Winner Creek Trail Bridge at Glacier Creek Feasibility Study

May 2021

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1.0 Introduction

1.1 Project Goal

The Girdwood Service Area (GSA) has initiated a feasibility study to determine options for a new pedestrian bridge across Glacier Creek near the Crow Creek Road end (lower trailhead) of the Winner Creek Trail in Girdwood, Alaska. The existing crossing of Glacier Creek at this location is a hand tram, which opened in 2001. Since 2001, there have been multiple accidents at the tram. In the summer of 2019, two people fell from the tram in two separate incidents that resulted in one fatality and one serious injury. The existing tram requires yearly maintenance.

Prior to installation of the hand tram, Glacier Creek was crossed using one of two cables spanning the creek upstream of the tram; these two cables are still present. In the early 1900s, Girdwood miners constructed a bridge downstream of the tram's current location; remnants of this bridge are still visible.

This feasibility study serves to identify feasible and favorable alignments for the bridge and to recommend structure types that meet general project requirements with respect to the following criteria:

- Provide pedestrian bridge to enhance access to regional trails.
- Least environmental impact.
- Aesthetics.
- Sustainability and minimum maintenance.
- Cost savings.
- Public safety.

1.2 Area Description

The existing Winner Creek Hand Tram crosses the Glacier Greek just north of the intersection with Winner Creek. It is anticipated that the new bridge will cross Glacier Creek near or at the same location. This current crossing is approximately one (1) mile from the Winner Creek Gorge Trail Head (on Crow Creek Road, four (4) miles north of Girdwood) and two (2) miles from Alyeska Resort.

The current hand tram crossing is approximately 52 feet above the creek and has a span of approximately 180 feet. The canyon at this location is rock faced on both sides. The existing west foundation is a concrete footing founded on undisturbed earth. The existing east foundation is a concrete footing anchored into hard rock.

2.0 Project Design Criteria

2.1 Project Design Criteria

Bridge: Based on community and other stakeholder input, the following criteria has been set for the new pedestrian bridge. The new bridge shall:

- Support pedestrian loading (width of 5'-0", 90 psf pedestrian load),
- Require low maintenance,
- Have low up-front capital costs,

- Deter vandalism, and
- Deter (bungee-) jumping from the bridge.

Note that for year-round functionality, all designs assume that snow is removed from the bridge for winter accessibility.

Alignment: Factors considered during the alignment study include:

- Routes that could provide shortest overall bridge lengths and minimum costs for crossing Winner Creek.
- Routes that could minimize existing vegetation, tree, trail and any sensitive area impacts.
- Routes that can provide construction access and staging and can facilitate erection of long-span structures such as steel trusses.
- Consideration was given to the maximum slope of the trail per local standards and requirements.

Trail: The Girdwood Valley Trails Management Plan (2020 Revision) classifies the section of Winner Creek Trail in the area of the Tram Crossing as a Class 3 Trail. Any realignment of the trail to connect to a new bridge location will require a design that meets the criteria provided in the Design Parameter Matrice below.

Designed Use Hiker/Pedestrian		**Trail Class 3**
Design Tread Width	Wilderness (Single Lane)	3 feet
Design Surface	Type	Native with onsite borrow or imported material where needed for stabilization, occasional grading. Intermittingly rough.
	Protrusions	< 3" - May be common, not continuous
	Obstacles (Maximum Height)	10"
Design Grade	Target Grade	3% - 12%
	Short Pitch Maximum	25%
	Maximum Pitch Density	10% - 20% of trail
Design Cross Slope	Target Cross Slope	5% - 10%
	Maximum Cross Slope	15%
Design Clearing	Height	7' – 8'
	Width	36" – 60"
	Shoulder Clearance	12" – 18"
Design Turn	Radius	3' – 6'

Figure 1 TRAIL CRITERIA

2.2 Bridge Options

Four options will be considered for this Feasibility Study:

a. Lower Trail Crossing: A steel truss bridge located approximately five (5) feet above the 50-year creek flood elevation and 40 feet (+/-) north of the existing hand tram, spanning 84 feet (+/-).

b. Upper Trail Crossing No. 1: A steel truss bridge located at the top of the canyon (roughly level with the existing hand tram), spanning 180 feet (+/-).

c. Upper Trail Crossing No. 2: A steel cable suspension bridge located at the top of the canyon (roughly level with the existing hand tram), spanning 180 feet (+/-).

d. Upper Trail Crossing No. 3: A relocated steel truss bridge located at the top of the canyon, relocated from its current location at the Parks Highway crossing Montana Creek, spanning 200 feet.

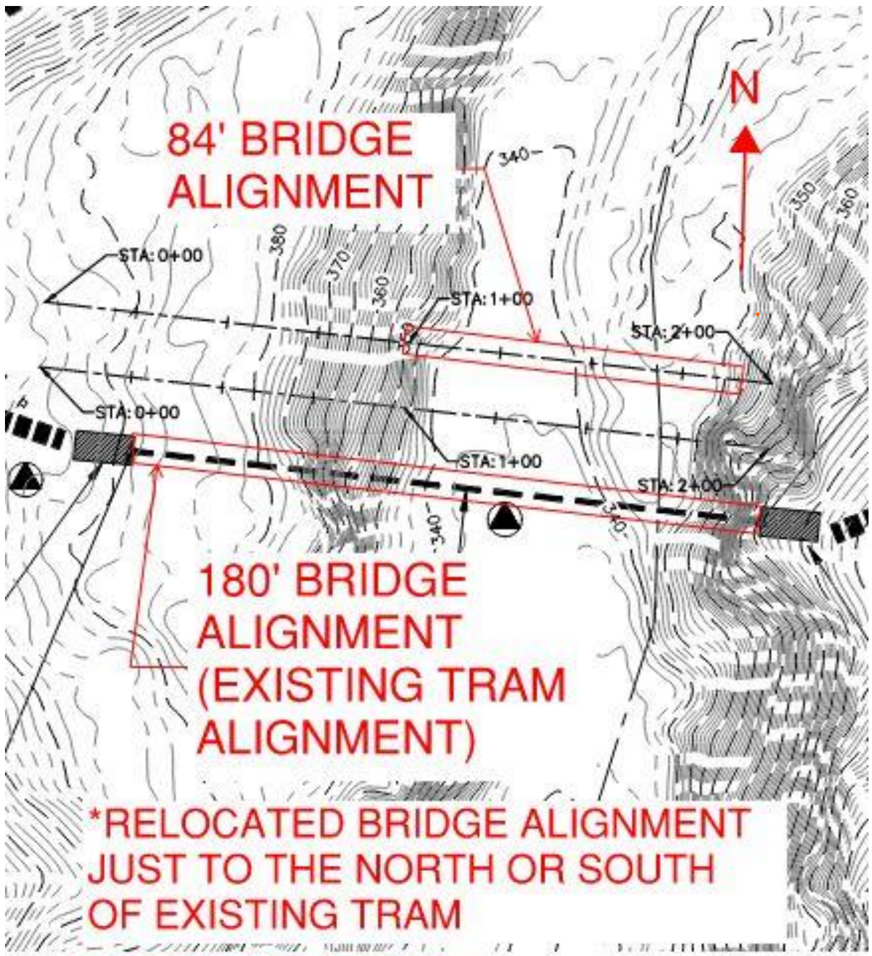


Figure 2 ALIGNMENTS

3.0 Methodology

3.1 Structural Design

The structural design for the steel truss bridges in this Feasibility Study are based on the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Specifications and Design of Pedestrian Bridges (Specifications). Note that pedestrian bridges with cable supports (for example, suspension bridges) are not specifically addressed in the Specifications, and additional references were used. For both the steel truss bridges and the suspension bridge, a pedestrian live load of 90 pounds per square foot (psf) was used. This is considered conservative for a trail bridge. Due to the 5-feet width limitation, vehicle traffic is not a design load. The relocated bridge has a wider width and was originally designed for vehicle traffic.

Preliminary designs for the steel truss bridges are based on hand calculations and confirmation using the computer program ETABS.

Preliminary design for the suspension bridge is based on hand calculations and published design guides.

3.2 Constructability and Accessibility

A significant consideration for a new bridge across Glacier Creek is constructability. Based on our site visits, the current tram crossing location appears to be best suited for the new crossing. The site is not accessible by road, but by trail, with trail widths varying from 3' to 6'. It is anticipated that an all-terrain vehicle (ATV) could be used to get equipment to the site, but no trucks or other construction vehicles would be able to deliver materials. During construction of the existing hand tram, a helicopter was used to provide materials to the site; therefore it is assumed that a helicopter will be used again for delivery of materials for the new bridge. A May 17, 2001 Turnagain Times news article indicated that 77 helicopter flights were needed to complete the hand tram.

The site is in a heavily wooded area; it is assumed that staging areas will be cleared from the existing forest on both sides of the river during construction.

Construction of a new bridge would likely be limited to the months of May through November, depending on the type of bridge selected, due to frozen soil before May and snowfall after November.

3.3 Site Visits

In preparation for this Feasibility Study, individuals representing The Boutet Company, Reid Middleton and Shannon and Wilson performed two (2) site visits to the proposed bridge crossing location. The first site visit was on October 7, 2020, and the second visit on February 18, 2021. During the October 2020 visit, Glacier Creek was flowing and the hand tram was closed due to safety concerns. During the February 2021 visit, Glacier Creek was covered by snow and ice and the hand tram was closed for the season. Although some of the tram foundation was covered in snow, as-built measurements of the existing foundations on both sides of the tram were taken.

3.4 References

The following references were used to prepare the bridge concept designs:

- 2009 LRFD Specifications for the Design of Pedestrian Bridges, with 2015 Interim Revisions (Specifications)
- 2007 AASHTO LRFD Bridge Design Specifications with 2008 and 2009 Interim Revisions
- Bridges to Prosperity Design Manual, 2nd Edition, 2011
- Base map of the area provided by TBC, dated January 26, 2021.

3.5 Geotechnical Data

A geotechnical study was prepared by S&W to support the Feasibility Study for a new bridge crossing over Glacier Creek. Based on site observations, foundation recommendations were developed for the 'lower trail crossing' and 'upper trail crossings' alternatives as well as the associated rock cuts for the 'lower trail crossing.'

Lower Trail Crossing: Depending the location of the lower crossing, there could be several suitable foundation approaches to the abutments. Shallow foundations could be used bearing on rock if the abutments are located against the edges of the gorge. We believe the preliminary guidance in Section 6.2.1 of the attached Geotechnical Report for the upper crossings is appropriate for shallow foundations bearing on rock at the lower crossing alternatives. However, the recommended slope setbacks will not be needed since the abutments are likely to be at or very close to the bottom of the gorge with a minimal foreslope in front of the foundations.

Shallow foundations bearing on alluvium or driven pile foundations could be used if the abutments are located away from the edges of the gorge. If foundations bear on alluvium, special consideration will be needed for accommodating potentially liquefiable soils and significant scour conditions during periods of high water. If pile foundations are used, it is likely that they will need to be connected in some way to bedrock as alluvium in the gorge bottom is likely too thin to accommodate lateral and uplift loading. For the purposes of this report, we assume that some form of deep foundation will be used on lower crossing alternatives due to the anticipated poor soil and scour conditions likely to exist in the bottom of the gorge.

Deep foundations for lower crossings will likely consist of open-ended driven pipe piles that could range in size from 8 to 24 inches in diameter depending on final design and latera/axial loading. For planning purposes, we recommend assuming that piles will need to be driven through alluvial soils and will need to be socketed into rock. The thickness of the alluvial soils is unknown, but for planning purposes we recommend assuming a thickness of approximately 20 feet. Additional depth into competent bedrock will be required for lateral and uplift resistance. If conventional socketing techniques are used (i.e. drilling beyond the pile tip and advancing a concrete shaft below the pipe pile) it is likely that lateral and uplift capacities will be achieved with approximately 10 feet of embedment into rock. Note that significant additional geotechnical explorations and engineering evaluation is needed to determine the required configuration of pile foundations for lower crossing alternatives.

Upper Trail Crossings: It is our opinion that the foundation and slope conditions for foundations associated with the upper crossing alternatives are favorable, however adequate setback from the crest of the rock slope below the abutments should be confirmed for final placement. Assuming strip footings bearing directly on rock will be used to support the crossing, it is recommended assuming a setback for the gorge-side edge of the footing of approximately 10 feet from the rock slope crest. These setbacks are based on our observations of rock structure and slope height in the slopes below the abutment. The dominant jointing on both sides of the creek appears to be steeply dipping and kinematically admissible failures appear to consist of toppling on the east side and planar and wedge failures on the west side. Further analysis will be required once a preferred crossing type is identified, a crossing location is selected, and foundation loading requirements are determined. It is possible that greater setbacks may be required or fore-slope stabilization may be needed.

Given the above recommended setbacks and assuming the footing bears directly on a clean, non-weathered rock surface, for preliminary purposes it was recommended assuming an unfactored bearing resistance of rock to be approximately 20 kips per square foot (ksf) and a minimum footing width of 2 feet. Resistance to lateral loading and uplift forces on the upper crossings will be gained by connecting the foundation footing to the rock through tensioned rock anchors. The actual configuration of the rock anchors will depend on the structural design of the abutment foundations. Designing the tensioned rock anchors (i.e. diameter of the rods and pre-tension loads) will depend on the magnitude of uplift and shear loading on the foundation, which are not known at the time of this report. For planning purposes, it was assumed 1.5 to 3-inch threaded bars will need to penetrate a minimum of 20 feet below the foundation with a minimum free-bonded length of 10 feet. Friction resistance along the base of the footings can be estimated using a friction coefficient of 0.4 between concrete and rock. The actual configuration and design of the foundations and anchors will require additional engineering analysis once a conceptual bridge design and loading requirements are determined. The anchors should incorporate the appropriate corrosion protection to ensure that they maintain capacity over the life of the structure.

Rock Cut Slopes: Rock cuts may be required, especially if a lower trail crossing is selected to establish access from existing trials to the gorge bottom. Establishing trail access to the gorge bottom will likely require benching a new trail into the gorge slopes. Based on our experience in the area and observations on site, we believe that the gorge slopes contain minimal organic and mineral soil overburden. Additionally, we believe that gorge slopes north of the existing tram crossing provide the most favorable conditions for establishing new trails. Establishing new benches for the trail should be achievable using conventional drill and blast techniques. For planning purposes, we recommend establishing a setback of at least 2 feet from the edge of the trail to the edge of slope to allow for a safety buffer and establishing a railing. Additional space for catchment of rockfall should be included on the upslope side of the bench. The width of rockfall catchment will depend on the height of the cut slope above the bench, but we believe that 2 to 4 feet should be sufficient for planning purposes. It is recommended a maximum rock cut slope angle of ¼ horizontal (H) to 1 vertical (V) be used in rock. Additional geotechnical analysis will be needed once trail alignments are established to determine appropriate rock cut slope angles and stabilization measures if needed.

4.0 Bridge Options

4.1 Lower Trail Crossing - 84' Steel Truss Bridge

An 84' steel truss bridge would be placed approximately 40' north of the existing tram bridge and be at elevation 350', roughly 12' above the creek. This option requires substantial civil work to bring the existing trail down the canyon wall from elevation 390' to approximately elevation 350'. Depending on the river 50-year-flood level (to be determined by separate hydrology and hydraulic analysis), the elevation of the bridge could vary, and the length of this option could vary from 80' to 100'.

Steel truss bridges are common for trail bridges in the Municipality of Anchorage (MOA) and can be seen along the Chester Creek, Campbell Creek and Coastal Trails. The steel truss bridge would be composed of HSS tube and wide-flange steel members. The deck would be metal deck grating. Both steel members and deck would be hot dip galvanized for protection and longevity.

The steel truss bridge could be covered. This roof can be designed to allow snow pass-through (allowing winter trail grooming) or to provide protection, while also deterring people from jumping off the bridge. Due to requirements for pedestrian and bike clearance, deck to underside of overhead covering minimum is 10'.

This steel truss bridge would likely be delivered via helicopter. The bridge could be delivered to the site in multiple sections and spliced on-site.

Abutments for the steel truss bridge will likely need to accommodate shallow rock, which may include concrete pads anchored to the ground using rock anchors.

Conceptual plan, elevation, and cross section are shown below:

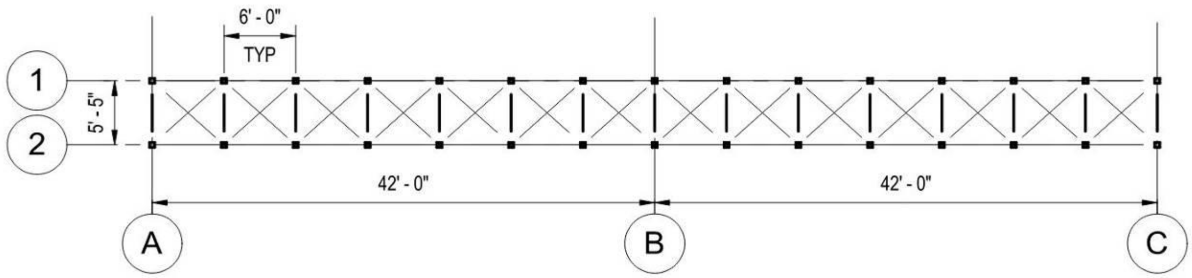


Figure 3 TRUSS BRIDGE PLAN, SPAN 84'

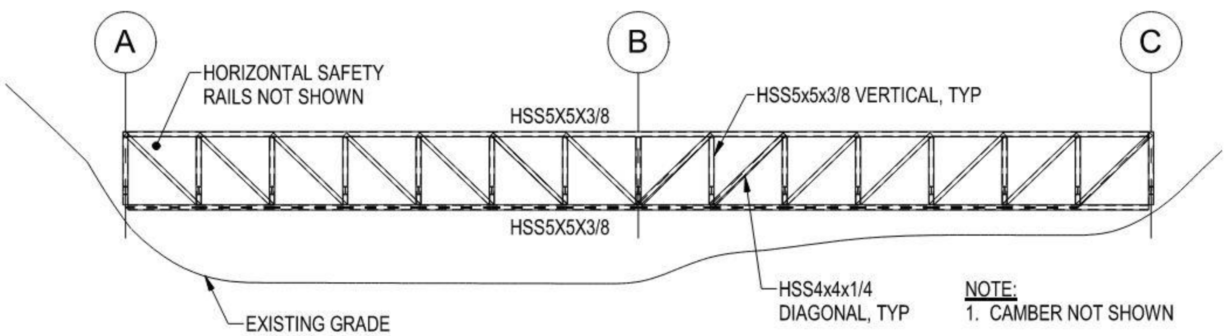


Figure 4 TRUSS BRIDGE ELEVATION, SPAN 84'

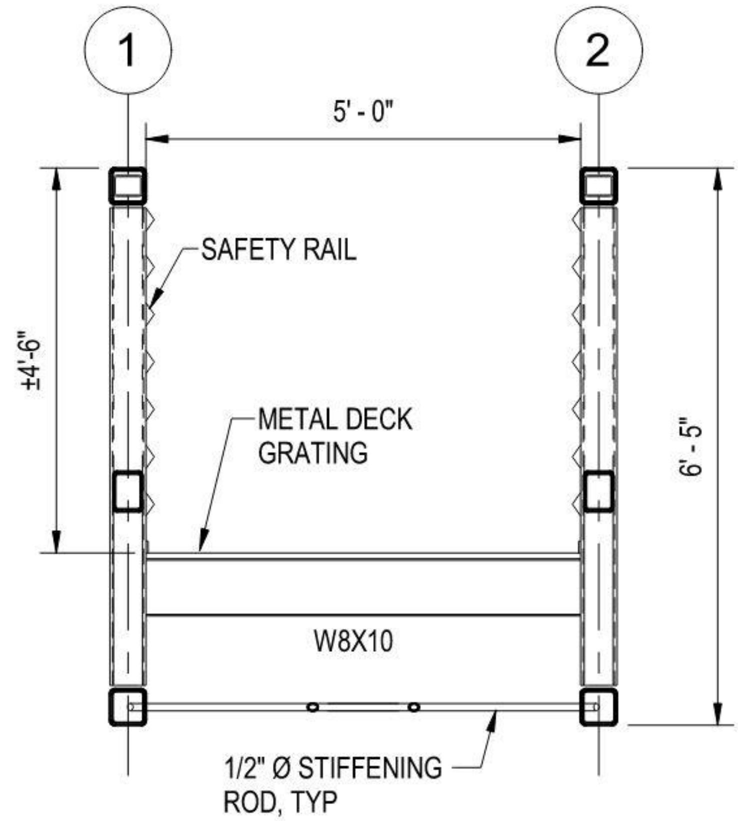


Figure 5 TRUSS BRIDGE SECTION, SPAN 84'

4.2 Upper Trail Crossing No. 1 - 180' Steel Truss Bridge

A 180' steel truss bridge would be placed along the alignment of the existing tram. This option requires minimal civil work as the existing trails meet up with the existing tram ends.

The existing tram concrete foundations would remain in place and be supplemented/expanded with additional concrete to support the new bridge cross section. New concrete would be connected to existing concrete with adhesive doweled reinforcement. New concrete will be anchored to rock to resist uplift loads.

Similar to the 84' Steel Truss Bridge option, the bridge would be composed of HSS tube and wide-flange steel members. The deck would be metal deck grating and both steel members and deck would be hot dip galvanized for protection and longevity. Moreover, the steel truss bridge would likely be delivered to the site via helicopter in multiple sections and spliced on-site.

Abutments for the steel truss bridge will be concrete pads anchored to the ground using rock anchors.

Conceptual plan, elevation, and cross section are shown on the following page.

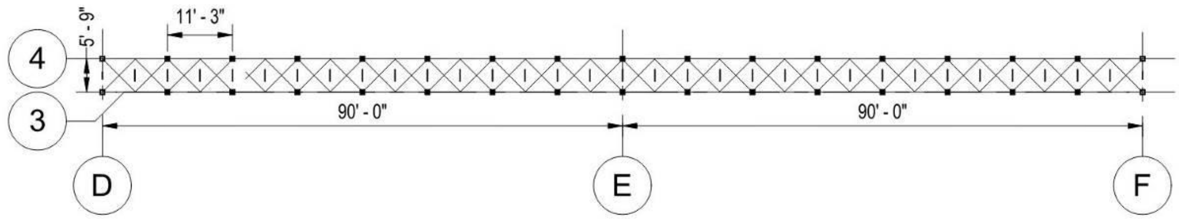


Figure 6 TRUSS BRIDGE PLAN, SPAN180'

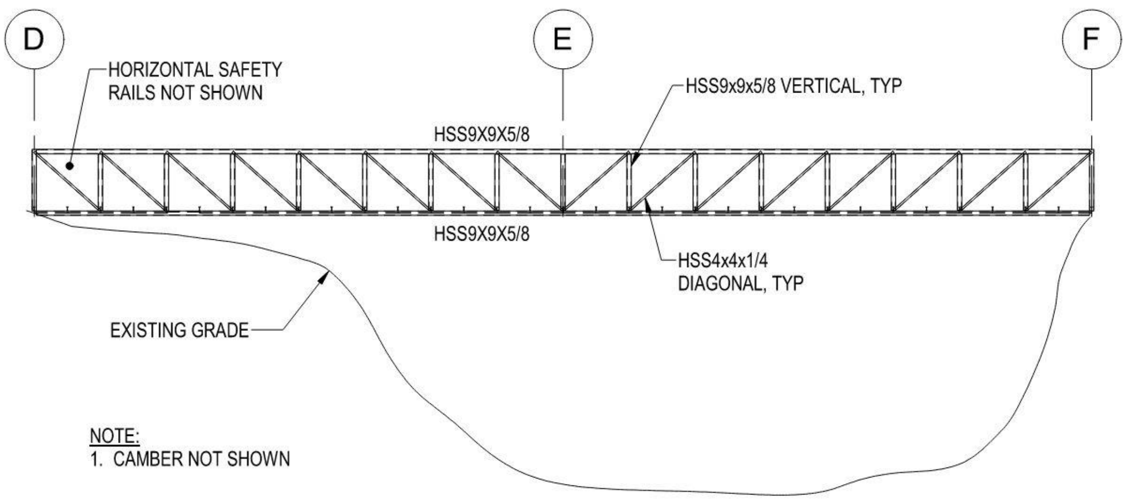


Figure 7 TRUSS BRIDGE ELEVATION, SPAN 180'

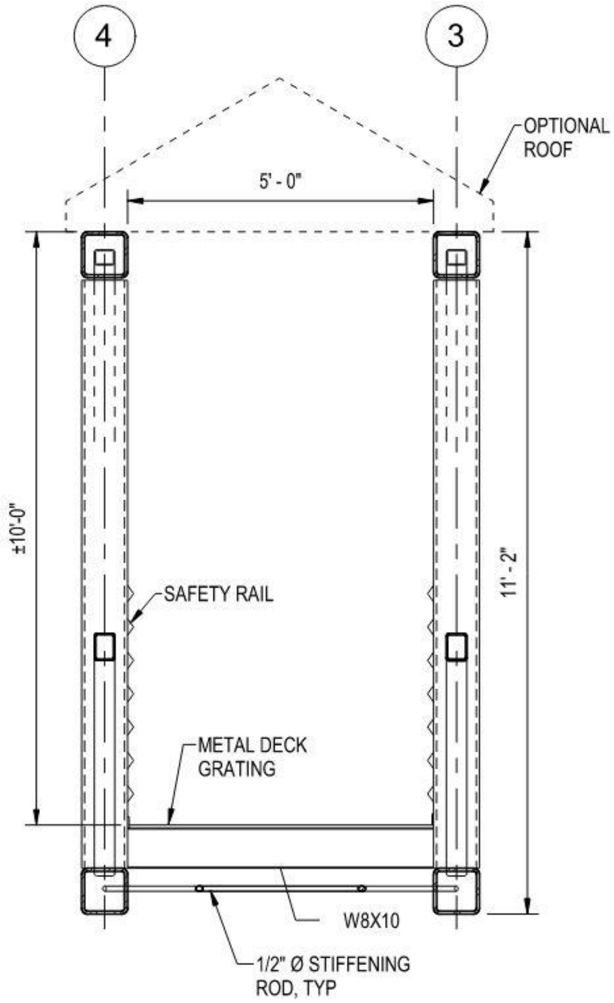


Figure 8 TRUSS BRIDGE CROSS SECTION, SPAN 180'

4.3 Upper Trail Crossing No. 2 - 180' Suspension Bridge

A cross-canyon suspension bridge is a common design for pedestrian bridges in remote areas across the world. At Glacier Creek, a 180' suspension bridge would be placed along the alignment of the existing tram. This option requires minimal civil work as the existing trails meet up with the tram bridge ends.

The existing tram concrete foundations would remain in place and be supplemented/expanded with additional concrete to support the new bridge cross section. New concrete would be connected to existing concrete with adhesive doweled reinforcement. New concrete will be anchored to rock to resist uplift loads.

The abutment towers will be anchored to the expanded concrete foundations. The main cables will be anchored to bedrock using rock anchors, as recommended by the geotechnical engineer. Minimum design load of 60 kips (unfactored) is required per main cable.

The new suspension bridge would be composed of one (1) main cable per side and suspender cables every 11'-3", each side. The deck would be metal grating. The safety rail would be a metal mesh to provide lightweight fall protection.

Conceptual plan, elevation, and cross sections are shown below.

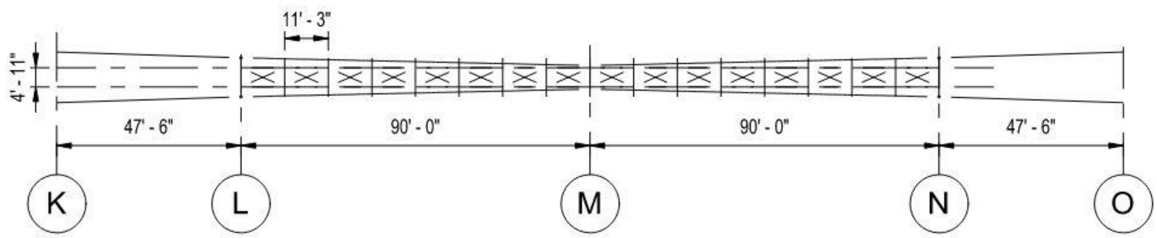


Figure 9 SUSPENSION BRIDGE PLAN

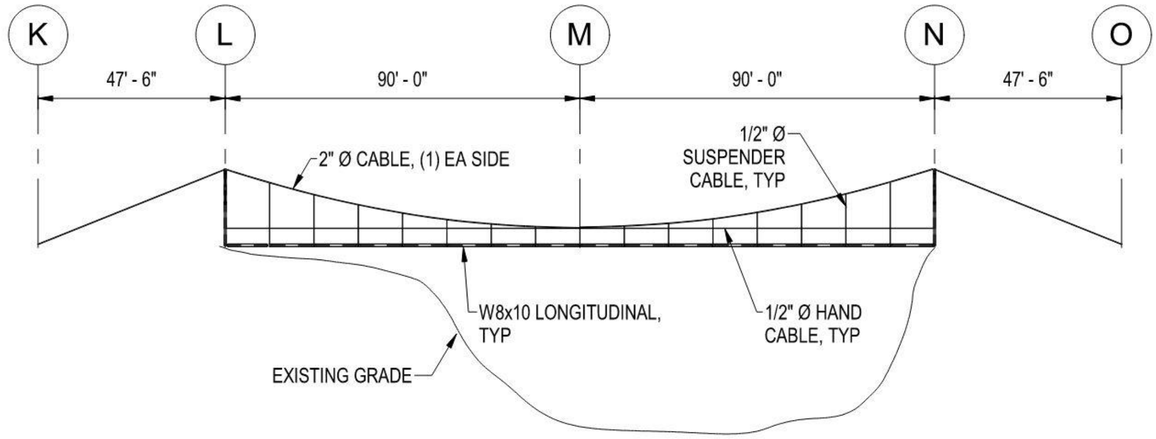


Figure 10 SUSPENSION BRIDGE ELEVATION

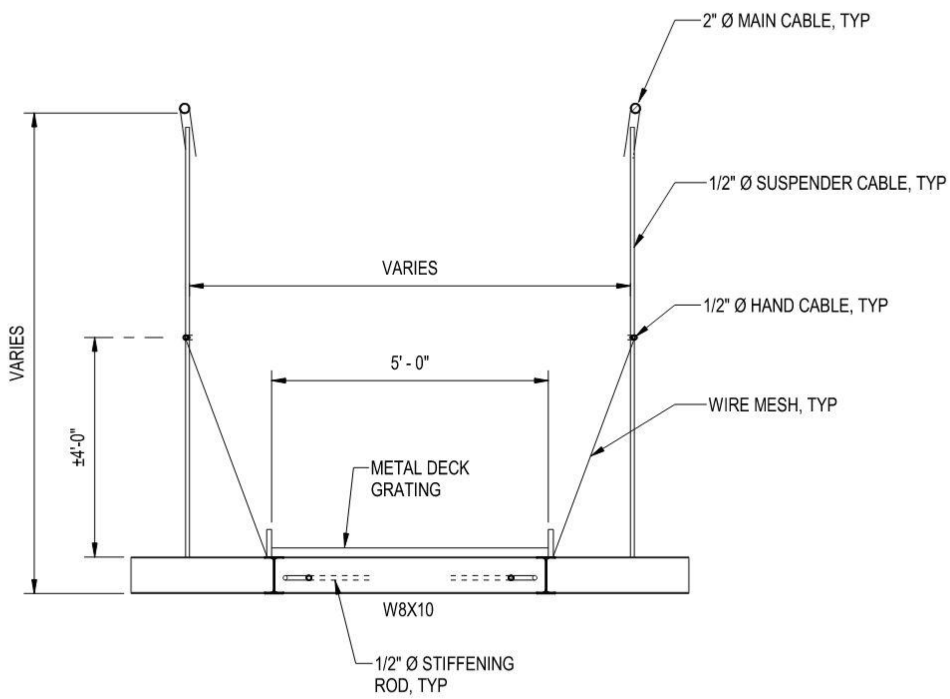


Figure 11 SUSPENSION BRIDGE CROSS SECTION

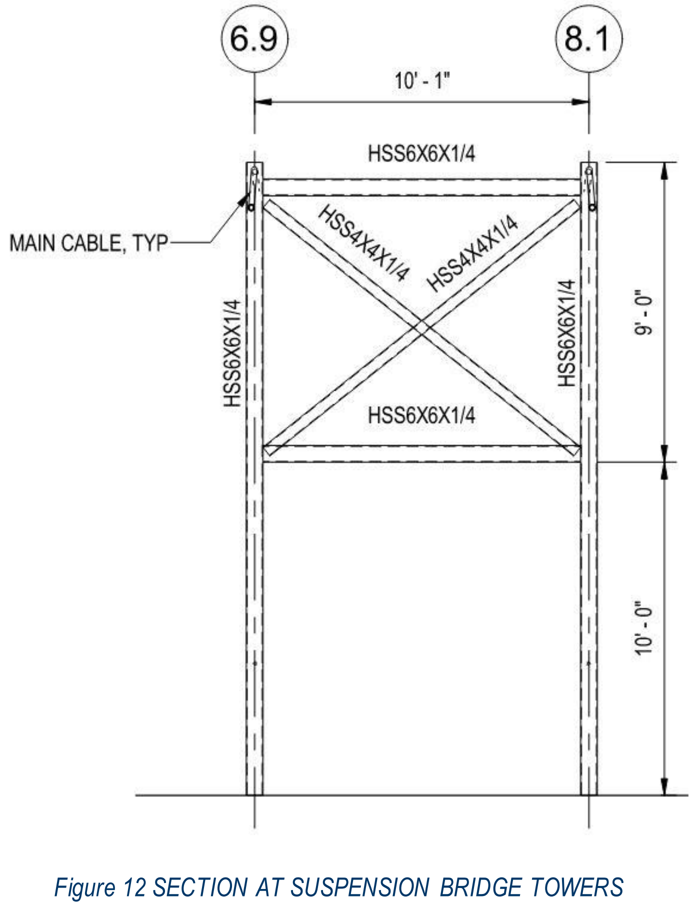


Figure 12 SECTION AT SUSPENSION BRIDGE TOWERS

4.4 Upper Trail Crossing No. 3 - Relocated Bridge

The current pedestrian crossing of Montana Creek at mile 96.5 of the Parks Highway is a 200' steel truss bridge. The Alaska Department of Transportation (ADOT) is planning on replacing the vehicle crossing of Montana Creek at this location and including a new pedestrian bridge with the new vehicle crossing. The existing pedestrian bridge will be removed and could be relocated to provide a crossing of Glacier Creek. This process is constrained by the ADOT timeline.

