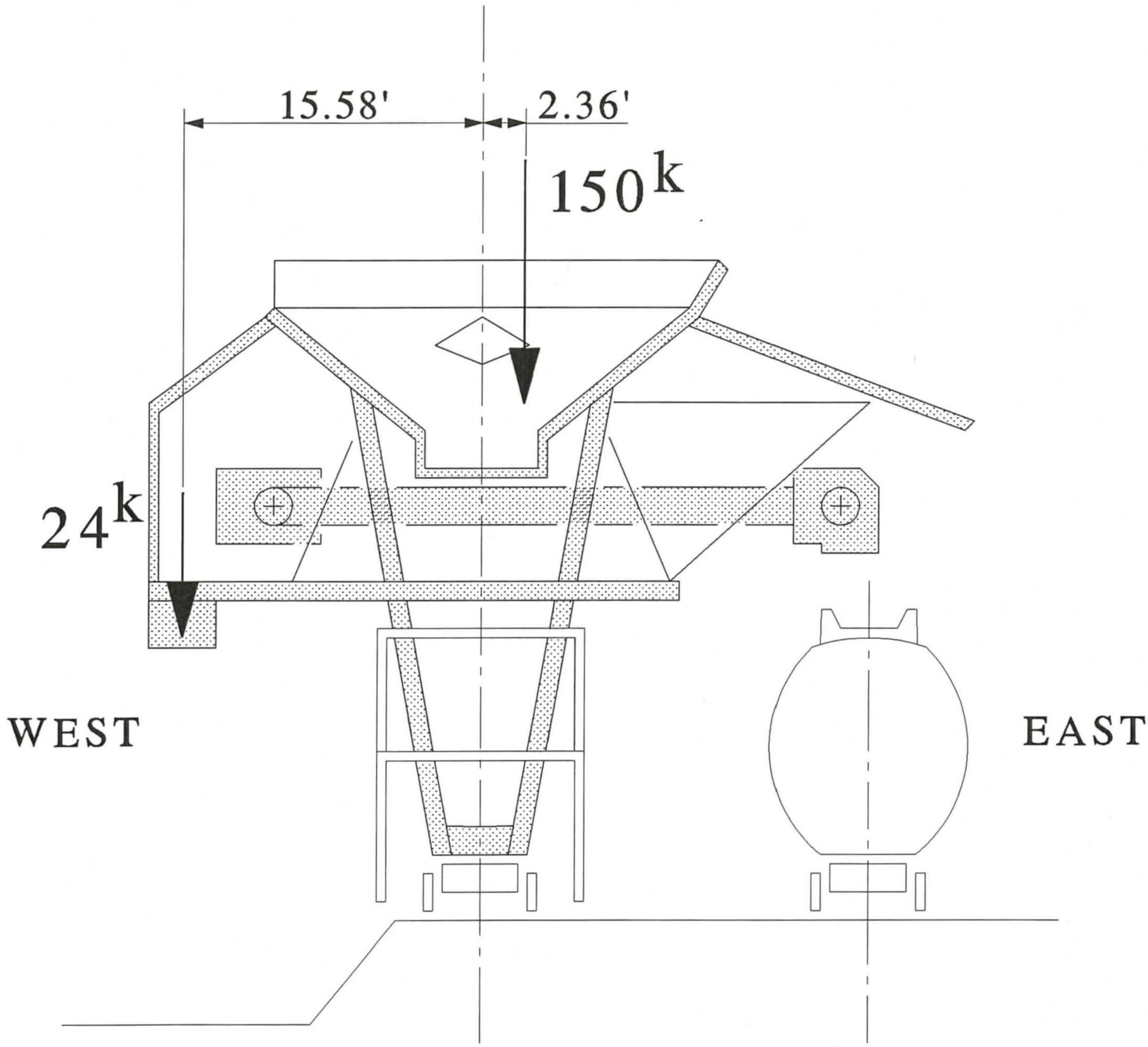
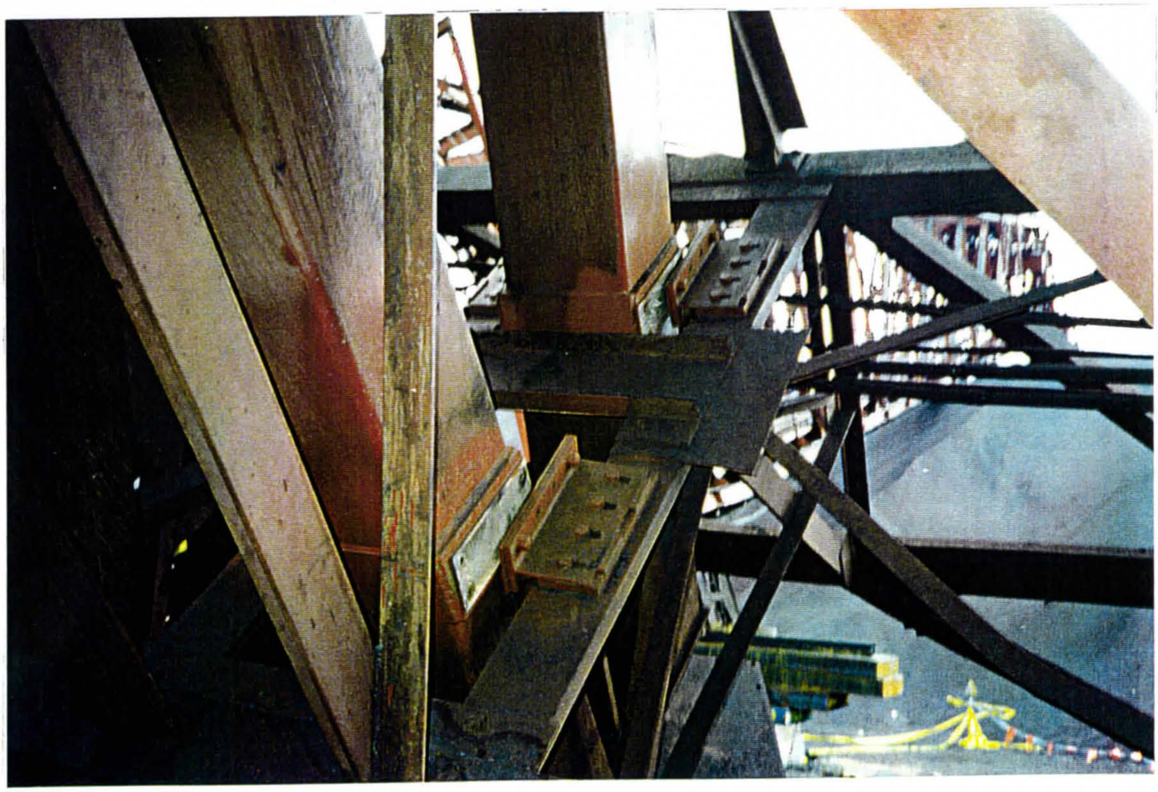


FIGURE 58: Hopper dead loads including the counterweight.

FIGURE 59: **View of the south hopper truck within the sill truss.**

FIGURE 60: **View of the guide arrangement at the top of the sill truss.**



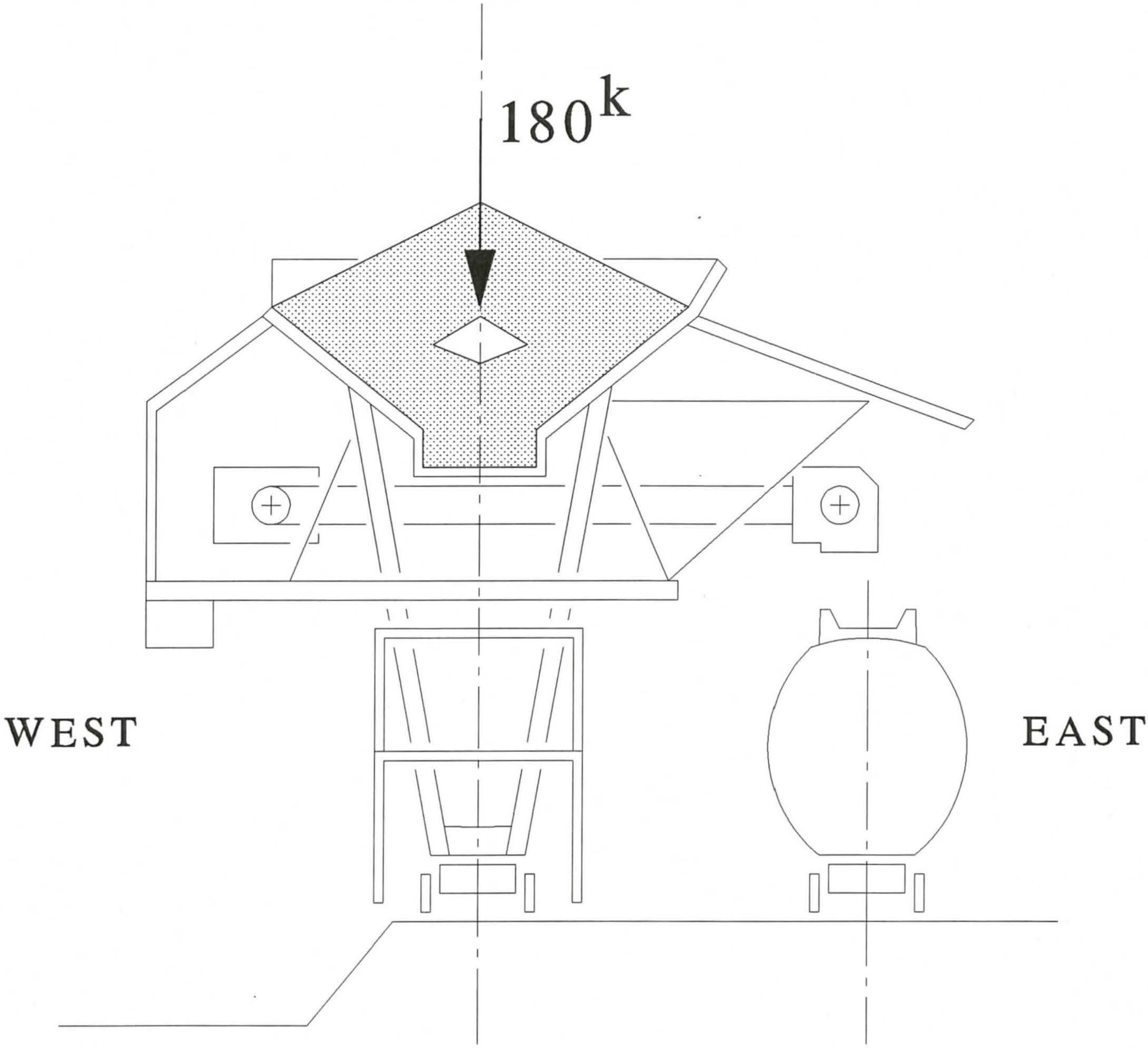


FIGURE 61: Hopper live load when completely filled.

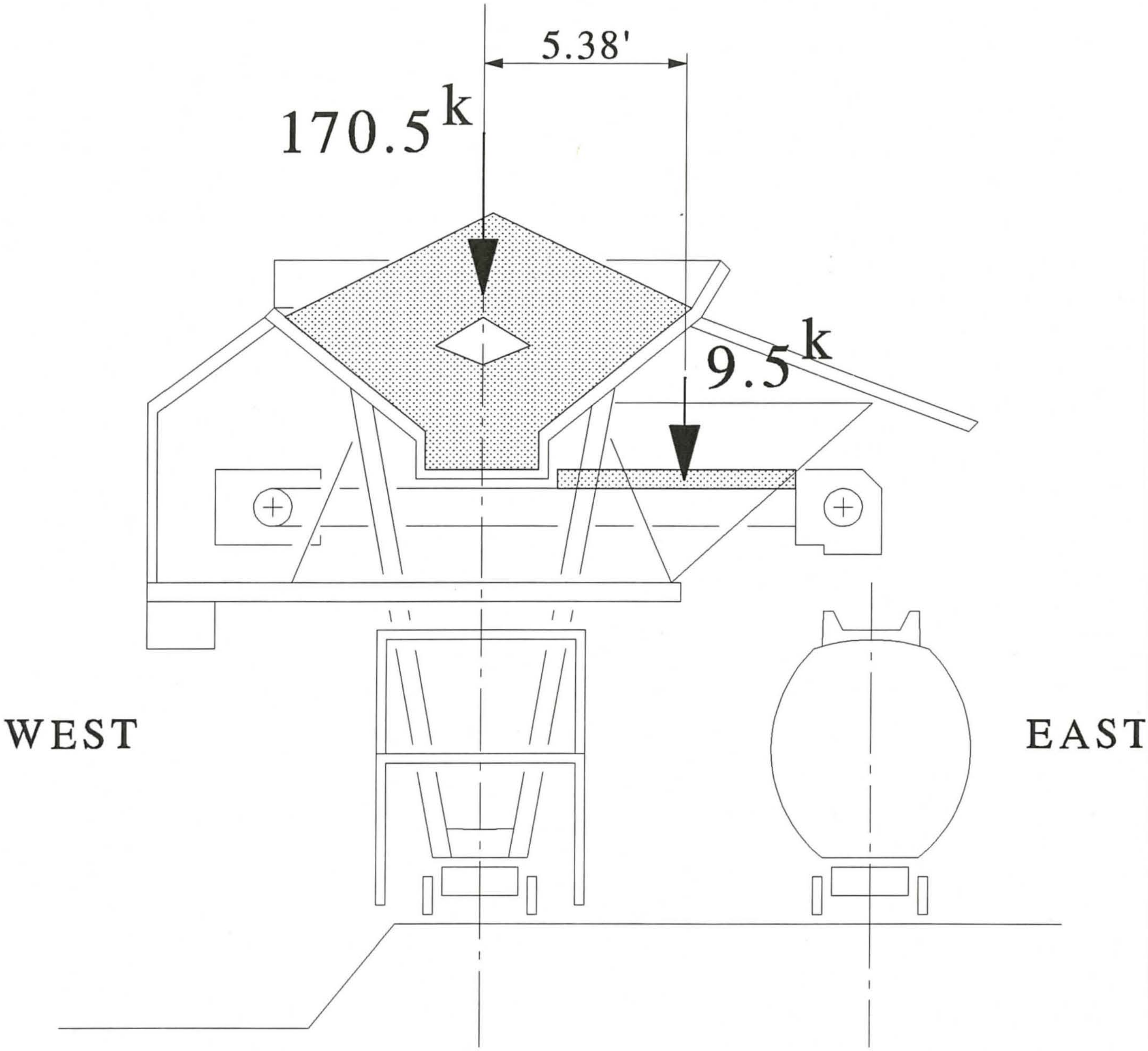


FIGURE 62: Hopper live load when hopper is just beginning to be emptied.

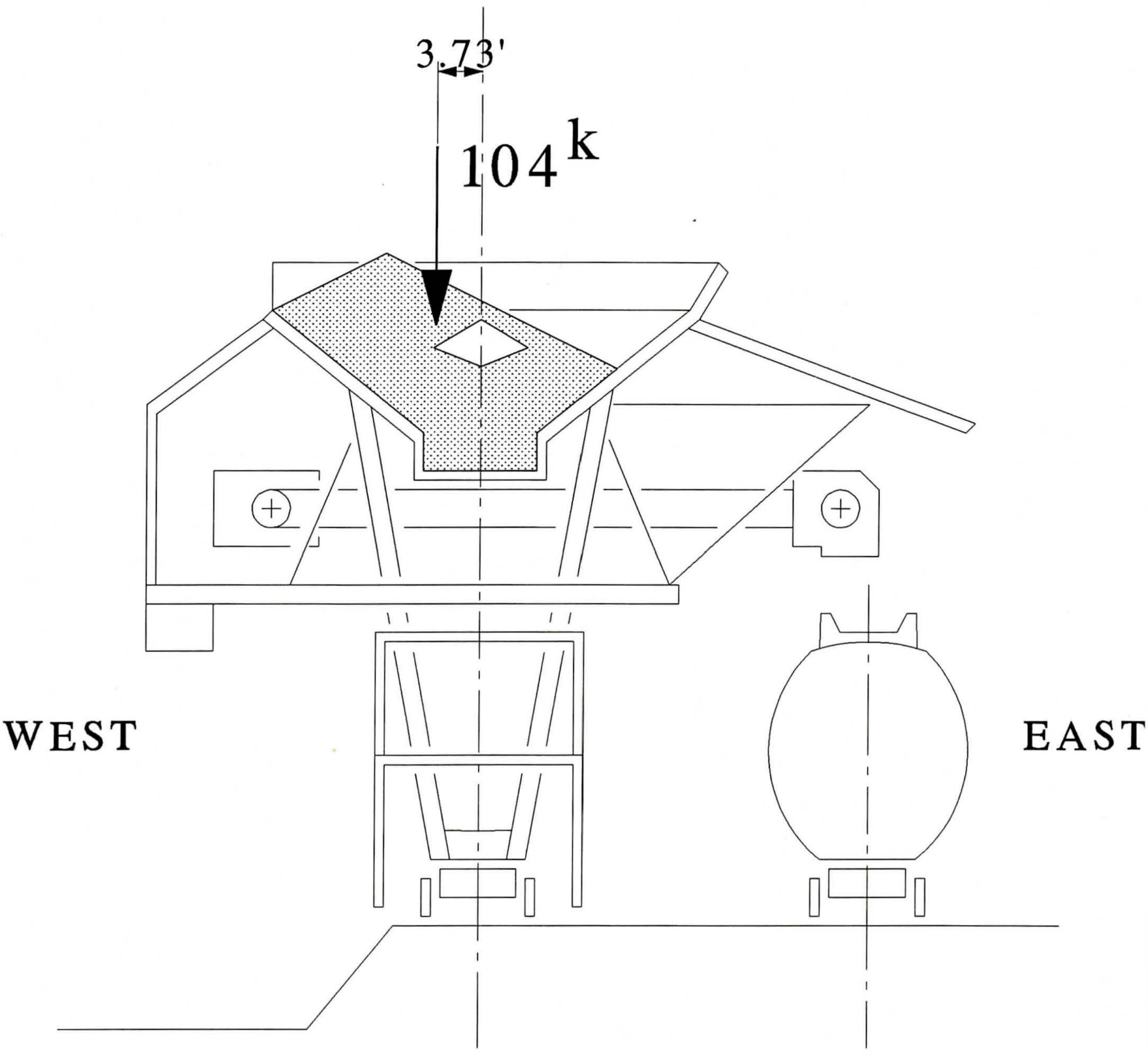


FIGURE 63: Hopper live load when it is filled off-centre.