Figure 13 EXISTING PEDESTRIAN BRIDGE AT MONTANA CREEK

The Montana Creek pedestrian bridge was designed and built in 2000. Both the plan drawings and shop drawings are available for review. The bridge is 200' long and has an inside width of 8'-2". The steel truss bridge is composed of galvanized pipe and wide-flange steel members. The decking is pre-galvanized metal bridge decking. The bridge was designed for both pedestrian and service truck (10,000 lb) loading.

Abutments for the relocated steel truss bridge will be concrete pads anchored to the ground using rock anchors. Due to length and width of the relocated bridge, reusing tram foundations is not feasible. The existing tram foundations could be converted to observation and/or picnic platforms.

Review of the 2018 inspection report for the Montana Creek pedestrian bridge shows the bridge in good condition. Minimal repair and paint of rusted pieces will need to be done prior to installation at Glacier Creek.

Anticipated process for relocation of the steel truss bridge from Montana Creek is as follows:

1. Remove any existing asphalt or wearing surfaces.
2. Saw cut existing deck at joints.
3. Disassemble bridge into three x 67' sections and transport to Girdwood airport. Maximum section weight = +/- 33,000 lbs.
4. Repair and paint rusted pieces.
5. Site cast new foundations just up or downriver from existing tram foundations.
6. Helicopter +/- 20,000 lbs. sections
7. Add suicide and jump protection (welded mesh sides) to bridge once in place.

Existing plan, elevation, and cross section are shown below:

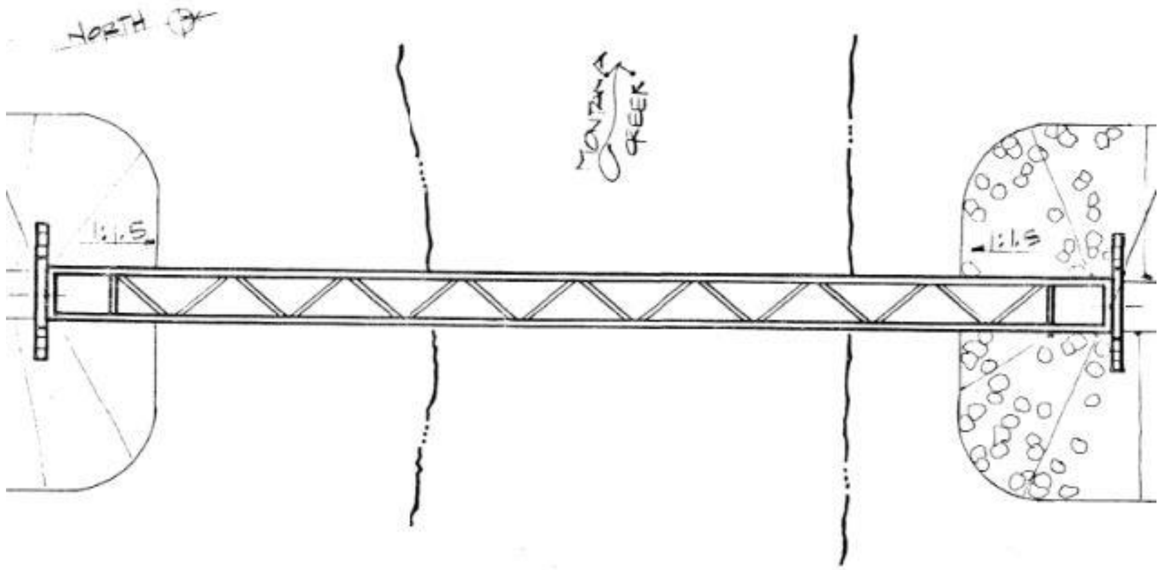


Figure 14 RELOCATED BRIDGE PLAN (SHOWN AT MONTANA CREEK)

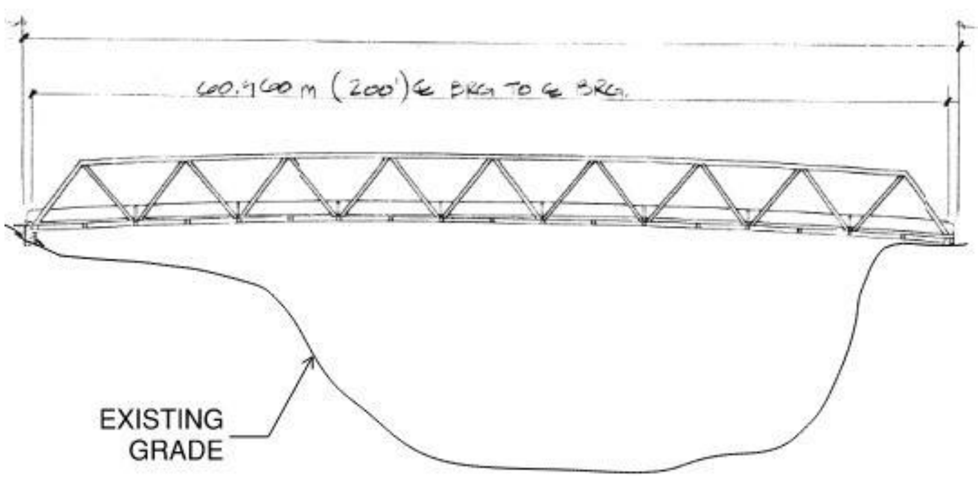


Figure 15 RELOCATED BRIDGE ELEVATION (GLACIER CREEK GRADE SHOWN)

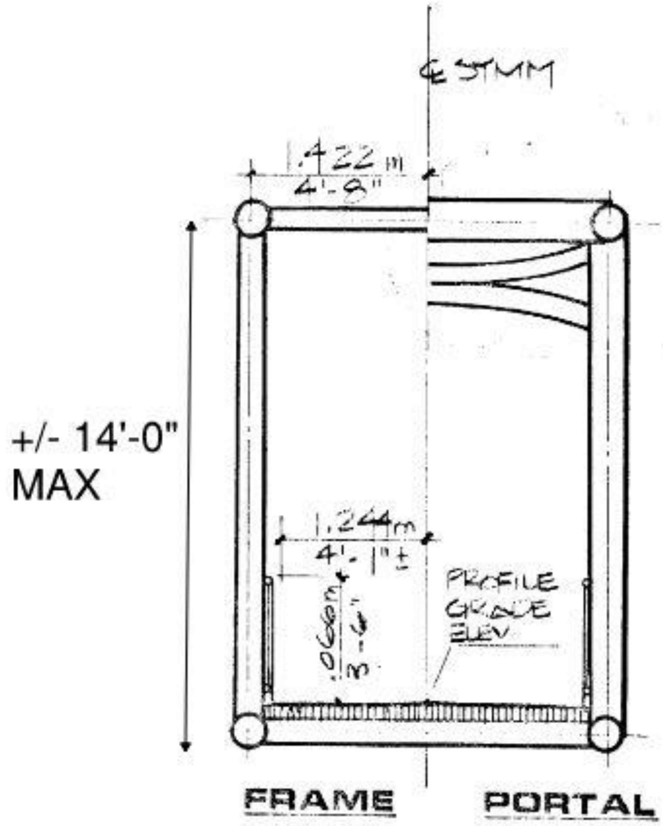


Figure 16 RELOCATED BRIDGE SECTION

5.0 Cost Estimates

5.1 Remote Construction Access

The remote environment of this project makes conventional bridge construction uneconomical. Conventional bridge construction method being bridge segments are prefabricated on- or off-site, transported to the project site via the road system, coupled together, and lifted into final position with heavy equipment. It was beyond the scope of this Feasibility Study to determine the preferred method of construction whether it be balanced cantilever, incremental launching, skyline rigging, etc. For a concept level evaluation, all bridge alternatives were assumed to use the services of a heavy lift helicopter to get the material on-site.

Heavy lift helicopters can work in remote areas when loads are unable to be transported via traditional means. Two heavy lift options were used individually and in combination to evaluate bridge costs. The first helicopter option was the Kaman K-Max with a lift payload of 5,800 pounds and the second was the Sikorsky S-64 Sky Crane with a lift capacity of 20,000 pounds. Typically helicopter service companies do not station sky crane resources in Alaska. If these larger helicopters are required, the project cost will include the very high mobilization/ demobilization cost that are in an order-of-magnitude between $400,000 and $500,000. This cost can be reduced if multiple projects within Alaska occur during the same season. Helicopter transport service can be refined as the design progress advances.

Another access challenge is the bridge site is located approximately 1 mile from the nearest road.

5.2 Lower Trail Crossing - 84' Steel Truss Bridge

A major challenge to the lower bridge crossing is the access to the crossing. The proposed bridge is on average 40 feet below the top of the gorge. Instead of climbing straight up the steep rocky gorge sidewalls, a "switchback" trail is proposed to make the ascend and descent hiking experience more manageable. The east and west switchback trails were evaluated using the following criteria:

- Target maximum grades: 10 to 12%
- 6-foot wide pathway, including handrail and safety buffer
- Cross slope: 2 to 5%
- Backslope: 0.5 (horizontal):1.0 (vertical)

The western switchback is approximately 400 feet in length requiring 1,300 cubic yards of rock and muck excavation and disposal. Rock excavation is assumed to involve blasting. Likewise, the eastern switchback trail is approximately 500 feet long with 2,700 cubic yards of excavation. Federal, State and local environmental regulations will not allow the excavated material to be disposed of within the floodplain of Glacier Creek. All clearing, grubbing, and excavated material must be hauled to an approved disposal site on top of the gorge or transported off-site. This is a major undertaking that makes the 84' lower bridge crossing one of the least desirable alternatives.

It was assumed a S-64 Sky Crane will be used to transport and place the bridge in two individual sections.

5.3 Upper Trail Crossing No. 1 - 180' Steel Truss Bridge

A steel truss bridge is commonly used where heavy equipment can be used to place it. With an estimated bridge weight of 100,000 to 120,000 pounds, the bridge is unable to be assembled and lifted by helicopter into place as a complete unit. For this project, the long span will be shipped in sections and coupled on-site. At minimum, the superstructure will be designed and constructed in 5 to 6 segments with each individual piece weighing under 20,000 pounds. The project complexity associated with assembling and installing the bridge in-place will be time and labor intense.

The new bridge will reuse the existing hand tram concrete abutments. The existing foundations will be retrofitted with additional concrete to accommodate the proposed bridge configuration.

5.4 Upper Trail Crossing No. 2 - 180' Suspension Bridge

According to the California State Parks *Trails Handbook (revised 2019)*:

> *Although a metal truss bridge can be purchased with a span over 200 feet long, this design is limited to use in sites with heavy equipment access. Generally, when the bridge span exceeds 120 feet, a suspension bridge becomes one of the most viable options, especially when the site is remote and not near a trailhead or road access.*

Remote long span suspension style bridges have three major benefits when compared to steel truss bridges - individual components can be packaged so a smaller, less expensive helicopter services can be used; estimated material weight is 25 to 35% of a steel truss bridge; and construction complexity is reduced. Keeping the maximum payload under 6,000 pounds allows the use of helicopter resources readily available in Alaska, thus greatly reducing the overall construction costs.

Similar to the 180’ steel truss bridge option, the new suspension bridge will retrofit the existing hand tram concrete abutments with additional concrete to accommodate the proposed bridge configuration.

5.5 Upper Trail Crossing No. 3 – Relocated Bridge

Although inspection reports indicate the Montana Creek bridge is in good condition, the existing bridge sections will need to be reconfigured to come under the 20,000-pound pick load for the S-64 helicopter. This will require both structural analysis and field fabrication retrofits. In addition, due to the length of the bridge, the existing hand tram concrete foundation are unable to be used as part of the bridge installation. New concrete abutments will need to be constructed.

Finally. it is difficult to determine the life expectancy of the existing Montana Creek bridge if it were to be rehabilitated and relocated. Using a 50-year design life for a new structure, the rehabilitated bridge is expected to realize an additional 25- to 30-year life span. The cost saving associated with relocating the bridge, the expected shortened life span, and the uncertainties related to moving, rehabilitating, and retrofitting an existing structure makes this alternative not recommended for further analysis.

5.6 Alternative Costs

Construction cost estimates in 2021 dollars for the various bridge types and their respective alignments are shown in the table on the following page. The estimated total project costs are provided to aid project budget planning and preparation. Refined details for construction cost estimates can be found in Appendix E.

Alignment Options	Lower Trail Crossing 84' Steel Truss Bridge	Upper Trail Crossing No. 1 180' Steel Truss Bridge	Upper Trail Crossing No. 2 180' Suspension Bridge	Upper Trail Crossing No. 3 Relocated Bridge
Construction Costs				
Contingency (30%)				
Design (30%)				
Inspection (15%)				
Estimated Project Total				

Figure 17 ALTERNATIVE COSTS

6.0 Comparison of Alternatives

Alignment Options	Lower Trail Crossing 84' Steel Truss Bridge	Upper Trail Crossing No. 1 180' Steel Truss Bridge	Upper Trail Crossing No. 2 180' Suspension Bridge	Upper Trail Crossing No. 3 Relocated Bridge
Overall Bridge Length	84'	180'	180'	180'
Landing Length (Access)	900'	Minimal	Minimal	Minimal
Environmental Impacts (existing trees, vegetation)	High	Mid	Low	Mid
Construction access, staging and constructability	High	Mid	Low	Mid
Estimated Project Cost				

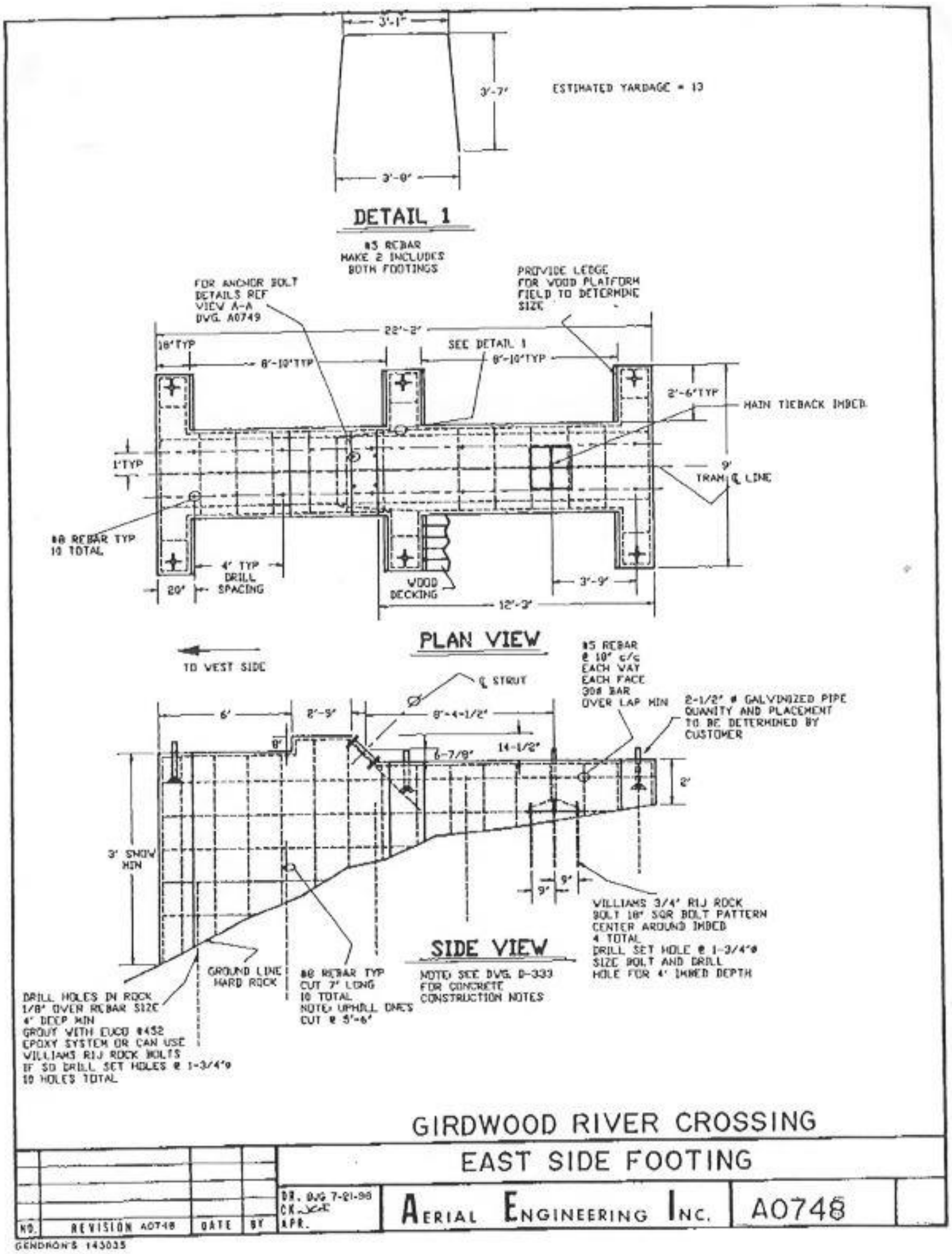
Figure 18 ALTERNATIVE COMPARISON

7.0 Conclusions

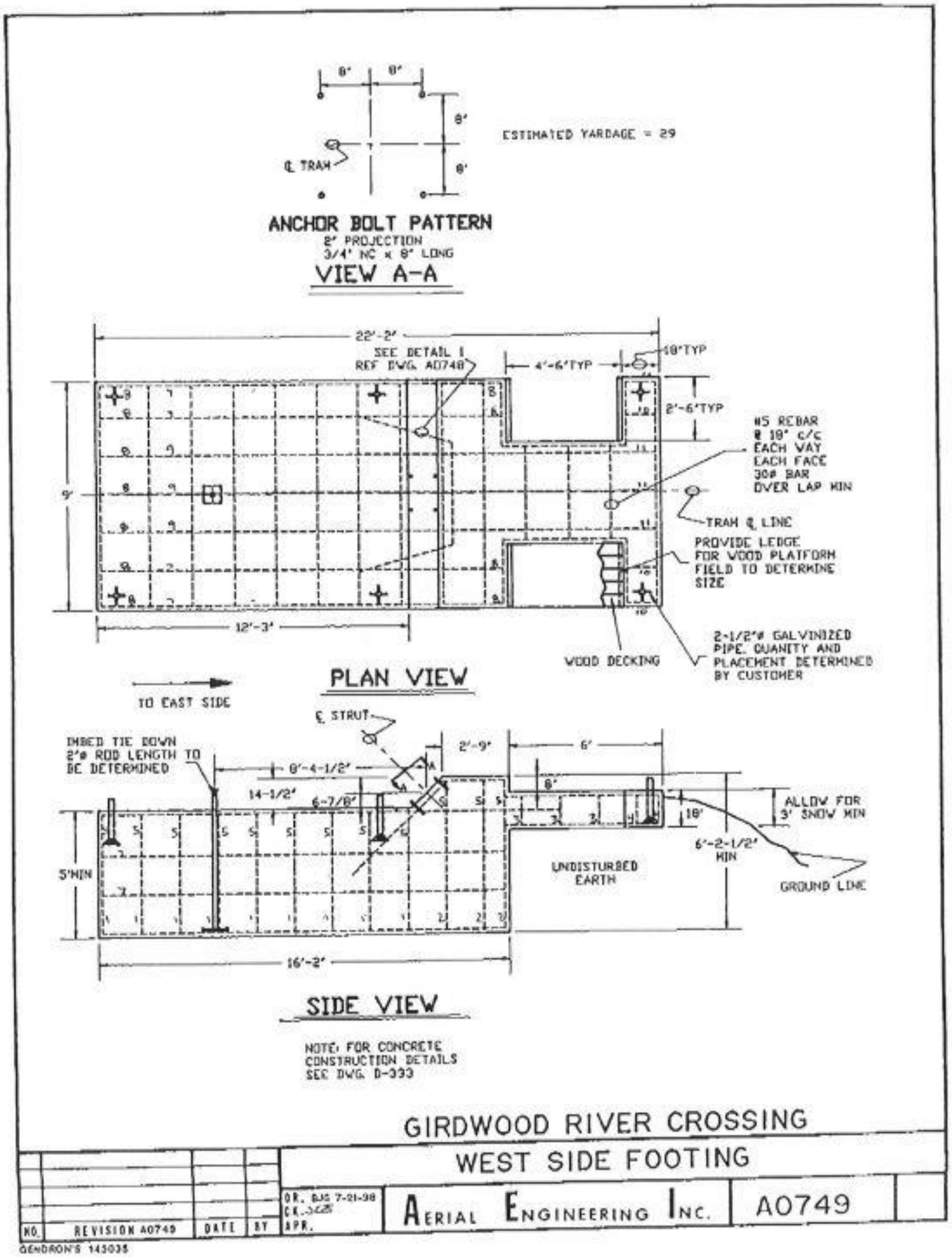
7.1 Summary

This Feasibility Study provides schematic level design for four (4) bridge options for the Winner Creek Trail crossing at Glacier Creek. All options would provide a safe, durable, year-round crossing of the creek.

Appendix A – Hand Tram Foundation Details

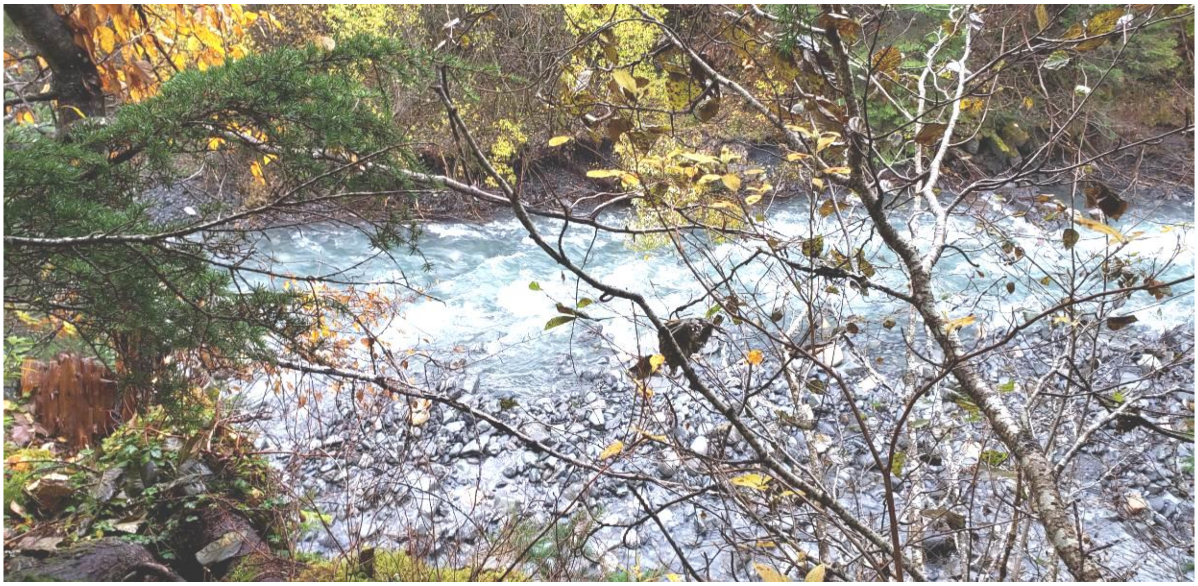


3'-1"
3'-7"
ESTIMATED YARDAGE = 13
3'-8"
DETAIL 1
#5 REBAR MAKE 2 INCLUDES BOTH FOOTINGS
FOR ANCHOR BOLT DETAILS REF VIEW A-A DWG. A0749
PROVIDE LEDGE FOR WOOD PLATFORM FIELD TO DETERMINE SIZE
22'-2"
SEE DETAIL 1
MAIN TIEBACK IMBED
TRAM ℄ LINE
#8 REBAR TYP 10 TOTAL
4" TYP DRILL SPACING
WOOD DECKING
12'-3"
PLAN VIEW
TO WEST SIDE
℄ STRUT
#5 REBAR @ 10" c/c EACH WAY EACH FACE 30# BAR OVER LAP MIN
2-1/2" # GALVINIZED PIPE QUANITY AND PLACEMENT TO BE DETERMINED BY CUSTOMER
3' SNOW MIN
WILLIAMS 3/4" RIJ ROCK BOLT 18" SQR BOLT PATTERN CENTER AROUND IMBED 4 TOTAL DRILL SET HOLE @ 1-3/4"ø SIZE BOLT AND DRILL HOLE FOR 4' IMBED DEPTH
SIDE VIEW
GROUND LINE HARD ROCK
#8 REBAR TYP CUT 7' LONG 10 TOTAL NOTE: UPHILL ONES CUT @ 5'-6"
NOTE: SEE DWG. D-333 FOR CONCRETE CONSTRUCTION NOTES
DRILL HOLES IN ROCK 1/8" OVER REBAR SIZE 4' DEEP MIN GROUT WITH EUCO #452 EPOXY SYSTEM OR CAN USE WILLIAMS RIJ ROCK BOLTS IF SO DRILL SET HOLES @ 1-3/4"ø 10 HOLES TOTAL
GIRDWOOD RIVER CROSSING
EAST SIDE FOOTING
AERIAL ENGINEERING INC.
A0748
NO. REVISION A0748 DATE BY
GENDRON'S 143035

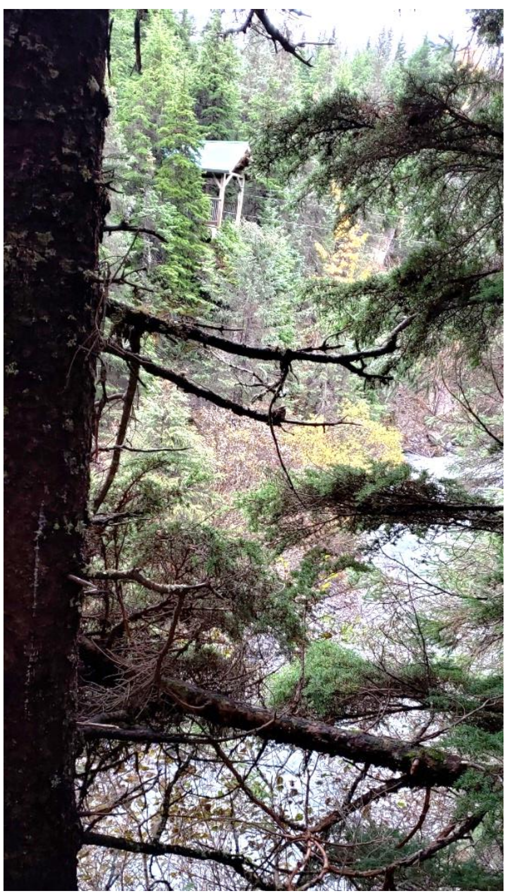

ESTIMATED YARDAGE = 29
ANCHOR BOLT PATTERN
2" PROJECTION
3/4" NC x 8" LONG
VIEW A-A
SEE DETAIL I
REF DWG. A0748
#5 REBAR
@ 18" c/c
EACH WAY
EACH FACE
30ø BAR
OVER LAP MIN
TRAM ℄ LINE
PROVIDE LEDGE
FOR WOOD PLATFORM
FIELD TO DETERMINE
SIZE
2-1/2"ø GALVINIZED
PIPE. QUANITY AND
PLACEMENT DETERMINED
BY CUSTOMER
WOOD DECKING
PLAN VIEW
TO EAST SIDE
℄ STRUT
IMBED TIE DOWN
2"ø ROD LENGTH TO
BE DETERMINED
ALLOW FOR
3' SNOW MIN
UNDISTURBED
EARTH
GROUND LINE
SIDE VIEW
NOTE: FOR CONCRETE
CONSTRUCTION DETAILS
SEE DWG. D-333
GIRDWOOD RIVER CROSSING
WEST SIDE FOOTING
AERIAL ENGINEERING INC.
A0749
REVISION A0749
GENDRON'S 143035

Appendix B – Site Photos

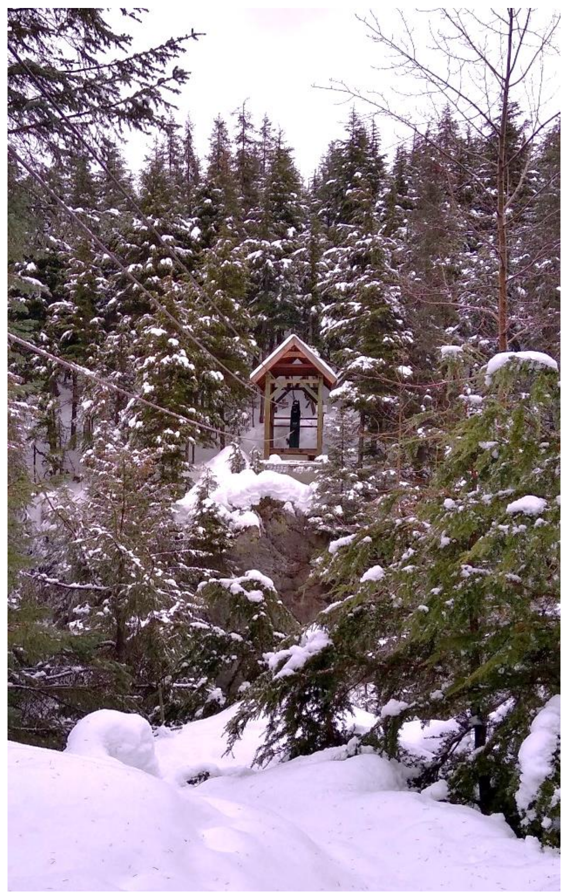
The following section contains photos from site visits on October 7, 2020 and February 18, 2021.



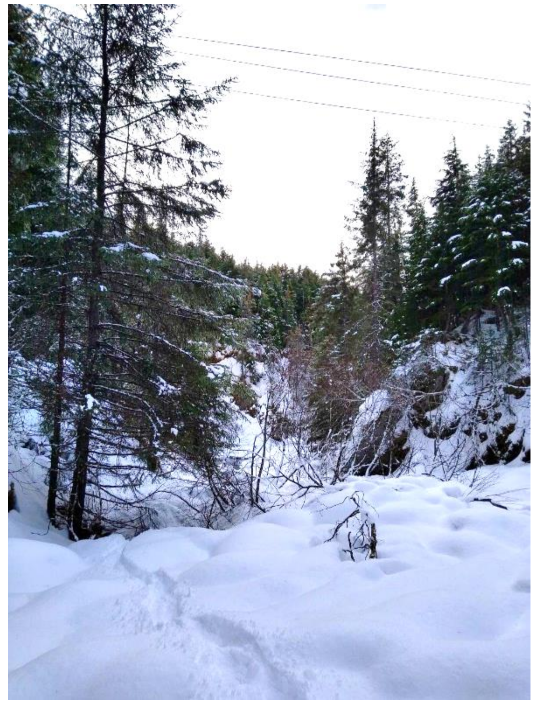
Glacier Creek (Oct 2020)



Existing Tram, Looking Southeast (Oct 2020)



Existing Tram, Looking East (Feb 2021)



Existing Tram, West Side (Feb 2021)

Existing Tram Rope (Feb 2021)

Appendix C – Geotechnical Investigation

The following section contains the findings of a limited geotechnical investigation.

SUBMITTED TO:
Municipality of Anchorage
Department of Public Works
4700 Elmore Road
Anchorage, Alaska 99507

BY:
Shannon & Wilson, Inc.
5430 Fairbanks Street, Suite 3
Anchorage, Alaska 99518

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AECC125

GEOTECHNICAL REPORT

Glacier Creek Crossing

GIRDWOOD, ALASKA

April 2021

Shannon & Wilson No: 106204-001

SHANNON & WILSON

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Submitted To: Municipality of Anchorage
Department of Public Works
4700 Elmore Road
Anchorage, Alaska 99507
Attn: Mr. Timothy Huntting

Subject: GEOTECHNICAL REPORT, GLACIER CREEK CROSSING, GIRDWOOD, ALASKA

Shannon & Wilson prepared this report and participated in this project as a consultant to the Municipality of Anchorage (MOA). Our scope of services was specified in our proposal dated October 10, 2020 and authorized via Purchase Order 2021000339 from Mr. Timothy Huntting, Contract Manager for MOA, on February 2, 2021. This report presents the results of our surface reconnaissance and was prepared by the undersigned.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON, INC.

Katra Wedeking, CPG
Senior Geologist

Kyle Brennan, PE
Vice President

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2 Site and Project Description 1
3 Geologic setting 2
4 Field Explorations .... **Error! Bookmark not defined.**
4.1 Surface Reconnaissance .... **Error! Bookmark not defined.**
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Figures

Figure 1: Vicinity Map
Figure 2: Site Plan
Figure 3: Site Photos

Appendices

Important Information

1 INTRODUCTION

This letter presents preliminary geotechnical design considerations for a proposed bridge crossing over Glacier Creek in Girdwood, Alaska. The new bridge crossing will replace the existing hand tram that currently crosses the gorge which has been closed for safety reasons. Our work includes conducting a site reconnaissance, review of existing information, and preparation of geotechnical design conditions for several bridge crossing options. Our information will be used to complete a feasibility study for the new bridge crossing that will also include input from structural and civil engineers. Our work was conducted in general conformance with our October 10, 2020 proposal.

2 SITE AND PROJECT DESCRIPTION

The project site as shown on Figure 1, is located on Glacier Creek between its confluence with Winner Creek and Crow Creek in an area where the creek flows through a narrow gorge. At this location, the creek flows roughly north to south, but the average trend of the creek in the valley is northeast to southwest. At the existing tram crossing, the gorge is approximately 50 feet deep and flanked on both sides by naturally steep slopes with exposed bedrock. The existing tram includes a hand-operated cart that travels along a 240-foot long cable, spanning the 140-foot wide gorge that contains Glacier Creek. The east and west terminals of the existing tram consist of timber and steel structures founded on monolithic concrete foundations. The site is accessed from the west by a trailhead on Crow Creek Road for the Iditarod National Historic Trail (approximately 1 mile from the site) and from east via the Winner Creek Trail with a trailhead at the Alyeska Hotel (approximately 2.5 miles from the site).

We understand that the purpose of this geotechnical study is to support a feasibility study for a new bridge crossing over Glacier Creek. The new bridge is to be located at the approximate location of the existing hand tram, but the precise location and configuration of the bridge is yet to be determined. Currently being considered is a lower crossing that would consist of a conventional steel bridge and an upper crossing that could consist of a conventional steel bridge or a suspension/cable bridge. Regardless of bridge type or location, we understand that the structure will be single span and will be intended to support pedestrian traffic and possibly small maintenance traffic (such as 4-wheelers or other lightly loaded vehicles).

3 GEOLOGIC SETTING

The project site lies within a flat, glacially-carved valley near the head of Turnagain Arm. Bedrock in the Girdwood area consists of a complex mixture of marine sedimentary rocks and igneous rocks. These rocks have been intensely deformed and metamorphosed by high temperature and pressure during the Chugach Mountain building processes and accretion from tectonic activity. Depth to bedrock ranges from exposure in the mountains (and in the valley) to over several hundred feet below the surface in wider river valleys and tidal areas. The entire sequence is known as the Valdez Group, and represents shallow to deep marine facies, which are characterized by shales, slate, argillite, and greywackes. Overlying the Valdez Group is a package of unconsolidated sediments of glacial and fluvial origin. Regionally, several major streams, including Glacier Creek, California Creek, and Virgin Creek, have created a thick package of alluvium that is complexly interbedded with the glacial deposits. The thickness of these deposits can vary significantly over short horizontal distances, however, the thickness of these deposits generally increases at lower elevations.

Seismicity in the area is dominated by the Aleutian Megathrust, where the Pacific Plate dives under the North American Plate. The largest sources of seismicity in the megathrust are along the Benioff Seismic Zone, between 30 and 100 kilometers below the ground surface. This complex is capable of producing large scale earthquakes of magnitude up to M9.2 with long period, strong ground shaking. Associated with this tectonic feature are many secondary faults and shear zones, some of which are visible on the ground surface.

The climate is predominantly cool maritime with mild winters, cool summers, and very heavy precipitation. Average annual precipitation is about 28 inches and average annual temperature is about 38 degrees Fahrenheit (F) with a mean January temperature of about 14 degrees F and a mean July temperature of almost 56 degrees F.

4 SURFACE RECONNAISSANCE

Field activities consisted of conducting a ground surface reconnaissance at the proposed crossing location. The locations of various field activities, rock mapping locations, and general observation points, shown on Figure 2, were recorded using a handheld Global Positioning System (GPS). Therefore, all locations provided for this project should be considered approximate.

On February 17, 2021, two representatives from Shannon & Wilson's Anchorage office geotechnical group conducted surface reconnaissance at the bridge abutment locations. The goal of the surface reconnaissance was to observe the general surface conditions at the site to evaluate geotechnical aspects that should be considered during the feasibility. While onsite, we also evaluated the areas around the existing hand tram terminals and in the general vicinity of the crossing. Note that snow cover on the order of several feet was present in the flat lying areas making direct observations of ground cover conditions impossible during our site visit.

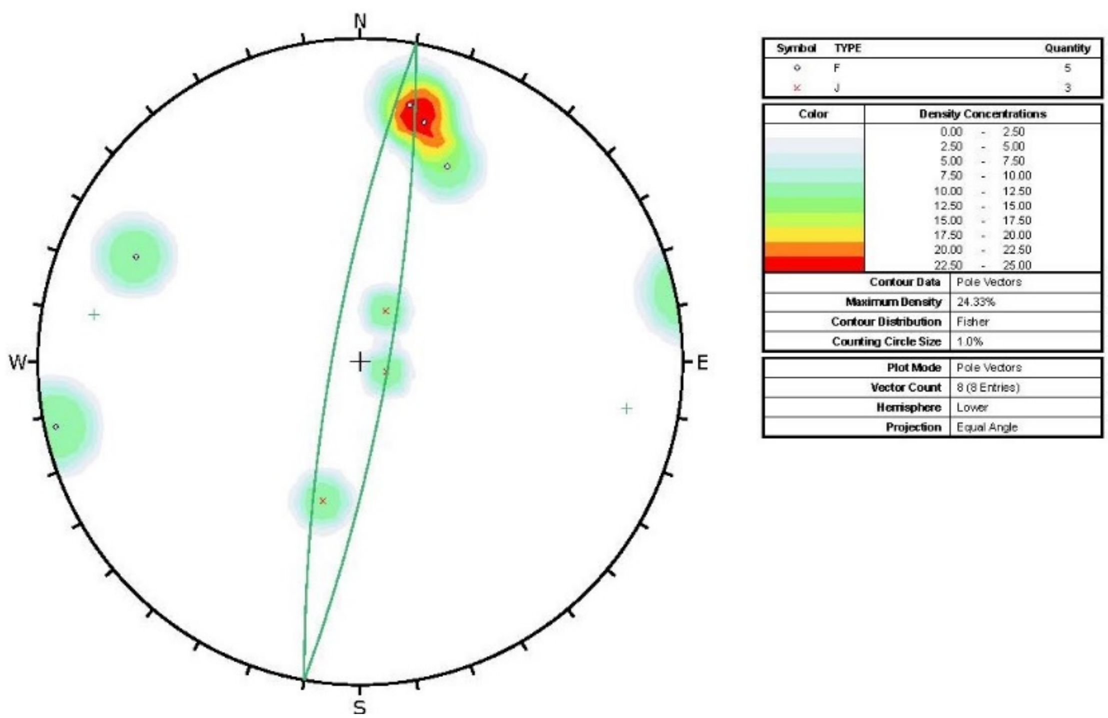
In general, the reconnaissance consisted of travelling to the site on foot. Location control in the field was maintained using a handheld GPS. While at the crossing site, we conducted rock structure mapping at several locations where in-tact bedrock was exposed at the ground surface. We collected rock mass structure information using the cell mapping technique as described by Hustrulid and others, 2000. This method includes the collection of structure (e.g. bedding, foliation, shear zones, joint sets, etc.) as well as other information such as feature length, persistence, separation, and roughness to characterize the rock mass for the purposes of slope stability evaluation and designing slope stabilization. The approximate mapping points are indicated on the site plan in Figure 2. A Stereo plot of the collected structure measurements are presented below in Section 5.

5 SITE OBSERVATIONS

Conditions observed at the site are included in the sections below. Photographs from our site visits are included in Figure 3 (Sheets 1 through 3). In general, vegetative cover at both bridge sites consisted of relatively dense spruce and alder trees. The ground surface at each site was not directly observable during our site visit, but based on prior experience in the area, we believe that the ground surface is covered with a relatively thin layer or organic material, generally firm, capable of supporting foot traffic, and well drained.

Rock exposure was present in the gorge slopes on both sides of the crossing, suggesting that if mineral soils exist under organic materials and over bedrock, it is relatively thin. Original design drawings of the existing tram terminals indicate that the east crossing structure is bearing directly on sloping bedrock and connected to rock with dowels. This information along with prior experience in the area suggests that soil overburden above the east side of the gorge is likely less than approximately 1 to 2 feet. The design drawing for the west tram terminal appears to show the foundation bearing on soil without positive connections to rock. If the drawing represents the as-built conditions, the west tram terminal is in an area where the soil overburden is at least 5 feet thick. If this condition exists at the west terminal, it is likely representative of an isolated area of thicker soil overburden and we do not anticipate soils in this area to be significantly thicker than 5 feet. Based on our observations around the west tram terminal and exposure on slopes below the terminal, soil overburden thickness on the west side of the gorge is likely less than 5 feet on average.

Rock exposure in the slopes on both sides of the gorge indicate that bedrock in the area consist of relatively competent slate. The gorge slopes currently stand at variable angles with the steepest slopes at or near vertical. The rock structure is dominated by foliation planes of the slate, which vary in spacing from less than one inch to up to 1 foot. Based on our structural measurements, there may be significant folding in the area as we observed a wide range of dip orientations (up to approximately 90 degrees difference). Secondary joint sets were observed roughly orthogonal to the controlling foliation with significantly wider spacing, giving the rock a relatively platy appearance. Generally speaking, foliation and joint structure observed in the rock exposure appeared to be relatively smooth and tight. We did not observe significant zones of seepage from the rock face during our site visit. A stereo pole plot of the structure measured during our site visit is included below. Note that the plot is a compilation of all structure measurements collected at the Observation Points (G1 through G4) shown on the site plan in Figure 2. Because of the observed variability, additional measurements and evaluation would need to be conducted to determine the full nature of the controlling rock structure at this site.



Structure Measurements – All structure orientations relative to magnetic north. Green great circles represent average gore slope orientations in immediate project area.

At the time of our visit, the bottom of the gorge was covered with snow and the surface conditions were not observable. Based on an earlier site visit made in the fall prior to the project, the gorge bottom appeared to be relatively flat with the river comprising approximately half of the total base of the gorge. The portions of the gorge bottom not submerged consisted of sand and gravel bars several feet above the water surface with sporadic alder vegetation growth on the higher, more stable bars. Though not observed during our site visits, it is likely that during periods of heavy rain or snow melt, the entirety of the gorge bottom is submerged. The alluvial deposits at the gorge bottom appear to consist of sand and gravel that is typically less than approximately 6 inches in diameter. Isolated areas of coarser material were observed with boulders up to approximately 2 feet in diameter. The thickness of these deposits is unknown, but it is likely less than 20 to 30 feet near the center of the gorge.

6 GEOTECHICAL CONSIDERATIONS

We anticipate that the information in this report will be used for creating conceptual designs for the crossing alternatives. Once a preferred crossing approach is determined, additional geotechnical explorations and analyses will be required to support final design. The design will need to consider the bearing support capabilities of the subgrade materials. For upper crossing alternatives, establishing setbacks from existing slopes or incorporating stabilization sufficient to address abutment fore-slope stability will also be needed. Based on our observations, we anticipate that the foundations of upper crossing alternatives will likely be founded on and/or connected directly to bedrock. Depending on the lower crossing alternative location, foundations will likely need to accommodate shallow rock, but may also need to consider overburden soil impacts.

6.1 Seismic Conditions

Estimation of the peak ground acceleration (PGA) and spectral accelerations provides parameters for the project design. These values may be estimated for the project site based upon regional seismicity studies performed by others, from a site-specific seismicity study, or applicable building codes/local standards of practice. To adhere to Chapter 3 of the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, a single ground motion was selected. Basal layer (or bedrock) motions at the site, in the form of PGA, were estimated from mapped PGA values provided on Figure 3.10.2.1-17 of AASHTO LRFD Bridge Design Specifications for a 7 percent probability of exceedance in 75 years (1,000-year return period).

According to the AASHTO Table 3.10.3.1-1, the upper and lower crossing locations should be considered Site Class B for rock (shear wave velocity of 2,500 to 5,000 feet per second [fps]). Note that these classifications are based on our visual observations of surface and near surface conditions at the site and our understanding of regional geology. It is possible that soils in the gorge bottom may be thick enough to warrant a Site Class C or D for the lower crossing location may be appropriate, however, more investigation is needed to confirm conditions. Seismic coefficients based on Site Class B are presented in the table below.

Seismic Design Coefficients

Seismic Coefficient (Site Class B)	Value	Source
Site Class B Acceleration Coefficient, (PGA)	0.52	AASHTO Figure 3.10.2.1-17
Spectral Acceleration Coefficient at Period of 0.2s, (S_s)	1.19	AASHTO Figure 3.10.2.1-18
Spectral Acceleration Coefficient at Period of 1.0s, (S_1)	0.50	AASHTO Figure 3.10.2.1-19
Site Factor at Zero Period, (F_{pga})	1.0	AASHTO Figure 3.10.3.2-1
Site Factor for Short Period, (F_a)	1.0	AASHTO Figure 3.10.3.2-2
Site Factor for Long Period, (F_v)	1.0	AASHTO Figure 3.10.3.2-3

6.2 Foundation Recommendations

Based on our site observations, we believe that bridge foundations for the upper crossing alternatives will bear directly on bedrock. Foundations at the lower crossing alternatives could bear on alluvial mineral soils or on bedrock depending on the location of the crossing and depth of alluvium.

6.2.1 Upper Crossing Alternatives

It is our opinion that the foundation and slope conditions for foundations associated with the upper crossing alternatives are favorable, however adequate setback from the crest of the rock slope below the abutments should be confirmed for final placement. Assuming strip footings bearing directly on rock will be used to support the crossing, we recommend assuming a setback for the gorge-side edge of the footing of approximately 10 feet from the rock slope crest. These setbacks are based on our observations of rock structure and slope height in the slopes below the abutment. The dominant jointing on both sides of the creek appears to be steeply dipping and kinematically admissible failures appear to consist of toppling on the east side and planar and wedge failures on the west side. Further analysis will be required once a preferred crossing type is identified, a crossing location is selected, and foundation loading requirements are determined. It is possible that greater setbacks may be required or fore-slope stabilization may be needed.

Given the above recommended setbacks and assuming the footing bears directly on a clean, non-weathered rock surface, for preliminary purposes we recommend assuming an unfactored bearing resistance of rock to be approximately 20 kips per square foot (ksf) and a minimum footing width of 2 feet. Resistance to lateral loading and uplift forces on the upper crossings will be gained by connecting the foundation footing to the rock through tensioned rock anchors. The actual configuration of the rock anchors will depend on the structural design of the abutment foundations. Designing the tensioned rock anchors (i.e. diameter of the rods and pre-tension loads) will depend on the magnitude of uplift and shear loading on the foundation, which are not known at the time of this report. For planning purposes, we recommend assuming 1.5 to 3-inch threaded bars will need to penetrate a minimum of 20 feet below the foundation with a minimum free-bonded length of 10 feet. Friction resistance along the base of the footings can be estimated using a friction coefficient of 0.4 between concrete and rock.

The actual configuration and design of the foundations and anchors will require additional engineering analysis once a conceptual bridge design and loading requirements are determined. The anchors should incorporate the appropriate corrosion protection to ensure that they maintain capacity over the life of the structure.

6.2.2 Lower Crossing Alternatives

Depending the location of the lower crossing, there could be several suitable foundation approaches to the abutments. Shallow foundations could be used bearing on rock if the abutments are located against the edges of the gorge. We believe the preliminary guidance in Section 6.2.1 for the upper crossings is appropriate for shallow foundations bearing on rock at the lower crossing alternatives. However, the recommended slope setbacks will not be needed since the abutments are likely to be at or very close to the bottom of the gorge with a minimal foreslope in front of the foundations.

Shallow foundations bearing on alluvium or driven pile foundations could be used if the abutments are located away from the edges of the gorge. If foundations bear on alluvium, special consideration will be needed for accommodating potentially liquefiable soils and significant scour conditions during periods of high water. If pile foundations are used, it is likely that they will need to be connected in some way to bedrock as alluvium in the gorge bottom is likely too thin to accommodate lateral and uplift loading. For the purposes of this report, we assume that some form of deep foundation will be used on lower crossing alternatives due to the anticipated poor soil and scour conditions likely to exist in the bottom of the gorge.

Deep foundations for lower crossings will likely consist of open-ended driven pipe piles that could range in size from 8 to 24 inches in diameter depending on final design and latera/axial loading. For planning purposes, we recommend assuming that piles will need to be driven through alluvial soils and will need to be socketed into rock. The thickness of the alluvial soils is unknown, but for planning purposes we recommend assuming a thickness of approximately 20 feet. Additional depth into competent bedrock will be required for lateral and uplift resistance. If conventional socketing techniques are used (i.e. drilling beyond the pile tip and advancing a concrete shaft below the pipe pile) it is likely that lateral and uplift capacities will be achieved with approximately 10 feet of embedment into rock. Note that significant additional geotechnical explorations and engineering evaluation is needed to determine the required configuration of pile foundations for lower crossing alternatives.

6.3 Rock Cut Slopes

Rock cuts may be required, especially if a lower trail crossing is selected to establish access from existing trials to the gorge bottom. Establishing trail access to the gorge bottom will likely require benching a new trail into the gorge slopes. Based on our experience in the area and observations on site, we believe that the gorge slopes contain minimal organic and mineral soil overburden. Additionally, we believe that gorge slopes north of the existing tram crossing provide the most favorable conditions for establishing new trails. Establishing new benches for the trail should be achievable using conventional drill and blast techniques. For planning purposes, we recommend establishing a setback of at least 2 feet from the edge of the trail to the edge of slope to allow for a safety buffer and establishing a railing. Additional space for catchment of rockfall should be included on the upslope side of the bench. The width of rockfall catchment will depend on the height of the cut slope above the bench, but we believe that 2 to 4 feet should be sufficient for planning purposes. We recommend assuming a maximum rock cut slope angle of ¼ horizontal (H) to 1 vertical (V) in rock. Additional geotechnical analysis will be needed once trail alignments are established to determine appropriate rock cut slope angles and stabilization measures if needed.

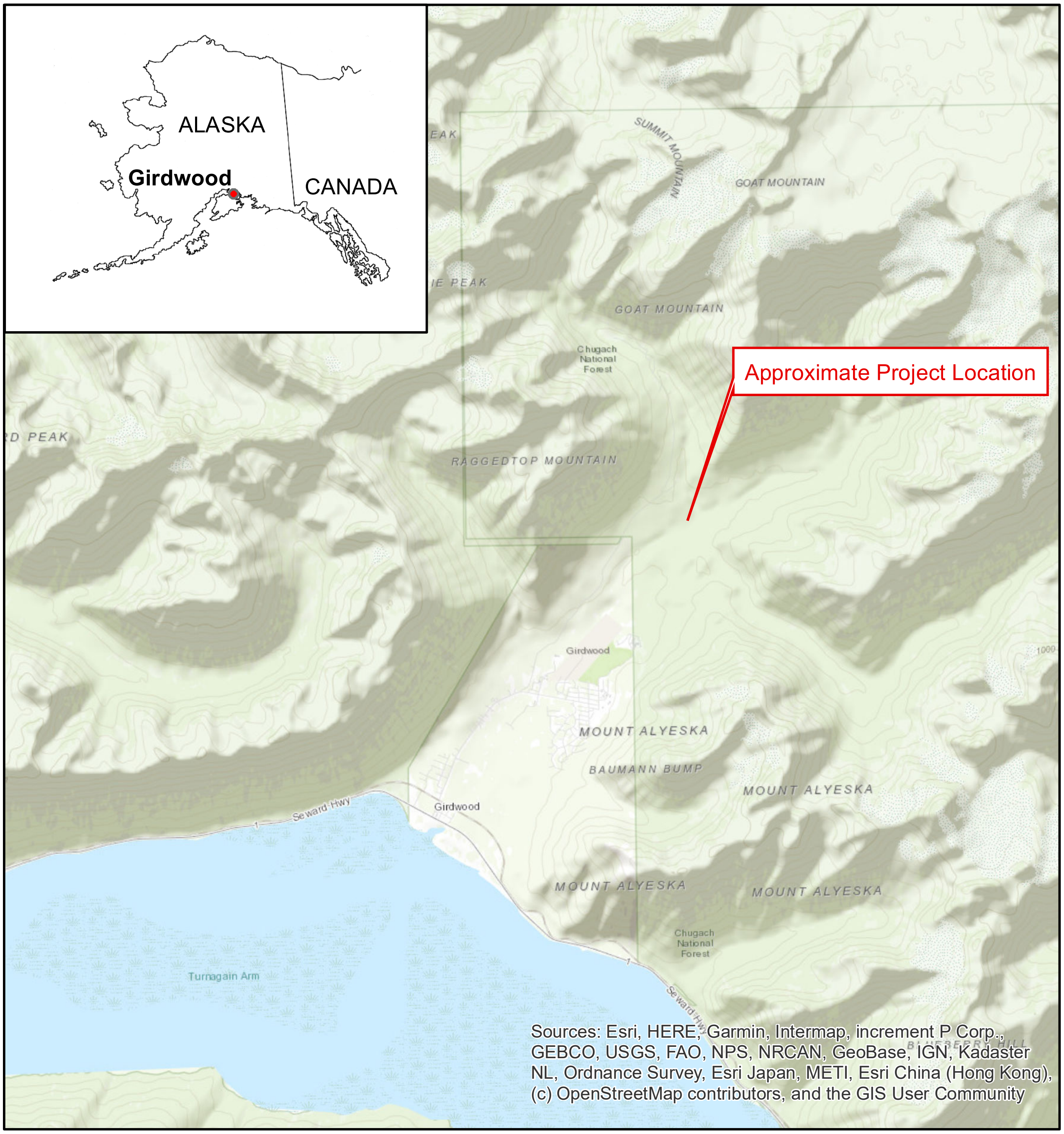
7 CLOSURES AND LIMITATIONS

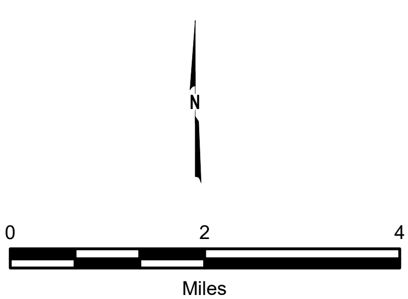
This report was prepared for the exclusive use of our client and their representatives for evaluating the site as it relates to the geotechnical aspects discussed herein. The conclusions contained in this report are based on site conditions as they presently exist. It is assumed that our observations are representative of the subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those interpreted from our surface observations.

If, during construction, subsurface conditions different from those inferred from our surface observations and described herein are observed or appear to be present, Shannon & Wilson, Inc. should be advised at once so that these conditions can be reviewed and recommendations can be reconsidered where necessary. If there is a substantial lapse of time between the submittal of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, it is recommended that this report be reviewed to determine the applicability of the conclusions and interpretations considering the changed conditions and time lapse.

Unanticipated conditions are commonly encountered and cannot fully be determined by merely making surface observations. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs. Shannon & Wilson has prepared the attachment *Important Information About Your Geotechnical/Environmental Report* to assist you and others in understanding the use and limitations of the reports.

Copies of documents that may be relied upon by our client are limited to the printed copies (also known as hard copies) that are signed or sealed by Shannon & Wilson with a wet, blue ink signature. Files provided in electronic media format are furnished solely for the convenience of the client. Any conclusion or information obtained or derived from such electronic files shall be at the user's sole risk. If there is a discrepancy between the electronic files and the hard copies, or you question the authenticity of the report please contact the undersigned.





Glacier Creek Crossing
Girdwood, Alaska

VICINITY MAP

April 2021 106204-001

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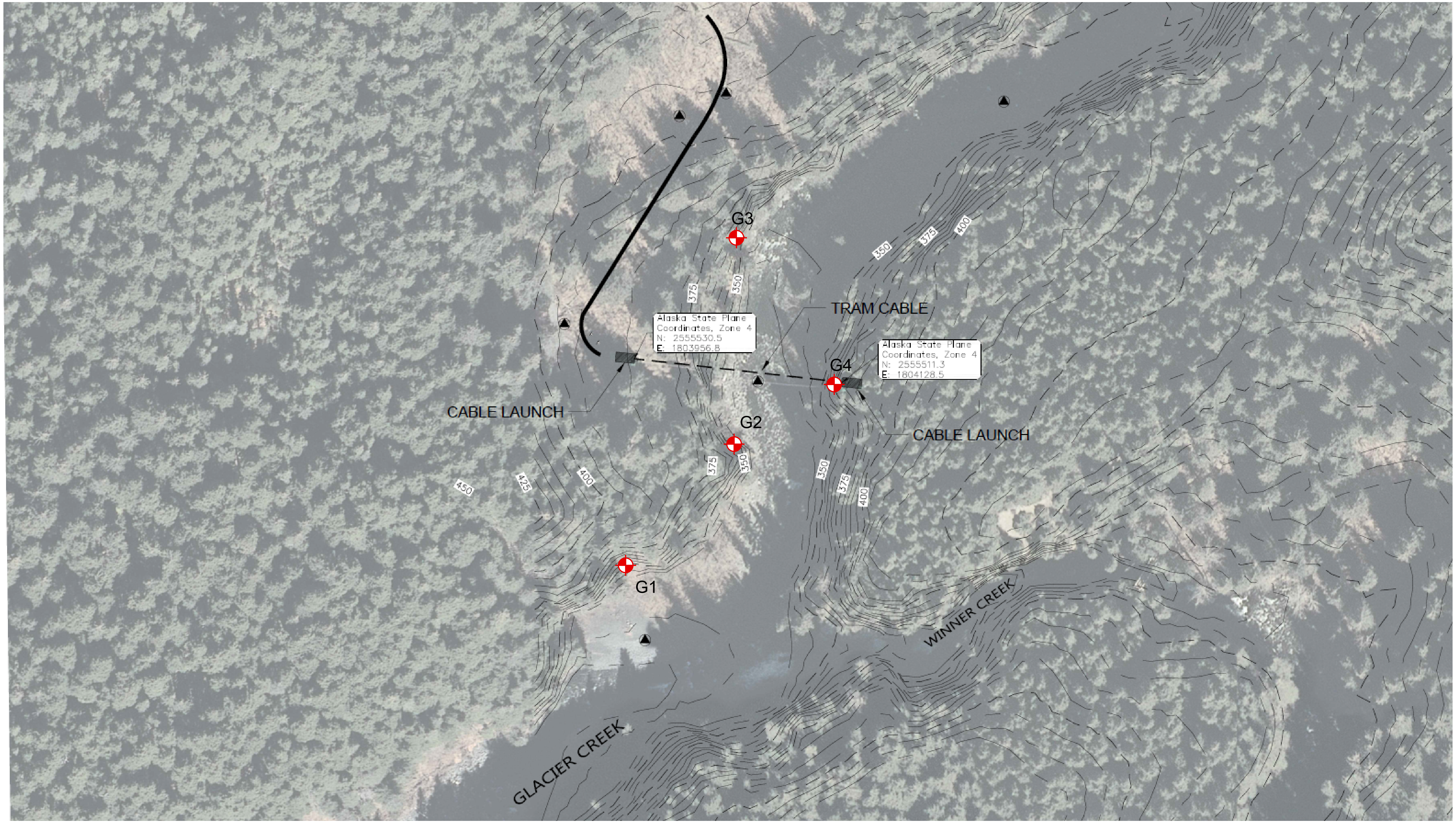
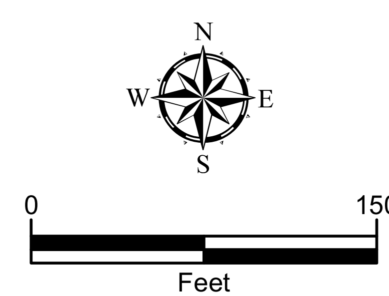
FIG. 1



Legend

Approximate Location of Observation Point G1 by Shannon & Wilson Inc. on Month Day, Year

G1



Glacier Creek Crossing
Girdwood, Alaska

SITE PLAN

April 2021 | 106204-001

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GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 2

Photo 1: Rock exposure at Observation Point G1

Photo 2: Rock exposure structural mapping at Observation Point G1.

Photo 3: Rock exposure at Observation Point G2.

Photo 4: Rock structure mapping at Observation Point G3.

Photo 5: Rock exposure mapping at Observation Point G4. Note east hand tram terminal at top of slope.

IMPORTANT INFORMATION

Important Information

About Your Geotechnical Report

IMPORTANT INFORMATION

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent

IMPORTANT INFORMATION

such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

Appendix D – Preliminary Structural Calculations

The following section contains calculations used to form the conclusions stated in this report.

Steel Truss Bridge - 84 ft span

Ref: AASHTO Ped Bridges and AASHTO LRFD (2020)

Determine size of HSS longitudinal members

PL =	90	psf (pedestrian loading, PL)
DC =	5	psf
DW =	5	psf
F_y =	46	ksi
w =	60	in
Span =	84	ft
h =	72	in (vertical dim, center to center of HSS)
L =	72	in, length between panel points
n =	14	number of panel points

Member:	HSS5x5x3/8	
A_g =	6.18	in^2
r_s =	1.87	in
b/t =	11.3	
Z =	10.6	in^3
I =	21.7	in^4
E =	29000	ksi

Strength I load cominbation: 1.75PL + 1.25DC + 1.5DW

Reference AASHTO LRFD for box section design

Mu =	755212.5	lbft
Mu/d =	125868.8	lb
1/2 * Mu/d =	62934	lb (= Tu = Cu)

ϕ = 0.95 (axial compression and tension yielding)

Tension, AASHTO 6.8.2

$\phi P_n = \phi\ F_y\ A_g > Tu$

Solve for A_g:

Min A_g = 1.44 in^2

DCR = 0.23

Compression, AASHTO 6.9.2 & 6.9.3, and Ped Bridge 7.1

$$C = \frac{E}{h^2[(h/3I_c) + (b/2I_b)]}$$

Ref Ped Bridge pg. 22 and Galambos, 1968

C =	2.25	k/in (assumes all members same section)
P_c =	83.7	k (1.33 x factored compressive load)
CL/P_c =	1.93	
n =	14	
1/K =	0.9	from Ped Bridge Table 7.1.2-1

Check slenderness

KL/r < 120	main members (6.9.3)
42.78075 <120	
Kl/r < 140	bracing members (6.9.3)
42.78075 <140	

ϕP_n:

$P_o = F_y A_g =$ 284.3 kip

$$P_e = \frac{\pi^2 E}{\left(\frac{Kl}{r_s}\right)^2} A_g$$

K = 1.111111

P_e = 966.47 k

Check slenderness per AASHTO Table 6.9.4.2.1-1

b/t =	11.3
limit =	35.2
slender?	no

P_o/P_e = 0.3

P_n = 251.35 kip eqn 6.9.4.1.1-1

ϕP_n = 238.78 kip

DCR = 0.26

Lateral force on post shall not be less than 0.01/K x average factored design compressive force

Check 0.01/K > 0.003 Section 7.1.1

0.01/K = 0.009 >0.003

if 0.01/K < 0.003, use 0.003

force = 566.4 lb (use max force rather than average, conservative)

min force = 188.8 lb

design force = 566.4 lb

quick check (does not include axial)

cantilever moment = force*h

factored moment = 3398.5 lbft

ϕ_f = 1.0

$\phi_f M_n$ = 487.6 kin

$\phi_f M_n$ = 40633.3 lbft

DCR =	0.084

Deck design:

1 3/4" x 1/8 bearing bars (grating pacific load tables)
19-W-4
Serrated and hot dip galvanized

Horizontal deck beams:

trib width = 6 ft

See enercalc

W8x10 is adequate

Force to diagonal members:

Max Tu = 16.3 k (ETABS analysis)

Tension, AASHTO 6.8.2

$\phi P_n = \phi F_y A_g > Tu$

Solve for A_g:

Min A_g = 0.37 in^2

Use 4x4x1/4 Ag = 3.37 in^2

DCR =	0.11

Table 7.1.2-1—Values of $1/K$ for Various Values of CL/P_c and n

$1/K$	$n = 4$	$n = 6$	$n = 8$	$n = 10$	$n = 12$	$n = 14$	$n = 16$
1.000	3.686	3.616	3.660	3.714	3.754	3.785	3.809
0.980		3.284	2.944	2.806	2.787	2.771	2.774
0.960		3.000	2.665	2.542	2.456	2.454	2.479
0.950			2.595				
0.940		2.754		2.303	2.252	2.254	2.282
0.920		2.643		2.146	2.094	2.101	2.121
0.900	3.352	2.593	2.263	2.045	1.951	1.968	1.981
0.850		2.460	2.013	1.794	1.709	1.681	1.694
0.800	2.961	2.313	1.889	1.629	1.480	1.456	1.465
0.750		2.147	1.750	1.501	1.344	1.273	1.262
0.700	2.448	1.955	1.595	1.359	1.200	1.111	1.088
0.650		1.739	1.442	1.236	1.087	0.988	0.940
0.600	2.035	1.639	1.338	1.133	0.985	0.878	0.808
0.550		1.517	1.211	1.007	0.860	0.768	0.708
0.500	1.750	1.362	1.047	0.847	0.750	0.668	0.600
0.450		1.158	0.829	0.714	0.624	0.537	0.500
0.400	1.232	0.886	0.627	0.555	0.454	0.428	0.383
0.350		0.530	0.434	0.352	0.323	0.292	0.280
0.300	0.121	0.187	0.249	0.170	0.203	0.183	0.187
0.293	0						
0.259		0					
0.250			0.135	0.107	0.103	0.121	0.112
0.200			0.045	0.068	0.055	0.053	0.070
0.180			0				
0.150				0.017	0.031	0.029	0.025
0.139				0			
0.114					0		
0.100						0.003	0.010
0.097						0	
0.085							0

7.1.3—Alternative Analysis Procedures

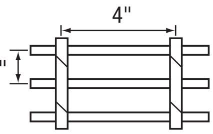
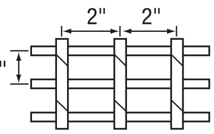
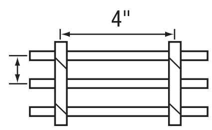
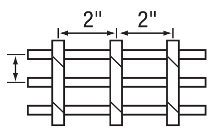
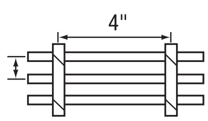
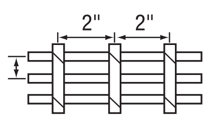
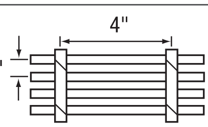
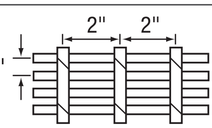
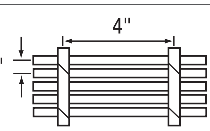
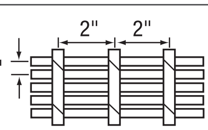
The use of a second-order numerical analysis procedure to evaluate the stability of the top chord of a half-through truss is acceptable in lieu of the procedure above, provided the following aspects are included in the model:

- Effects of initial out-of-straightness, both between panel points and across the entire length of the compression chord
- Effects of residual stresses in compression members due to fabrication and construction
- Effects of the stiffness of vertical to floorbeam connections

C7.1.3

Given the increasing availability of software that is capable of second order analyses, such an analysis is a practical alternative to the method given in Article 7.1.2. However, the design equations in *AASHTO LRFD* account for the issues identified, and any alternative method should also address these. One method that might be followed would be to use the second order elastic analysis to determine the K factor for a given chord size and panel point frame stiffness, and then the design equations of *AASHTO LRFD* to determine the corresponding resistance.