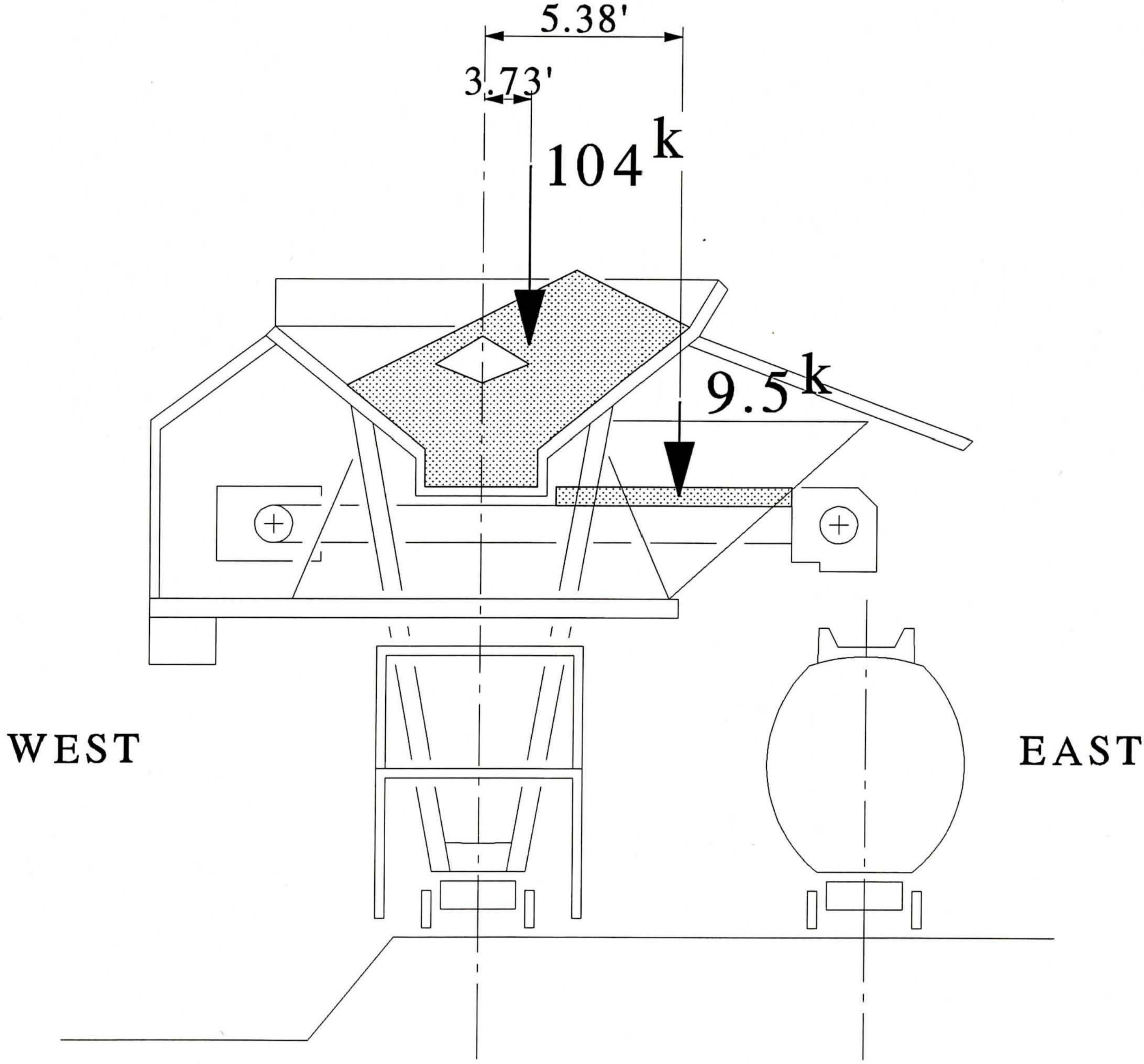


FIGURE 64: Hopper live load when it is being emptied.

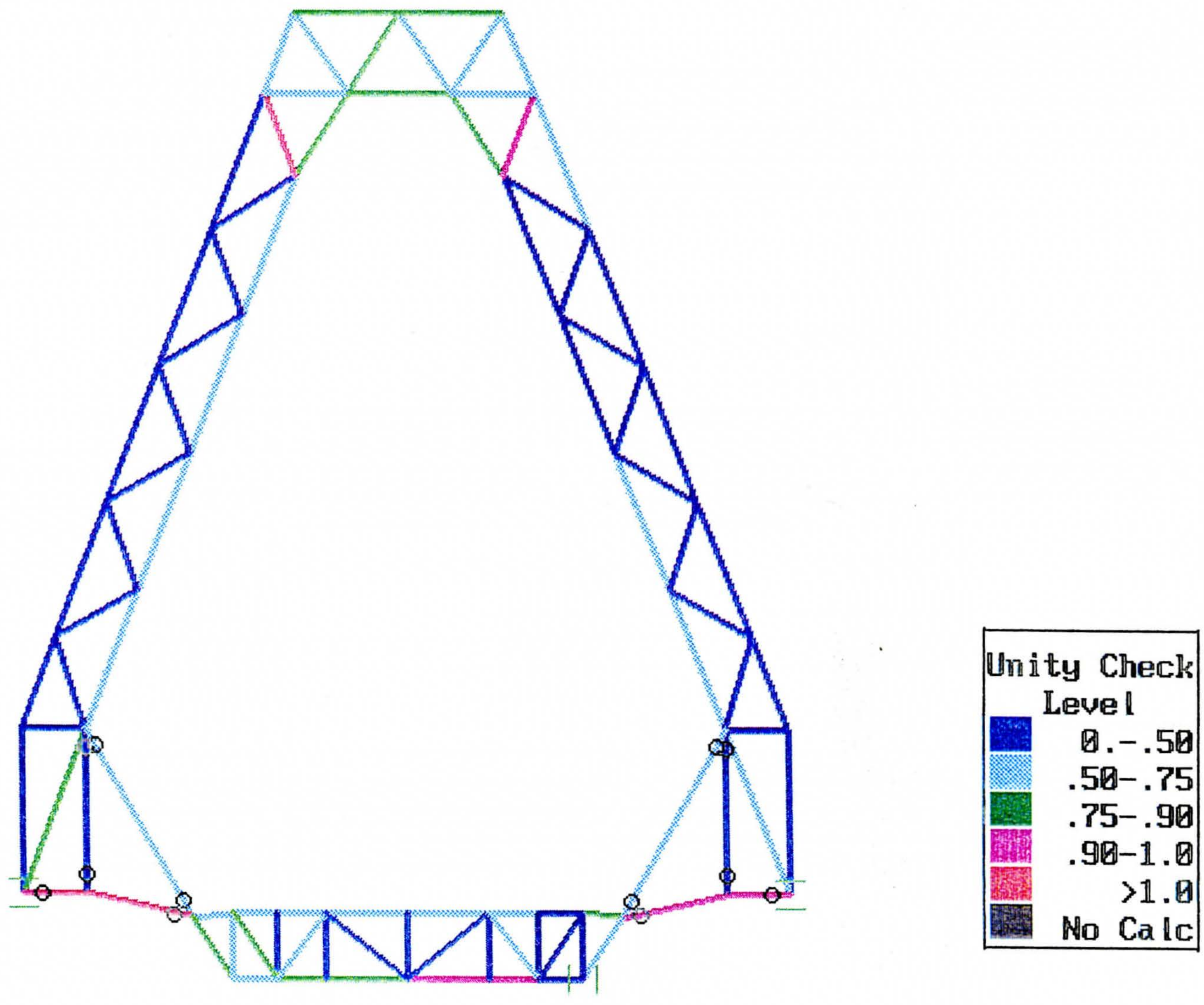


FIGURE 65: Results of a two-dimensional structural design check of the Shear Leg of the No.2 Ore Bridge with the hopper addition.

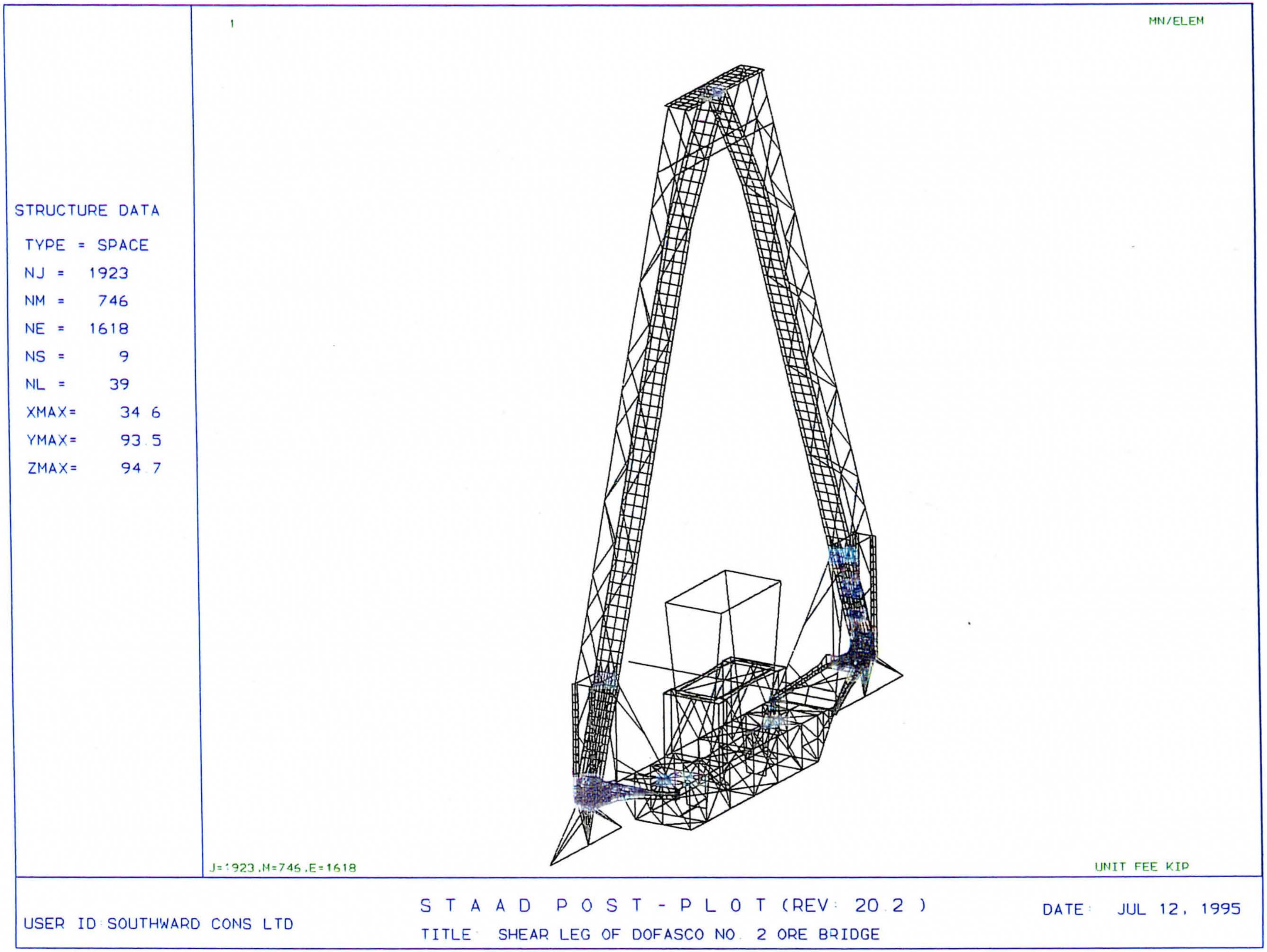
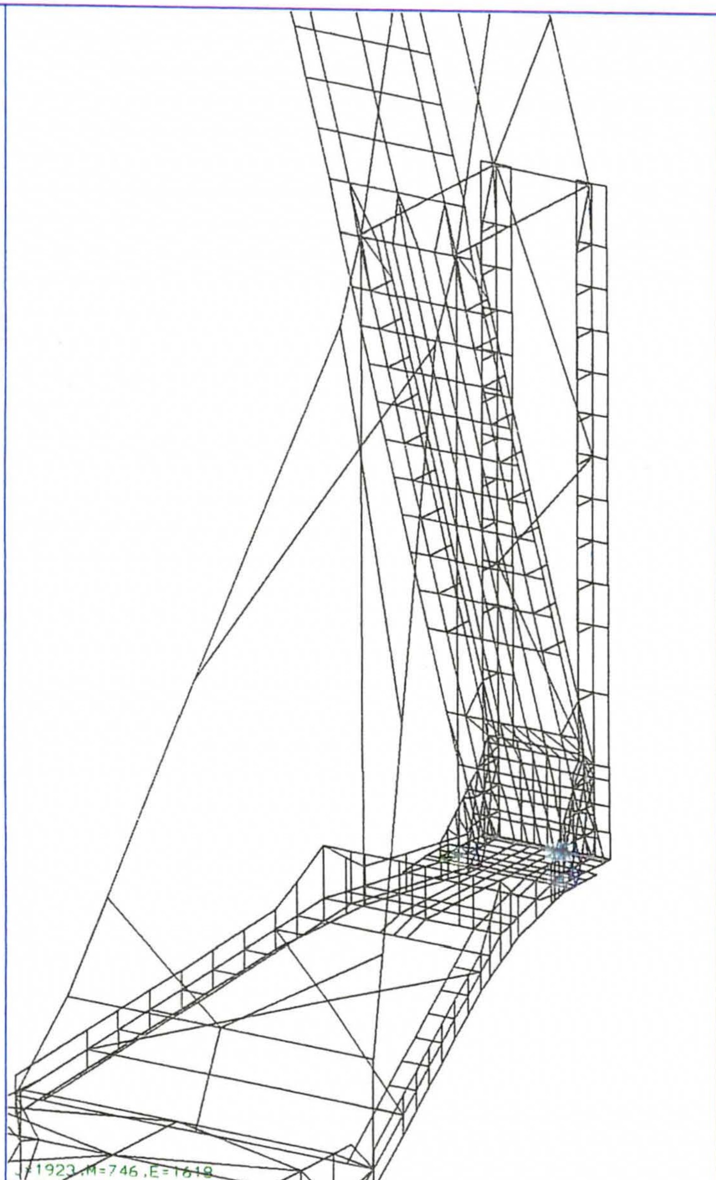
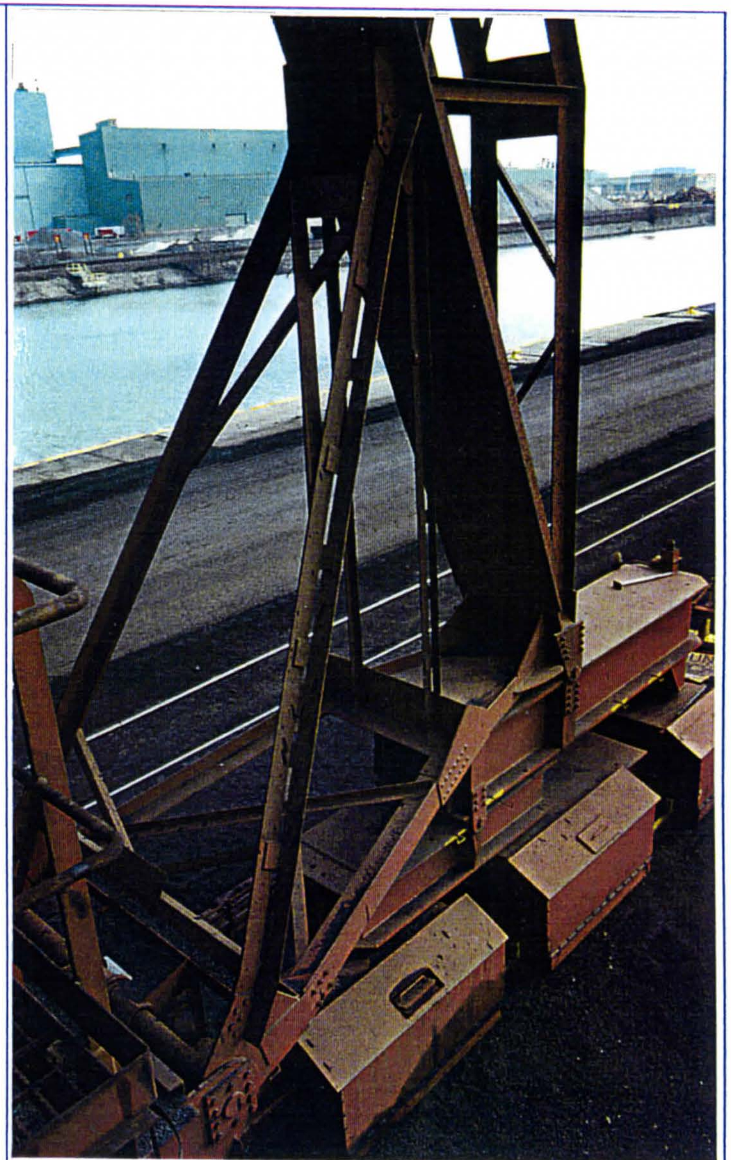


FIGURE 66a: Three-dimensional computer model of Shear Leg.

STRUCTURE DATA

TYPE = SPACE
NJ = 1923
NM = 746
NE = 1618
NS = 9
NL = 39
XMAX= 34.6
YMAX= 93.5
ZMAX= 94.7





UNIT FEE KIP

USER ID SOUTHWARD CONS LTD

S T A A D P O S T - P L O T (REV: 20.2)

TITLE: SHEAR LEG OF DOFASCO NO. 2 ORE BRIDGE

DATE: JUL 11, 1995

FIGURE 66b: Closeup of finite element mesh at the base of the South Leg of the Shear Leg.

FIGURE 66c: Closeup view of the base of the South Leg of the Shear Leg.

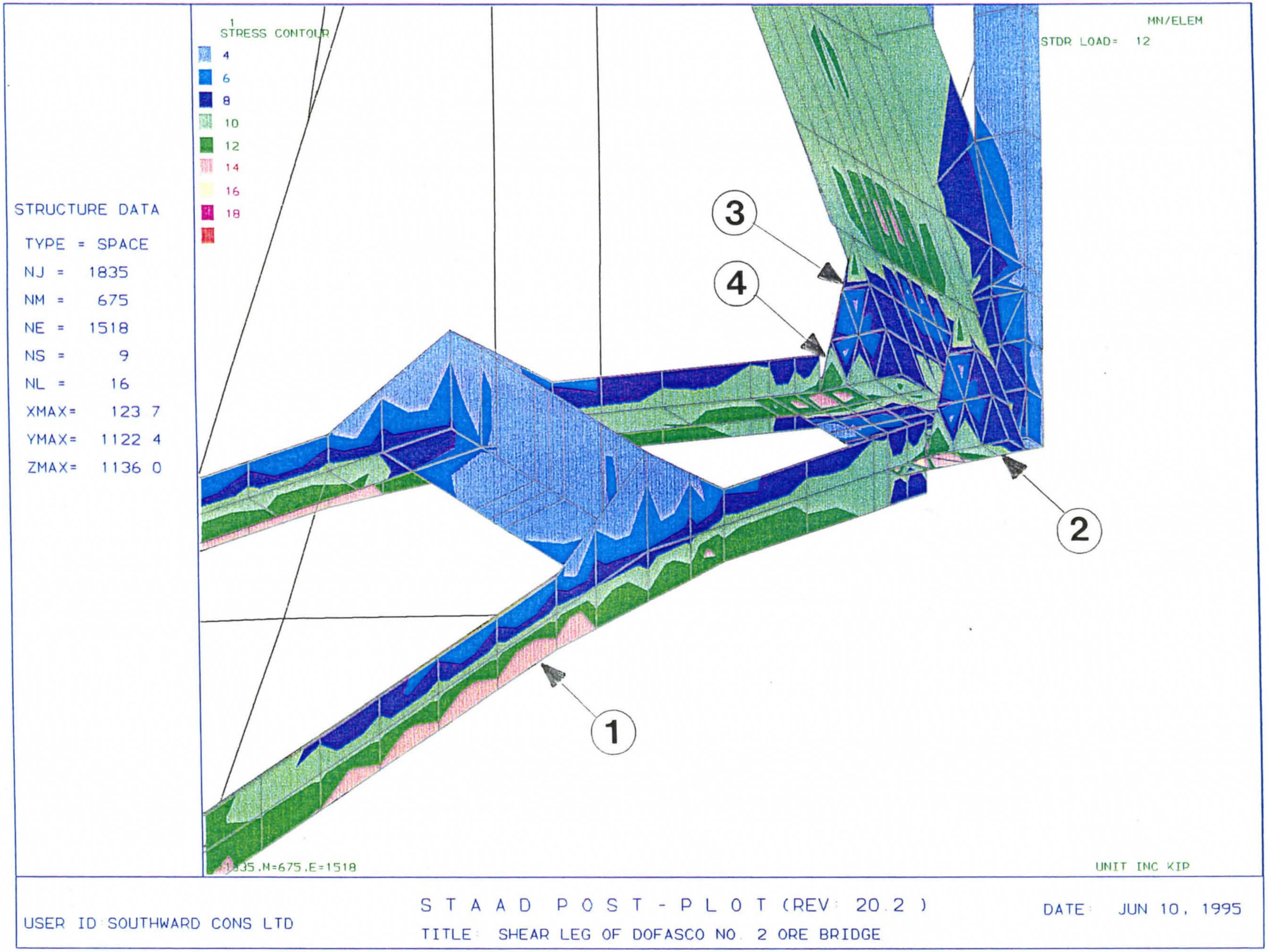


FIGURE 67: **Finite element analysis results at the base of the South Leg in its original configuration. (See text for notes)**

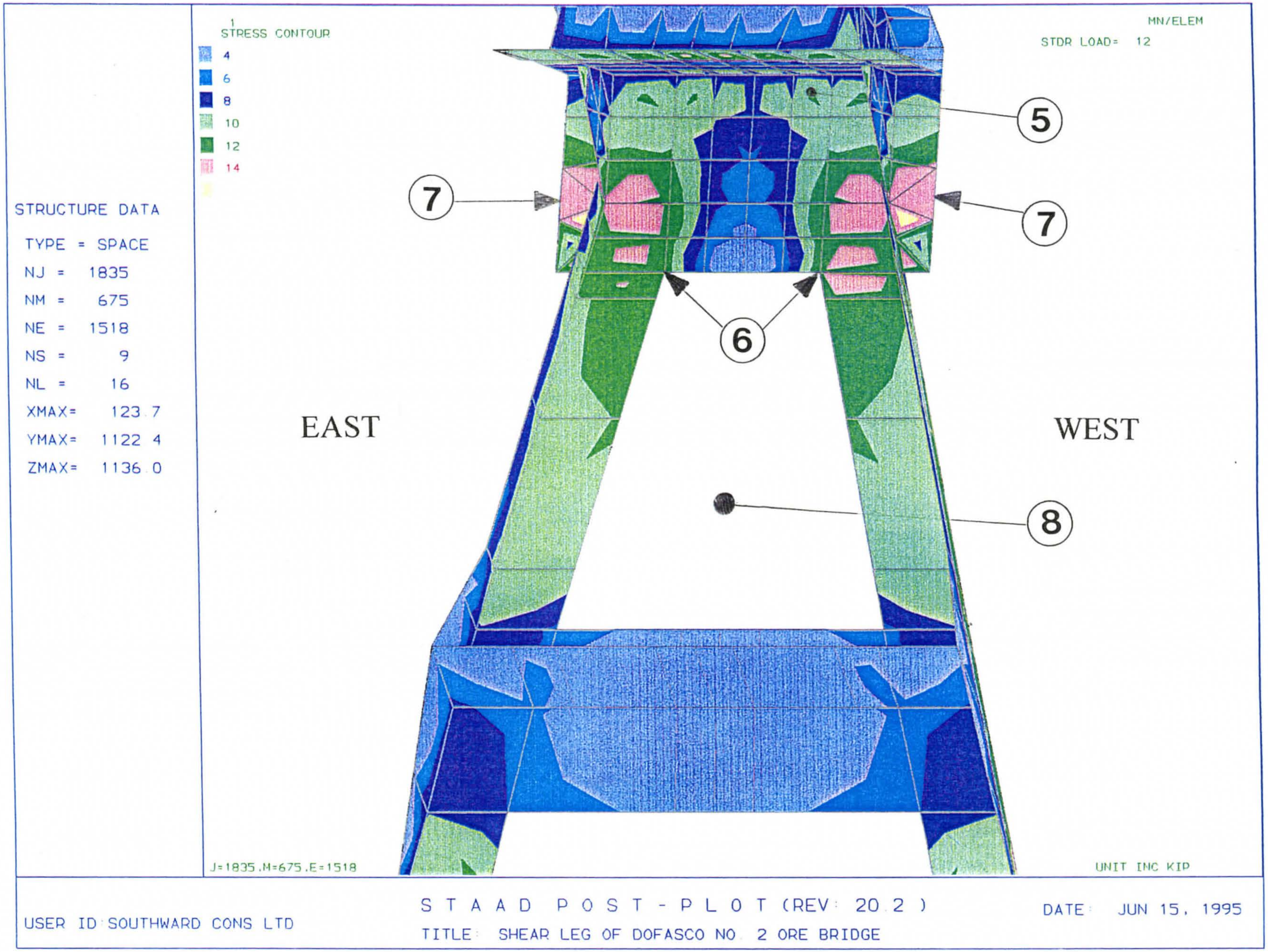


FIGURE 68: **Finite element analysis results at the base of the South Leg in its original configuration as viewed from above. (See text for notes)**

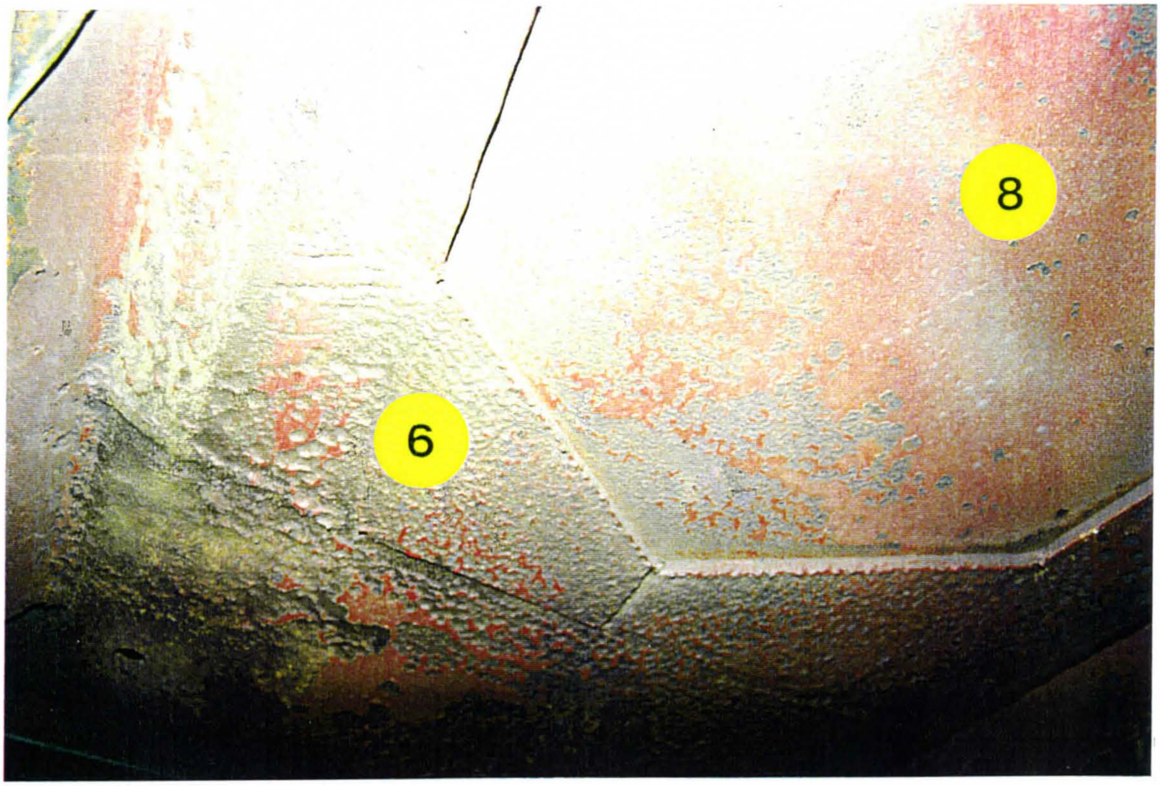


FIGURE 69: **View of underside of south tie-plate showing additional reinforcing plates. (See text for notes)**

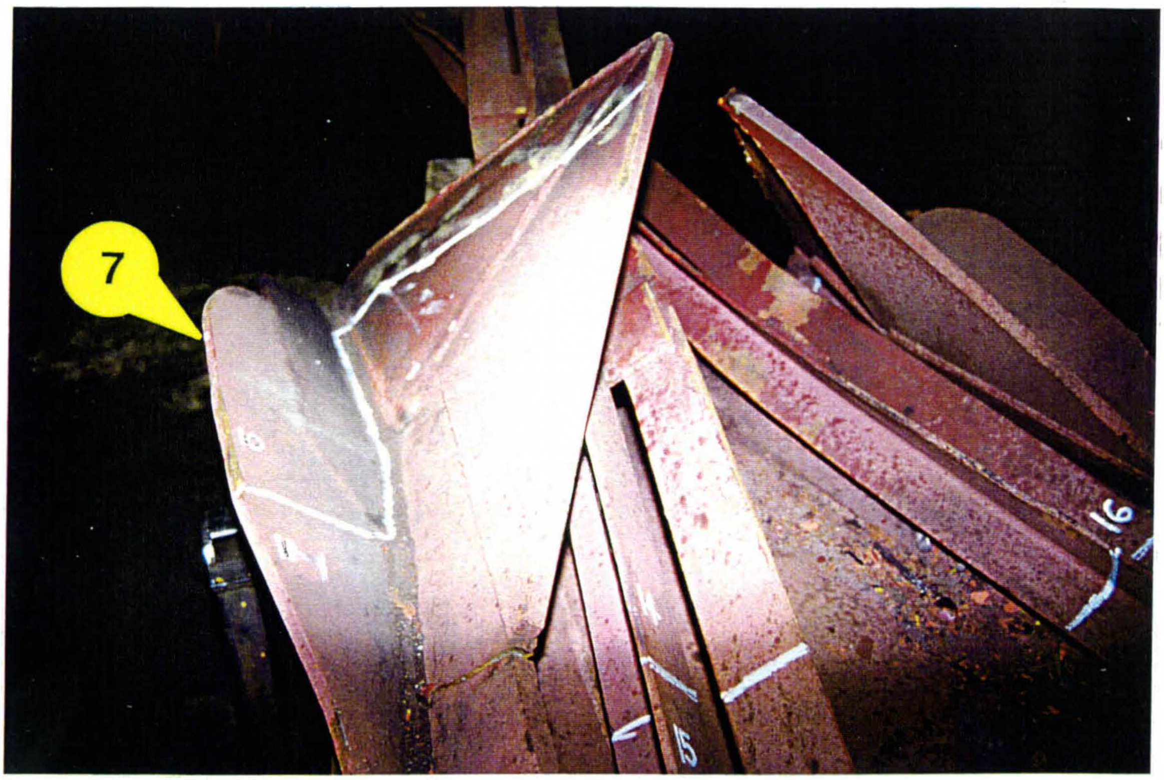


FIGURE 70: **View of additional reinforcing plates which were added to the edges of the tie-plate. (See text for note)**

FIGURE 71: View of base of South Leg of Shear Leg of the No.1 Ore Bridge. (Arrow notes the gusset extensions which were added)