Steel Grating Table of Spacings

Part No.	Spacing	Open Area*	
19-W-4 19-DT-4 19-SL-4	1-3/16" 	78%	Bearing bars spaced at 1-3/16" on center and cross bars at 4" on center. The workhorse of industrial flooring, popular for platforms, catwalks, mezzanines, and stairways.
19-W-2 19-DT-2 19-SL-2	1-3/16" 	73%	Bearing bars spaced at 1-3/16" on center and cross bars at 2" on center. Excellent for short spans and applications where small wheeled carts continuously cross the grating surface.
15-W-4 15-DT-4 15-SL-4	15/16" 	75%	Bearing bars spaced at 15/16" on center and cross bars at 4" on center. The closer spaced bearing bars increase load capacity by more than 26% when compared to similar gratings produced with bearing bars at 1-3/16" on center.
15-W-2 15-DT-2 15-SL-2	15/16" 	69%	Bearing bars spaced at 15/16" on center and cross bars at 2" on center. The closer spaced bearing bars and cross bars provide additional flooring surface to support pedestrian and wheeled traffic.
11-W-4 11-DT-4 11-SL-4	11/16" 	68%	Bearing bars spaced at 11/16" on center and cross bars at either 4" or 2" on center. Types 11-4 and 11-2 with 3/16" thick bearing bars comply with the spacing requirements of the Americans with Disabilities Act. For ADA installations, specify that the bearing bars span perpendicular to the normal flow of traffic.
11-W-2 11-DT-2 11-SL-2	11/16" 	63%	
8-W-4 8-DT-4 8-SL-4	1/2" 	58%	Bearing bars spaced at 1/2" on center and cross bars at 4" or 2" on center. Types 8-4 and 8-2 comply with ADA spacing requirements. These products are popular for material handling platforms and mezzanines subject to continuous cart and dolly traffic.
8-W-2 8-DT-2 8-SL-2	1/2" 	54%	
7-W-4 7-DT-4 7-SL-4	7/16" 	53%	Bearing bars spaced at 7/16" on center and cross bars at 4" or 2" on center. Types 7-4 and 7-2 comply with ADA spacing requirements and are popular for applications in the public way. When specified with 3/16" thick bearing bars, 7-4 and 7-2 gratings have a net 1/4" clear opening between the bearing bars and commonly reject intrusion by high heeled shoes.
7-W-2 7-DT-2 7-SL-2	7/16" 	49%	

* Percentage of open area is based upon 3/16" thick bearing bars and .275" cross bars. Contact Grating Pacific if exact open area calculation is required for alternative bearing bar thicknesses or cross bar sizes.

How to Specify Steel Bar Grating

1. Select type of grating
   - **"W"** for welded steel grating
   - **"DT"** for dovetail pressure locked grating
   - **"SL"** for swage locked grating
2. Select bar spacing from table above
3. Select bearing bar size (consult load tables on pages 6-10 considering service loads and clear spans)
4. Specify plain, serrated, or Algrip surface
5. Specify banding or additional trim required
6. Specify finish
   - Bare steel (no finish)
   - Painted (red, black, silver, other)
   - Hot dip galvanized (per ASTM A-123)
   - Other
7. Specify fasteners (if required) – see page 59

19 Space
(1-3/16") Load Table

Use this table when evaluating spans and loads for the following types of steel grating:
19-W-4, 19-W-2, 19-DT-4, 19-DT-2, 19-SL-4, & 19-SL-2

Bearing Bar Size (inches)	Approx. Weight psf *	Max. Ped. Span**	Sec. Prop.*** Sx in³ Ix in⁴		Unsupported Span 2'-0	2'-6	3'-0	3'-6	4'-0	4'-6	5'-0	5'-6	6'-0	6'-6	7'-0	8'-0	9'-0
3/4 x 1/8	3.9	3'-5"	0.118	U	355	227	158	116	89	70							
				D	0.099	0.155	0.223	0.304	0.397	0.503							
			0.044	C	355	284	237	203	178	158							
				D	0.079	0.124	0.179	0.243	0.318	0.402							
3/4 x 3/16	5.6	3'-10"	0.178	U	533	341	237	174	133	105	85						
				D	0.099	0.155	0.223	0.304	0.397	0.503	0.621						
			0.067	C	533	426	355	305	266	237	213						
				D	0.079	0.124	0.179	0.243	0.318	0.402	0.497						
1 x 1/8	5.0	4'-3"	0.211	U	632	404	281	206	158	125	101	84					
				D	0.074	0.116	0.168	0.228	0.298	0.377	0.466	0.563					
			0.105	C	632	505	421	361	316	281	253	230					
				D	0.060	0.093	0.134	0.182	0.238	0.302	0.372	0.451					
1 x 3/16	7.2	4'-9"	0.316	U	947	606	421	309	237	187	152	125	105				
				D	0.074	0.116	0.168	0.228	0.298	0.377	0.466	0.563	0.670				
			0.158	C	947	758	632	541	474	421	379	345	316				
				D	0.060	0.093	0.134	0.182	0.238	0.302	0.372	0.451	0.536				
1-1/4 x 1/8	6.1	5'-1"	0.329	U	987	632	439	322	247	195	158	131	110	93			
				D	0.060	0.093	0.134	0.182	0.238	0.302	0.372	0.451	0.536	0.629			
			0.206	C	987	790	658	564	493	439	395	359	329	304			
				D	0.048	0.074	0.107	0.146	0.191	0.241	0.298	0.360	0.429	0.504			
1-1/4 x 3/16	8.9	5'-7"	0.493	U	1,480	947	658	483	370	292	237	196	165	140	121		
				D	0.060	0.093	0.134		0.238	0.302	0.372	0.451	0.536	0.629	0.730		
			0.308	C	1,480	1,184	987	846	740	658	592	538	493	456	423		
				D	0.048	0.074	0.107	0.146	0.191	0.241	0.298	0.360	0.429	0.504	0.584		
1-1/2 x 1/8	7.2	5'-10"	0.474	U	1,421	910	632	464	355	281	227	188	158	135	116		
				D	0.050	0.078	0.112	0.152	0.199	0.251	0.310	0.376	0.447	0.524	0.608		
			0.355	C	1,421	1,137	947	812	711	632	568	517	474	437	406		
				D	0.040	0.062	0.089	0.122	0.159	0.201	0.248	0.300	0.358	0.420	0.487		
1-1/2 x 3/16	10.7	6'-5"	0.711	U	2,132	1,364	947	696	533	421	341	282	237	202	174	133	
				D	0.050	0.078	0.112	0.152	0.199	0.251	0.310	0.376	0.447	0.524	0.608	0.794	
			0.533	C	2,132	1,705	1,421	1,218	1,066	947	853	775	711	656	609	533	
				D	0.040	0.062	0.089	0.122	0.159	0.201	0.248	0.300	0.358	0.420	0.487	0.636	
1-3/4 x 1/8	8.5	6'-6"	0.645	U	1,934	1,238	860	632	484	382	310	256	215	183	158	121	96
				D	0.043	0.067	0.096	0.130	0.170	0.215	0.266	0.322	0.383	0.450	0.521	0.681	0.862
			0.564	C	1,934	1,547	1,290	1,105	967	860	774	703	645	595	553	484	430
				D	0.034	0.053	0.077	0.104	0.136	0.172	0.213	0.257	0.306	0.360	0.417	0.545	0.689
1-3/4 x 3/16	12.3	7'-3"	0.967	U	2,901	1,857	1,290	947	725	573	464	384	322	275	237	181	143
				D	0.043	0.067	0.096	0.130	0.170	0.215	0.266	0.322	0.383	0.450	0.521	0.681	0.862
			0.846	C	2,901	2,321	1,934	1,658	1,451	1,290	1,161	1,055	967	893	829	725	645
				D	0.034	0.053	0.077	0.104	0.136	0.172	0.213	0.257	0.306	0.360	0.417	0.545	0.689
2 x 1/8	9.6	7'-4"	0.842	U	2,526	1,617	1,123	825	632	499	404	334	281	239	206	158	125
				D	0.037	0.058	0.084	0.114	0.149	0.189	0.233	0.282	0.335	0.393	0.456	0.596	0.754
			0.842	C	2,526	2,021	1,684	1,444	1,263	1,123	1,011	919	842	777	722	632	561
				D	0.030	0.047	0.067	0.091	0.119	0.151	0.186	0.225	0.268	0.315	0.365	0.477	0.603
2 x 3/16	13.9	8'-0"	1.263	U	3,790	2,425	1,684	1,237	947	749	606	501	421	359	309	237	187
				D	0.037	0.058	0.084	0.114	0.149	0.189	0.233	0.282	0.335	0.393	0.456	0.596	0.754
			1.263	C	3,790	3,032	2,526	2,165	1,895	1,684	1,516	1,378	1,263	1,166	1,083	947	842
				D	0.030	0.047	0.067	0.091	0.119	0.151	0.186	0.225	0.268	0.315	0.365	0.477	0.603
2-1/4 x 3/16	15.6	8'-9"	1.599	U	4,796	3,070	2,132	1,566	1,199	947	767	634	533	454	392	300	237
				D	0.033	0.052	0.074	0.101	0.132	0.168	0.207	0.250	0.298	0.350	0.406	0.530	0.670
			1.799	C	4,796	3,837	3,197	2,741	2,398	2,132	1,918	1,744	1,599	1,476	1,370	1,199	1,066
				D	0.026	0.041	0.060	0.081	0.106	0.134	0.166	0.200	0.238	0.280	0.324	0.424	0.536
2-1/2 x 3/16	17.2	9'-5"	1.974	U	5,921	3,790	2,632	1,933	1,480	1,170	947	783	658	561	483	370	292
				D	0.030	0.047	0.067	0.091	0.119	0.151	0.186	0.225	0.268	0.315	0.365	0.477	0.603
			2.467	C	5,921	4,737	3,947	3,384	2,961	2,632	2,368	2,153	1,974	1,822	1,692	1,480	1,316
				D	0.024	0.037	0.054	0.073	0.095	0.121	0.149	0.180	0.215	0.252	0.292	0.381	0.483

All loads and deflections are theoretical and based upon the gross sections of the bearing bars, using a fiber stress of 18,000 psi.

The values are not intended to be absolute since the actual load capacity will be affected by the slight variations in mill and manufacturing tolerances.

Grating for spans to the left of the heavy line have a deflection ≤ 1/4" for uniform loads of 100 psf.

U = uniform load in pounds/sq. ft.
C = concentrated load in pounds/ft. of grating width
D = deflection in inches

* Weight per square foot based upon 19-W-4 grating. Add .60 psf for 2" on center cross bars. ** Maximum pedestrian load is defined as a 100# uniform load with deflection ≤ 1/4 inch. (The 1/4" maximum deflection criteria is considered consistent with pedestrian comfort, but may be exceeded for other loading conditions at the discretion of the specifying authority.) *** Section properties per foot of width.

Note: When gratings with serrated surface are specified, the depth of the grating required for a specific load will be 1/4" greater than that shown in these tables.

Panel Widths

Grating panels are available from stock in nominal 24", 36" and 48" widths. When considering alternative widths, consult this table to select widths that will maintain uniform "out-to-out" spacing of the bearing bars. Specified widths deviating from this table will be fabricated to size with side banding and the bar spacing on one side of the finished panel will vary from the spacing throughout the remainder of the panel.

Number of Bearing Bars	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Panel Width	1-3/8"	2-9/16"	3-3/4"	4-15/16"	6-1/8"	7-5/16"	8-1/2"	9-11/16"	10-7/8"	12-1/16"	13-1/4"	14-7/16"	15-5/8"	16-13/16"	18"
Number of Bearing Bars	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
Panel Width	19-3/16"	20-3/8"	21-9/16"	22-3/4"	23-15/16"	25-1/8"	26-5/16"	27-1/2"	28-11/16"	29-7/8"	31-1/16"	32-1/4"	33-7/16"	34-5/8"	35-13/16"
Number of Bearing Bars	32	33	34	35	36	37	38	39	40	41					
Panel Width	37"	38-3/16"	39-3/8"	40-9/16"	41-3/4"	42-15/16"	44-1/8"	45-5/16"	46-1/2"	47-11/16"					

Panel widths indicated are for gratings with 3/16" thick bearing bars. For 1/8" thick bearing bars deduct 1/16" from the stated values.

Indicates stock panel widths.

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Project ID:
Project Descr:

Steel Beam

File: 21-01-28_Winner creek_eh.ec6

Lic. # : KW-06001667 REID MIDDLETON, INC.

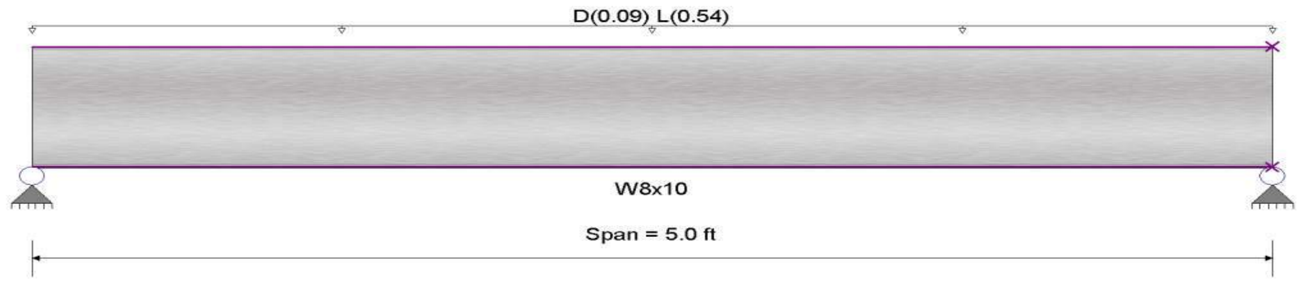
DESCRIPTION: Horizontal deck beam

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.0150, L = 0.090 ksf, Tributary Width = 6.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.090** : 1	Maximum Shear Stress Ratio =	**0.059** : 1
Section used for this span	**W8x10**	Section used for this span	**W8x10**
Ma : Applied	1.969 k-ft	Va : Applied	1.575 k
Mn / Omega : Allowable	21.870 k-ft	Vn/Omega : Allowable	26.826 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	2.500 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.009 in	Ratio =	7,025 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.010 in	Ratio =	6022 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+H													
Dsgn. L = 5.00 ft	1	0.013	0.008	0.28		0.28	36.52	21.87	1.00	1.00	0.23	40.24	26.83
+D+L+H													
Dsgn. L = 5.00 ft	1	0.090	0.059	1.97		1.97	36.52	21.87	1.00	1.00	1.58	40.24	26.83
+D+Lr+H													
Dsgn. L = 5.00 ft	1	0.013	0.008	0.28		0.28	36.52	21.87	1.00	1.00	0.23	40.24	26.83
+D+S+H													
Dsgn. L = 5.00 ft	1	0.013	0.008	0.28		0.28	36.52	21.87	1.00	1.00	0.23	40.24	26.83
+D+0.750Lr+0.750L+H													
Dsgn. L = 5.00 ft	1	0.071	0.046	1.55		1.55	36.52	21.87	1.00	1.00	1.24	40.24	26.83
+D+0.750L+0.750S+H													
Dsgn. L = 5.00 ft	1	0.071	0.046	1.55		1.55	36.52	21.87	1.00	1.00	1.24	40.24	26.83
+D+0.60W+H													
Dsgn. L = 5.00 ft	1	0.013	0.008	0.28		0.28	36.52	21.87	1.00	1.00	0.23	40.24	26.83
+D+0.70E+H													
Dsgn. L = 5.00 ft	1	0.013	0.008	0.28		0.28	36.52	21.87	1.00	1.00	0.23	40.24	26.83
+D+0.750Lr+0.750L+0.450W+H													
Dsgn. L = 5.00 ft	1	0.071	0.046	1.55		1.55	36.52	21.87	1.00	1.00	1.24	40.24	26.83
+D+0.750L+0.750S+0.450W+H													
Dsgn. L = 5.00 ft	1	0.071	0.046	1.55		1.55	36.52	21.87	1.00	1.00	1.24	40.24	26.83
+D+0.750L+0.750S+0.5250E+H													
Dsgn. L = 5.00 ft	1	0.071	0.046	1.55		1.55	36.52	21.87	1.00	1.00	1.24	40.24	26.83
+0.60D+0.60W+0.60H													
Dsgn. L = 5.00 ft	1	0.008	0.005	0.17		0.17	36.52	21.87	1.00	1.00	0.14	40.24	26.83

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Project Descr:

Steel Beam

File: 21-01-28_Winner creek_eh.ec6

Lic. # : KW-06001667
REID MIDDLETON, INC.

DESCRIPTION: Horizontal deck beam

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+0.60D+0.70E+0.60H													
Dsgn. L = 5.00 ft	1	0.008	0.005	0.17		0.17	36.52	21.87	1.00	1.00	0.14	40.24	26.83

Overall Maximum Deflections

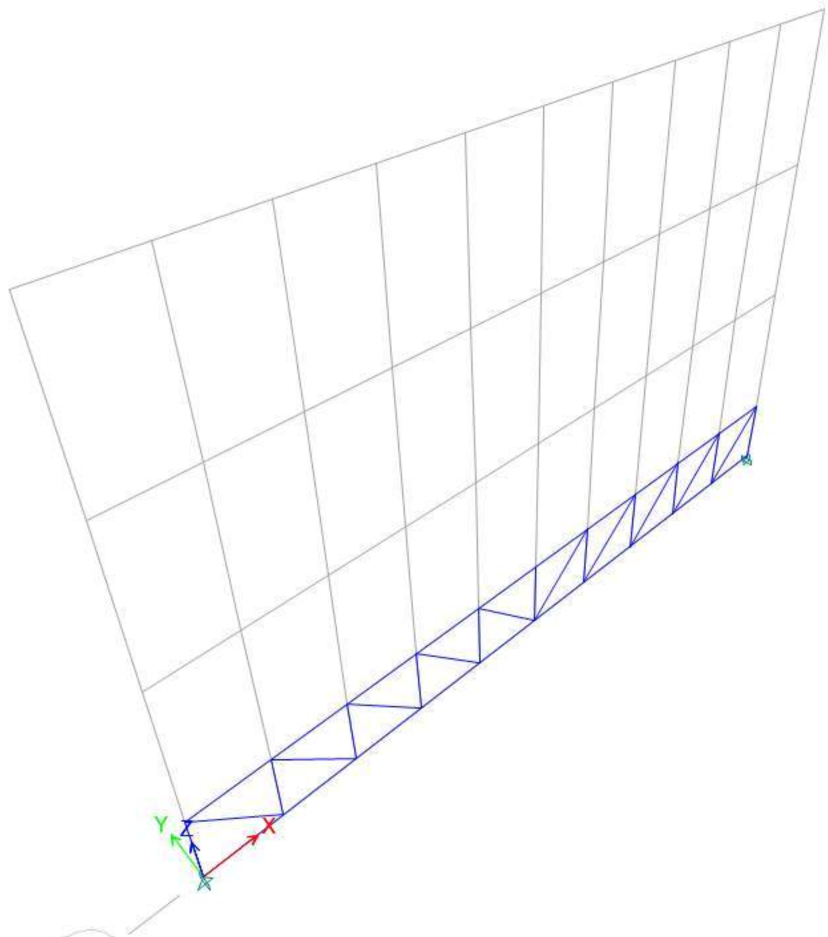
Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.0100	2.514		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.575	1.575
Overall MINimum	0.135	0.135
+D+H	0.225	0.225
+D+L+H	1.575	1.575
+D+Lr+H	0.225	0.225
+D+S+H	0.225	0.225
+D+0.750Lr+0.750L+H	1.238	1.238
+D+0.750L+0.750S+H	1.238	1.238
+D+0.60W+H	0.225	0.225
+D+0.70E+H	0.225	0.225
+D+0.750Lr+0.750L+0.450W+H	1.238	1.238
+D+0.750L+0.750S+0.450W+H	1.238	1.238
+D+0.750L+0.750S+0.5250E+H	1.238	1.238
+0.60D+0.60W+0.60H	0.135	0.135
+0.60D+0.70E+0.60H	0.135	0.135
D Only	0.225	0.225
L Only	1.350	1.350
H Only		



Project Report

Model File: 21-02-22_60 ft steel bridge_eh, Revision 0
3/17/2021

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1 Structure Data

This chapter provides model geometry information, including items such as story levels, point coordinates, and element connectivity.

1.1 Story Data

Table 1.1 - Story Definitions

Tower	Name	Height ft	Master Story	Similar To	Splice Story	Color
T1	Story4	12	Yes	None	No	Magenta
T1	Story3	12	No	Story4	No	Yellow
T1	Story2	12	No	Story4	No	Gray8Dark
T1	Story1	6	No	Story4	No	Blue

1.2 Grid Data

Table 1.2 - Grid Definitions - General

Tower	Name	Type	Ux ft	Uy ft	Rz deg	Story Range	Bubble Size in	Color
T1	G1	Cartesian	0	0	0	Default	60	Gray6

Table 1.3 - Grid Definitions - Grid Lines

Name	Grid Line Type	ID	Ordinate ft	Bubble Location	Visible
G1	X (Cartesian)	A	0	End	Yes
G1	X (Cartesian)	A.1	6	End	Yes
G1	X (Cartesian)	A.2	12	End	Yes
G1	X (Cartesian)	A.3	18	End	Yes
G1	X (Cartesian)	A.4	24	End	Yes
G1	X (Cartesian)	A.5	30	End	Yes
G1	X (Cartesian)	A.6	36	End	Yes
G1	X (Cartesian)	A.7	42	End	Yes
G1	X (Cartesian)	A.8	48	End	Yes
G1	X (Cartesian)	A.9	54	End	Yes
G1	X (Cartesian)	B	60	End	Yes
G1	Y (Cartesian)	1	0	Start	Yes
G1	Y (Cartesian)	2	5	Start	Yes

1.3 Point Coordinates

Table 1.4 - Point Bays

Label	Is Auto Point	X ft	Y ft	DZBelow ft
1	No	0	0	0
5	No	60	0	0
6	No	6	0	0
7	No	12	0	0
8	No	18	0	0
9	No	24	0	0

Table 1.4 - Point Bays (continued)

Label	Is Auto Point	X ft	Y ft	DZBelow ft
10	No	30	0	0
11	No	36	0	0
12	No	42	0	0
13	No	48	0	0
14	No	54	0	0

1.4 Line Connectivity

Table 1.5 - Column Bays

Label	PointBayI	PointBayJ	IEndStory
C1	1	1	Below
C2	6	6	Below
C3	7	7	Below
C4	8	8	Below
C5	9	9	Below
C6	10	10	Below
C7	11	11	Below
C8	12	12	Below
C9	13	13	Below
C10	14	14	Below
C11	5	5	Below

Table 1.6 - Beam Bays

Label	PointBayI	PointBayJ
B3	1	5

Table 1.7 - Brace Bays

Label	PointBayI	PointBayJ	IEndStory
D2	6	1	Below
D3	7	6	Below
D4	8	7	Below
D5	9	8	Below
D6	10	9	Below
D7	10	11	Below
D8	11	12	Below
D9	12	13	Below
D10	13	14	Below
D11	14	5	Below

1.5 Mass

Table 1.8 - Mass Source Definition

Name	Is Default	Include Lateral Mass?	Include Vertical Mass?	Lump Mass?	Source Self Mass?	Source Added Mass?	Source Load Patterns?	Move Mass Centroid?
MsSrc1	Yes	Yes	No	Yes	Yes	Yes	No	No

Table 1.9 - Mass Summary by Story

Story	UX lb-s2/ft	UY lb-s2/ft	UZ lb-s2/ft
Story4	0	0	0
Story3	0	0	0
Story2	0	0	0
Story1	92.33	92.33	0
Base	92.63	92.63	0

Table 1.10 - Mass Summary by Group

Group	Self Mass lb-s2/ft	Self Weight kip	Mass X lb-s2/ft	Mass Y lb-s2/ft	Mass Z lb-s2/ft
All	184.96	0	184.96	184.96	0

1.6 Groups

Table 1.11 - Group Definitions

Name	Color	Steel Design?	Concrete Design?	Composite Design?
All	Yellow	No	No	No

2 Properties

This chapter provides property information for materials, frame sections, shell sections, and links.

2.1 Materials

Table 2.1 - Material Properties - General

Material	Type	SymType	Grade	Color	Notes
4000Psi	Concrete	Isotropic	f'c 4000 psi	Gray8Dark	
A416Gr270	Tendon	Uniaxial	Grade 270	Green	
A615Gr60	Rebar	Uniaxial	Grade 60	Blue	
A992Fy50	Steel	Isotropic	Grade 50	Yellow	

3 Assignments

This chapter provides a listing of the assignments applied to the model.

3.1 Joint Assignments

Table 3.1 - Joint Assignments - Summary

Story	Label	UniqueName	Diaphragm	Restraints
Story1	1	5	From Area	
Story1	5	6	From Area	
Story1	6	10	From Area	
Story1	7	12	From Area	
Story1	8	14	From Area	
Story1	9	16	From Area	
Story1	10	18	From Area	
Story1	11	20	From Area	
Story1	12	22	From Area	
Story1	13	24	From Area	
Story1	14	26	From Area	
Base	1	7	From Area	UX; UY; UZ; RX; RZ
Base	5	8	From Area	UY; UZ; RX; RZ
Base	6	9	From Area	
Base	7	11	From Area	
Base	8	13	From Area	
Base	9	15	From Area	
Base	10	17	From Area	
Base	11	19	From Area	
Base	12	21	From Area	
Base	13	23	From Area	
Base	14	25	From Area	

3.2 Frame Assignments

Table 3.2 - Frame Assignments - Summary

Story	Label	UniqueName	Design Type	Length ft	Analysis Section	Design Section	Max Station Spacing ft	Min Number Stations	Releases
Story1	B3	3	Beam	60	HSS5x5x3/8	N/A	2		Yes
Story1	C1	5	Column	6	Vertical	N/A		3	
Story1	C2	7	Column	6	Vertical	N/A		3	Yes
Story1	C3	8	Column	6	Vertical	N/A		3	Yes
Story1	C4	9	Column	6	Vertical	N/A		3	Yes
Story1	C5	10	Column	6	Vertical	N/A		3	Yes
Story1	C6	11	Column	6	Vertical	N/A		3	Yes
Story1	C7	12	Column	6	Vertical	N/A		3	Yes
Story1	C8	13	Column	6	Vertical	N/A		3	Yes
Story1	C9	14	Column	6	Vertical	N/A		3	Yes
Story1	C10	15	Column	6	Vertical	N/A		3	Yes
Story1	C11	16	Column	6	Vertical	N/A		3	

Table 3.2 - Frame Assignments - Summary (continued)

Story	Label	UniqueName	Design Type	Length ft	Analysis Section	Design Section	Max Station Spacing ft	Min Number Stations	Releases
Story1	D2	36	Brace	8.4853	diagonal	N/A		3	Yes
Story1	D3	37	Brace	8.4853	diagonal	N/A		3	Yes
Story1	D4	38	Brace	8.4853	diagonal	N/A		3	Yes
Story1	D5	39	Brace	8.4853	diagonal	N/A		3	Yes
Story1	D6	40	Brace	8.4853	diagonal	N/A		3	Yes
Story1	D7	41	Brace	8.4853	diagonal	N/A		3	Yes
Story1	D8	42	Brace	8.4853	diagonal	N/A		3	Yes
Story1	D9	43	Brace	8.4853	diagonal	N/A		3	Yes
Story1	D10	44	Brace	8.4853	diagonal	N/A		3	Yes
Story1	D11	45	Brace	8.4853	diagonal	N/A		3	Yes
Base	B3	34	Beam	60	HSS5x5x3/8	N/A	2		Yes

4 Loads

This chapter provides loading information as applied to the model.

4.1 Load Patterns

Table 4.1 - Load Pattern Definitions

Name	Is Auto Load	Type	Self Weight Multiplier
~LLRF	Yes	Other	0
Dead	No	Dead	0
Live	No	Live	0

4.2 Applied Loads

4.2.1 Line Loads

Table 4.2 - Frame Loads Assignments - Point

Story	Label	UniqueName	Load Pattern	Load Type	Direction	Distance Type	Relative Distance	Absolute Distance ft	Force kip
Story1	C1	5	Dead	Force	Gravity	Relative	0.25	1.5	0.075
Story1	C2	7	Dead	Force	Gravity	Relative	0.25	1.5	0.15
Story1	C3	8	Dead	Force	Gravity	Relative	0.25	1.5	0.15
Story1	C4	9	Dead	Force	Gravity	Relative	0.25	1.5	0.15
Story1	C5	10	Dead	Force	Gravity	Relative	0.25	1.5	0.15
Story1	C6	11	Dead	Force	Gravity	Relative	0.25	1.5	0.15
Story1	C7	12	Dead	Force	Gravity	Relative	0.25	1.5	0.15
Story1	C8	13	Dead	Force	Gravity	Relative	0.25	1.5	0.15
Story1	C9	14	Dead	Force	Gravity	Relative	0.25	1.5	0.15
Story1	C10	15	Dead	Force	Gravity	Relative	0.25	1.5	0.15
Story1	C11	16	Dead	Force	Gravity	Relative	0.25	1.5	0.075
Story1	C1	5	Live	Force	Gravity	Relative	0.25	1.5	0.675
Story1	C2	7	Live	Force	Gravity	Relative	0.25	1.5	1.35
Story1	C3	8	Live	Force	Gravity	Relative	0.25	1.5	1.35
Story1	C4	9	Live	Force	Gravity	Relative	0.25	1.5	1.35
Story1	C5	10	Live	Force	Gravity	Relative	0.25	1.5	1.35
Story1	C6	11	Live	Force	Gravity	Relative	0.25	1.5	1.35
Story1	C7	12	Live	Force	Gravity	Relative	0.25	1.5	1.35
Story1	C8	13	Live	Force	Gravity	Relative	0.25	1.5	1.35
Story1	C9	14	Live	Force	Gravity	Relative	0.25	1.5	1.35
Story1	C10	15	Live	Force	Gravity	Relative	0.25	1.5	1.35
Story1	C11	16	Live	Force	Gravity	Relative	0.25	1.5	0.675

4.3 Load Cases

Table 4.3 - Load Case Definitions - Summary

Name	Type
Dead	Linear Static
Live	Linear Static
Modal	Modal - Eigen

4.4 Load Combinations

Table 4.4 - Load Combination Definitions

Name	Type	Is Auto	Load Name	SF	Notes
AASHTO D + L	Linear Add	No	Dead	1.375	
AASHTO D + L			Live	1.75	

5 Analysis Results

This chapter provides analysis results.