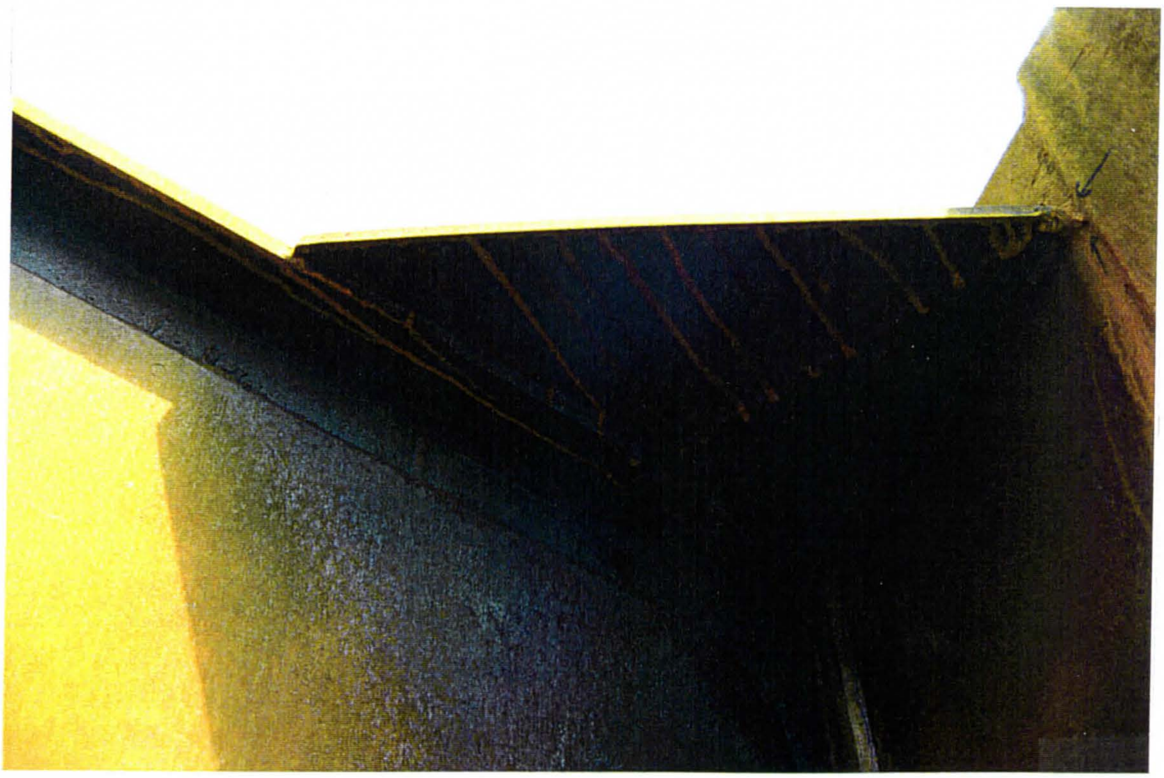


FIGURE 72: Closeup view of the gusset extension at the west side of the base of the North Leg of the Shear Leg of the No.2 Ore Bridge.

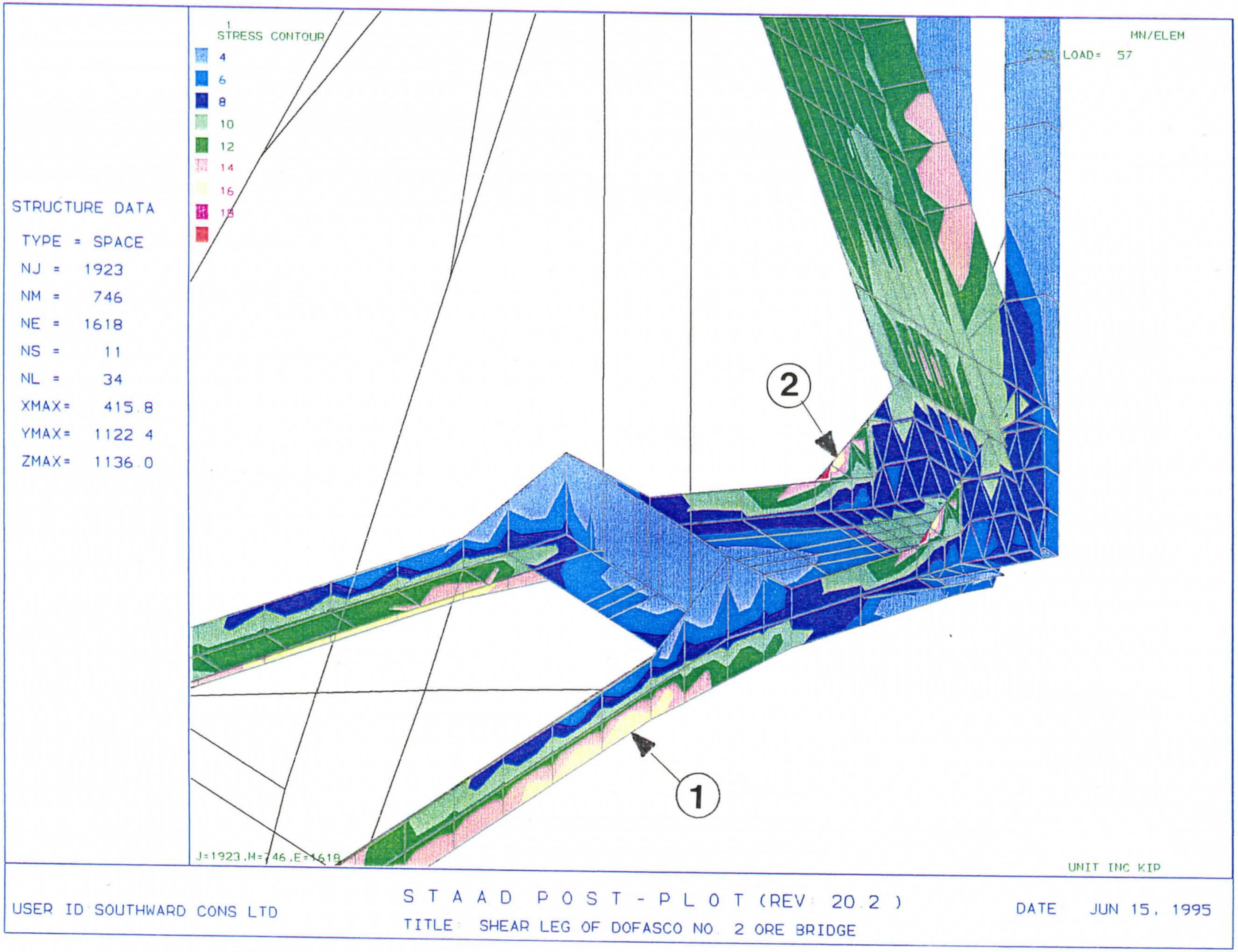
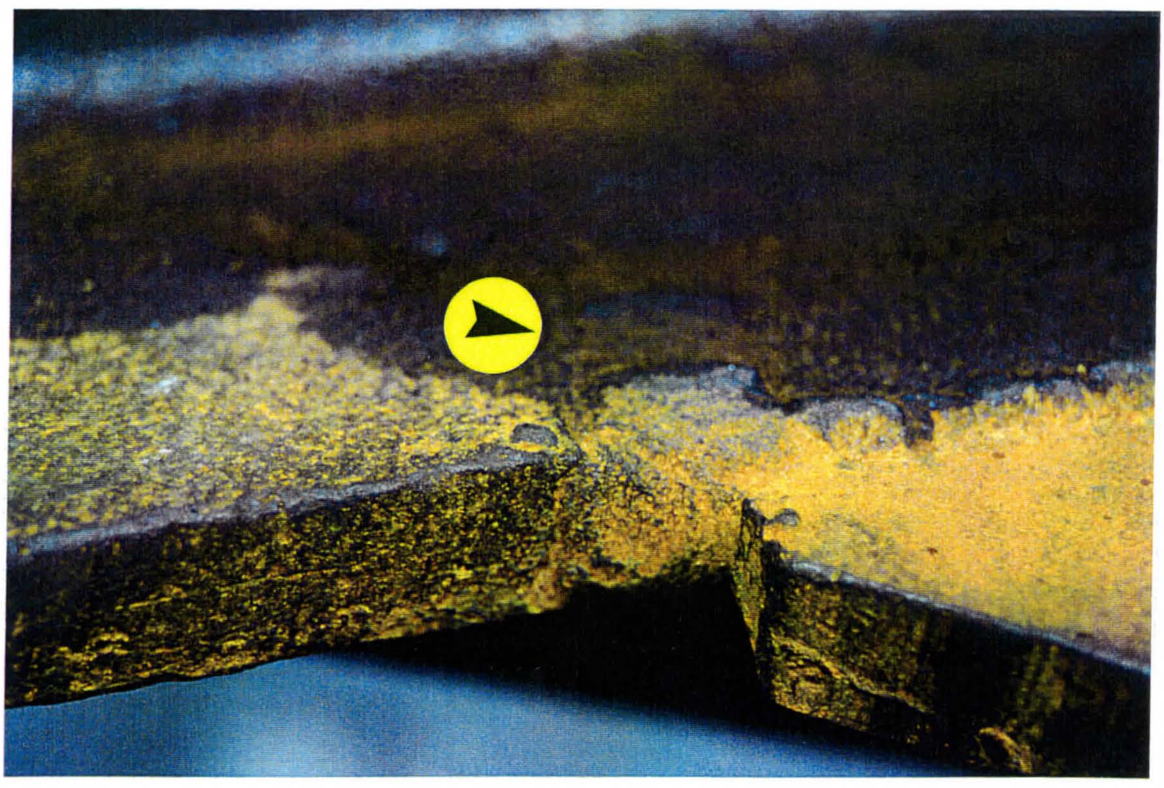


FIGURE 73: Finite element analysis results at the base of the South Leg of the Shear Leg once the additional reinforcements were added. (See text for notes)

FIGURE 74: **Closeup view of weld repairs in the tie-plate region.**

FIGURE 75: **Closeup view of a weld repair in the east tie-brace flange.**



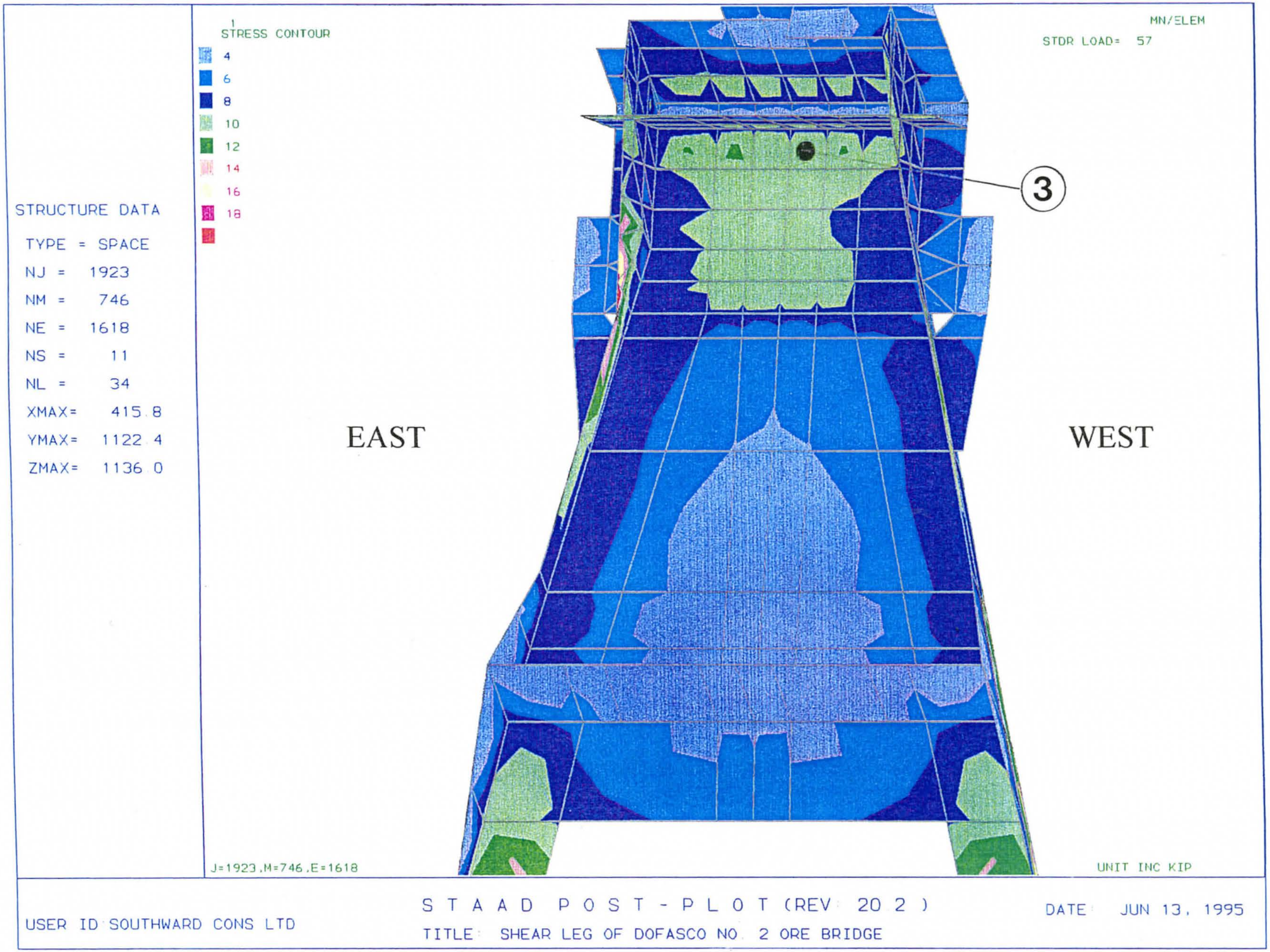


FIGURE 76: **Finite element analysis results at the base of the South Leg including the additional reinforcements under gravity loading. (See text for note)**

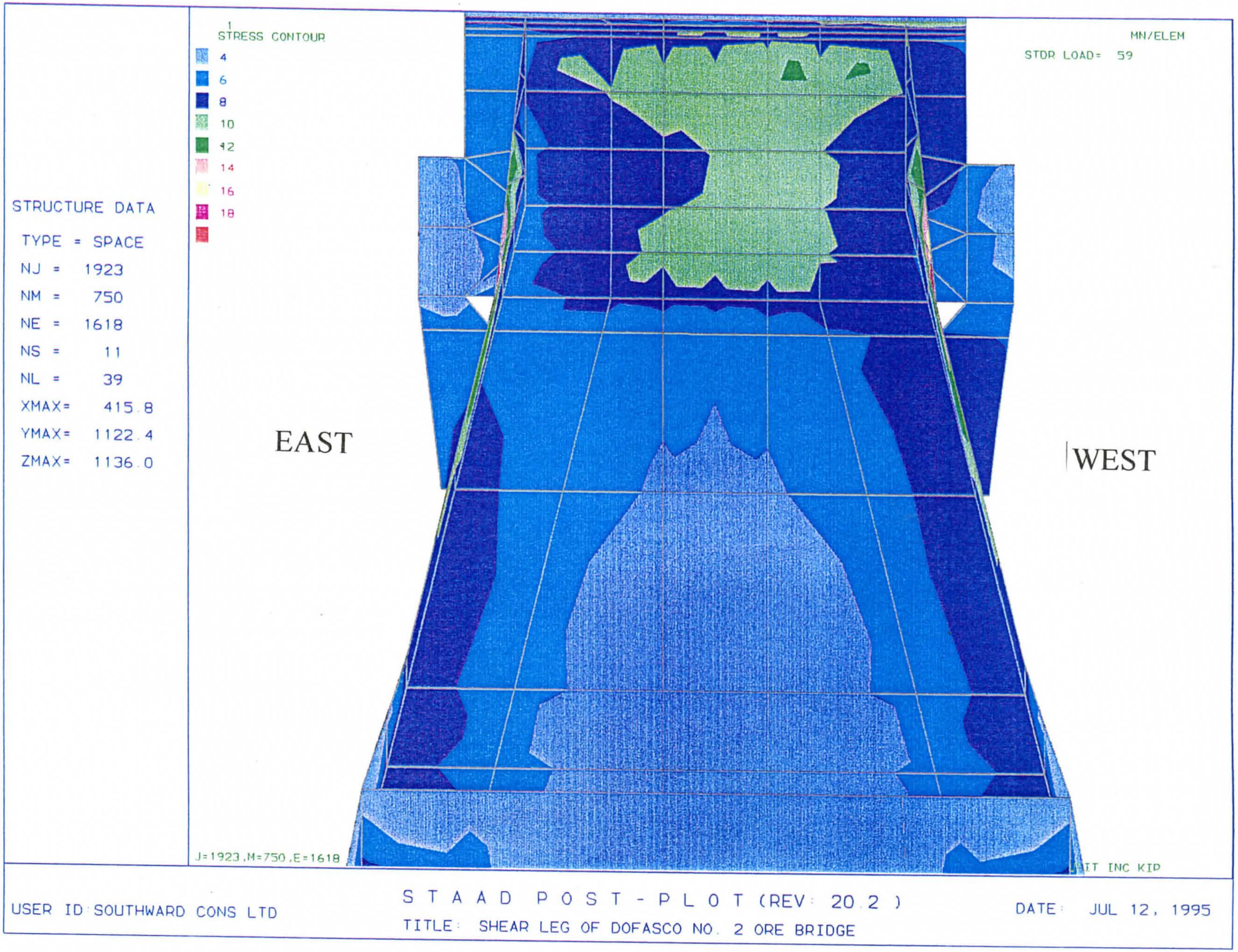


FIGURE 77: Finite element analysis results at the base of the South Leg including the effect of the unbalanced hopper loading