5.1 Structure Results

Table 5.1 - Base Reactions

Output Case	Case Type	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft	X ft	Y ft	Z ft
Dead	LinStatic	0	0	1.5	0	-45	0	0	0	0
Live	LinStatic	0	0	13.5	0	-405	0	0	0	0
AASHTO D + L	Combination	0	0	25.687	0	-770.625	0	0	0	0

5.2 Point Results

Table 5.2 - Joint Reactions

Story	Label	Unique Name	Output Case	Case Type	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft
Base	1	7	Dead	LinStatic	0	0	0.75	0	0	0
Base	1	7	Live	LinStatic	0	0	6.75	0	0	0
Base	1	7	AASHTO D + L	Combination	0	0	12.844	0	0	0
Base	5	8	Dead	LinStatic	0	0	0.75	0	0	0
Base	5	8	Live	LinStatic	0	0	6.75	0	0	0
Base	5	8	AASHTO D + L	Combination	0	0	12.844	0	0	0

5.3 Line Results

Table 5.3 - Element Forces - Columns (Part 1 of 2)

Story	Column	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft
Story1	C1	5	Dead	LinStatic	0	-0.748	-6.157E-05	0	0	0
Story1	C1	5	Dead	LinStatic	1.5	-0.748	-6.157E-05	0	0	0
Story1	C1	5	Dead	LinStatic	1.5	-0.673	-6.157E-05	0	0	0
Story1	C1	5	Dead	LinStatic	2.7917	-0.673	-6.157E-05	0	0	0
Story1	C1	5	Dead	LinStatic	5.5833	-0.673	-6.157E-05	0	0	0
Story1	C1	5	Live	LinStatic	0	-6.736	-0.001	0	0	0
Story1	C1	5	Live	LinStatic	1.5	-6.736	-0.001	0	0	0
Story1	C1	5	Live	LinStatic	1.5	-6.061	-0.001	0	0	0
Story1	C1	5	Live	LinStatic	2.7917	-6.061	-0.001	0	0	0
Story1	C1	5	Live	LinStatic	5.5833	-6.061	-0.001	0	0	0
Story1	C1	5	AASHTO D + L	Combination	0	-12.817	-0.001	0	0	0
Story1	C1	5	AASHTO D + L	Combination	1.5	-12.817	-0.001	0	0	0
Story1	C1	5	AASHTO D + L	Combination	1.5	-11.533	-0.001	0	0	0
Story1	C1	5	AASHTO D + L	Combination	2.7917	-11.533	-0.001	0	0	0
Story1	C1	5	AASHTO D + L	Combination	5.5833	-11.533	-0.001	0	0	0
Story1	C2	7	Dead	LinStatic	0	-0.673	0	0	0	0
Story1	C2	7	Dead	LinStatic	1.5	-0.673	0	0	0	0
Story1	C2	7	Dead	LinStatic	1.5	-0.523	0	0	0	0
Story1	C2	7	Dead	LinStatic	3	-0.523	0	0	0	0
Story1	C2	7	Dead	LinStatic	6	-0.523	0	0	0	0
Story1	C2	7	Live	LinStatic	0	-6.054	0	0	0	0

Table 5.3 - Element Forces - Columns (Part 1 of 2, continued)

Story	Column	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft
Story1	C2	7	Live	LinStatic	1.5	-6.054	0	0	0	0
Story1	C2	7	Live	LinStatic	1.5	-4.704	0	0	0	0
Story1	C2	7	Live	LinStatic	3	-4.704	0	0	0	0
Story1	C2	7	Live	LinStatic	6	-4.704	0	0	0	0
Story1	C2	7	AASHTO D + L	Combination	0	-11.519	0	0	0	0
Story1	C2	7	AASHTO D + L	Combination	1.5	-11.519	0	0	0	0
Story1	C2	7	AASHTO D + L	Combination	1.5	-8.95	0	0	0	0
Story1	C2	7	AASHTO D + L	Combination	3	-8.95	0	0	0	0
Story1	C2	7	AASHTO D + L	Combination	6	-8.95	0	0	0	0
Story1	C3	8	Dead	LinStatic	0	-0.524	0	0	0	0
Story1	C3	8	Dead	LinStatic	1.5	-0.524	0	0	0	0
Story1	C3	8	Dead	LinStatic	1.5	-0.374	0	0	0	0
Story1	C3	8	Dead	LinStatic	3	-0.374	0	0	0	0
Story1	C3	8	Dead	LinStatic	6	-0.374	0	0	0	0
Story1	C3	8	Live	LinStatic	0	-4.717	0	0	0	0
Story1	C3	8	Live	LinStatic	1.5	-4.717	0	0	0	0
Story1	C3	8	Live	LinStatic	1.5	-3.367	0	0	0	0
Story1	C3	8	Live	LinStatic	3	-3.367	0	0	0	0
Story1	C3	8	Live	LinStatic	6	-3.367	0	0	0	0
Story1	C3	8	AASHTO D + L	Combination	0	-8.975	0	0	0	0
Story1	C3	8	AASHTO D + L	Combination	1.5	-8.975	0	0	0	0
Story1	C3	8	AASHTO D + L	Combination	1.5	-6.406	0	0	0	0
Story1	C3	8	AASHTO D + L	Combination	3	-6.406	0	0	0	0
Story1	C3	8	AASHTO D + L	Combination	6	-6.406	0	0	0	0
Story1	C4	9	Dead	LinStatic	0	-0.374	0	0	0	0
Story1	C4	9	Dead	LinStatic	1.5	-0.374	0	0	0	0
Story1	C4	9	Dead	LinStatic	1.5	-0.224	0	0	0	0
Story1	C4	9	Dead	LinStatic	3	-0.224	0	0	0	0
Story1	C4	9	Dead	LinStatic	6	-0.224	0	0	0	0
Story1	C4	9	Live	LinStatic	0	-3.367	0	0	0	0
Story1	C4	9	Live	LinStatic	1.5	-3.367	0	0	0	0
Story1	C4	9	Live	LinStatic	1.5	-2.017	0	0	0	0
Story1	C4	9	Live	LinStatic	3	-2.017	0	0	0	0
Story1	C4	9	Live	LinStatic	6	-2.017	0	0	0	0
Story1	C4	9	AASHTO D + L	Combination	0	-6.406	0	0	0	0
Story1	C4	9	AASHTO D + L	Combination	1.5	-6.406	0	0	0	0
Story1	C4	9	AASHTO D + L	Combination	1.5	-3.838	0	0	0	0
Story1	C4	9	AASHTO D + L	Combination	3	-3.838	0	0	0	0
Story1	C4	9	AASHTO D + L	Combination	6	-3.838	0	0	0	0
Story1	C5	10	Dead	LinStatic	0	-0.225	0	0	0	0
Story1	C5	10	Dead	LinStatic	1.5	-0.225	0	0	0	0
Story1	C5	10	Dead	LinStatic	1.5	-0.075	0	0	0	0
Story1	C5	10	Dead	LinStatic	3	-0.075	0	0	0	0
Story1	C5	10	Dead	LinStatic	6	-0.075	0	0	0	0
Story1	C5	10	Live	LinStatic	0	-2.021	0	0	0	0

Table 5.3 - Element Forces - Columns (Part 1 of 2, continued)

Story	Column	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft
Story1	C5	10	Live	LinStatic	1.5	-2.021	0	0	0	0
Story1	C5	10	Live	LinStatic	1.5	-0.671	0	0	0	0
Story1	C5	10	Live	LinStatic	3	-0.671	0	0	0	0
Story1	C5	10	Live	LinStatic	6	-0.671	0	0	0	0
Story1	C5	10	AASHTO D + L	Combination	0	-3.846	0	0	0	0
Story1	C5	10	AASHTO D + L	Combination	1.5	-3.846	0	0	0	0
Story1	C5	10	AASHTO D + L	Combination	1.5	-1.277	0	0	0	0
Story1	C5	10	AASHTO D + L	Combination	3	-1.277	0	0	0	0
Story1	C5	10	AASHTO D + L	Combination	6	-1.277	0	0	0	0
Story1	C6	11	Dead	LinStatic	0	-0.15	0	0	0	0
Story1	C6	11	Dead	LinStatic	1.5	-0.15	0	0	0	0
Story1	C6	11	Dead	LinStatic	1.5	0.0002683	0	0	0	0
Story1	C6	11	Dead	LinStatic	3	0.0002683	0	0	0	0
Story1	C6	11	Dead	LinStatic	6	0.0002683	0	0	0	0
Story1	C6	11	Live	LinStatic	0	-1.348	0	0	0	0
Story1	C6	11	Live	LinStatic	1.5	-1.348	0	0	0	0
Story1	C6	11	Live	LinStatic	1.5	0.002	0	0	0	0
Story1	C6	11	Live	LinStatic	3	0.002	0	0	0	0
Story1	C6	11	Live	LinStatic	6	0.002	0	0	0	0
Story1	C6	11	AASHTO D + L	Combination	0	-2.564	0	0	0	0
Story1	C6	11	AASHTO D + L	Combination	1.5	-2.564	0	0	0	0
Story1	C6	11	AASHTO D + L	Combination	1.5	0.005	0	0	0	0
Story1	C6	11	AASHTO D + L	Combination	3	0.005	0	0	0	0
Story1	C6	11	AASHTO D + L	Combination	6	0.005	0	0	0	0
Story1	C7	12	Dead	LinStatic	0	-0.225	0	0	0	0
Story1	C7	12	Dead	LinStatic	1.5	-0.225	0	0	0	0
Story1	C7	12	Dead	LinStatic	1.5	-0.075	0	0	0	0
Story1	C7	12	Dead	LinStatic	3	-0.075	0	0	0	0
Story1	C7	12	Dead	LinStatic	6	-0.075	0	0	0	0
Story1	C7	12	Live	LinStatic	0	-2.021	0	0	0	0
Story1	C7	12	Live	LinStatic	1.5	-2.021	0	0	0	0
Story1	C7	12	Live	LinStatic	1.5	-0.671	0	0	0	0
Story1	C7	12	Live	LinStatic	3	-0.671	0	0	0	0
Story1	C7	12	Live	LinStatic	6	-0.671	0	0	0	0
Story1	C7	12	AASHTO D + L	Combination	0	-3.846	0	0	0	0
Story1	C7	12	AASHTO D + L	Combination	1.5	-3.846	0	0	0	0
Story1	C7	12	AASHTO D + L	Combination	1.5	-1.277	0	0	0	0
Story1	C7	12	AASHTO D + L	Combination	3	-1.277	0	0	0	0
Story1	C7	12	AASHTO D + L	Combination	6	-1.277	0	0	0	0
Story1	C8	13	Dead	LinStatic	0	-0.374	0	0	0	0
Story1	C8	13	Dead	LinStatic	1.5	-0.374	0	0	0	0
Story1	C8	13	Dead	LinStatic	1.5	-0.224	0	0	0	0
Story1	C8	13	Dead	LinStatic	3	-0.224	0	0	0	0
Story1	C8	13	Dead	LinStatic	6	-0.224	0	0	0	0
Story1	C8	13	Live	LinStatic	0	-3.367	0	0	0	0

Table 5.3 - Element Forces - Columns (Part 1 of 2, continued)

Story	Column	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft
Story1	C8	13	Live	LinStatic	1.5	-3.367	0	0	0	0
Story1	C8	13	Live	LinStatic	1.5	-2.017	0	0	0	0
Story1	C8	13	Live	LinStatic	3	-2.017	0	0	0	0
Story1	C8	13	Live	LinStatic	6	-2.017	0	0	0	0
Story1	C8	13	AASHTO D + L	Combination	0	-6.406	0	0	0	0
Story1	C8	13	AASHTO D + L	Combination	1.5	-6.406	0	0	0	0
Story1	C8	13	AASHTO D + L	Combination	1.5	-3.838	0	0	0	0
Story1	C8	13	AASHTO D + L	Combination	3	-3.838	0	0	0	0
Story1	C8	13	AASHTO D + L	Combination	6	-3.838	0	0	0	0
Story1	C9	14	Dead	LinStatic	0	-0.524	0	0	0	0
Story1	C9	14	Dead	LinStatic	1.5	-0.524	0	0	0	0
Story1	C9	14	Dead	LinStatic	1.5	-0.374	0	0	0	0
Story1	C9	14	Dead	LinStatic	3	-0.374	0	0	0	0
Story1	C9	14	Dead	LinStatic	6	-0.374	0	0	0	0
Story1	C9	14	Live	LinStatic	0	-4.717	0	0	0	0
Story1	C9	14	Live	LinStatic	1.5	-4.717	0	0	0	0
Story1	C9	14	Live	LinStatic	1.5	-3.367	0	0	0	0
Story1	C9	14	Live	LinStatic	3	-3.367	0	0	0	0
Story1	C9	14	Live	LinStatic	6	-3.367	0	0	0	0
Story1	C9	14	AASHTO D + L	Combination	0	-8.975	0	0	0	0
Story1	C9	14	AASHTO D + L	Combination	1.5	-8.975	0	0	0	0
Story1	C9	14	AASHTO D + L	Combination	1.5	-6.406	0	0	0	0
Story1	C9	14	AASHTO D + L	Combination	3	-6.406	0	0	0	0
Story1	C9	14	AASHTO D + L	Combination	6	-6.406	0	0	0	0
Story1	C10	15	Dead	LinStatic	0	-0.673	0	0	0	0
Story1	C10	15	Dead	LinStatic	1.5	-0.673	0	0	0	0
Story1	C10	15	Dead	LinStatic	1.5	-0.523	0	0	0	0
Story1	C10	15	Dead	LinStatic	3	-0.523	0	0	0	0
Story1	C10	15	Dead	LinStatic	6	-0.523	0	0	0	0
Story1	C10	15	Live	LinStatic	0	-6.054	0	0	0	0
Story1	C10	15	Live	LinStatic	1.5	-6.054	0	0	0	0
Story1	C10	15	Live	LinStatic	1.5	-4.704	0	0	0	0
Story1	C10	15	Live	LinStatic	3	-4.704	0	0	0	0
Story1	C10	15	Live	LinStatic	6	-4.704	0	0	0	0
Story1	C10	15	AASHTO D + L	Combination	0	-11.519	0	0	0	0
Story1	C10	15	AASHTO D + L	Combination	1.5	-11.519	0	0	0	0
Story1	C10	15	AASHTO D + L	Combination	1.5	-8.95	0	0	0	0
Story1	C10	15	AASHTO D + L	Combination	3	-8.95	0	0	0	0
Story1	C10	15	AASHTO D + L	Combination	6	-8.95	0	0	0	0
Story1	C11	16	Dead	LinStatic	0	-0.748	6.157E-05	0	0	0
Story1	C11	16	Dead	LinStatic	1.5	-0.748	6.157E-05	0	0	0
Story1	C11	16	Dead	LinStatic	1.5	-0.673	6.157E-05	0	0	0
Story1	C11	16	Dead	LinStatic	2.7917	-0.673	6.157E-05	0	0	0
Story1	C11	16	Dead	LinStatic	5.5833	-0.673	6.157E-05	0	0	0
Story1	C11	16	Live	LinStatic	0	-6.736	0.001	0	0	0

Table 5.3 - Element Forces - Columns (Part 1 of 2, continued)

Story	Column	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft
Story1	C11	16	Live	LinStatic	1.5	-6.736	0.001	0	0	0
Story1	C11	16	Live	LinStatic	1.5	-6.061	0.001	0	0	0
Story1	C11	16	Live	LinStatic	2.7917	-6.061	0.001	0	0	0
Story1	C11	16	Live	LinStatic	5.5833	-6.061	0.001	0	0	0
Story1	C11	16	AASHTO D + L	Combination	0	-12.817	0.001	0	0	0
Story1	C11	16	AASHTO D + L	Combination	1.5	-12.817	0.001	0	0	0
Story1	C11	16	AASHTO D + L	Combination	1.5	-11.533	0.001	0	0	0
Story1	C11	16	AASHTO D + L	Combination	2.7917	-11.533	0.001	0	0	0
Story1	C11	16	AASHTO D + L	Combination	5.5833	-11.533	0.001	0	0	0

Table 5.3 - Element Forces - Columns (Part 2 of 2)

M3 kip-ft	Element	Elem Station ft	Location
0	5	0	
0.0001	5	1.5	Before
0.0001	5	1.5	After
0.0002	5	2.7917	
0.0003	5	5.5833	
0	5	0	
0.0008	5	1.5	Before
0.0008	5	1.5	After
0.0015	5	2.7917	
0.0031	5	5.5833	
0	5	0	
0.0016	5	1.5	Before
0.0016	5	1.5	After
0.0029	5	2.7917	
0.0059	5	5.5833	
0	7	0	
0	7	1.5	Before
0	7	1.5	After
0	7	3	
0	7	6	
0	7	0	
0	7	1.5	Before
0	7	1.5	After
0	7	3	
0	7	6	
0	7	0	
0	7	1.5	Before
0	7	1.5	After
0	7	3	
0	7	6	
0	8	0	

Table 5.3 - Element Forces - Columns (Part 2 of 2, continued)

M3 kip-ft	Element	Elem Station ft	Location
0	8	1.5	Before
0	8	1.5	After
0	8	3	
0	8	6	
0	8	0	
0	8	1.5	Before
0	8	1.5	After
0	8	3	
0	8	6	
0	8	0	
0	8	1.5	Before
0	8	1.5	After
0	8	3	
0	8	6	
0	9	0	
0	9	1.5	Before
0	9	1.5	After
0	9	3	
0	9	6	
0	9	0	
0	9	1.5	Before
0	9	1.5	After
0	9	3	
0	9	6	
0	9	0	
0	9	1.5	Before
0	9	1.5	After
0	9	3	
0	9	6	
0	10	0	
0	10	1.5	Before
0	10	1.5	After
0	10	3	
0	10	6	
0	10	0	
0	10	1.5	Before
0	10	1.5	After
0	10	3	
0	10	6	
0	10	0	
0	10	1.5	Before
0	10	1.5	After
0	10	3	
0	10	6	

Table 5.3 - Element Forces - Columns (Part 2 of 2, continued)

M3 kip-ft	Element	Elem Station ft	Location
0	11	0	
0	11	1.5	Before
0	11	1.5	After
0	11	3	
0	11	6	
0	11	0	
0	11	1.5	Before
0	11	1.5	After
0	11	3	
0	11	6	
0	11	0	
0	11	1.5	Before
0	11	1.5	After
0	11	3	
0	11	6	
0	12	0	
0	12	1.5	Before
0	12	1.5	After
0	12	3	
0	12	6	
0	12	0	
0	12	1.5	Before
0	12	1.5	After
0	12	3	
0	12	6	
0	12	0	
0	12	1.5	Before
0	12	1.5	After
0	12	3	
0	12	6	
0	13	0	
0	13	1.5	Before
0	13	1.5	After
0	13	3	
0	13	6	
0	13	0	
0	13	1.5	Before
0	13	1.5	After
0	13	3	
0	13	6	
0	13	0	
0	13	1.5	Before
0	13	1.5	After
0	13	3	

Table 5.3 - Element Forces - Columns (Part 2 of 2, continued)

M3 kip-ft	Element	Elem Station ft	Location
0	13	6	
0	14	0	
0	14	1.5	Before
0	14	1.5	After
0	14	3	
0	14	6	
0	14	0	
0	14	1.5	Before
0	14	1.5	After
0	14	3	
0	14	6	
0	14	0	
0	14	1.5	Before
0	14	1.5	After
0	14	3	
0	14	6	
0	15	0	
0	15	1.5	Before
0	15	1.5	After
0	15	3	
0	15	6	
0	15	0	
0	15	1.5	Before
0	15	1.5	After
0	15	3	
0	15	6	
0	15	0	
0	15	1.5	Before
0	15	1.5	After
0	15	3	
0	15	6	
0	16	0	
-0.0001	16	1.5	Before
-0.0001	16	1.5	After
-0.0002	16	2.7917	
-0.0003	16	5.5833	
0	16	0	
-0.0008	16	1.5	Before
-0.0008	16	1.5	After
-0.0015	16	2.7917	
-0.0031	16	5.5833	
0	16	0	
-0.0016	16	1.5	Before
-0.0016	16	1.5	After

Table 5.3 - Element Forces - Columns (Part 2 of 2, continued)

M3 kip-ft	Element	Elem Station ft	Location
-0.0029	16	2.7917	
-0.0059	16	5.5833	

Table 5.4 - Element Forces - Beams (Part 1 of 2)

Story	Beam	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft
Story1	B3	3	Dead	LinStatic	0.2083	-0.672	-0.002	0	0	0
Story1	B3	3	Dead	LinStatic	2.1389	-0.672	-0.002	0	0	0
Story1	B3	3	Dead	LinStatic	4.0694	-0.672	-0.002	0	0	0
Story1	B3	3	Dead	LinStatic	6	-0.672	-0.002	0	0	0
Story1	B3	3	Dead	LinStatic	6	-1.196	-0.0003841	0	0	0
Story1	B3	3	Dead	LinStatic	8	-1.196	-0.0003841	0	0	0
Story1	B3	3	Dead	LinStatic	10	-1.196	-0.0003841	0	0	0
Story1	B3	3	Dead	LinStatic	12	-1.196	-0.0003841	0	0	0
Story1	B3	3	Dead	LinStatic	12	-1.57	-0.001	0	0	0
Story1	B3	3	Dead	LinStatic	14	-1.57	-0.001	0	0	0
Story1	B3	3	Dead	LinStatic	16	-1.57	-0.001	0	0	0
Story1	B3	3	Dead	LinStatic	18	-1.57	-0.001	0	0	0
Story1	B3	3	Dead	LinStatic	18	-1.794	-0.0002036	0	0	0
Story1	B3	3	Dead	LinStatic	20	-1.794	-0.0002036	0	0	0
Story1	B3	3	Dead	LinStatic	22	-1.794	-0.0002036	0	0	0
Story1	B3	3	Dead	LinStatic	24	-1.794	-0.0002036	0	0	0
Story1	B3	3	Dead	LinStatic	24	-1.869	-0.0001342	0	0	0
Story1	B3	3	Dead	LinStatic	26	-1.869	-0.0001342	0	0	0
Story1	B3	3	Dead	LinStatic	28	-1.869	-0.0001342	0	0	0
Story1	B3	3	Dead	LinStatic	30	-1.869	-0.0001342	0	0	0
Story1	B3	3	Dead	LinStatic	30	-1.869	0.0001342	0	0	0
Story1	B3	3	Dead	LinStatic	32	-1.869	0.0001342	0	0	0
Story1	B3	3	Dead	LinStatic	34	-1.869	0.0001342	0	0	0
Story1	B3	3	Dead	LinStatic	36	-1.869	0.0001342	0	0	0
Story1	B3	3	Dead	LinStatic	36	-1.794	0.0002036	0	0	0
Story1	B3	3	Dead	LinStatic	38	-1.794	0.0002036	0	0	0
Story1	B3	3	Dead	LinStatic	40	-1.794	0.0002036	0	0	0
Story1	B3	3	Dead	LinStatic	42	-1.794	0.0002036	0	0	0
Story1	B3	3	Dead	LinStatic	42	-1.57	0.001	0	0	0
Story1	B3	3	Dead	LinStatic	44	-1.57	0.001	0	0	0
Story1	B3	3	Dead	LinStatic	46	-1.57	0.001	0	0	0
Story1	B3	3	Dead	LinStatic	48	-1.57	0.001	0	0	0
Story1	B3	3	Dead	LinStatic	48	-1.196	0.0003841	0	0	0
Story1	B3	3	Dead	LinStatic	50	-1.196	0.0003841	0	0	0
Story1	B3	3	Dead	LinStatic	52	-1.196	0.0003841	0	0	0
Story1	B3	3	Dead	LinStatic	54	-1.196	0.0003841	0	0	0
Story1	B3	3	Dead	LinStatic	54	-0.672	0.002	0	0	0
Story1	B3	3	Dead	LinStatic	55.9306	-0.672	0.002	0	0	0

Table 5.4 - Element Forces - Beams (Part 1 of 2, continued)

Story	Beam	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft
Story1	B3	3	Dead	LinStatic	57.8611	-0.672	0.002	0	0	0
Story1	B3	3	Dead	LinStatic	59.7917	-0.672	0.002	0	0	0
Story1	B3	3	Live	LinStatic	0.2083	-6.046	-0.016	0	0	0
Story1	B3	3	Live	LinStatic	2.1389	-6.046	-0.016	0	0	0
Story1	B3	3	Live	LinStatic	4.0694	-6.046	-0.016	0	0	0
Story1	B3	3	Live	LinStatic	6	-6.046	-0.016	0	0	0
Story1	B3	3	Live	LinStatic	6	-10.762	-0.003	0	0	0
Story1	B3	3	Live	LinStatic	8	-10.762	-0.003	0	0	0
Story1	B3	3	Live	LinStatic	10	-10.762	-0.003	0	0	0
Story1	B3	3	Live	LinStatic	12	-10.762	-0.003	0	0	0
Story1	B3	3	Live	LinStatic	12	-14.126	-0.006	0	0	0
Story1	B3	3	Live	LinStatic	14	-14.126	-0.006	0	0	0
Story1	B3	3	Live	LinStatic	16	-14.126	-0.006	0	0	0
Story1	B3	3	Live	LinStatic	18	-14.126	-0.006	0	0	0
Story1	B3	3	Live	LinStatic	18	-16.147	-0.002	0	0	0
Story1	B3	3	Live	LinStatic	20	-16.147	-0.002	0	0	0
Story1	B3	3	Live	LinStatic	22	-16.147	-0.002	0	0	0
Story1	B3	3	Live	LinStatic	24	-16.147	-0.002	0	0	0
Story1	B3	3	Live	LinStatic	24	-16.819	-0.001	0	0	0
Story1	B3	3	Live	LinStatic	26	-16.819	-0.001	0	0	0
Story1	B3	3	Live	LinStatic	28	-16.819	-0.001	0	0	0
Story1	B3	3	Live	LinStatic	30	-16.819	-0.001	0	0	0
Story1	B3	3	Live	LinStatic	30	-16.819	0.001	0	0	0
Story1	B3	3	Live	LinStatic	32	-16.819	0.001	0	0	0
Story1	B3	3	Live	LinStatic	34	-16.819	0.001	0	0	0
Story1	B3	3	Live	LinStatic	36	-16.819	0.001	0	0	0
Story1	B3	3	Live	LinStatic	36	-16.147	0.002	0	0	0
Story1	B3	3	Live	LinStatic	38	-16.147	0.002	0	0	0
Story1	B3	3	Live	LinStatic	40	-16.147	0.002	0	0	0
Story1	B3	3	Live	LinStatic	42	-16.147	0.002	0	0	0
Story1	B3	3	Live	LinStatic	42	-14.126	0.006	0	0	0
Story1	B3	3	Live	LinStatic	44	-14.126	0.006	0	0	0
Story1	B3	3	Live	LinStatic	46	-14.126	0.006	0	0	0
Story1	B3	3	Live	LinStatic	48	-14.126	0.006	0	0	0
Story1	B3	3	Live	LinStatic	48	-10.762	0.003	0	0	0
Story1	B3	3	Live	LinStatic	50	-10.762	0.003	0	0	0
Story1	B3	3	Live	LinStatic	52	-10.762	0.003	0	0	0
Story1	B3	3	Live	LinStatic	54	-10.762	0.003	0	0	0
Story1	B3	3	Live	LinStatic	54	-6.046	0.016	0	0	0
Story1	B3	3	Live	LinStatic	55.9306	-6.046	0.016	0	0	0
Story1	B3	3	Live	LinStatic	57.8611	-6.046	0.016	0	0	0
Story1	B3	3	Live	LinStatic	59.7917	-6.046	0.016	0	0	0
Story1	B3	3	AASHTO D + L	Combination	0.2083	-11.504	-0.03	0	0	0
Story1	B3	3	AASHTO D + L	Combination	2.1389	-11.504	-0.03	0	0	0
Story1	B3	3	AASHTO D + L	Combination	4.0694	-11.504	-0.03	0	0	0

Table 5.4 - Element Forces - Beams (Part 1 of 2, continued)

Story	Beam	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft
Story1	B3	3	AASHTO D + L	Combination	6	-11.504	-0.03	0	0	0
Story1	B3	3	AASHTO D + L	Combination	6	-20.477	-0.007	0	0	0
Story1	B3	3	AASHTO D + L	Combination	8	-20.477	-0.007	0	0	0
Story1	B3	3	AASHTO D + L	Combination	10	-20.477	-0.007	0	0	0
Story1	B3	3	AASHTO D + L	Combination	12	-20.477	-0.007	0	0	0
Story1	B3	3	AASHTO D + L	Combination	12	-26.879	-0.011	0	0	0
Story1	B3	3	AASHTO D + L	Combination	14	-26.879	-0.011	0	0	0
Story1	B3	3	AASHTO D + L	Combination	16	-26.879	-0.011	0	0	0
Story1	B3	3	AASHTO D + L	Combination	18	-26.879	-0.011	0	0	0
Story1	B3	3	AASHTO D + L	Combination	18	-30.724	-0.003	0	0	0
Story1	B3	3	AASHTO D + L	Combination	20	-30.724	-0.003	0	0	0
Story1	B3	3	AASHTO D + L	Combination	22	-30.724	-0.003	0	0	0
Story1	B3	3	AASHTO D + L	Combination	24	-30.724	-0.003	0	0	0
Story1	B3	3	AASHTO D + L	Combination	24	-32.003	-0.002	0	0	0
Story1	B3	3	AASHTO D + L	Combination	26	-32.003	-0.002	0	0	0
Story1	B3	3	AASHTO D + L	Combination	28	-32.003	-0.002	0	0	0
Story1	B3	3	AASHTO D + L	Combination	30	-32.003	-0.002	0	0	0
Story1	B3	3	AASHTO D + L	Combination	30	-32.003	0.002	0	0	0
Story1	B3	3	AASHTO D + L	Combination	32	-32.003	0.002	0	0	0
Story1	B3	3	AASHTO D + L	Combination	34	-32.003	0.002	0	0	0
Story1	B3	3	AASHTO D + L	Combination	36	-32.003	0.002	0	0	0
Story1	B3	3	AASHTO D + L	Combination	36	-30.724	0.003	0	0	0
Story1	B3	3	AASHTO D + L	Combination	38	-30.724	0.003	0	0	0
Story1	B3	3	AASHTO D + L	Combination	40	-30.724	0.003	0	0	0
Story1	B3	3	AASHTO D + L	Combination	42	-30.724	0.003	0	0	0
Story1	B3	3	AASHTO D + L	Combination	42	-26.879	0.011	0	0	0
Story1	B3	3	AASHTO D + L	Combination	44	-26.879	0.011	0	0	0
Story1	B3	3	AASHTO D + L	Combination	46	-26.879	0.011	0	0	0
Story1	B3	3	AASHTO D + L	Combination	48	-26.879	0.011	0	0	0
Story1	B3	3	AASHTO D + L	Combination	48	-20.477	0.007	0	0	0
Story1	B3	3	AASHTO D + L	Combination	50	-20.477	0.007	0	0	0
Story1	B3	3	AASHTO D + L	Combination	52	-20.477	0.007	0	0	0
Story1	B3	3	AASHTO D + L	Combination	54	-20.477	0.007	0	0	0
Story1	B3	3	AASHTO D + L	Combination	54	-11.504	0.03	0	0	0
Story1	B3	3	AASHTO D + L	Combination	55.9306	-11.504	0.03	0	0	0
Story1	B3	3	AASHTO D + L	Combination	57.8611	-11.504	0.03	0	0	0
Story1	B3	3	AASHTO D + L	Combination	59.7917	-11.504	0.03	0	0	0
Base	B3	34	Dead	LinStatic	0	6.157E-05	-0.002	0	0	0
Base	B3	34	Dead	LinStatic	2	6.157E-05	-0.002	0	0	0
Base	B3	34	Dead	LinStatic	4	6.157E-05	-0.002	0	0	0
Base	B3	34	Dead	LinStatic	6	6.157E-05	-0.002	0	0	0
Base	B3	34	Dead	LinStatic	6	0.672	-0.001	0	0	0
Base	B3	34	Dead	LinStatic	8	0.672	-0.001	0	0	0
Base	B3	34	Dead	LinStatic	10	0.672	-0.001	0	0	0
Base	B3	34	Dead	LinStatic	12	0.672	-0.001	0	0	0

Table 5.4 - Element Forces - Beams (Part 1 of 2, continued)

Story	Beam	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft
Base	B3	34	Dead	LinStatic	12	1.196	-0.001	0	0	0
Base	B3	34	Dead	LinStatic	14	1.196	-0.001	0	0	0
Base	B3	34	Dead	LinStatic	16	1.196	-0.001	0	0	0
Base	B3	34	Dead	LinStatic	18	1.196	-0.001	0	0	0
Base	B3	34	Dead	LinStatic	18	1.57	-0.0002771	0	0	0
Base	B3	34	Dead	LinStatic	20	1.57	-0.0002771	0	0	0
Base	B3	34	Dead	LinStatic	22	1.57	-0.0002771	0	0	0
Base	B3	34	Dead	LinStatic	24	1.57	-0.0002771	0	0	0
Base	B3	34	Dead	LinStatic	24	1.794	-0.0002124	0	0	0
Base	B3	34	Dead	LinStatic	26	1.794	-0.0002124	0	0	0
Base	B3	34	Dead	LinStatic	28	1.794	-0.0002124	0	0	0
Base	B3	34	Dead	LinStatic	30	1.794	-0.0002124	0	0	0
Base	B3	34	Dead	LinStatic	30	1.794	0.0002124	0	0	0
Base	B3	34	Dead	LinStatic	32	1.794	0.0002124	0	0	0
Base	B3	34	Dead	LinStatic	34	1.794	0.0002124	0	0	0
Base	B3	34	Dead	LinStatic	36	1.794	0.0002124	0	0	0
Base	B3	34	Dead	LinStatic	36	1.57	0.0002771	0	0	0
Base	B3	34	Dead	LinStatic	38	1.57	0.0002771	0	0	0
Base	B3	34	Dead	LinStatic	40	1.57	0.0002771	0	0	0
Base	B3	34	Dead	LinStatic	42	1.57	0.0002771	0	0	0
Base	B3	34	Dead	LinStatic	42	1.196	0.001	0	0	0
Base	B3	34	Dead	LinStatic	44	1.196	0.001	0	0	0
Base	B3	34	Dead	LinStatic	46	1.196	0.001	0	0	0
Base	B3	34	Dead	LinStatic	48	1.196	0.001	0	0	0
Base	B3	34	Dead	LinStatic	48	0.672	0.001	0	0	0
Base	B3	34	Dead	LinStatic	50	0.672	0.001	0	0	0
Base	B3	34	Dead	LinStatic	52	0.672	0.001	0	0	0
Base	B3	34	Dead	LinStatic	54	0.672	0.001	0	0	0
Base	B3	34	Dead	LinStatic	54	6.157E-05	0.002	0	0	0
Base	B3	34	Dead	LinStatic	56	6.157E-05	0.002	0	0	0
Base	B3	34	Dead	LinStatic	58	6.157E-05	0.002	0	0	0
Base	B3	34	Dead	LinStatic	60	6.157E-05	0.002	0	0	0
Base	B3	34	Live	LinStatic	0	0.001	-0.014	0	0	0
Base	B3	34	Live	LinStatic	2	0.001	-0.014	0	0	0
Base	B3	34	Live	LinStatic	4	0.001	-0.014	0	0	0
Base	B3	34	Live	LinStatic	6	0.001	-0.014	0	0	0
Base	B3	34	Live	LinStatic	6	6.046	-0.005	0	0	0
Base	B3	34	Live	LinStatic	8	6.046	-0.005	0	0	0
Base	B3	34	Live	LinStatic	10	6.046	-0.005	0	0	0
Base	B3	34	Live	LinStatic	12	6.046	-0.005	0	0	0
Base	B3	34	Live	LinStatic	12	10.762	-0.005	0	0	0
Base	B3	34	Live	LinStatic	14	10.762	-0.005	0	0	0
Base	B3	34	Live	LinStatic	16	10.762	-0.005	0	0	0
Base	B3	34	Live	LinStatic	18	10.762	-0.005	0	0	0
Base	B3	34	Live	LinStatic	18	14.126	-0.002	0	0	0

Table 5.4 - Element Forces - Beams (Part 1 of 2, continued)

Story	Beam	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft
Base	B3	34	Live	LinStatic	20	14.126	-0.002	0	0	0
Base	B3	34	Live	LinStatic	22	14.126	-0.002	0	0	0
Base	B3	34	Live	LinStatic	24	14.126	-0.002	0	0	0
Base	B3	34	Live	LinStatic	24	16.147	-0.002	0	0	0
Base	B3	34	Live	LinStatic	26	16.147	-0.002	0	0	0
Base	B3	34	Live	LinStatic	28	16.147	-0.002	0	0	0
Base	B3	34	Live	LinStatic	30	16.147	-0.002	0	0	0
Base	B3	34	Live	LinStatic	30	16.147	0.002	0	0	0
Base	B3	34	Live	LinStatic	32	16.147	0.002	0	0	0
Base	B3	34	Live	LinStatic	34	16.147	0.002	0	0	0
Base	B3	34	Live	LinStatic	36	16.147	0.002	0	0	0
Base	B3	34	Live	LinStatic	36	14.126	0.002	0	0	0
Base	B3	34	Live	LinStatic	38	14.126	0.002	0	0	0
Base	B3	34	Live	LinStatic	40	14.126	0.002	0	0	0
Base	B3	34	Live	LinStatic	42	14.126	0.002	0	0	0
Base	B3	34	Live	LinStatic	42	10.762	0.005	0	0	0
Base	B3	34	Live	LinStatic	44	10.762	0.005	0	0	0
Base	B3	34	Live	LinStatic	46	10.762	0.005	0	0	0
Base	B3	34	Live	LinStatic	48	10.762	0.005	0	0	0
Base	B3	34	Live	LinStatic	48	6.046	0.005	0	0	0
Base	B3	34	Live	LinStatic	50	6.046	0.005	0	0	0
Base	B3	34	Live	LinStatic	52	6.046	0.005	0	0	0
Base	B3	34	Live	LinStatic	54	6.046	0.005	0	0	0
Base	B3	34	Live	LinStatic	54	0.001	0.014	0	0	0
Base	B3	34	Live	LinStatic	56	0.001	0.014	0	0	0
Base	B3	34	Live	LinStatic	58	0.001	0.014	0	0	0
Base	B3	34	Live	LinStatic	60	0.001	0.014	0	0	0
Base	B3	34	AASHTO D + L	Combination	0	0.001	-0.026	0	0	0
Base	B3	34	AASHTO D + L	Combination	2	0.001	-0.026	0	0	0
Base	B3	34	AASHTO D + L	Combination	4	0.001	-0.026	0	0	0
Base	B3	34	AASHTO D + L	Combination	6	0.001	-0.026	0	0	0
Base	B3	34	AASHTO D + L	Combination	6	11.504	-0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	8	11.504	-0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	10	11.504	-0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	12	11.504	-0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	12	20.477	-0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	14	20.477	-0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	16	20.477	-0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	18	20.477	-0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	18	26.879	-0.005	0	0	0
Base	B3	34	AASHTO D + L	Combination	20	26.879	-0.005	0	0	0
Base	B3	34	AASHTO D + L	Combination	22	26.879	-0.005	0	0	0
Base	B3	34	AASHTO D + L	Combination	24	26.879	-0.005	0	0	0
Base	B3	34	AASHTO D + L	Combination	24	30.724	-0.004	0	0	0
Base	B3	34	AASHTO D + L	Combination	26	30.724	-0.004	0	0	0