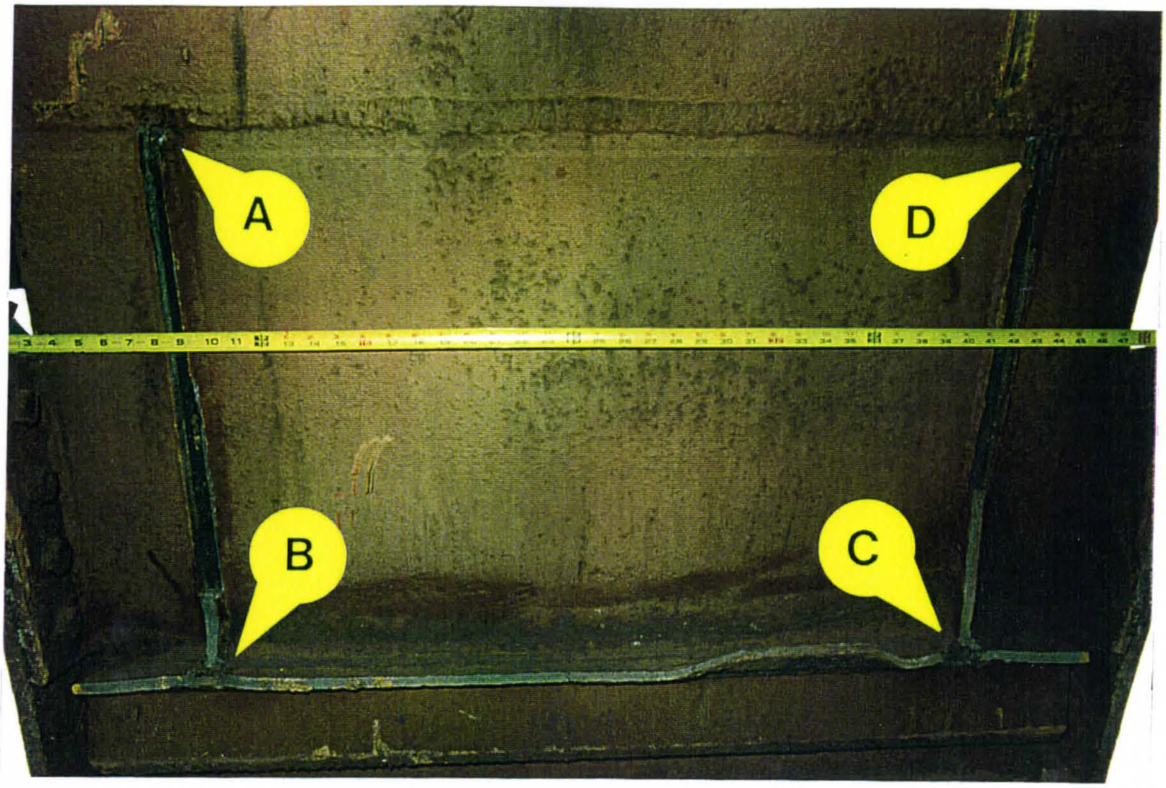


FIGURE 78: View of tie-plate and gusset fracture face at the inside face of South Leg of Shear Leg. (See text for notes)

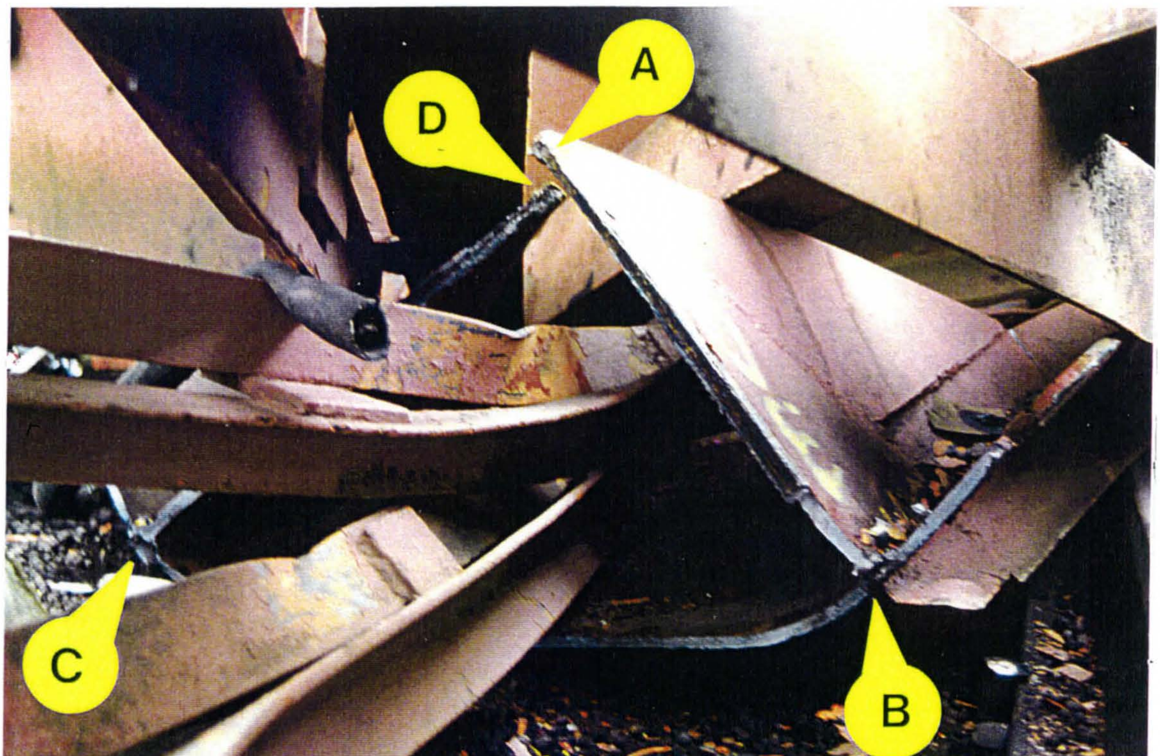


FIGURE 79: View of tie-plate and gusset fracture face. (See text for notes)

FIGURE 80: Closeup view of fracture face where the tie-plate intersects the east gusset.

FIGURE 81: Closeup view of fracture face where the tie-plate intersects the west gusset.

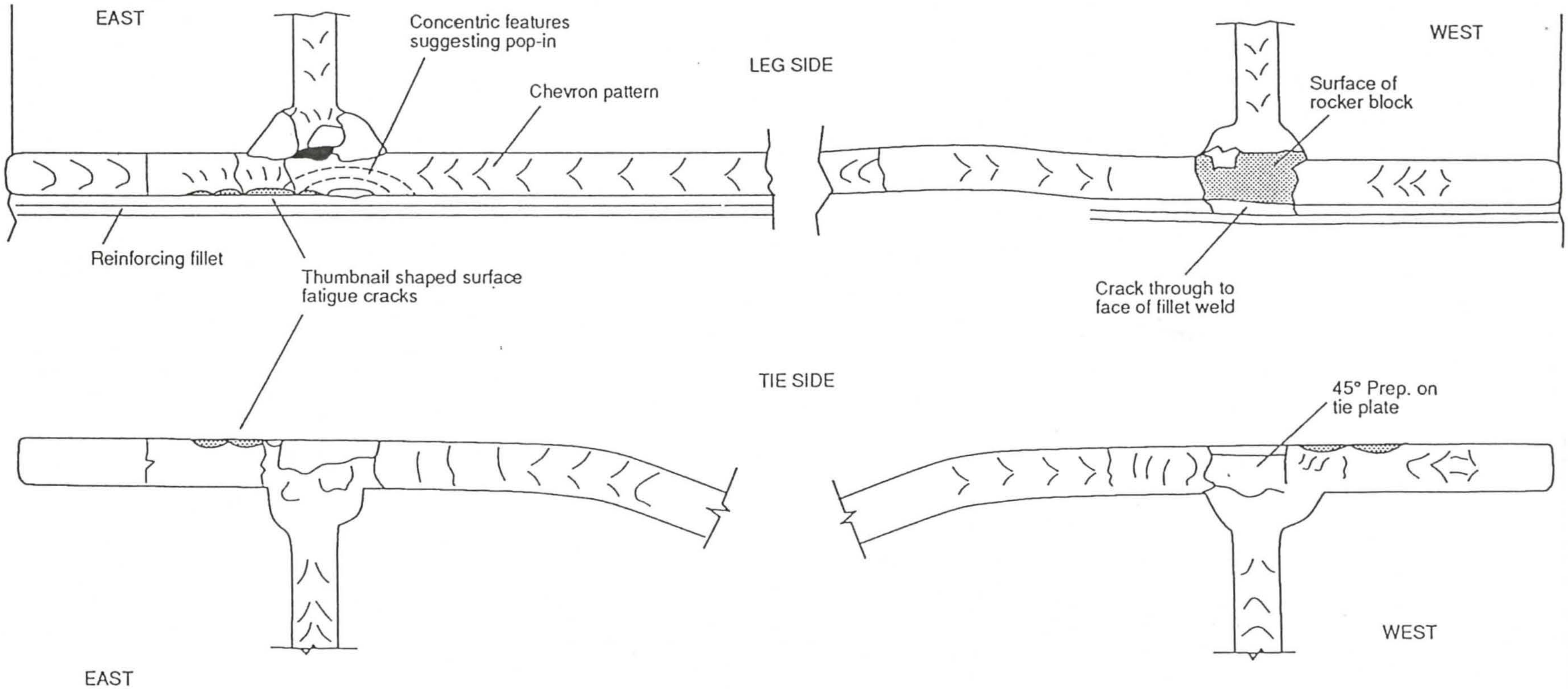


FIGURE 82: Sketch illustrating main features on the fracture faces. (Graville, 1995)

FIGURE 83: Closeup view of fracture face where the tie-plate intersects the west gusset. (Wood, 1995)

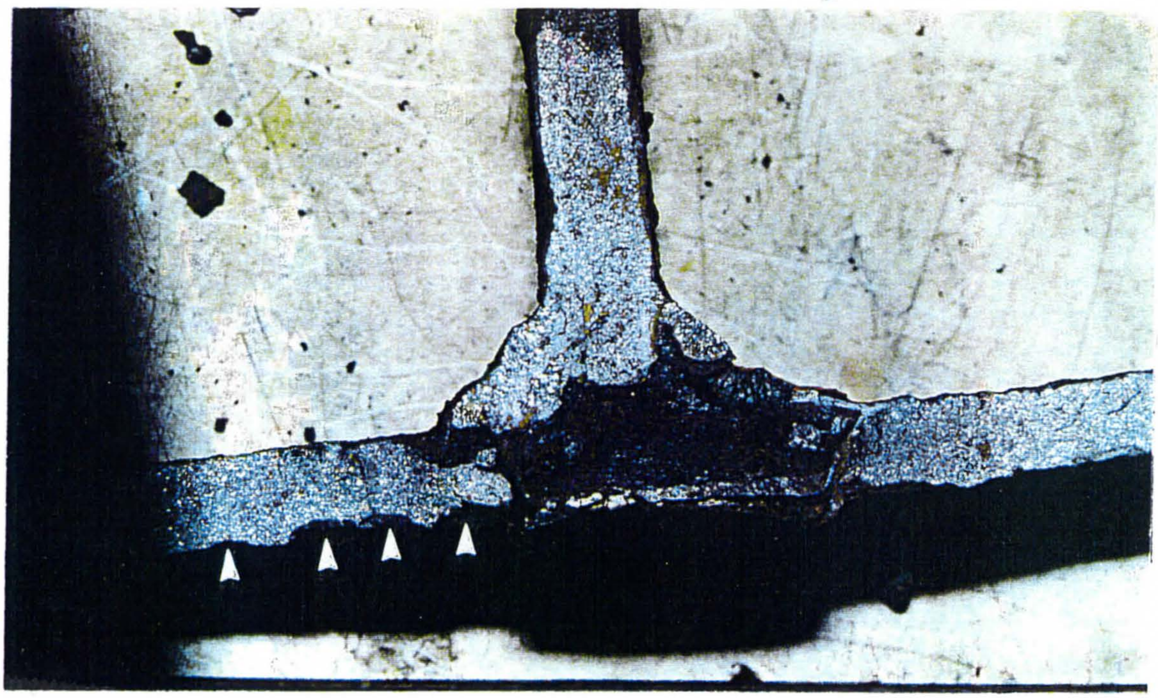


FIGURE 84: Closeup view of fracture face where the tie-plate intersects the west gusset. (Arrows note corrosion pits and fatigue cracks) (Wood, 1995)

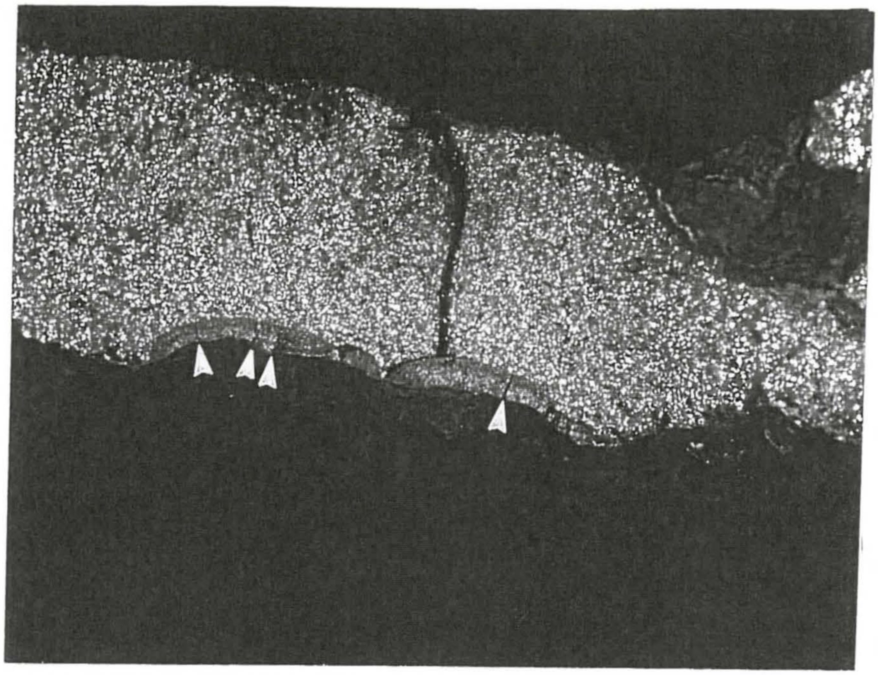


FIGURE 85: Magnified view of corrosion pits and fatigue cracks at the underside of the tie-plate. (Arrows identify ratchet marks) (Wood, 1995)

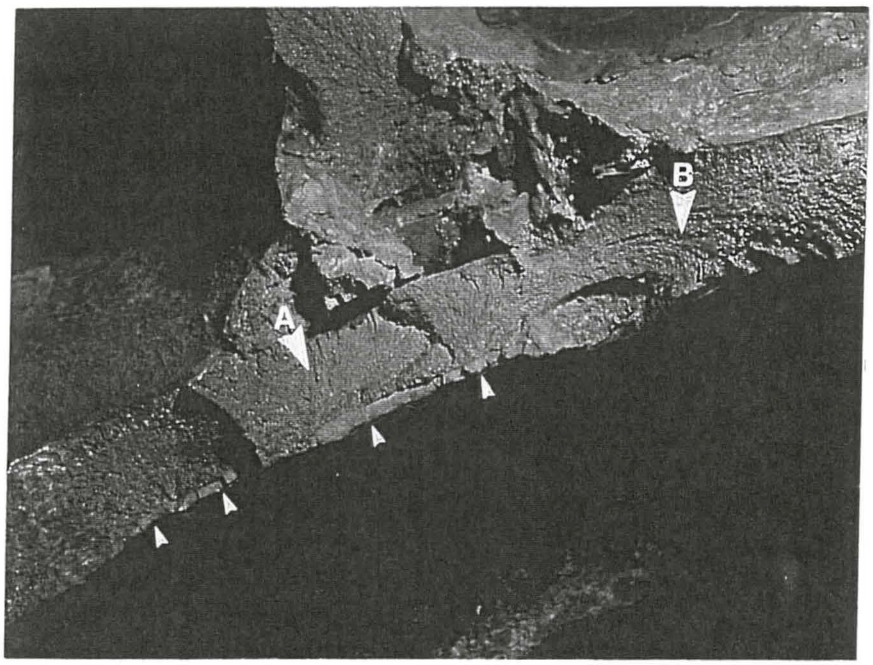


FIGURE 86: Closeup view of fracture face where the tie-plate intersects the east gusset under oblique lighting. (See text for notes) (Wood, 1995)