Table 5.4 - Element Forces - Beams (Part 1 of 2, continued)

Story	Beam	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft
Base	B3	34	AASHTO D + L	Combination	28	30.724	-0.004	0	0	0
Base	B3	34	AASHTO D + L	Combination	30	30.724	-0.004	0	0	0
Base	B3	34	AASHTO D + L	Combination	30	30.724	0.004	0	0	0
Base	B3	34	AASHTO D + L	Combination	32	30.724	0.004	0	0	0
Base	B3	34	AASHTO D + L	Combination	34	30.724	0.004	0	0	0
Base	B3	34	AASHTO D + L	Combination	36	30.724	0.004	0	0	0
Base	B3	34	AASHTO D + L	Combination	36	26.879	0.005	0	0	0
Base	B3	34	AASHTO D + L	Combination	38	26.879	0.005	0	0	0
Base	B3	34	AASHTO D + L	Combination	40	26.879	0.005	0	0	0
Base	B3	34	AASHTO D + L	Combination	42	26.879	0.005	0	0	0
Base	B3	34	AASHTO D + L	Combination	42	20.477	0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	44	20.477	0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	46	20.477	0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	48	20.477	0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	48	11.504	0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	50	11.504	0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	52	11.504	0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	54	11.504	0.01	0	0	0
Base	B3	34	AASHTO D + L	Combination	54	0.001	0.026	0	0	0
Base	B3	34	AASHTO D + L	Combination	56	0.001	0.026	0	0	0
Base	B3	34	AASHTO D + L	Combination	58	0.001	0.026	0	0	0
Base	B3	34	AASHTO D + L	Combination	60	0.001	0.026	0	0	0

Table 5.4 - Element Forces - Beams (Part 2 of 2)

M3 kip-ft	Element	Elem Station ft	Location
0	3-1	0.2083	
0.0034	3-1	2.1389	
0.0068	3-1	4.0694	
0.0103	3-1	6	
0.0103	3-2	0	
0.011	3-2	2	
0.0118	3-2	4	
0.0126	3-2	6	
0.0126	3-3	0	
0.0138	3-3	2	
0.0151	3-3	4	
0.0163	3-3	6	
0.0163	3-4	0	
0.0167	3-4	2	
0.0171	3-4	4	
0.0175	3-4	6	
0.0175	3-5	0	
0.0178	3-5	2	

Table 5.4 - Element Forces - Beams (Part 2 of 2, continued)

M3 kip-ft	Element	Elem Station ft	Location
0.0181	3-5	4	
0.0183	3-5	6	
0.0183	3-6	0	
0.0181	3-6	2	
0.0178	3-6	4	
0.0175	3-6	6	
0.0175	3-7	0	
0.0171	3-7	2	
0.0167	3-7	4	
0.0163	3-7	6	
0.0163	3-8	0	
0.0151	3-8	2	
0.0138	3-8	4	
0.0126	3-8	6	
0.0126	3-9	0	
0.0118	3-9	2	
0.011	3-9	4	
0.0103	3-9	6	
0.0103	3-10	0	
0.0068	3-10	1.9306	
0.0034	3-10	3.8611	
0	3-10	5.7917	
0	3-1	0.2083	
0.0308	3-1	2.1389	
0.0616	3-1	4.0694	
0.0924	3-1	6	
0.0924	3-2	0	
0.0993	3-2	2	
0.1063	3-2	4	
0.1132	3-2	6	
0.1132	3-3	0	
0.1244	3-3	2	
0.1356	3-3	4	
0.1468	3-3	6	
0.1468	3-4	0	
0.1504	3-4	2	
0.1541	3-4	4	
0.1578	3-4	6	
0.1578	3-5	0	
0.1602	3-5	2	
0.1626	3-5	4	
0.165	3-5	6	
0.165	3-6	0	
0.1626	3-6	2	

Table 5.4 - Element Forces - Beams (Part 2 of 2, continued)

M3 kip-ft	Element	Elem Station ft	Location
0.1602	3-6	4	
0.1578	3-6	6	
0.1578	3-7	0	
0.1541	3-7	2	
0.1504	3-7	4	
0.1468	3-7	6	
0.1468	3-8	0	
0.1356	3-8	2	
0.1244	3-8	4	
0.1132	3-8	6	
0.1132	3-9	0	
0.1063	3-9	2	
0.0993	3-9	4	
0.0924	3-9	6	
0.0924	3-10	0	
0.0616	3-10	1.9306	
0.0308	3-10	3.8611	
0	3-10	5.7917	
0	3-1	0.2083	
0.0586	3-1	2.1389	
0.1172	3-1	4.0694	
0.1759	3-1	6	
0.1759	3-2	0	
0.189	3-2	2	
0.2022	3-2	4	
0.2153	3-2	6	
0.2153	3-3	0	
0.2366	3-3	2	
0.2579	3-3	4	
0.2793	3-3	6	
0.2793	3-4	0	
0.2862	3-4	2	
0.2932	3-4	4	
0.3002	3-4	6	
0.3002	3-5	0	
0.3048	3-5	2	
0.3094	3-5	4	
0.314	3-5	6	
0.314	3-6	0	
0.3094	3-6	2	
0.3048	3-6	4	
0.3002	3-6	6	
0.3002	3-7	0	
0.2932	3-7	2	

Table 5.4 - Element Forces - Beams (Part 2 of 2, continued)

M3 kip-ft	Element	Elem Station ft	Location
0.2862	3-7	4	
0.2793	3-7	6	
0.2793	3-8	0	
0.2579	3-8	2	
0.2366	3-8	4	
0.2153	3-8	6	
0.2153	3-9	0	
0.2022	3-9	2	
0.189	3-9	4	
0.1759	3-9	6	
0.1759	3-10	0	
0.1172	3-10	1.9306	
0.0586	3-10	3.8611	
0	3-10	5.7917	
0	34-1	0	
0.0031	34-1	2	
0.0062	34-1	4	
0.0093	34-1	6	
0.0093	34-2	0	
0.0105	34-2	2	
0.0117	34-2	4	
0.0128	34-2	6	
0.0128	34-3	0	
0.014	34-3	2	
0.0151	34-3	4	
0.0162	34-3	6	
0.0162	34-4	0	
0.0167	34-4	2	
0.0173	34-4	4	
0.0178	34-4	6	
0.0178	34-5	0	
0.0183	34-5	2	
0.0187	34-5	4	
0.0191	34-5	6	
0.0191	34-6	0	
0.0187	34-6	2	
0.0183	34-6	4	
0.0178	34-6	6	
0.0178	34-7	0	
0.0173	34-7	2	
0.0167	34-7	4	
0.0162	34-7	6	
0.0162	34-8	0	
0.0151	34-8	2	

Table 5.4 - Element Forces - Beams (Part 2 of 2, continued)

M3 kip-ft	Element	Elem Station ft	Location
0.014	34-8	4	
0.0128	34-8	6	
0.0128	34-9	0	
0.0117	34-9	2	
0.0105	34-9	4	
0.0093	34-9	6	
0.0093	34-10	0	
0.0062	34-10	2	
0.0031	34-10	4	
0	34-10	6	
0	34-1	0	
0.0278	34-1	2	
0.0556	34-1	4	
0.0835	34-1	6	
0.0835	34-2	0	
0.0942	34-2	2	
0.1049	34-2	4	
0.1156	34-2	6	
0.1156	34-3	0	
0.1256	34-3	2	
0.1355	34-3	4	
0.1455	34-3	6	
0.1455	34-4	0	
0.1505	34-4	2	
0.1555	34-4	4	
0.1605	34-4	6	
0.1605	34-5	0	
0.1643	34-5	2	
0.1682	34-5	4	
0.172	34-5	6	
0.172	34-6	0	
0.1682	34-6	2	
0.1643	34-6	4	
0.1605	34-6	6	
0.1605	34-7	0	
0.1555	34-7	2	
0.1505	34-7	4	
0.1455	34-7	6	
0.1455	34-8	0	
0.1355	34-8	2	
0.1256	34-8	4	
0.1156	34-8	6	
0.1156	34-9	0	
0.1049	34-9	2	

Table 5.4 - Element Forces - Beams (Part 2 of 2, continued)

M3 kip-ft	Element	Elem Station ft	Location
0.0942	34-9	4	
0.0835	34-9	6	
0.0835	34-10	0	
0.0556	34-10	2	
0.0278	34-10	4	
0	34-10	6	
0	34-1	0	
0.0529	34-1	2	
0.1059	34-1	4	
0.1588	34-1	6	
0.1588	34-2	0	
0.1792	34-2	2	
0.1995	34-2	4	
0.2199	34-2	6	
0.2199	34-3	0	
0.2389	34-3	2	
0.2579	34-3	4	
0.2769	34-3	6	
0.2769	34-4	0	
0.2864	34-4	2	
0.2959	34-4	4	
0.3054	34-4	6	
0.3054	34-5	0	
0.3127	34-5	2	
0.32	34-5	4	
0.3272	34-5	6	
0.3272	34-6	0	
0.32	34-6	2	
0.3127	34-6	4	
0.3054	34-6	6	
0.3054	34-7	0	
0.2959	34-7	2	
0.2864	34-7	4	
0.2769	34-7	6	
0.2769	34-8	0	
0.2579	34-8	2	
0.2389	34-8	4	
0.2199	34-8	6	
0.2199	34-9	0	
0.1995	34-9	2	
0.1792	34-9	4	
0.1588	34-9	6	
0.1588	34-10	0	
0.1059	34-10	2	

Table 5.4 - Element Forces - Beams (Part 2 of 2, continued)

M3 kip-ft	Element	Elem Station ft	Location
0.0529	34-10	4	
0	34-10	6	

Table 5.5 - Element Forces - Braces (Part 1 of 2)

Story	Brace	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft	M3 kip-ft
Story1	D2	36	Dead	LinStatic	0	0.95	0	0	0	0	0
Story1	D2	36	Dead	LinStatic	4.2426	0.95	0	0	0	0	0
Story1	D2	36	Dead	LinStatic	8.4853	0.95	0	0	0	0	0
Story1	D2	36	Live	LinStatic	0	8.549	0	0	0	0	0
Story1	D2	36	Live	LinStatic	4.2426	8.549	0	0	0	0	0
Story1	D2	36	Live	LinStatic	8.4853	8.549	0	0	0	0	0
Story1	D2	36	AASHTO D + L	Combination	0	16.267	0	0	0	0	0
Story1	D2	36	AASHTO D + L	Combination	4.2426	16.267	0	0	0	0	0
Story1	D2	36	AASHTO D + L	Combination	8.4853	16.267	0	0	0	0	0
Story1	D3	37	Dead	LinStatic	0	0.741	0	0	0	0	0
Story1	D3	37	Dead	LinStatic	4.2426	0.741	0	0	0	0	0
Story1	D3	37	Dead	LinStatic	8.4853	0.741	0	0	0	0	0
Story1	D3	37	Live	LinStatic	0	6.67	0	0	0	0	0
Story1	D3	37	Live	LinStatic	4.2426	6.67	0	0	0	0	0
Story1	D3	37	Live	LinStatic	8.4853	6.67	0	0	0	0	0
Story1	D3	37	AASHTO D + L	Combination	0	12.691	0	0	0	0	0
Story1	D3	37	AASHTO D + L	Combination	4.2426	12.691	0	0	0	0	0
Story1	D3	37	AASHTO D + L	Combination	8.4853	12.691	0	0	0	0	0
Story1	D4	38	Dead	LinStatic	0	0.529	0	0	0	0	0
Story1	D4	38	Dead	LinStatic	4.2426	0.529	0	0	0	0	0
Story1	D4	38	Dead	LinStatic	8.4853	0.529	0	0	0	0	0
Story1	D4	38	Live	LinStatic	0	4.758	0	0	0	0	0
Story1	D4	38	Live	LinStatic	4.2426	4.758	0	0	0	0	0
Story1	D4	38	Live	LinStatic	8.4853	4.758	0	0	0	0	0
Story1	D4	38	AASHTO D + L	Combination	0	9.053	0	0	0	0	0
Story1	D4	38	AASHTO D + L	Combination	4.2426	9.053	0	0	0	0	0
Story1	D4	38	AASHTO D + L	Combination	8.4853	9.053	0	0	0	0	0
Story1	D5	39	Dead	LinStatic	0	0.318	0	0	0	0	0
Story1	D5	39	Dead	LinStatic	4.2426	0.318	0	0	0	0	0
Story1	D5	39	Dead	LinStatic	8.4853	0.318	0	0	0	0	0
Story1	D5	39	Live	LinStatic	0	2.858	0	0	0	0	0
Story1	D5	39	Live	LinStatic	4.2426	2.858	0	0	0	0	0
Story1	D5	39	Live	LinStatic	8.4853	2.858	0	0	0	0	0
Story1	D5	39	AASHTO D + L	Combination	0	5.438	0	0	0	0	0
Story1	D5	39	AASHTO D + L	Combination	4.2426	5.438	0	0	0	0	0
Story1	D5	39	AASHTO D + L	Combination	8.4853	5.438	0	0	0	0	0
Story1	D6	40	Dead	LinStatic	0	0.106	0	0	0	0	0
Story1	D6	40	Dead	LinStatic	4.2426	0.106	0	0	0	0	0

Table 5.5 - Element Forces - Braces (Part 1 of 2, continued)

Story	Brace	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft	M3 kip-ft
Story1	D6	40	Dead	LinStatic	8.4853	0.106	0	0	0	0	0
Story1	D6	40	Live	LinStatic	0	0.95	0	0	0	0	0
Story1	D6	40	Live	LinStatic	4.2426	0.95	0	0	0	0	0
Story1	D6	40	Live	LinStatic	8.4853	0.95	0	0	0	0	0
Story1	D6	40	AASHTO D + L	Combination	0	1.808	0	0	0	0	0
Story1	D6	40	AASHTO D + L	Combination	4.2426	1.808	0	0	0	0	0
Story1	D6	40	AASHTO D + L	Combination	8.4853	1.808	0	0	0	0	0
Story1	D7	41	Dead	LinStatic	0	0.106	0	0	0	0	0
Story1	D7	41	Dead	LinStatic	4.2426	0.106	0	0	0	0	0
Story1	D7	41	Dead	LinStatic	8.4853	0.106	0	0	0	0	0
Story1	D7	41	Live	LinStatic	0	0.95	0	0	0	0	0
Story1	D7	41	Live	LinStatic	4.2426	0.95	0	0	0	0	0
Story1	D7	41	Live	LinStatic	8.4853	0.95	0	0	0	0	0
Story1	D7	41	AASHTO D + L	Combination	0	1.808	0	0	0	0	0
Story1	D7	41	AASHTO D + L	Combination	4.2426	1.808	0	0	0	0	0
Story1	D7	41	AASHTO D + L	Combination	8.4853	1.808	0	0	0	0	0
Story1	D8	42	Dead	LinStatic	0	0.318	0	0	0	0	0
Story1	D8	42	Dead	LinStatic	4.2426	0.318	0	0	0	0	0
Story1	D8	42	Dead	LinStatic	8.4853	0.318	0	0	0	0	0
Story1	D8	42	Live	LinStatic	0	2.858	0	0	0	0	0
Story1	D8	42	Live	LinStatic	4.2426	2.858	0	0	0	0	0
Story1	D8	42	Live	LinStatic	8.4853	2.858	0	0	0	0	0
Story1	D8	42	AASHTO D + L	Combination	0	5.438	0	0	0	0	0
Story1	D8	42	AASHTO D + L	Combination	4.2426	5.438	0	0	0	0	0
Story1	D8	42	AASHTO D + L	Combination	8.4853	5.438	0	0	0	0	0
Story1	D9	43	Dead	LinStatic	0	0.529	0	0	0	0	0
Story1	D9	43	Dead	LinStatic	4.2426	0.529	0	0	0	0	0
Story1	D9	43	Dead	LinStatic	8.4853	0.529	0	0	0	0	0
Story1	D9	43	Live	LinStatic	0	4.758	0	0	0	0	0
Story1	D9	43	Live	LinStatic	4.2426	4.758	0	0	0	0	0
Story1	D9	43	Live	LinStatic	8.4853	4.758	0	0	0	0	0
Story1	D9	43	AASHTO D + L	Combination	0	9.053	0	0	0	0	0
Story1	D9	43	AASHTO D + L	Combination	4.2426	9.053	0	0	0	0	0
Story1	D9	43	AASHTO D + L	Combination	8.4853	9.053	0	0	0	0	0
Story1	D10	44	Dead	LinStatic	0	0.741	0	0	0	0	0
Story1	D10	44	Dead	LinStatic	4.2426	0.741	0	0	0	0	0
Story1	D10	44	Dead	LinStatic	8.4853	0.741	0	0	0	0	0
Story1	D10	44	Live	LinStatic	0	6.67	0	0	0	0	0
Story1	D10	44	Live	LinStatic	4.2426	6.67	0	0	0	0	0
Story1	D10	44	Live	LinStatic	8.4853	6.67	0	0	0	0	0
Story1	D10	44	AASHTO D + L	Combination	0	12.691	0	0	0	0	0
Story1	D10	44	AASHTO D + L	Combination	4.2426	12.691	0	0	0	0	0
Story1	D10	44	AASHTO D + L	Combination	8.4853	12.691	0	0	0	0	0
Story1	D11	45	Dead	LinStatic	0	0.95	0	0	0	0	0
Story1	D11	45	Dead	LinStatic	4.2426	0.95	0	0	0	0	0

Table 5.5 - Element Forces - Braces (Part 1 of 2, continued)

Story	Brace	Unique Name	Output Case	Case Type	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft	M3 kip-ft
Story1	D11	45	Dead	LinStatic	8.4853	0.95	0	0	0	0	0
Story1	D11	45	Live	LinStatic	0	8.549	0	0	0	0	0
Story1	D11	45	Live	LinStatic	4.2426	8.549	0	0	0	0	0
Story1	D11	45	Live	LinStatic	8.4853	8.549	0	0	0	0	0
Story1	D11	45	AASHTO D + L	Combination	0	16.267	0	0	0	0	0
Story1	D11	45	AASHTO D + L	Combination	4.2426	16.267	0	0	0	0	0
Story1	D11	45	AASHTO D + L	Combination	8.4853	16.267	0	0	0	0	0

Table 5.5 - Element Forces - Braces (Part 2 of 2)

Element	Elem Station ft	Location
36	0	
36	4.2426	
36	8.4853	
36	0	
36	4.2426	
36	8.4853	
36	0	
36	4.2426	
36	8.4853	
37	0	
37	4.2426	
37	8.4853	
37	0	
37	4.2426	
37	8.4853	
37	0	
37	4.2426	
37	8.4853	
38	0	
38	4.2426	
38	8.4853	
38	0	
38	4.2426	
38	8.4853	
38	0	
38	4.2426	
38	8.4853	
39	0	
39	4.2426	
39	8.4853	
39	0	
39	4.2426	
39	8.4853	

Table 5.5 - Element Forces - Braces (Part 2 of 2, continued)

Element	Elem Station ft	Location
39	0	
39	4.2426	
39	8.4853	
40	0	
40	4.2426	
40	8.4853	
40	0	
40	4.2426	
40	8.4853	
40	0	
40	4.2426	
40	8.4853	
41	0	
41	4.2426	
41	8.4853	
41	0	
41	4.2426	
41	8.4853	
41	0	
41	4.2426	
41	8.4853	
42	0	
42	4.2426	
42	8.4853	
42	0	
42	4.2426	
42	8.4853	
42	0	
42	4.2426	
42	8.4853	
43	0	
43	4.2426	
43	8.4853	
43	0	
43	4.2426	
43	8.4853	
43	0	
43	4.2426	
43	8.4853	
44	0	
44	4.2426	
44	8.4853	
44	0	
44	4.2426	

Table 5.5 - Element Forces - Braces (Part 2 of 2, continued)

Element	Elem Station ft	Location
44	8.4853	
44	0	
44	4.2426	
44	8.4853	
45	0	
45	4.2426	
45	8.4853	
45	0	
45	4.2426	
45	8.4853	
45	0	
45	4.2426	
45	8.4853	

Steel Truss Bridge - 180 ft span

Ref: AASHTO Ped Bridges and AASHTO LRFD (2020)

Determine size of HSS longitudinal members

PL =	90	psf (pedestrian loading, PL)	Member:	HSS9x9x5/8	
DC =	5	psf	A_g =	18.70	in^2
DW =	5	psf	r_s =	3.4	in
			b/t =	12.5	
F_y =	46	ksi	Z =	58.1	in^3
			I =	216	in^4
w =	72	in (assumed)	E =	29000	ksi
Span =	180	ft			
h =	120	in (vertical dim, center to center of HSS)			
L =	135	in, length between panel points			
n =	16	number of panel points			

Strength I load cominbation: 1.75PL + 1.25DC + 1.5DW

Reference AASHTO LRFD for box section design

Mu =	4161375	lbft
Mu/d =	416137.5	lb
1/2 * Mu/d =	208069	lb (= Tu = Cu)

ϕ = 0.95 (axial compression and tension yielding)

Tension, AASHTO 6.8.2

$\phi P_n = \phi\ F_y\ A_g > Tu$

Solve for A_g:

Min A_g = 4.76 in^2

DCR = 0.25

Compression, AASHTO 6.9.2 & 6.9.3, and Ped Bridge 7.1

$$C = \frac{E}{h^2[(h/3I_c) + (b/2I_b)]}$$

Ref Ped Bridge pg. 22 and Galambos, 1968

C =	5.72	k/in (assumes all members same section)
P_c =	276.7	k (1.33 x factored compressive load)
CL/P_c =	2.79	
n =	16	
1/K =	0.98	from Ped Bridge Table 7.1.2-1

Check slenderness

KL/r < 120 main members (6.9.3)

40.51621 <120

Kl/r < 140 bracing members (6.9.3)

40.51621 <140

ϕP_n:

$P_o = F_y A_g =$ 860.2 kip

$$P_e = \frac{\pi^2 E}{\left(\frac{Kl}{r_s}\right)^2} A_g$$

K = 1.020408

$P_e =$ 3260.48 k

Check slenderness per AASHTO Table 6.9.4.2.1-1

b/t =	12.5
limit =	35.2
slender?	no

$P_o/P_e =$ 0.3

$P_n =$ 770.27 kip eqn 6.9.4.1.1-1

$\phi P_n =$ 731.76 kip

DCR = 0.28

Lateral force on post shall not be less than 0.01/K x average factored design compressive force

Check 0.01/K > 0.003 Section 7.1.1

0.01/K = 0.0098 >0.003

if 0.01/K < 0.003, use 0.003

force =	2039.1	lb (use max force rather than average, conservative)
min force =	624.2	lb
design force =	2039.1	lb

quick check (does not include axial)

cantilever moment = force*h

factored moment =	20390.7	lbft
ϕ_f =	1.0	
$\phi_f M_n$ =	2672.6	kin
$\phi_f M_n$ =	222716.7	lbft

DCR = 0.092

Suspension Bridge option w/towers

Length of Main Span L_m	180	ft		
Length of Back Spans D	47.5	ft (L_b = 2.5*H)		
Flare; F/D = 3 %	1.425	ft		
Cradle, k	2	ft		
Total Height of Tower above Grade, H	19	ft		
Bridge Width	5	ft		
Tower Width	2	ft		
Dead Load	11	plf		(DL per side)
Water	0	plf		
LL (psf)	90	psf		
Live Load (plf)	225	plf		(LL per side)
E of Cable=	12000	ksi		
# cables per side =	1			

Main cable: Cable diameter = **2** **in** DCR = 0.94 FOS = 5

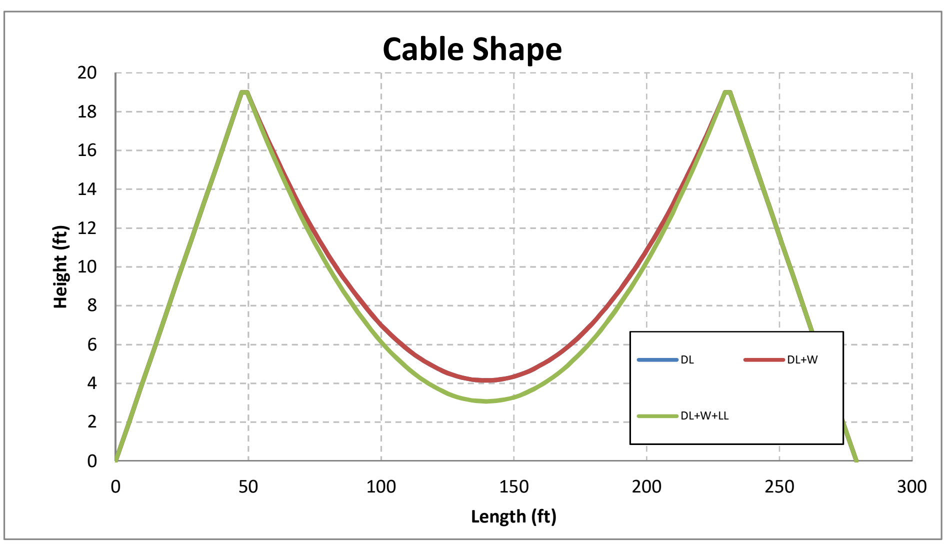
	Tension in Cable k	Max Sag ft	Length ft	Strain in/in	Delta L ft	Cable Length ft
DL	3	14.85	289.5334	0.0001	0.038502	289.4949
DL+Water	3	14.85	289.5334	0.0001	0.038502	289.4949
DL+LL	60	15.93	290.0096	0.0027	0.7713	289.2383

Force in towers: At towers, sum of the forces in vertical = zero, therefore P (tower force) = 2Tcosθ
m (rise/run) = wL/(T2)

w =	236	plf
L =	180	ft
T =	60000	lbs
m =	0.35	
tan θ = 1/m, θ =	70.51	degrees
P =	40045	lb (compression force in 1 tower)

**Note: assumes force in cable to the anchor = force in main span cable

theta back span =	68.20	degrees
delta degrees =	2.31	degrees



Hanger Cables:

Hanger Spacing 11.25 ft

Hanger #	Location	DL sag	hanger length	hanger length
	ft	ft	ft	m
0	0	0.00	19.00	
1	11.25	3.48	15.52	4.7316
2	22.5	6.50	12.50	3.8119
3	33.75	9.05	9.95	3.0338
4	45	11.14	7.86	2.3971
5	56.25	12.76	6.24	1.9019
6	67.5	13.92	5.08	1.5482
7	78.75	14.62	4.38	1.336
8	90	14.85	4.15	1.2652

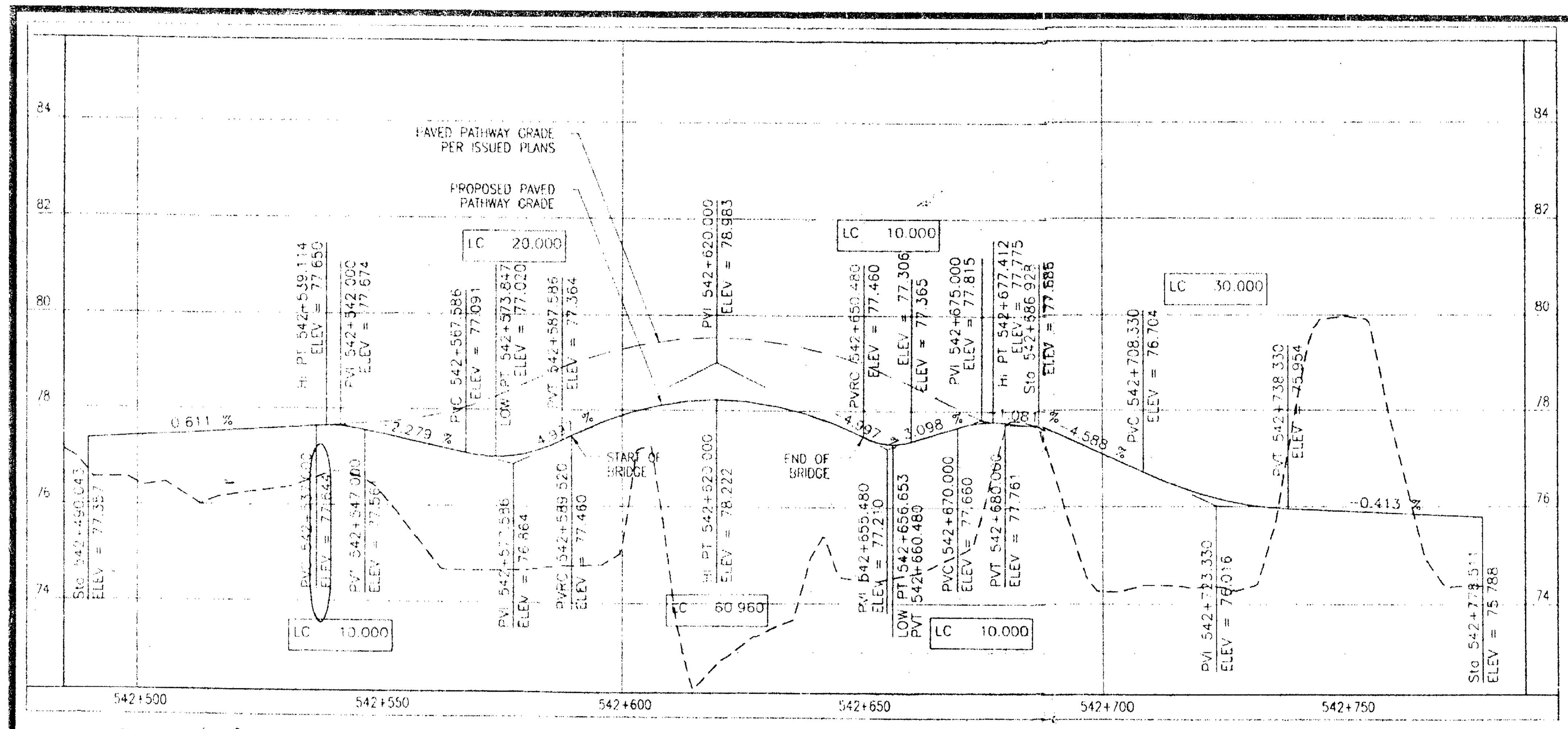
Load on hanger = 2655 lbs

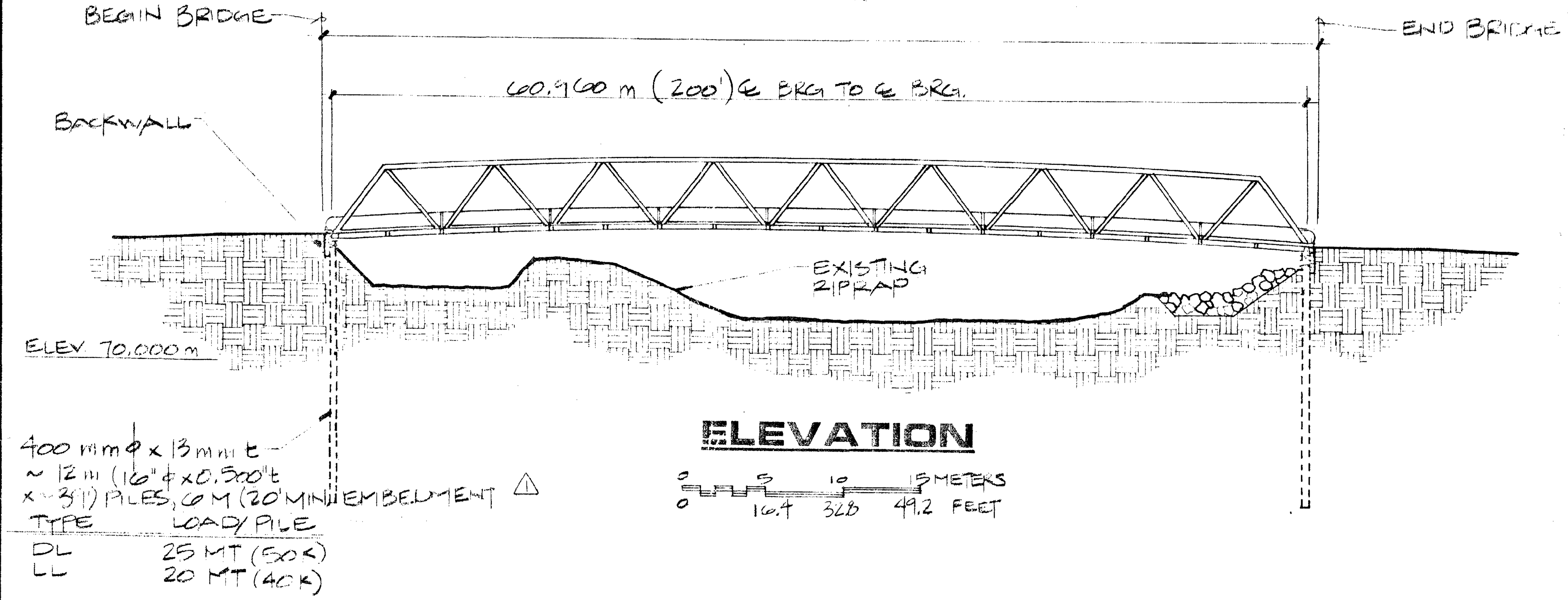
cable diameter = 3/8 in

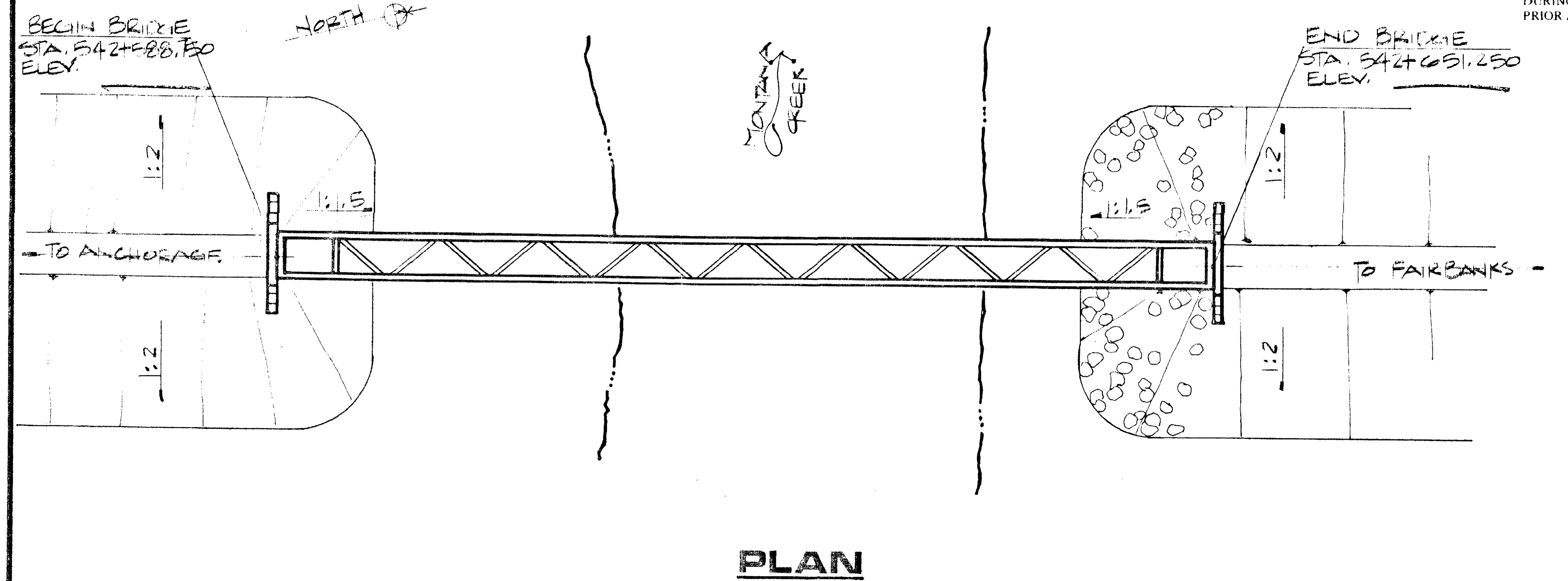
DCR = 0.92

Appendix E – Montana Creek Pedestrian References (Relocated Bridge Option)

The following section contains design drawings, shop drawings, and inspection report for the Montana Creek Pedestrian Bridge.