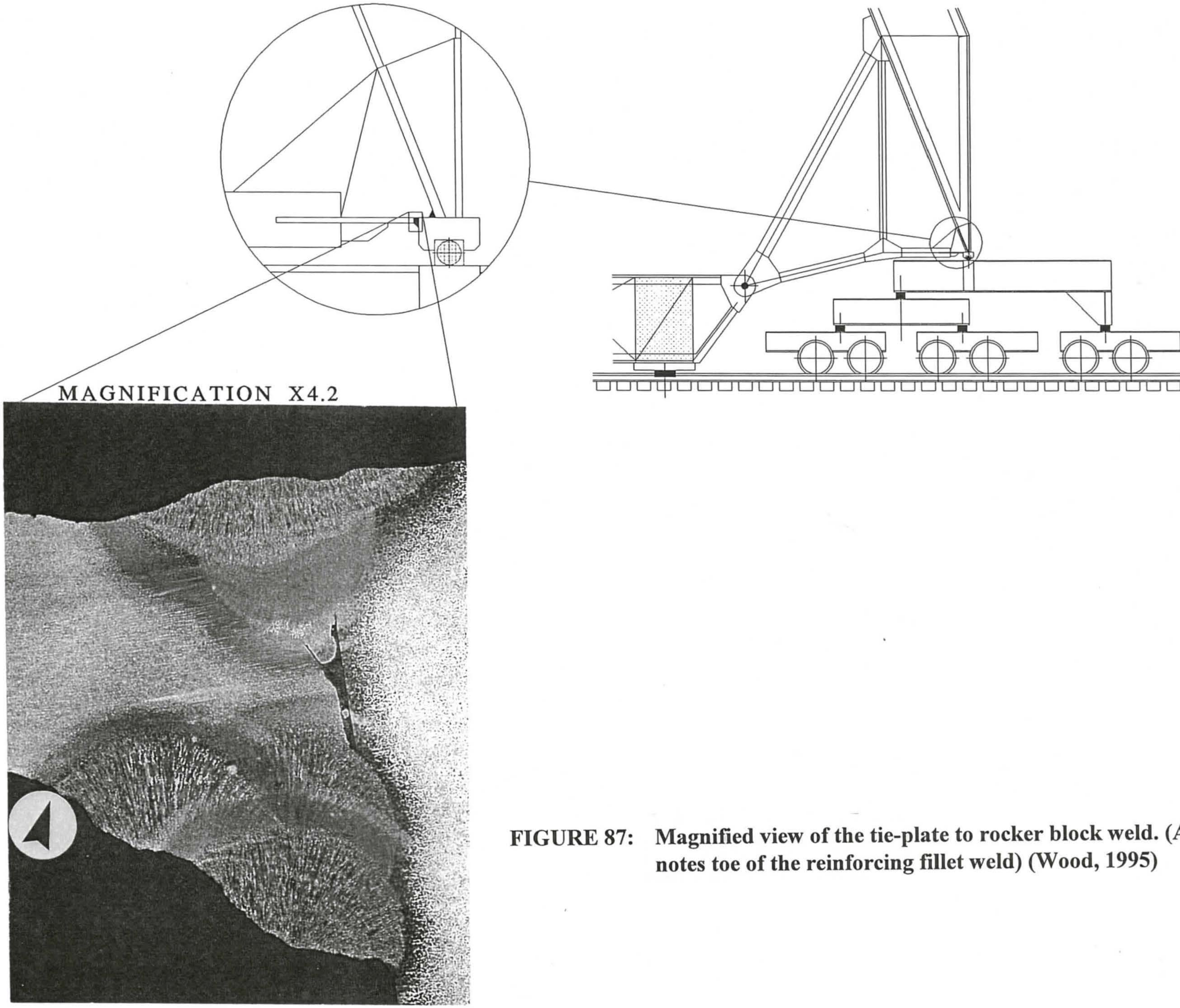


FIGURE 87: Magnified view of the tie-plate to rocker block weld. (Arrow notes toe of the reinforcing fillet weld) (Wood, 1995)

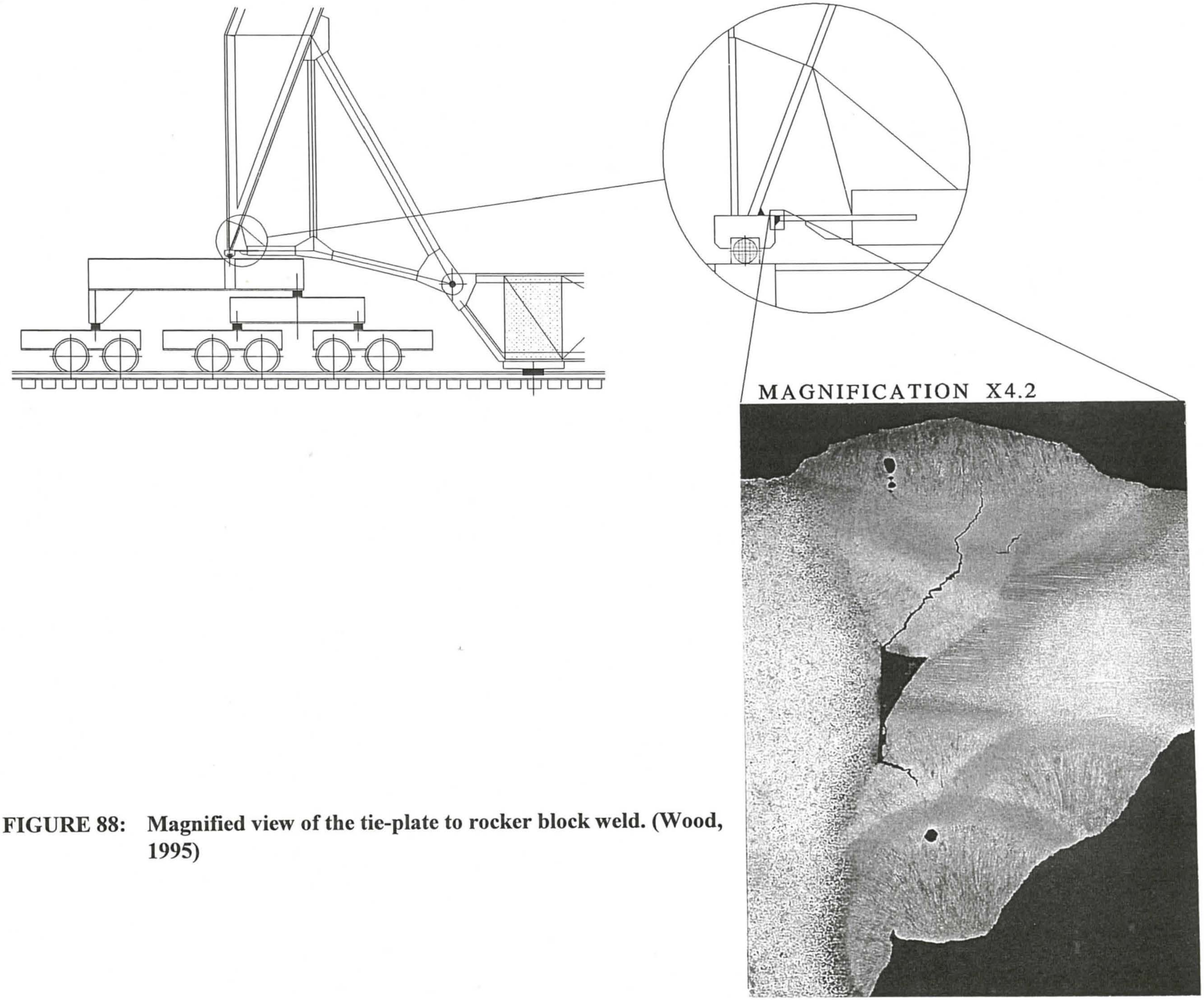


FIGURE 88: Magnified view of the tie-plate to rocker block weld. (Wood, 1995)

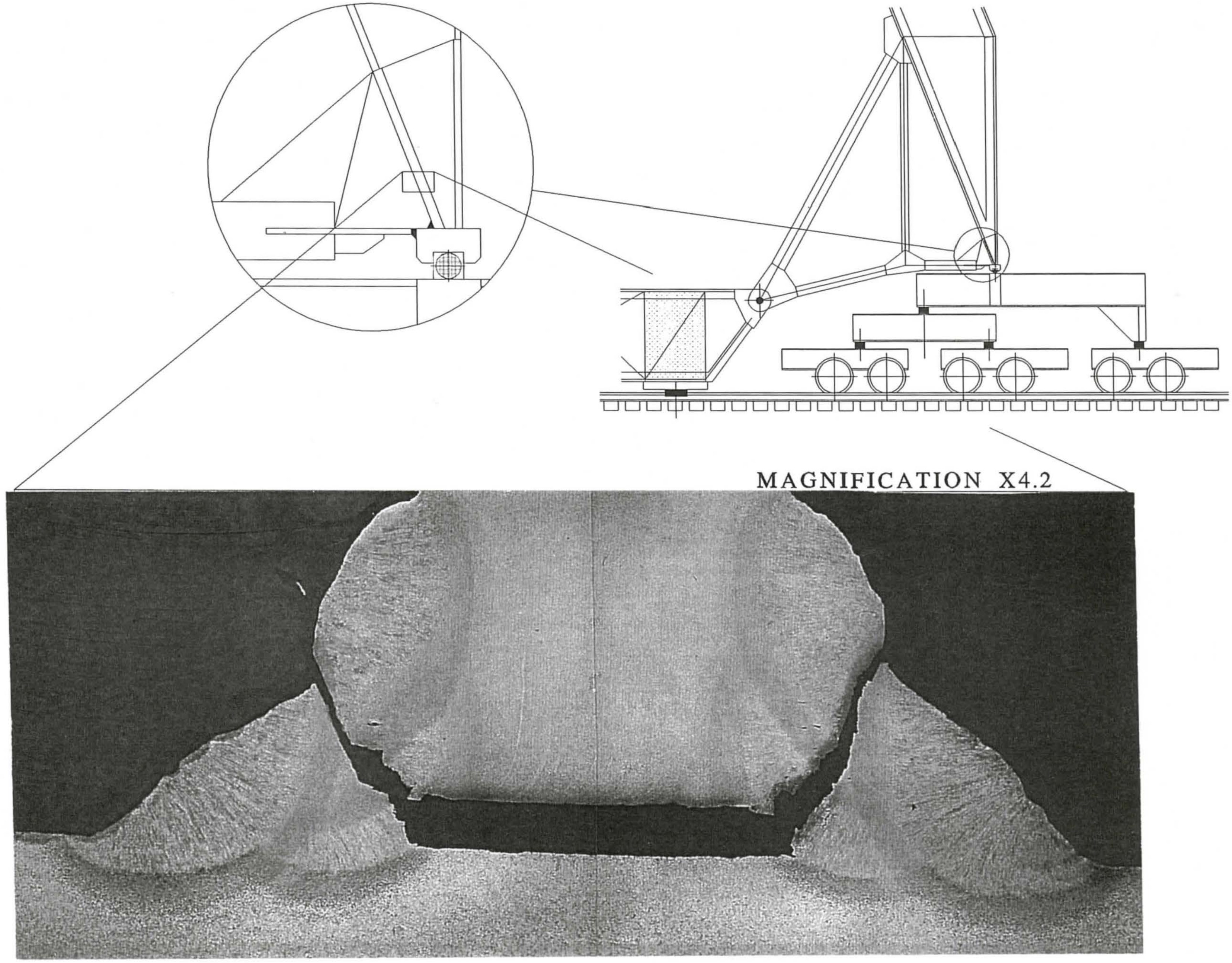


FIGURE 89: Magnified view of west gusset welds to inside face of the South Leg. (Wood, 1995)

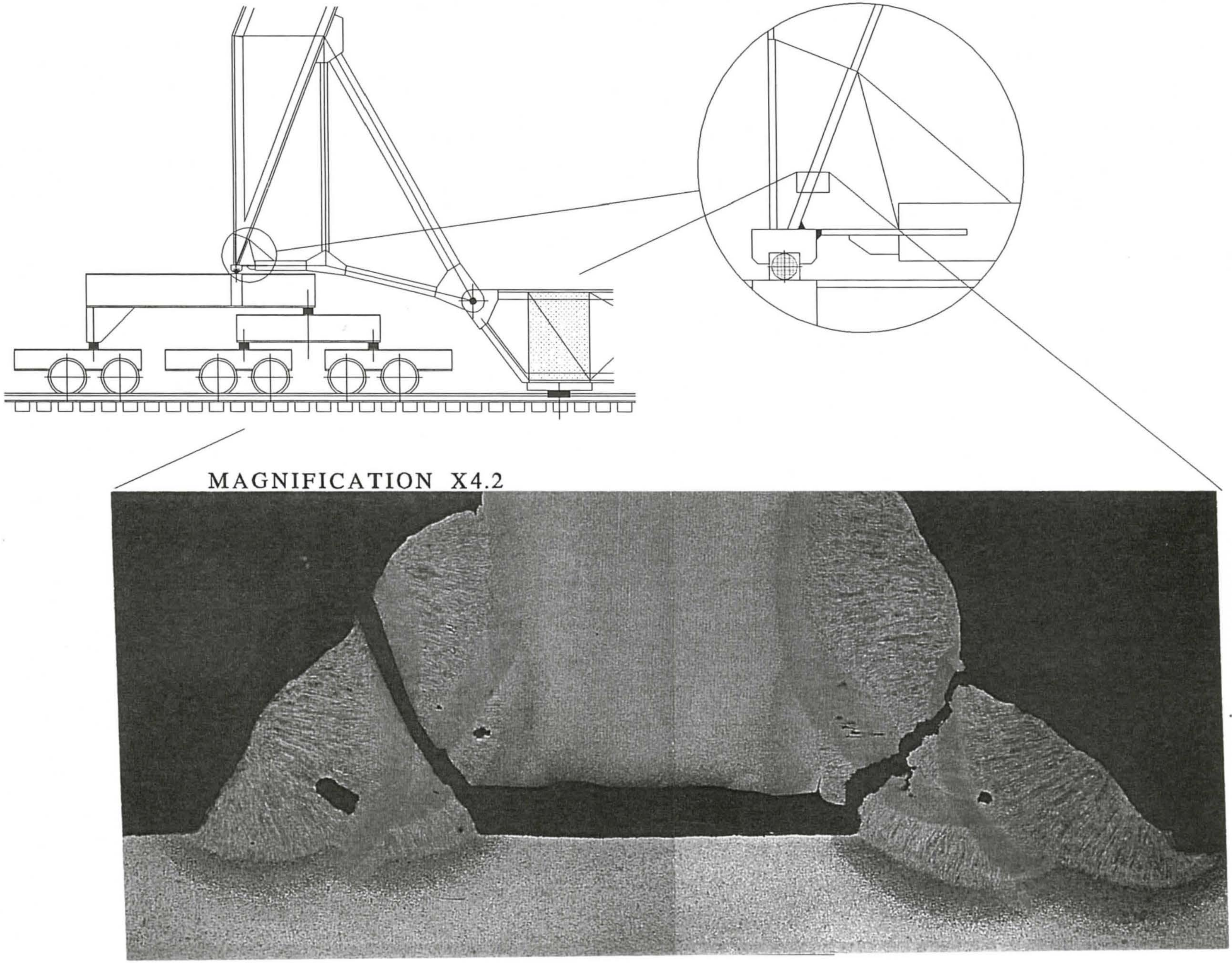


FIGURE 90: **Magnified view of east gusset welds to inside face of the South Leg. (Wood, 1995)**

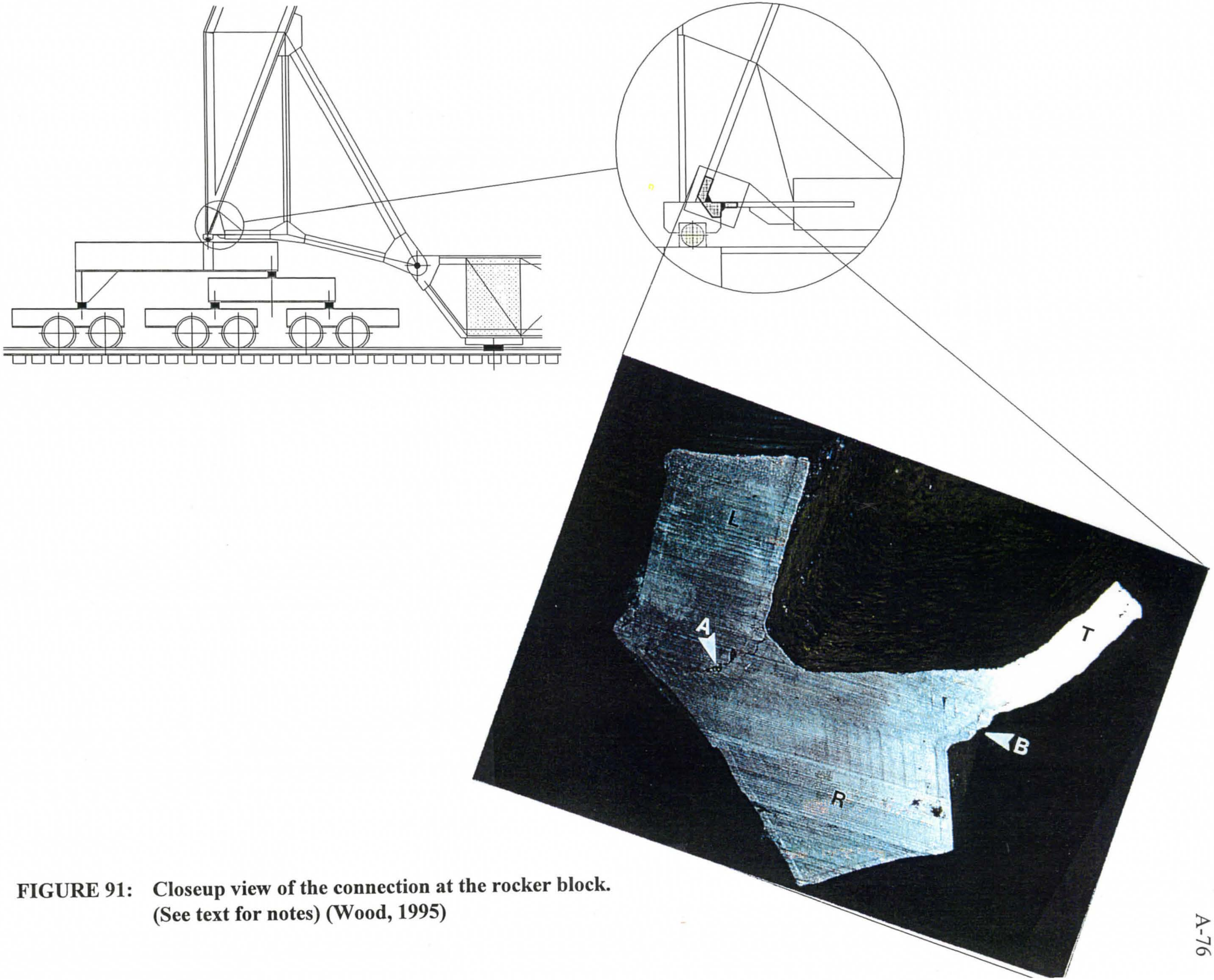


FIGURE 91: Closeup view of the connection at the rocker block. (See text for notes) (Wood, 1995)

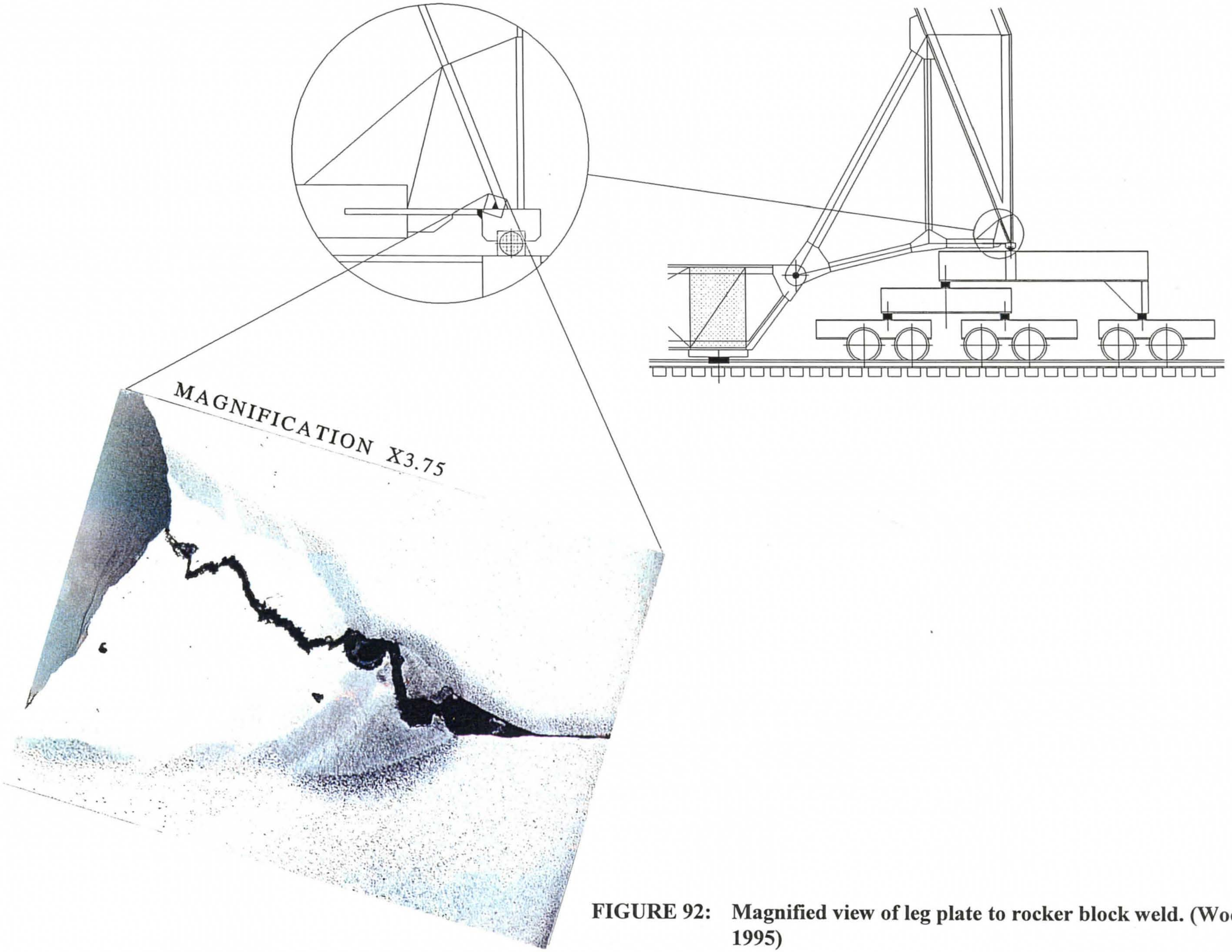


FIGURE 92: Magnified view of leg plate to rocker block weld. (Wood, 1995)

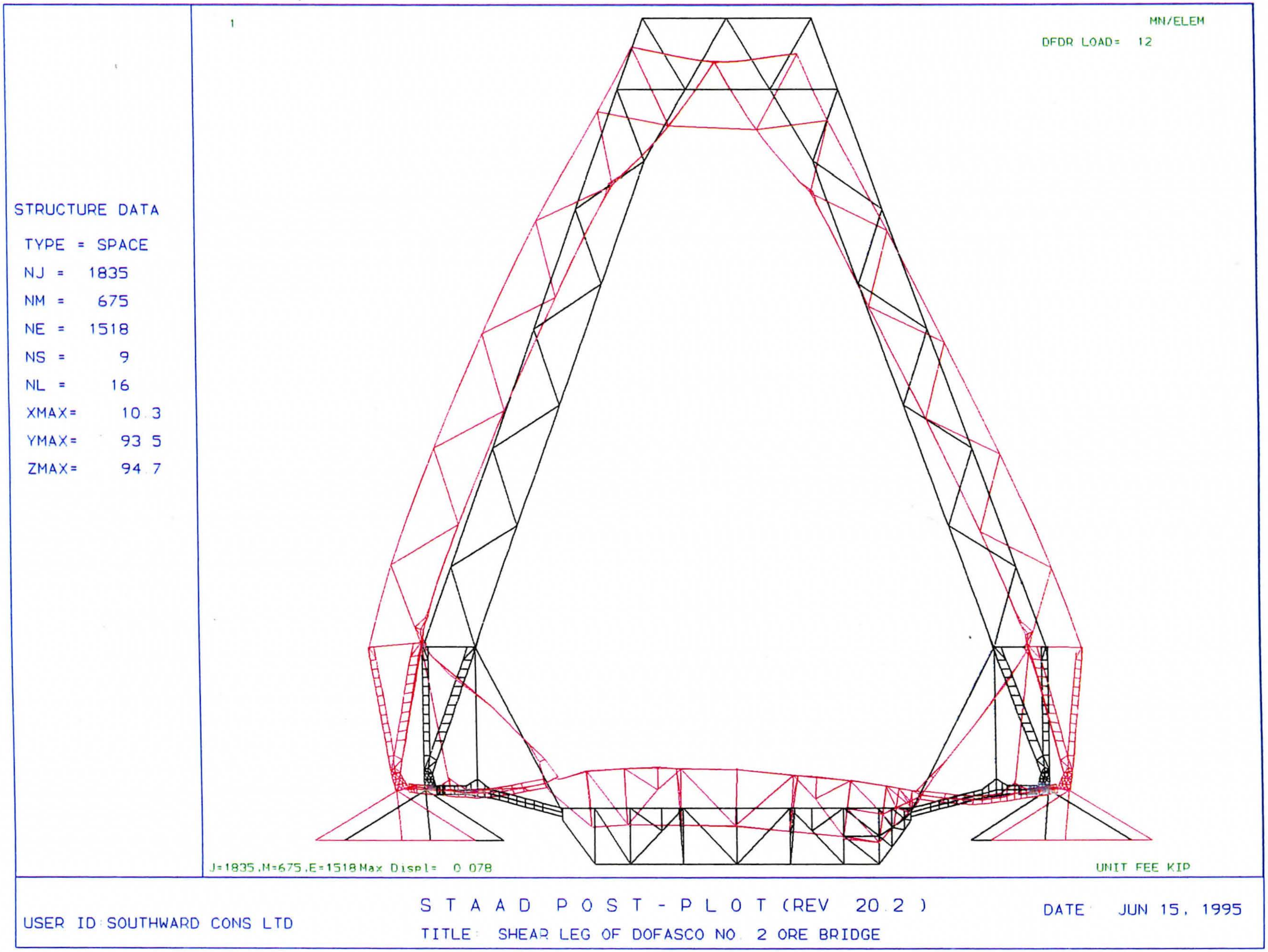


FIGURE 93: **View of deflected shape of the Shear Leg (red) under gravity loading without the hopper relative to its undeformed shape (black).**

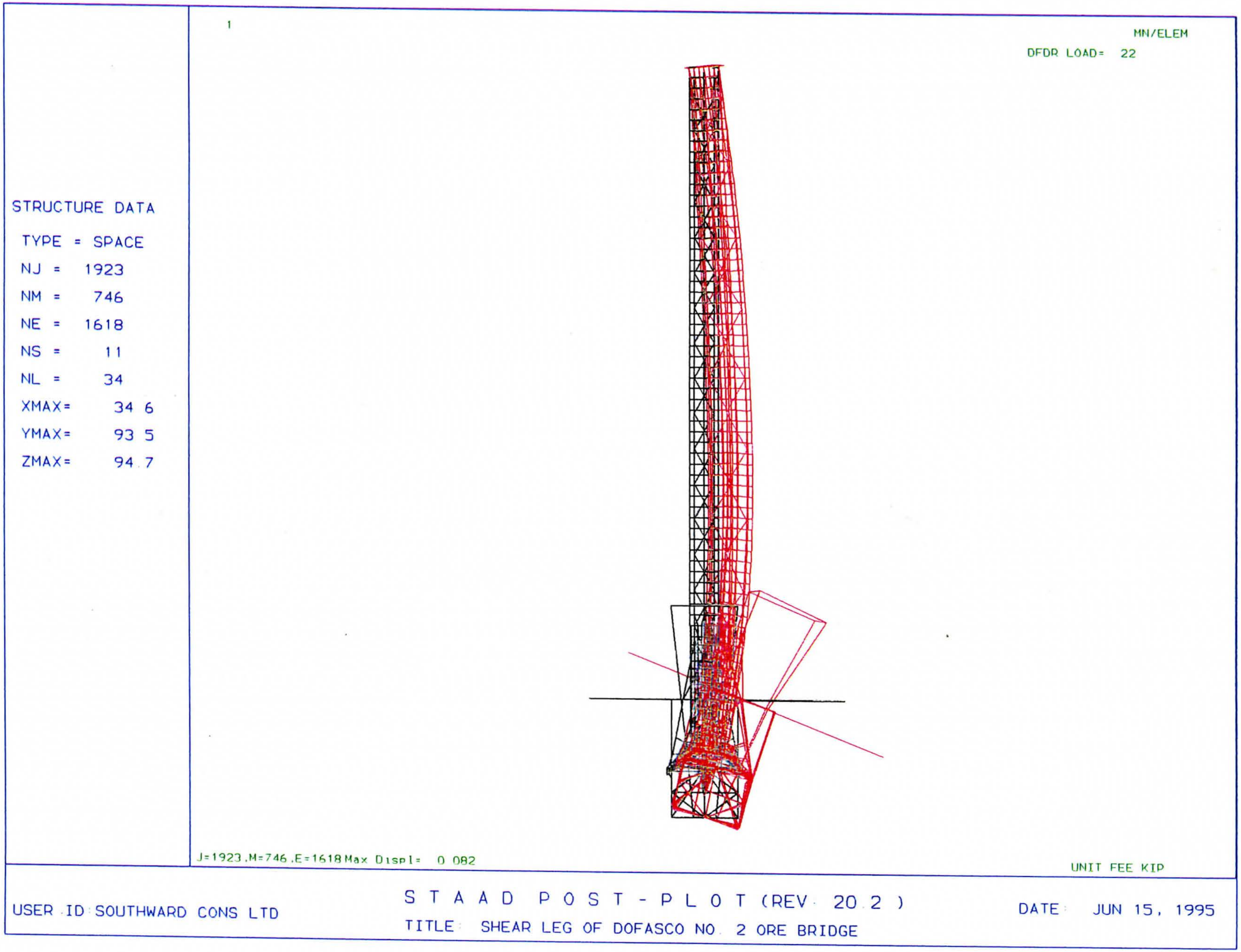


FIGURE 94: **View of lateral displacements of the Shear Leg (red) under lateral loading from the hopper relative to its undeformed shape (black).**

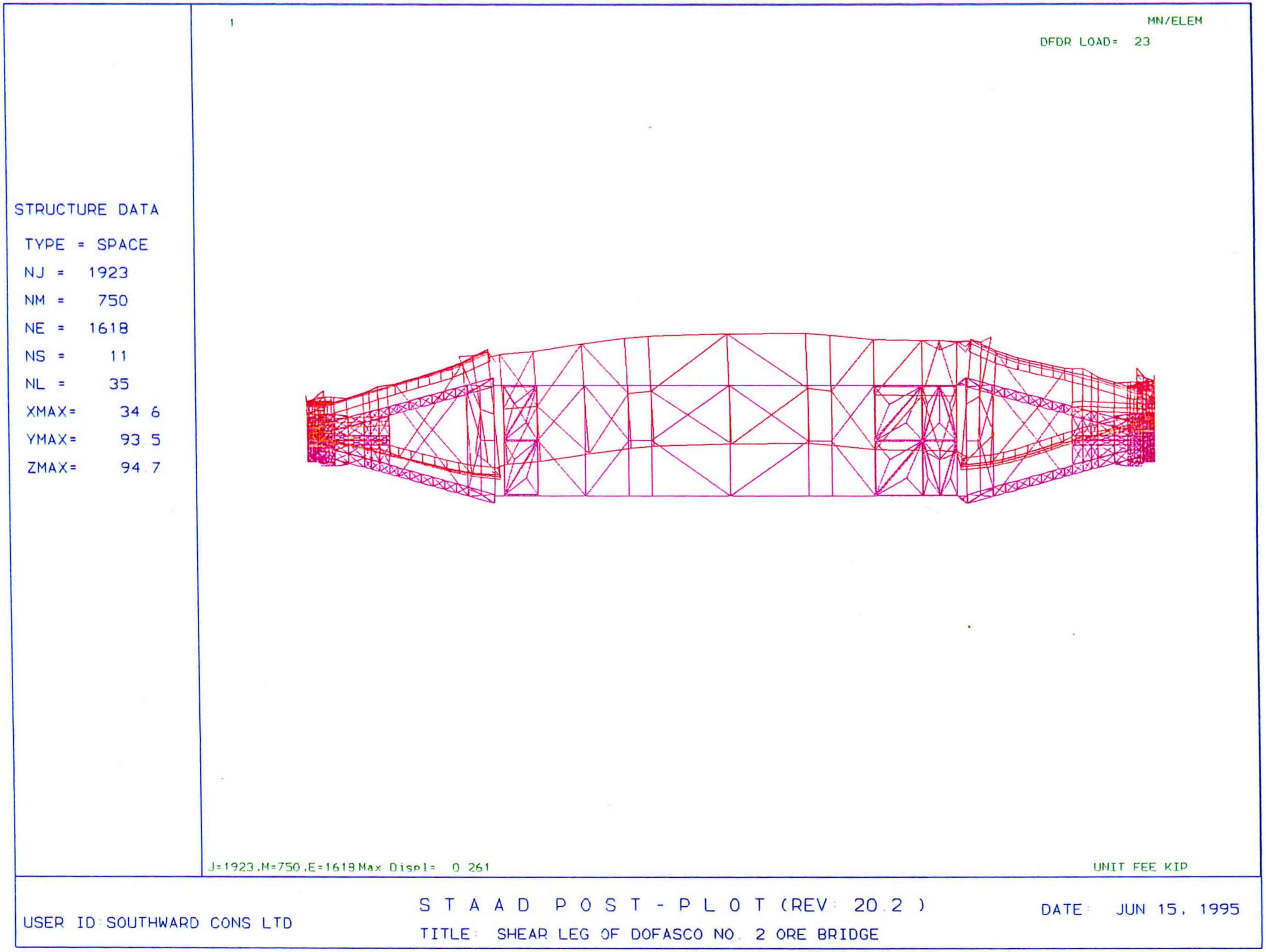


FIGURE 95: **View of the lateral bending of the top of the sill truss (red) under lateral loading from the hopper (purple).**