GENERAL NOTES

DESIGN CRITERIA

DESIGN SHALL CONFORM TO 1998 AASHTO AND 1998 ALASKA DOT/PF STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION METRIC EDITION (HEREAFTER REFERRED TO AS ADOT STANDARD) WITH ALL APPLICABLE ADDENDUMS.

PEDESTRIAN BRIDGE
LIVE LOAD 4.1 KPa (85 PSF)
45 kN (10,000#) SERVICE TRUCK
WIND LOAD 3.6 KPa (75 PSF)
LIVE LOAD DEFLECTION LIMIT L/1000
PEDESTRIAN BRIDGE RAIL 75 Kg/M (50 PLF)

SEISMIC LOAD AASHTO ZONE 4, NON-ESSENTIAL
ACCELERATION COEFFICIENT A=0.469
SOIL TYPE II S=1.2
INCLUDE 480 Pa (10 PSF) SNOW LOAD WITH SEISMIC DEAD LOAD.

PILES

PILES SHALL BE FURNISHED, DRIVEN, AND INSTALLED ACCORDING TO ADOT STANDARD SECTION 505 "PILING", SECTION 715 "STEEL FOR PILING", AND PROJECT SPECIAL PROVISIONS SECTION 505 UNLESS NOTED OTHERWISE. SEE SOIL LOGS FOR DRIVING INFORMATION.

UNCOATED STEEL PIPE PILES SHALL CONFORM TO ASTM A252 GRADE 2 OR 3 WITH ASTM A36 CHEMISTRY SUITABLE FOR WELDING. ALL PILES SHALL BE FITTED WITH AN INSIDE FLANGE OPEN DRIVING SHOE CONFORMING TO ASTM A27 GRADE 65-35 OR 148 GRADE 90-60. PROVIDE PILE MILL CERTS AND SUBMIT CARBON EQUIVALENCY FOR REVIEW.

PIPE PILES MAY BE PERMITTED TO BE SET AND INITIALLY EMBEDDED FOR 15 FEET WITH A VIBRATORY HAMMER AFTER WHICH PILES SHALL BE DRIVEN TO FINAL EMBEDMENT WITH AN IMPACT HAMMER. MINIMUM RATED HAMMER ENERGY SHALL BE 4000 Kg-M (30,000 FT-LB) AND MINIMUM RAM WEIGHT SHALL BE 1800 Kg (4000 LB). SUBMIT HAMMER INFORMATION TO ENGINEER FOR WAVE EQUATION ANALYSIS.

ALL PIPE PILE INSTALLATION SHALL BE CONDUCTED WITH THE ENGINEER PRESENT. THE CONTRACTOR SHALL ASSIST THE ENGINEER IN RECORDING BLOW COUNTS. PILE CAPACITIES SHALL BE DETERMINED SOLELY BY THE ENGINEER BY THE MODIFIED ENR FORMULA, WITH A MINIMUM FACTOR OF SAFETY OF 2.75 TIMES LOADS SHOWN ON THESE DRAWINGS. CONFIRM EQUIPMENT AND MATERIAL COMPATABILITY WITH WAVE EQUATION PRIOR TO MOBILIZING EQUIPMENT. DRIVING DIFFICULTIES MAY REQUIRE CONTRACTOR TO SPUD OR PRE-BORE HOLES PRIOR TO DRIVING.

THE CONTRACTOR SHALL MARK EACH PILE AT 300 MILLIMETER INTERVALS (1 FOOT) WITH 1500 MILLIMETER (5 FOOT) INTERVALS MARKED AND INDICATED. CONTRACTOR SHALL MARK 1-INCH INCREMENTS DURING FINAL DRIVING.

ALL PILES SHALL BE DRIVEN TO WITHIN 50 mm (2 INCHES) OF SPECIFIED LOCATION AT CUT-OFF AND 75mm (3 INCHES) AT GROUND ELEVATION. PILES SHALL BE WITHIN 1% OF ALIGNMENT.

STRUCTURAL STEEL

ALL STEEL MATERIALS, FABRICATION, CONSTRUCTION, AND ERECTION SHALL CONFORM TO ADOT STANDARD 504, "STEEL STRUCTURES", AND 716, "STRUCTURAL STEEL". WELDING AND FABRICATIONS SHALL CONFORM TO THE 2000 AWS D1.1.

305 mmφx12.7mm (12"φx0.500t") DIAMETER PIPE - Fy=275 MPa (40 KSI), Fu=415 MPa (60 KSI), $E_{2"}$ =15%
ALL OTHER PIPE - Fy=240 MPa (35 KSI), Fu=415 MPa (60 KSI), $E_{2"}$ =15%
ALL PIPE SHALL HAVE CHEMISTRY SUITABLE FOR WELDING, PIPE MAY BE ASTM A53 GRADE B, A106 GRADE C, A252 GRADE 3, API 5L GRADE B OR X42, OR ENGINEER APROVED EQUAL, MEETING ADDITIONAL MINIMUM REQUIREMENTS AS INDICATED. SPIRAL WELDED PIPE SHALL NOT BE PERMITTED.
TUBING SHALL BE ASTM A500 GRADE B.
SPLICE AND BEARING PLATES SHALL BE A572 OR OTHER SUITABLE GRADE 345 MPa (50 KSI).
STEEL CAP BEAMS SHALL CONFORM TO ASTM A709 OR A572, GR. 345 MPa (50 ksi).
MISCELLANEOUS STEEL PLATES, ANGLES AND CHANNELS SHALL BE ASTM A36.

METAL BRIDGE DECKING SHALL BE PRE-GALVANIZED ASTM A607 340 MPa (50 KSI) OR ASTM A570 310 MPa (45 KSI) MIN., PREGALVANIZED PER ASTM A924. WELD EACH PIECE TO ALL STRINGERS WITH DECK THICKNESS x 75mm (3") FILLET WELDS. WELD BOTH SIDES AND ENDS OF OVERLAPPING SIDE JOINT SLOTS WITH DECK THICKNESS FILLET WELDS. SLOTS SHALL BE 17.5mmx64mm @400mm O.C. (11/16"x2.5"@ 16"O.C.). END SPLICES ARE TO BE STAGGERED AND LOCATED OVER STRINGERS. CRESTS OF END SPLICES ARE TO BE BUTT-WELDED. OVERLAPPING SIDE JOINTS MAY BE BOLTED WITH 15.9mm (5/8") DIAMETER A325 HIGH STRENGTH BOLTS. REPAIR GALVANIZING AT WELDS AS INDICATED BELOW FOR FIELD REPAIR. PROVIDE THE FOLLOWING MINIMUM DECK PROPERTIES.

DECK TYPE	THICKNESS	S cm^3/M (IN^3/FT)	I cm^4/M (IN^4/FT)
108x305 (4.25"x12")	4.5mm (7 Ga - 0.179")	233 (4.34)	1407 (10.34)

DURING CONSTRUCTION DO NOT LOAD INDIVIDUAL DECK SPANS GREATER THAN 9.1 MT (20 KIPS) WITHOUT PRIOR APROVAL OF THE ENGINEER.

WELDING FILLERS SHALL BE AS FOLLOWS: SMA - E7018-1 OR E8018. FCAW - E71T8-Ni 1% or Ni 2%. OR ENGINEER APPROVED EQUAL. PROVIDE MINIMUM PREHEAT FOR FIELD WELDS OF 40 DEG. C (100 DEG. F).

HIGH STRENGTH BOLTS (HSB) FOR SPLICE CONNECTIONS SHALL BE ASTM A320 L7 OR L43 OR A193 B7, WITH THREADS EXCLUDED FROM THE SHEAR PLANE, WITH CLIPPED CIRCULAR HARDENED WASHERS AND LOAD INDICATING WASHERS. PROVIDE LOAD INDICATING WASHERS CONFORMING TO ASTM F959. PROVIDE HEAVY HEX NUTS CONFORMING TO ASTM A194 GRADE 4 OR 7. BOLTS AND NUTS SHALL BE FACTORY LUBRICATED. SPLICE CONNECTION HOLES SHALL BE SUB-PUNCHED AND REAMED TO FIT DURRING PARTIAL ASSEMBLY FIT-UP PRIOR TO SHIPMENT.

OTHER HIGH STRENGTH BOLTS SHALL BE ASTM A325 TYPE 3, A193 B7, OR A320 L7 OR L43, WITH THREADS EXCLUDED FROM THE SHEAR PLANE AND CIRCULAR HARDENED WASHERS CONFORMING TO ASTM F436. PROVIDE HEAVY HEX NUTS TO ASTM A194 GRADE 4 OR 7, OR ASTM A563 CLASS DH3, AS APPROPRIATE TO MATCH BOLTS.

ALL STRUCTURAL STEEL, STEEL CONNECTIONS AND STEEL HARDWARE SHALL BE GALVANIZED, SPRAY METALIZED, OR STAINLESS STEEL. ALL EXPOSED EDGES SHALL BE GROUND TO A SMOOTH RADIUS OR CONTOUR. ALL TUBULAR SECTIONS SHALL BE PROVIDED WITH END PLATES AND SHALL BE SEAL WELDED.

RUBBER BEARINGS SHALL BE 50 DUROMETER SHORE HARDNESS, GRADE E. PROVIDE REINFORCING AS SHOWN ON DRAWINGS.

SUPERSTRUCTURE SHALL BE FABRICATED BY AISC CERTIFIED MAJOR STEEL BRIDGE FABRICATOR. AN AWS CWI EXPERIENCED WITH VISUAL INSPECTION OF TUBULAR STRUCTURES SHALL PERFORM PIPE FIT-UP INSPECTION IN ADDITION TO POST-WELD INSPECTION. SUBMIT SHOP DRAWINGS AND ERECTION PLAN FOR REVIEW.

GALVANIZING

CONTRACTOR SHALL SUBMIT GALVANIZING PLAN TO THE ENGINEER FOR APPROVAL, INCLUDING INTENDED LOCATION FOR VENTING HOLES AND METHODS FOR REPAIR OF HOLES. SPRAY METALIZING MAY BE USED IN LIEU OF HOT-DIP GALVANIZING AS APPROVED BY THE ENGINEER. GALVANIZE IN ACCORDANCE WITH ASTM A525 G185. STEEL HARDWARE SHALL BE GALVANIZED PER ASTM A153 WITH A CLASS A OR B COATING. PROVIDE A MINIMUM OF 610 GRAMS PER SQUARE METER (2 OZ. PER SQUARE FOOT) UNLESS NOTED OTHERWISE. PROVIDE SEALER PER SSPC GUIDE 23.

FIELD REPAIR ALL DAMAGED GALVANIZED AREAS WITH "GALV STICKS" OR EQUAL, AND WITH A 5 MIL TOPCOAT OF COLD GALVANIZING PAINT, OR SPRAY METALIZE. SUBMIT REPAIR PROCEDURE TO ENGINEER FOR APPROVAL.

CONCRETE

CONCRETE SHALL CONFORM TO ADOT STANDARD SPECIFICATIONS SECTION 501 "STRUCTURAL CONCRETE",UNLESS NOTED OTHERWISE.

CLASS "A" - PRECAST CONCRETE. MINIMUM DESIGN STRENGTH OF 28 MPa (4000 PSI). MAXIMUM WATER CEMENT RATIO SHALL BE 0.40. PROVIDE 4-7 % AIR ENTRAINMENT.

PROVIDE A SMOOTH RUBBED FINISH ON ALL EXPOSED VERTICAL OR OVERHEAD SURFACES. PROVIDE A HEAVY BROOMED FINISH ON ALL HORIZONTAL WALKING SURFACES. PROVIDE 45 DEG. x 19mm (3/4 INCH) CHAMFER AT ALL EXPOSED EDGES UNLESS NOTED OTHERWISE.

REINFORCING STEEL SHALL CONFORM TO ASTM A615 GR. 400 MPa (60 KSI). CONCRETE AND REINFORCEMENT TOLERANCES SHALL CONFORM TO ACI 117.

SUBMIT: 1) CONCRETE MIX SPECIFICATIONS INCLUDING SUPPORTING TEST DOCUMENTATION FOR ALL CLASSES OF CONCRETE, 2) CONCRETE SHOP DRAWINGS FOR REVIEW.

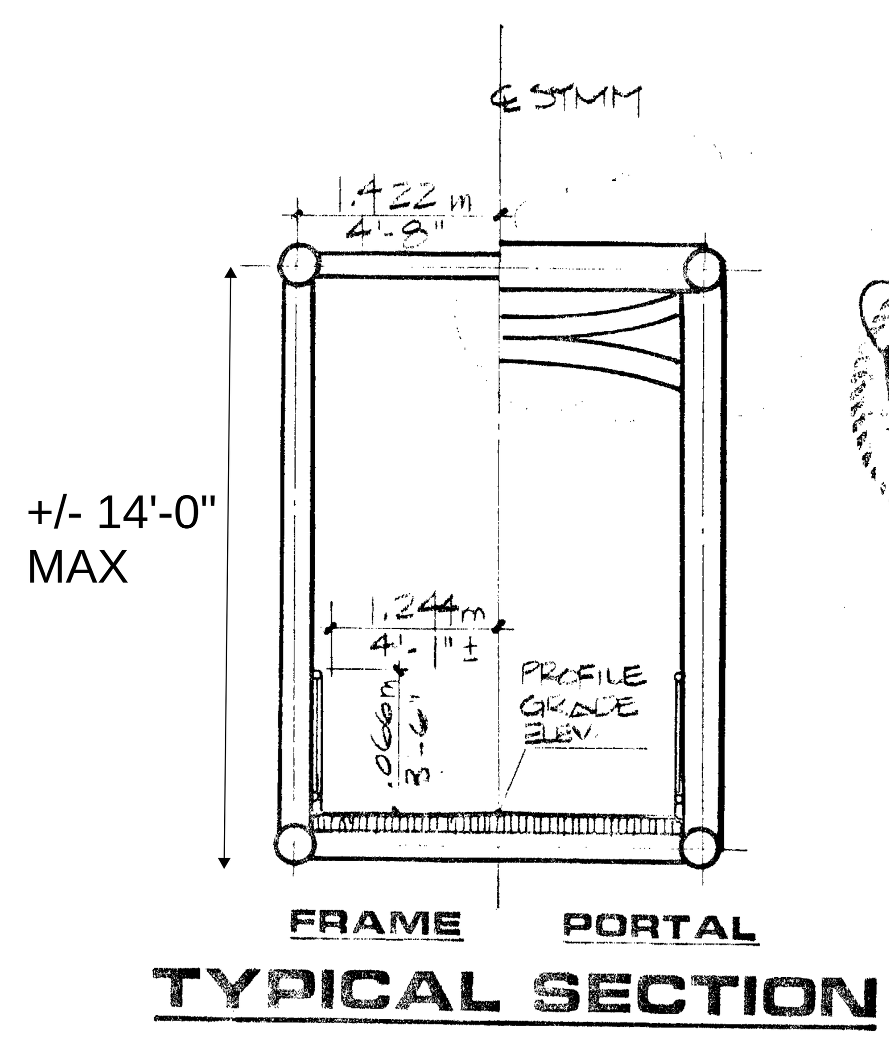
ASPHALT CONCRETE

PROVIDE ASPHALT CONCRETE PAVEMENT ON BRIDGE PER JOB SPECIFICATIONS AND ADOT STANDARD, WITH THE FOLLOWING MODIFICATIONS. PROVIDE BITUMINOUS TACK COAT ON METAL BRIDGE DECK PRIOR TO PAVING. DO NOT USE VIBRATORY ROLLER ON BRIDGE. CLEAN OVERSPRAY OFF STRUCTURE.

ESTIMATE OF BRIDGE QUANTITIES

ITEM	QUANTITY
PILING	4 EA.
STEEL PILECAPS	0.56 MT
PRECAST BACKWALLS	4 CM
GALVANIZED STRUCTURAL STEEL	41 MT
BRIDGE RAILING	125 LM
METAL DECKING	150 LM
ASPHALT CONCRETE	13.3 CM

MONTANA CREEK BRIDGE DESIGN DRAWINGS



APPROVED AS NOTED
BRIDGE DESIGN SECTION
ALASKA DEPARTMENT OF TRANSPORTATION AND PUBLIC FACILITIES
Reviewed for conformance with basic details and dimensions of the contract plans. Accuracy of information supplied by the fabricator is not guaranteed.
By EEM Date APR 14 2000

MONTANA CREEK PEDESTRIAN BRIDGE
BRIDGE NO. 1954

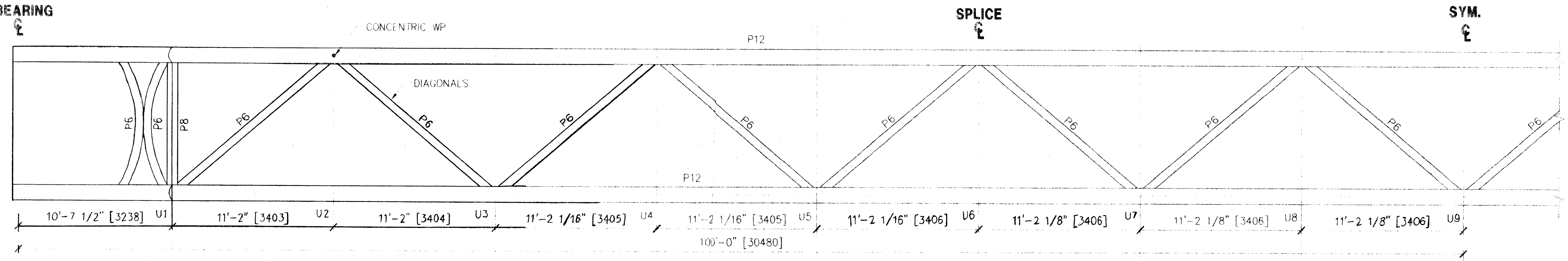
Designed: GH/DN
Drawn: ANE
Checked: DN/GB
Project No. 99110
Date: FEB. 2000
Scale: -

Peratrovich, Nottingham & Drage, Inc.
Engineering Consultants
1506 West 36th Avenue,
Anchorage, Alaska 99503
(907) 561-1011

LAYOUT & NOTES

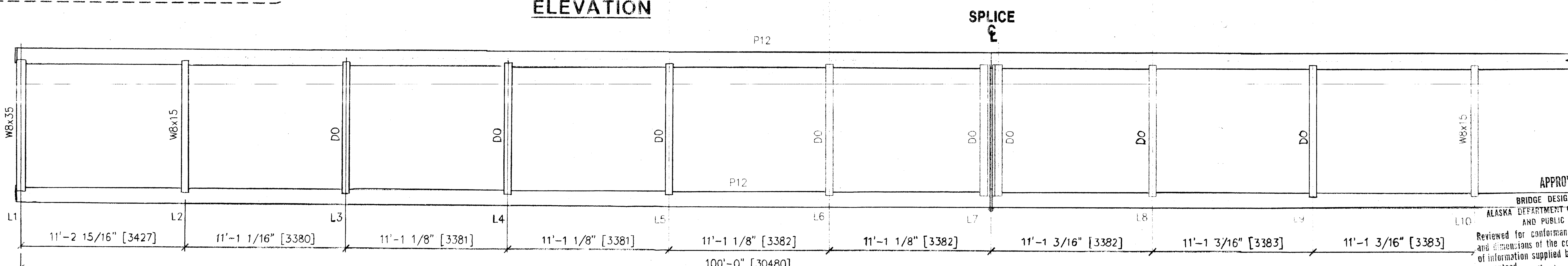
Sheet S1 of 4

△ REV. MARCH 2000 - GH



TOP CHORD FRAMING PLAN

ELEVATION



MEMBER	SECTION	METRIC
P12	12"ø×0.500"t	300ø×12.7
P8	8"ø×0.322"t	200ø×8.2
P6	6"ø×0.280"t	150ø×7.1
W8x15	W8x15	W200x22.5
W8x35	W8x35	W200x52

NOTE: METAL DECK AND CROSS BRACES NOT SHOWN FOR CLARITY

BOTTOM CHORD FRAMING PLAN

APPROVED AS NOTED
BRIDGE DESIGN SECTION
ALASKA DEPARTMENT OF TRANSPORTATION AND PUBLIC FACILITIES
Reviewed for conformance with basic details and dimensions of the contract plans. Accuracy of information supplied by the fabricator is not guaranteed.
By EBH Date APR 14 2000

STATE OF ALASKA 49TH D. Nottingham No. 1204-E

MONTANA CREEK PEDESTRIAN BRIDGE

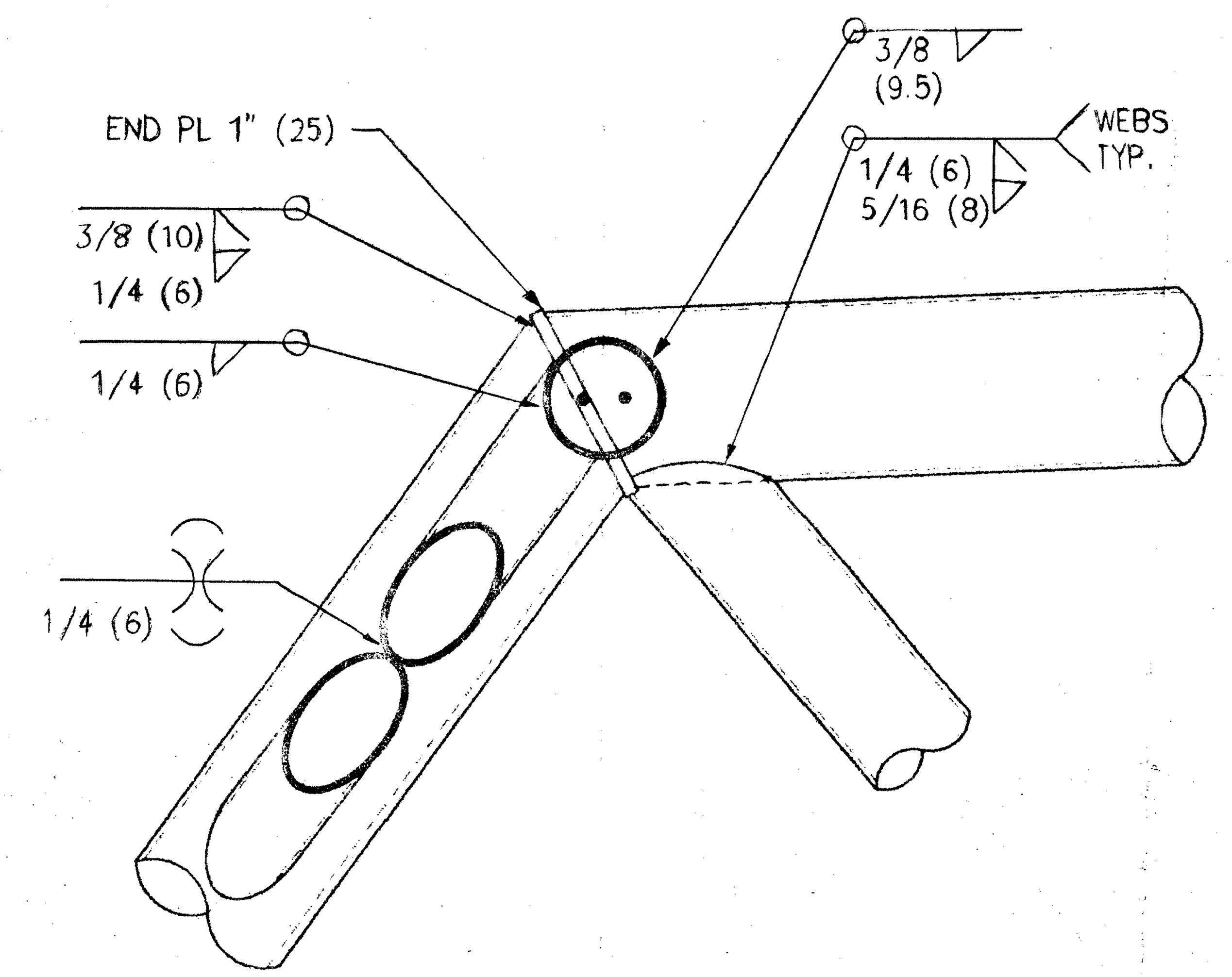
BRIDGE NO. 997

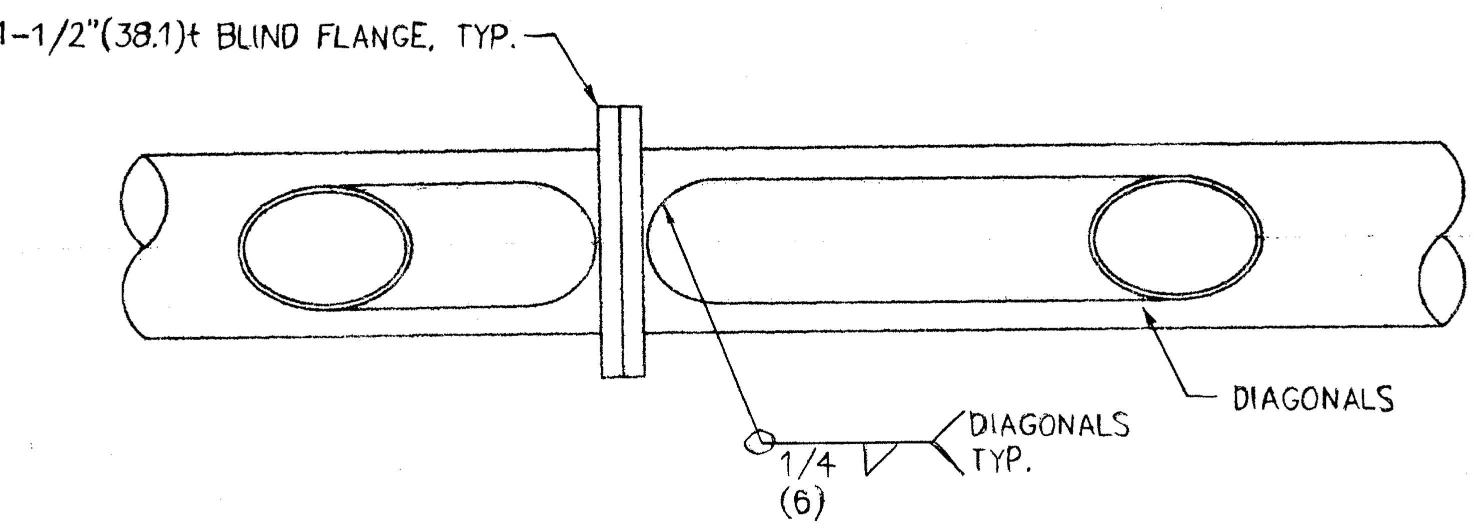
Designed: DN/GH
Drawn: GH
Checked: DN/SB
Project No: 99110
Date: FEB 2000
Scale:

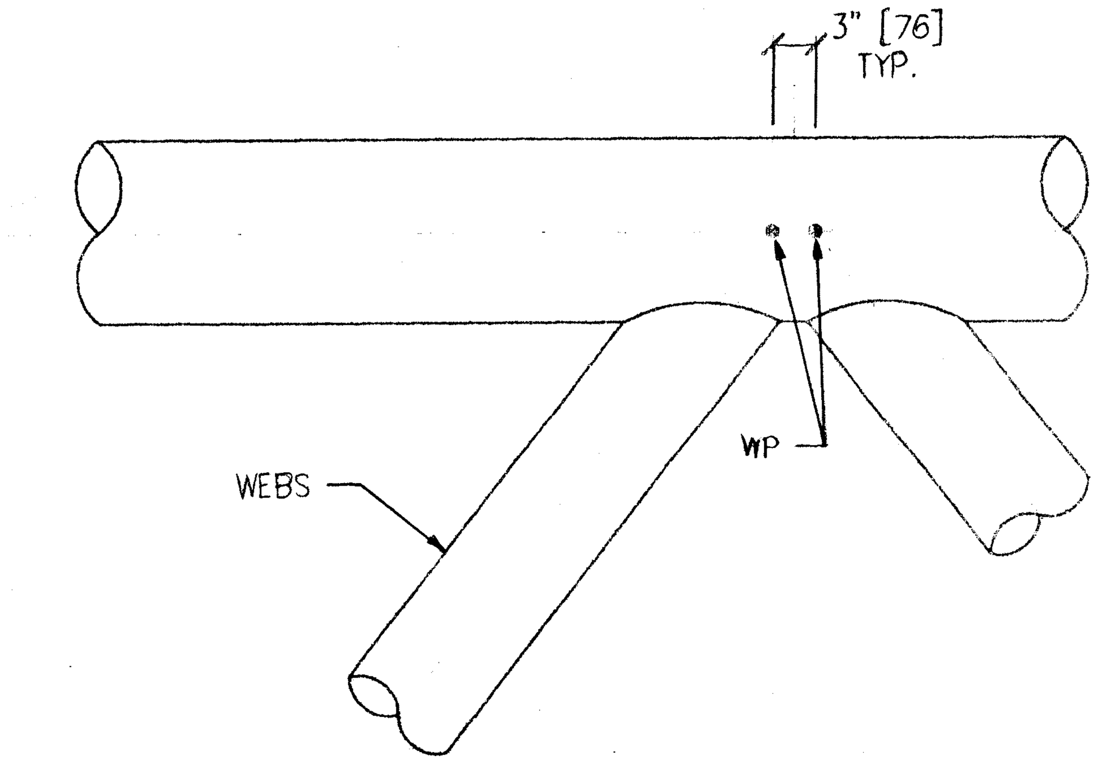
Peratrovich, Nottingham & Drage, Inc.
Engineering Consultants
1506 West 36th Avenue, Anchorage, Alaska 99503 (907) 561-1011 FAX (907) 563-4220