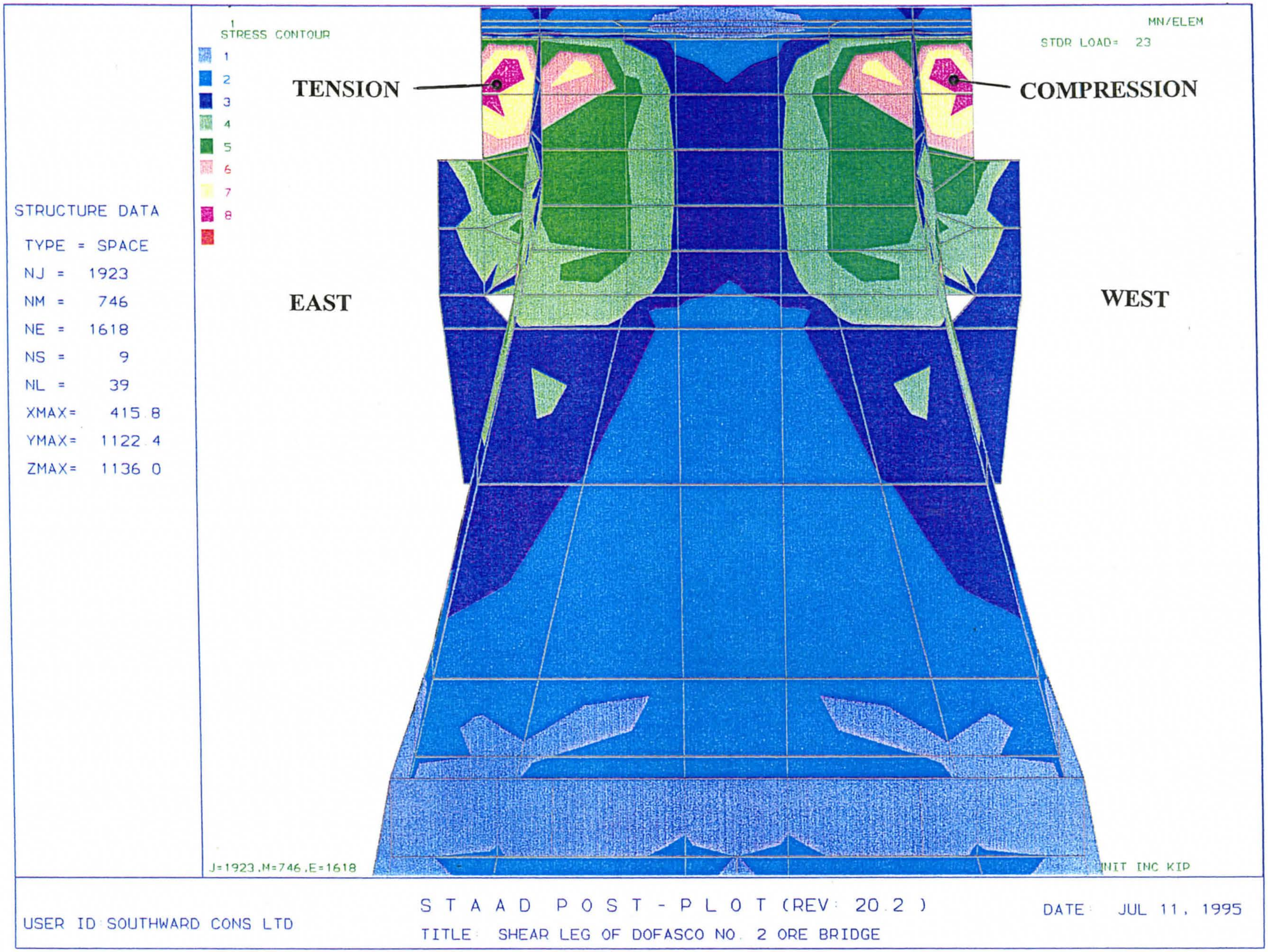


FIGURE 96: **Finite element analysis results at the base of the South Leg showing maximum principal stresses at the edges of the tie-plate.**

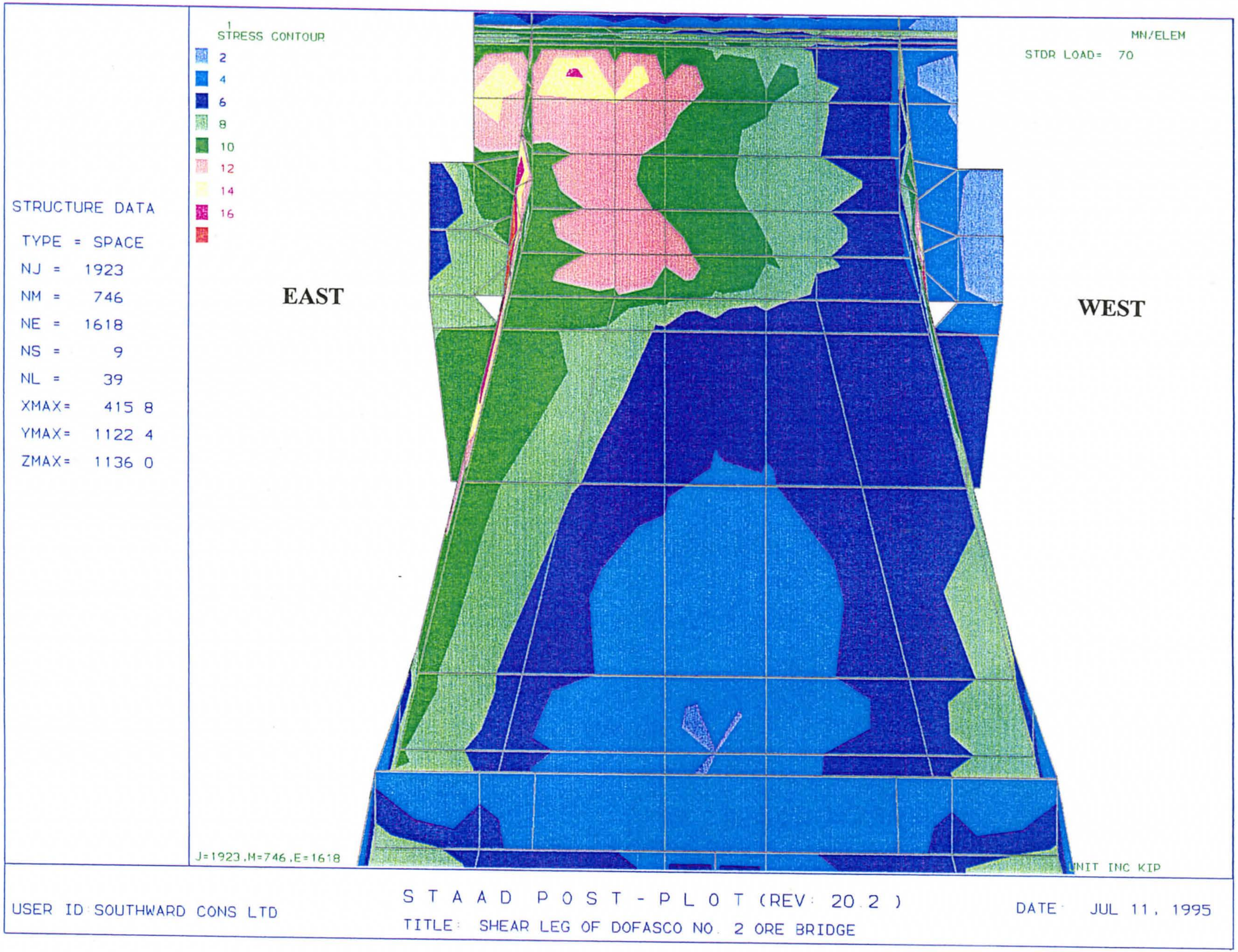


FIGURE 97: **Finite element analysis results at the base of the South Leg showing maximum principal stresses by combining gravity loading with a forced lateral displacement of the hopper.**

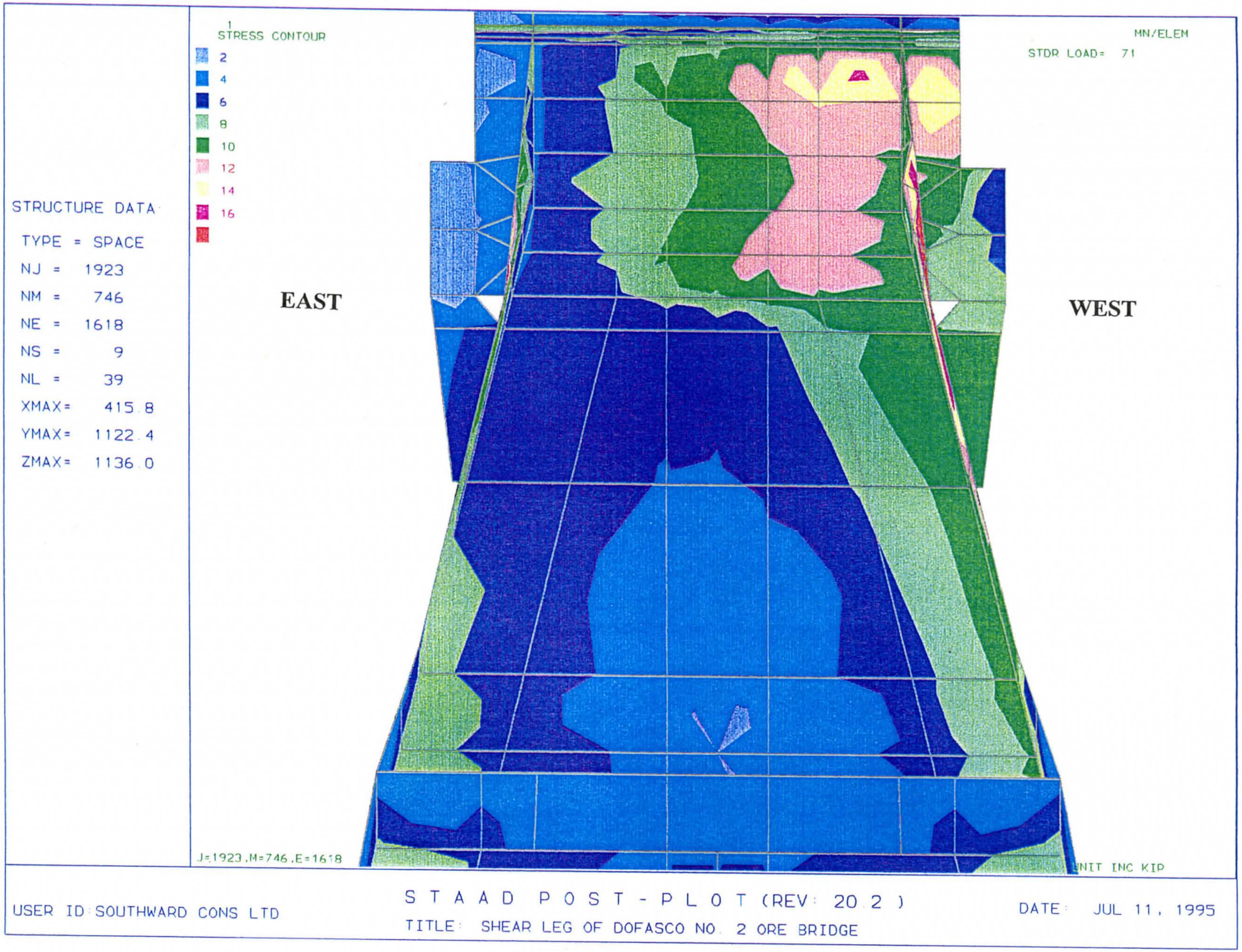


FIGURE 98: **Finite element analysis results at the base of the South Leg showing maximum principal stresses by displacing the hopper in the opposite direction.**

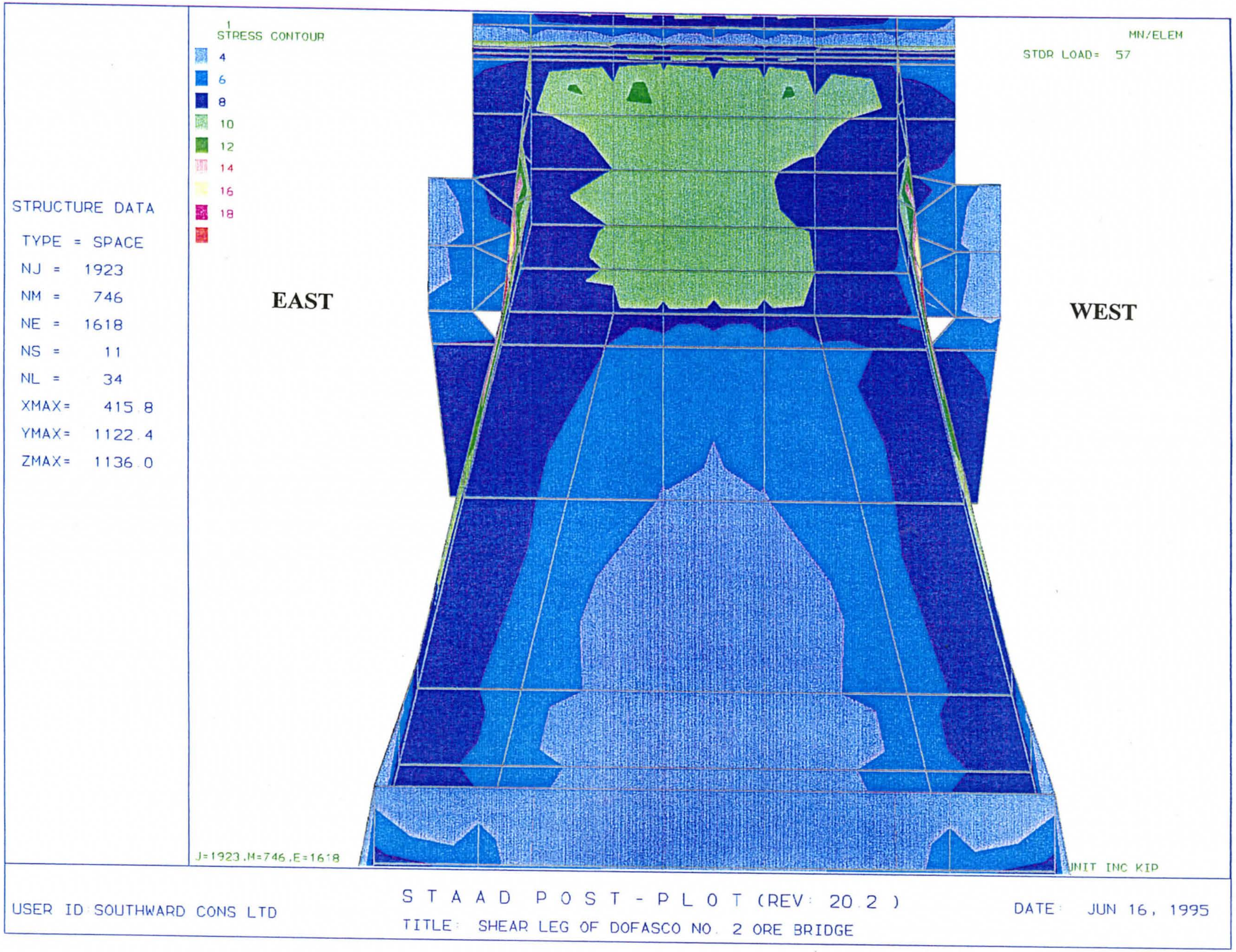


FIGURE 99: **Finite element analysis results at the base of the South Leg showing maximum principal stresses in the tie-plate from uniform gravity loading.**