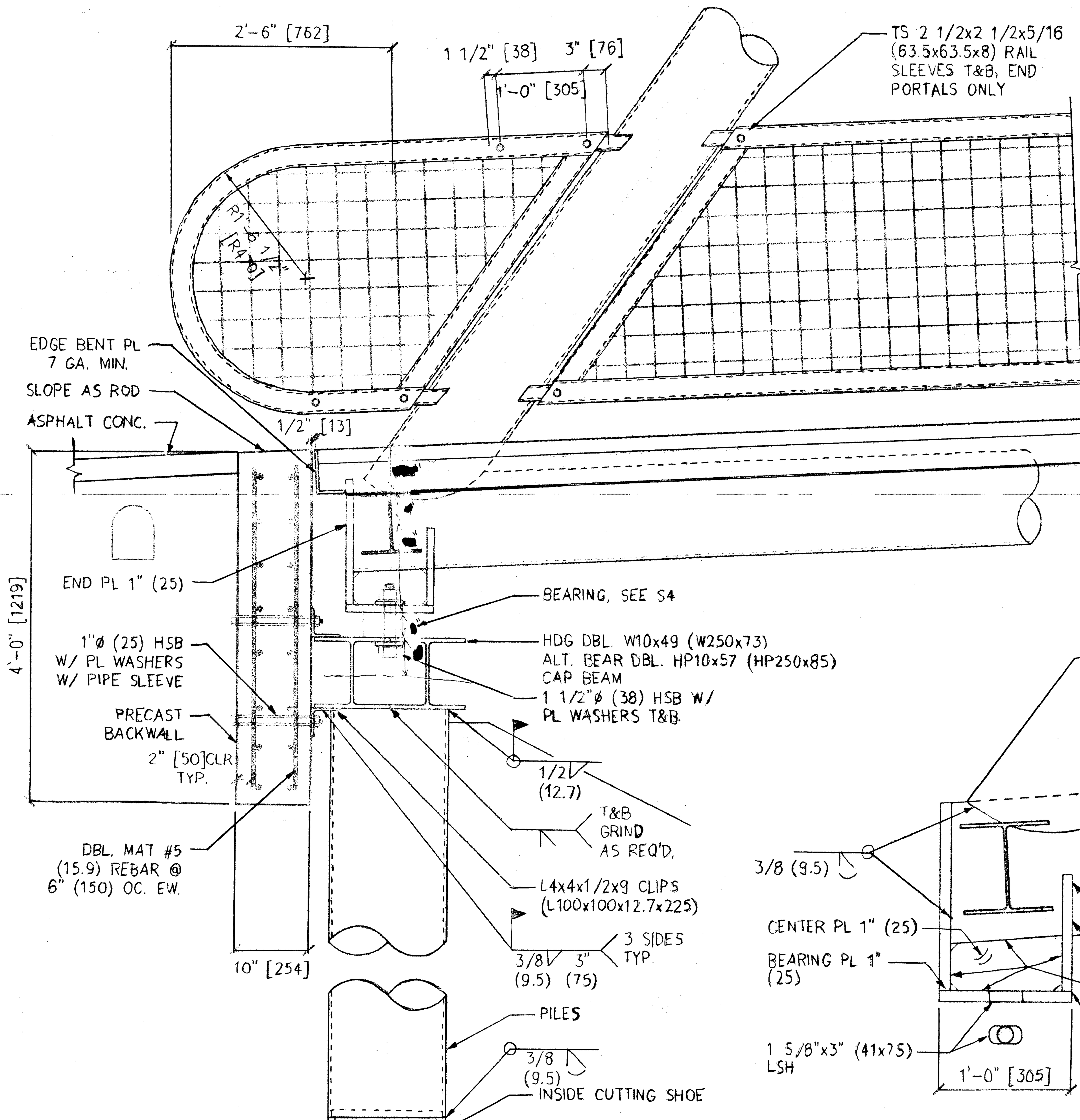
PLANS and ELEVATION

S2 4









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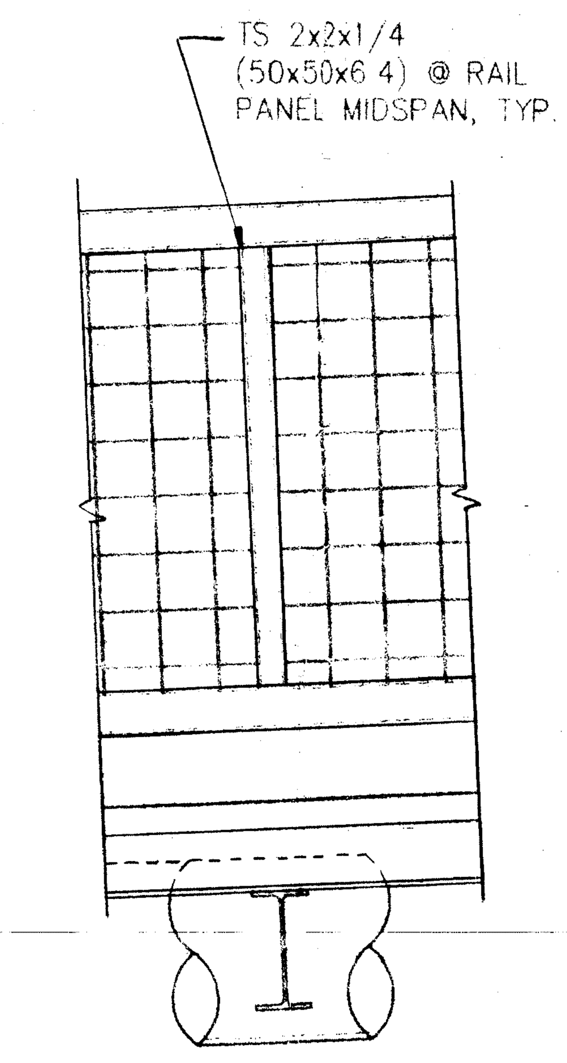

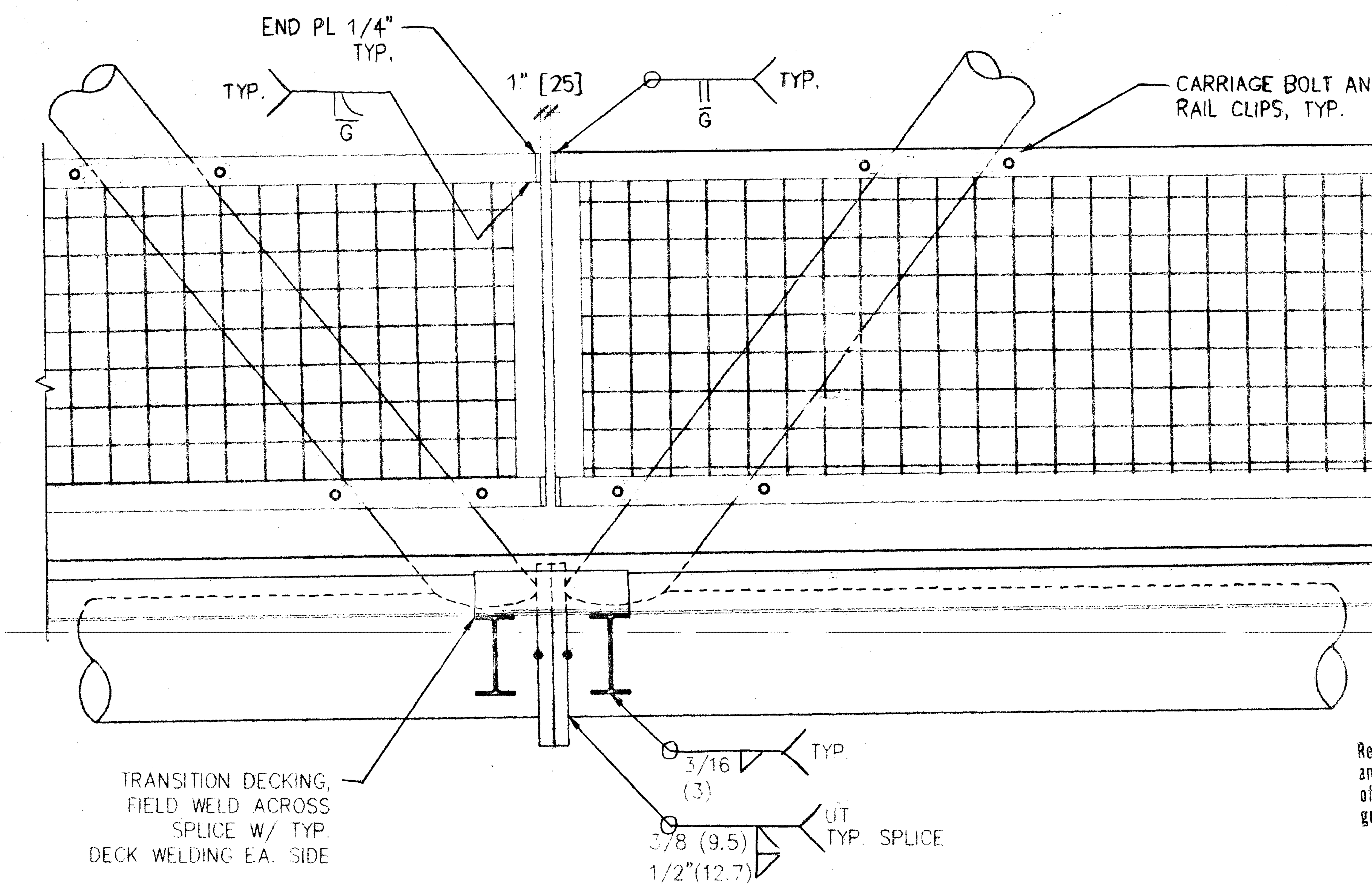

SPLICE

APPROVED AS NOTED
BRIDGE DESIGN SECTION
ALASKA DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES
Reviewed for conformance with basic details and dimensions of the contract plans. Accuracy of information supplied by the fabricator is not guaranteed.
By [signature] Date APR 14 2000

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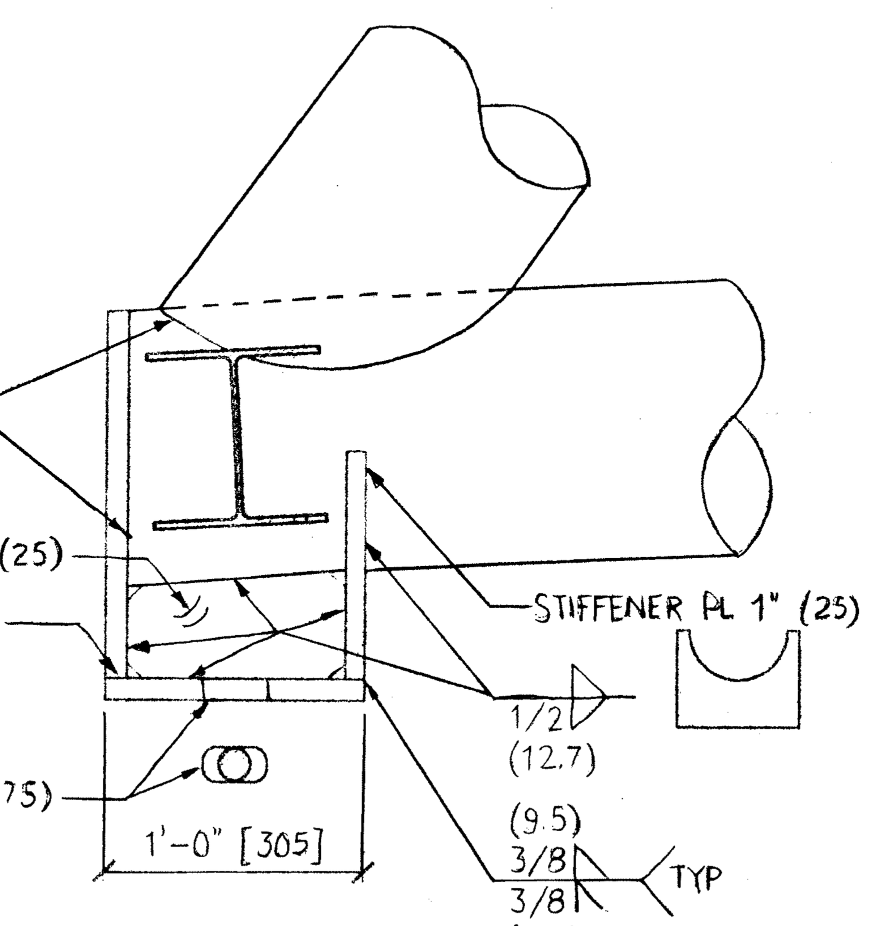
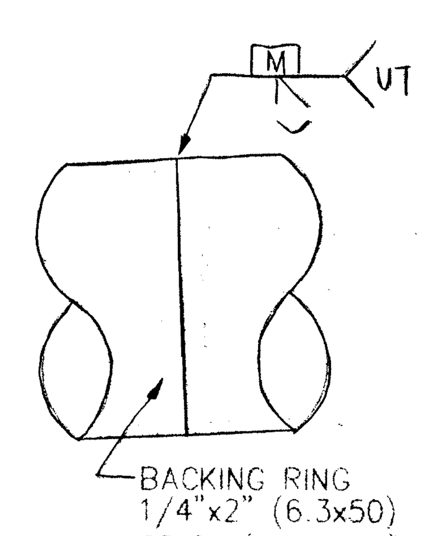
1 ADD SPLICE DETAIL, SIMPLIFY WELDS



BEARING SHOE



CHORD SPLICE

STATE OF ALASKA
49TH
B. Nottingham
No. 1204-E

MONTANA CREEK PEDESTRIAN BRIDGE

BRIDGE NO. 1934

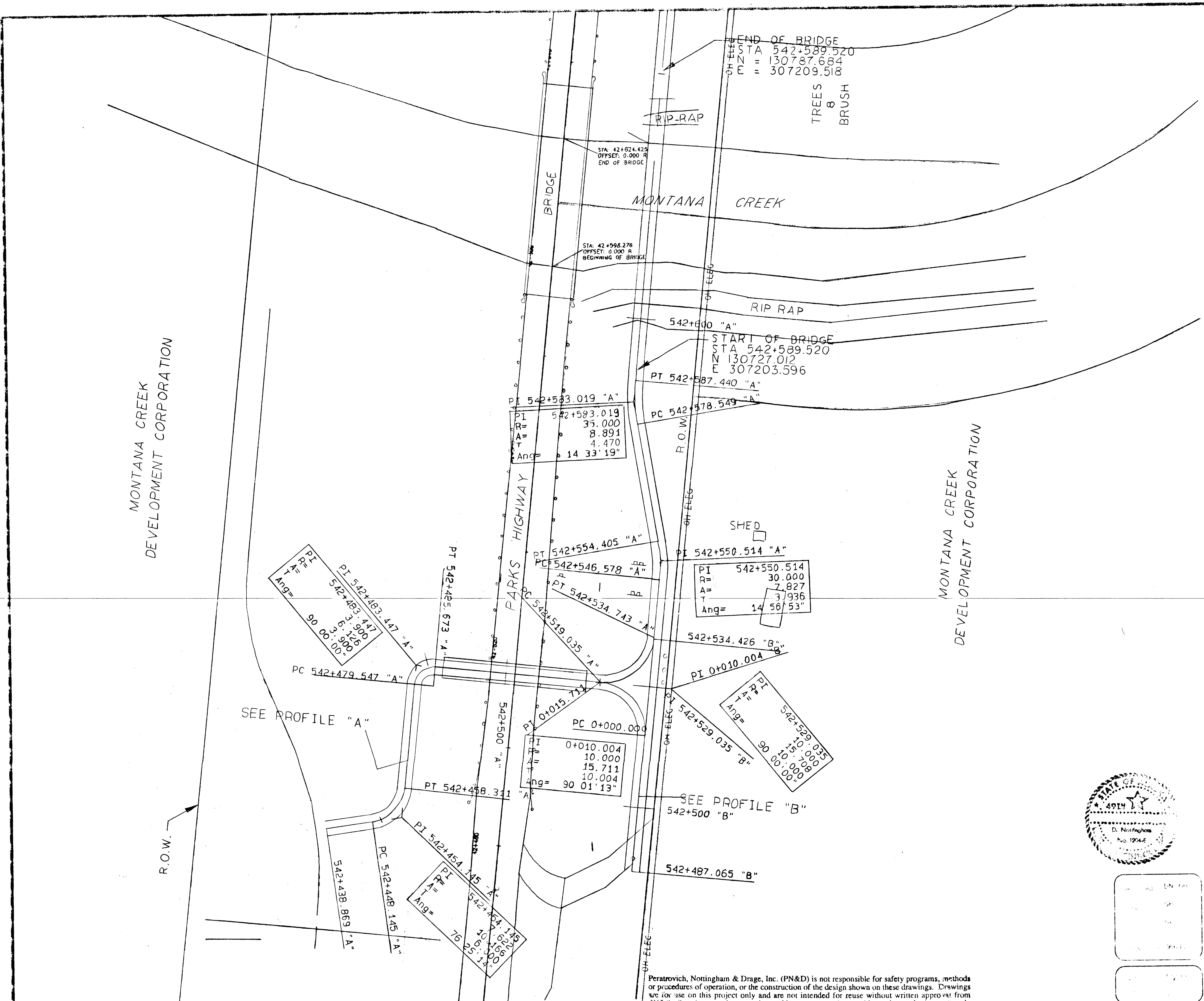
Designed:	DN/GH
Drawn:	GH
Checked:	DN
Project No:	99110
Date:	FEB 2000
Scale:	

Peratrovich, Nottingham & Drage, Inc.
Engineering Consultants
1506 West 36th Avenue,
Anchorage, Alaska 99503 (907) 561-1011 FAX (907) 563-4220

DETAILS

Sheet S3 4

STATE	YEAR	PROJECT DESIGNATION	NO	SHEETS
ALASKA	1999	53956		



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STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES
PROJECT NO. 53956

MONTANA CREEK PEDESTRIAN BRIDGE

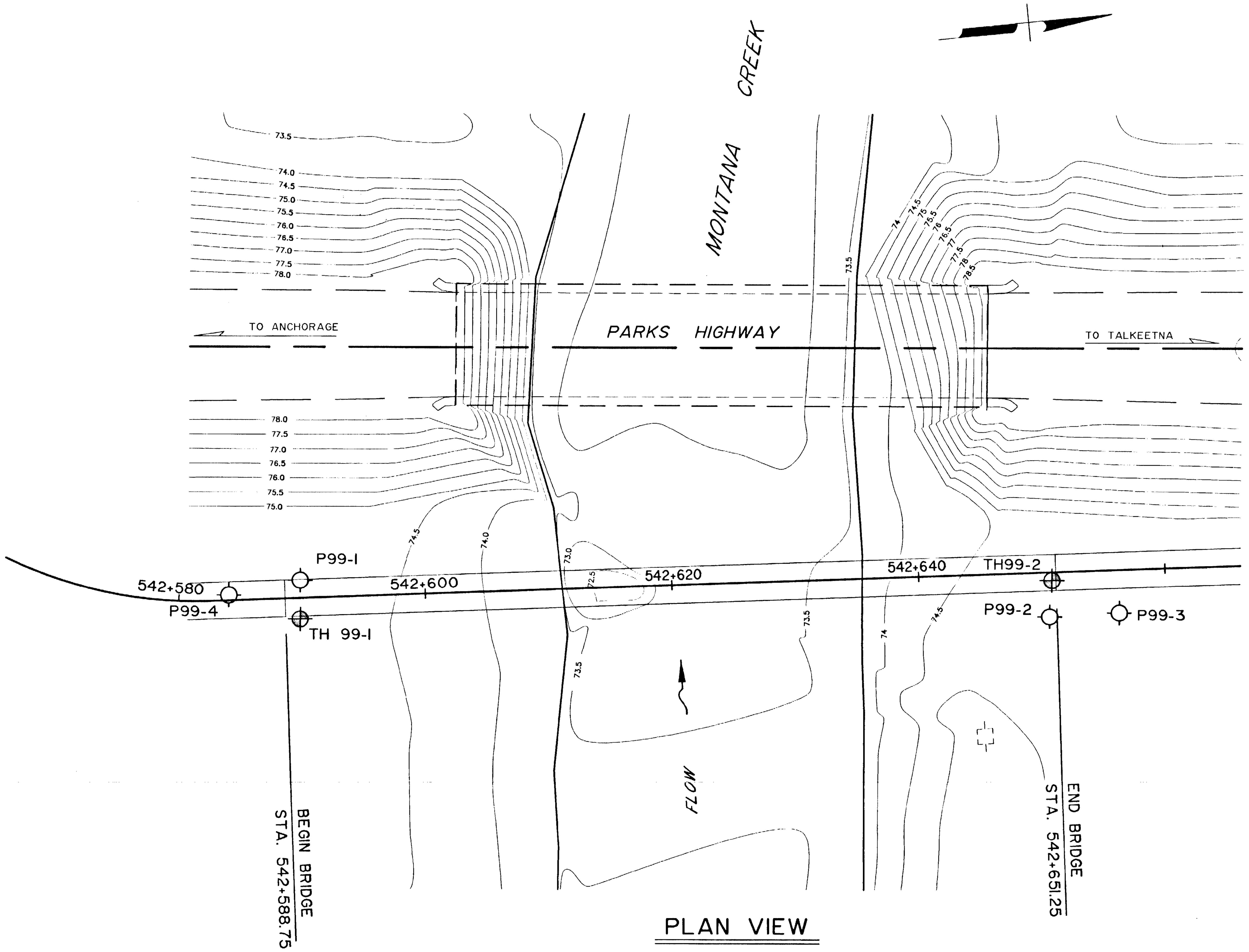
Peratrovich, Nottingham & Drage, Inc.
Engineering Consultants

1506 West 36th Avenue,
Anchorage, Alaska 99503 (907) 561-1011 FAX (907) 563-4220

PLANS and ELEVATION

C1 2

STATE	PROJECT DESIGNATION	YEAR	SHEET NO.	TOTAL SHEETS
ALASKA	TEA-OA4-1(17)/53956	1999	THI	24



PLAN VIEW

0 25 50 75 100
GRAPHIC SCALE

TEST HOLE LOGS AND LOCATIONS

PARKS HIGHWAY
MONTANA CREEK
PEDESTRIAN BRIDGE

GENERAL LAYOUT

GENERAL NOTES:

1. HORIZONTAL AND VERTICAL GEOMETRY WITH TOPOGRAPHIC DATA FURNISHED BY THE CENTRAL REGION HIGHWAY DESIGN SECTION JUNE 1999.
2. THE TEST HOLE(S) DEPICTED ARE A COMBINATION OF THE ORIGINAL FIELD LOG(S), AND AN OFFICE EXAMINATION OF THE FIELD LOG(S), SOIL SAMPLE(S) AND/OR ROCK CORE(S).
3. THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES OR RELATIVE DENSITIES. THE TRANSITION MAY BE GRADUAL.
4. WHERE CASING OR HOLLOW STEM IS INDICATED, THE NEED WAS PREDICATED BY THE POSITION OF THE GROUNDWATER TABLE OR BY CAVING GROUND CONDITIONS. THE CASING OR HOLLOW STEM WAS INSTALLED TO PROVIDE TEMPORARY SOIL SUPPORT AND/OR PROVIDE FOR DRILL FLUID CIRCULATION.
5. FIELD MOISTURE DESCRIPTIONS (DRY, MOIST, AND WET) ARE BASED ON THE FOLLOWING FIELD OBSERVATIONS:
   A. DRY-A SOIL WITH NO VISIBLE MOISTURE, FEELS DRY WHEN HELD IN THE HAND. WILL NOT FORM A CAST.
   B. MOIST-A SOIL WITH VISIBLE MOISTURE, FEELS MOIST IN THE HAND, WILL FORM A CAST.
   C. WET-A SOIL WITH VISIBLE WATER, WETS THE HAND WHEN HELD, HAS FREE WATER WHEN SHAKEN.
   A COMBINATION OF THESE TERMS MAY BE USED TO DESCRIBE THE SOIL MOISTURE CONDITION.

SOIL GRAIN SIZE DEFINITIONS

BOULDER	>305 mm DIAMETER
COBBLE	75 mm to 305 mm DIAMETER
BROKEN ROCK (ANGULAR)	>2.0 mm DIAMETER
GRAVEL (ROUNDED); STONE (ANGULAR)	2.0 mm to 75 mm
SAND	0.075 mm to 2.0 mm
SILT/CLAY	<0.075 mm

NOTE: SOIL CLASSIFICATIONS ARE VISUAL ONLY UNLESS AASHTO SOIL CLASS IS SHOWN ON THE LOG.

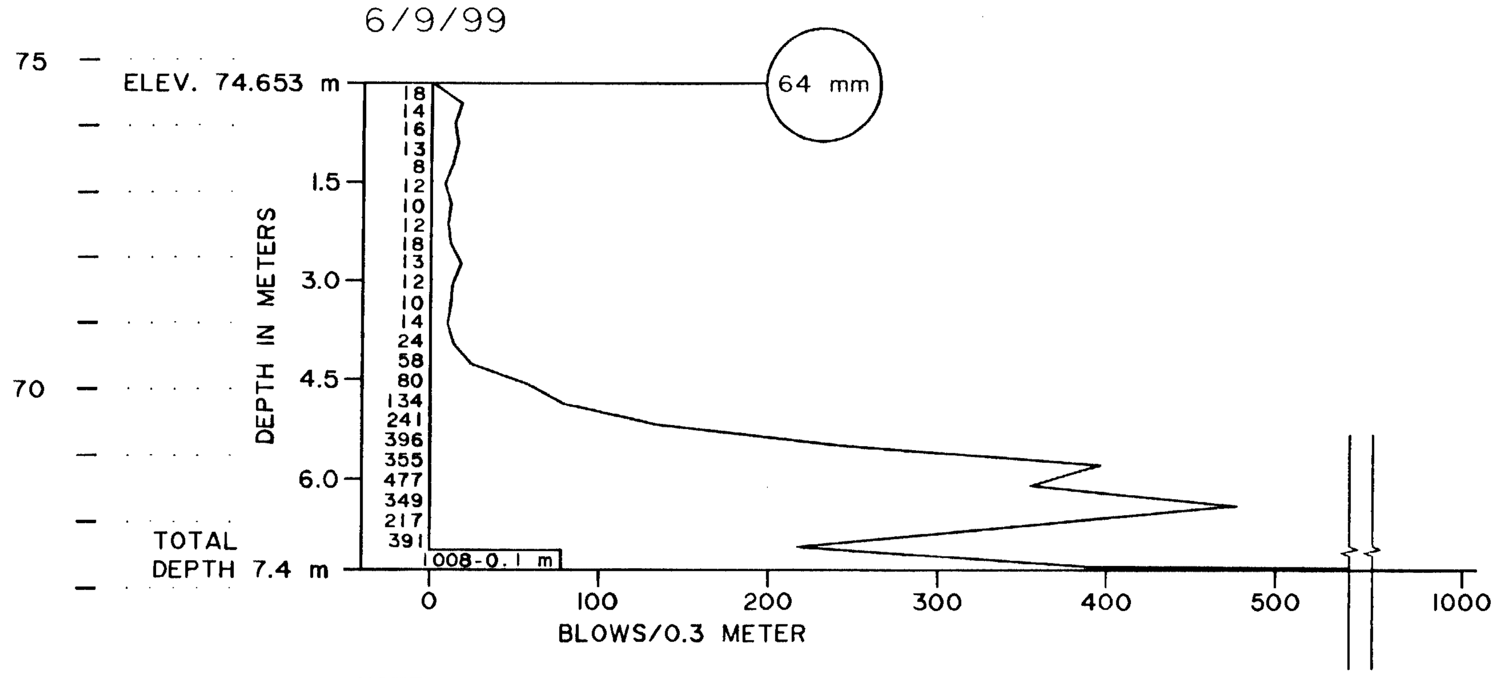
STATE OF ALASKA
49TH
Keith J. Kerri
CE 5126
08.10.99
REGISTERED PROFESSIONAL ENGINEER

State of Alaska
DEPARTMENT of TRANSPORTATION
and
PUBLIC FACILITIES

Bridge No: 1984
Drawing No: 1 OF 3

STATE	PROJECT DESIGNATION	YEAR	SHEET NO.	TOTAL SHEETS
ALASKA	TEA-0A4-1(17)/53956	1999	TH2	24

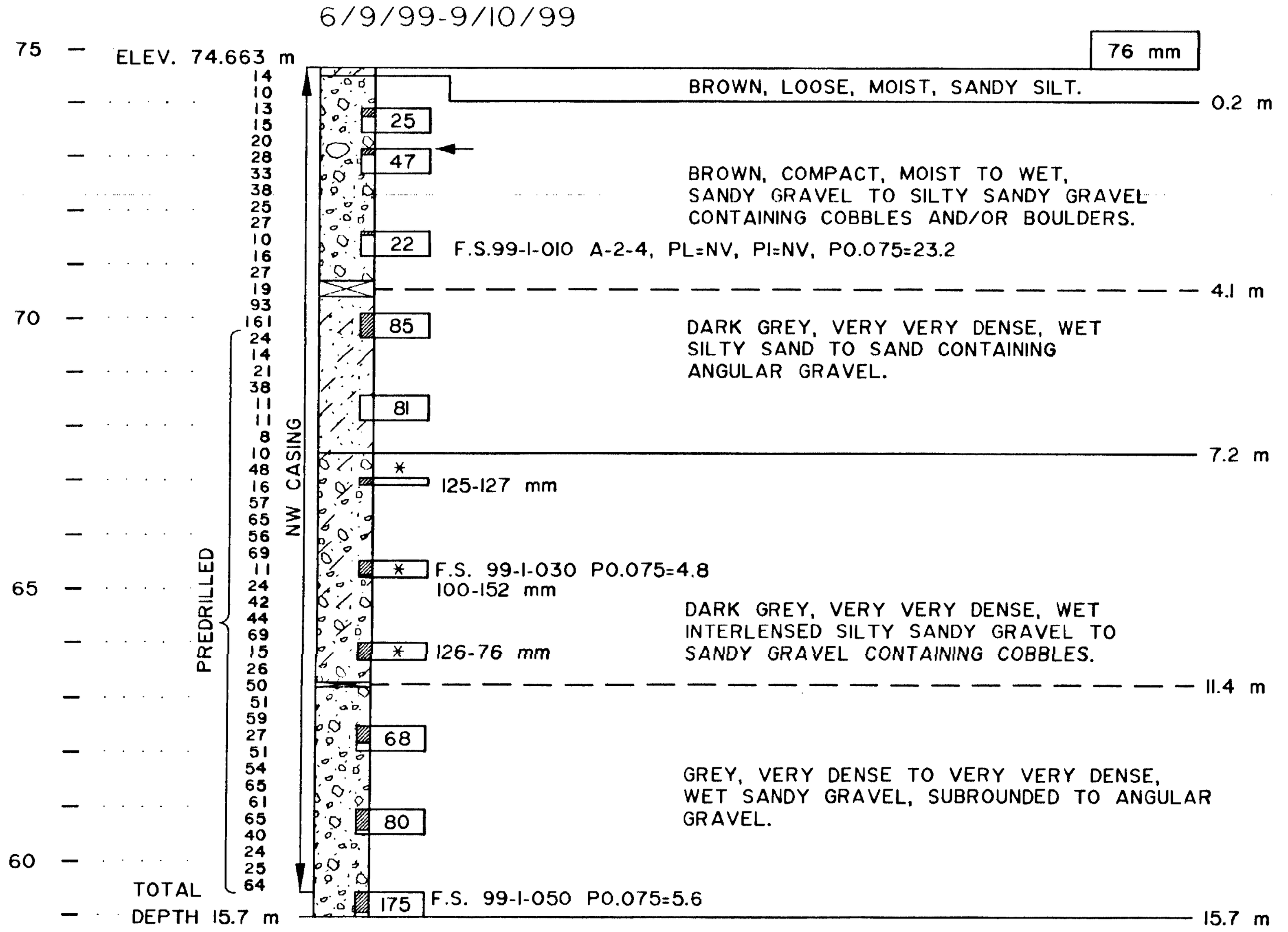
PENETROMETER 99-4
STA. 542+584.054, 0.436 m LT.
6/9/99



NOTE:

PENETROMETER ROD PULLOUT BREAK FORCE AFTER DRIVING TO 7.4 m IN DEPTH WAS 183,400 NEWTONS.

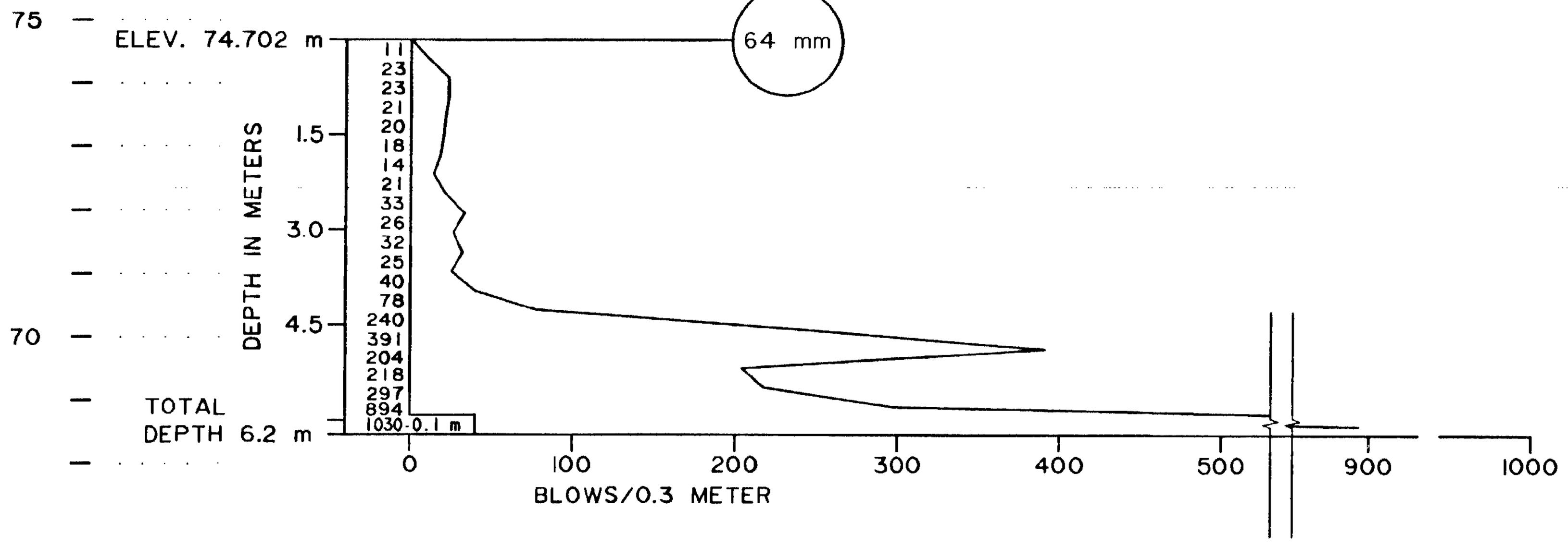
TEST HOLE 99-1
STA. 542+589.760, 1.728 m RT.
6/9/99-9/10/99



NOTE:

NW CASING PULLOUT BREAK FORCE IMMDEIATELY AFTER DRIVING WAS 88,100 NEWTONS.

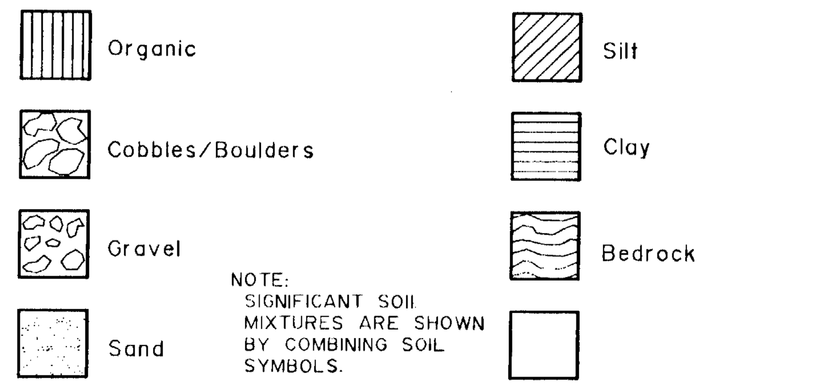
PENETROMETER 99-1
STA. 542+589.861, 1.443 m LT.
6/7/99-6/8/99



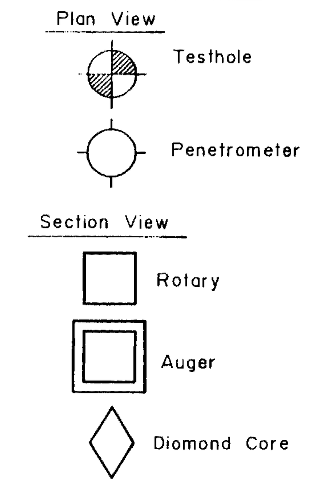
NOTE:

PENETROMETER ROD PULLOUT BREAK FORCE 16 HOURS AFTER DRIVING TO 6.2 m WAS 235,800 NEWTONS.

BASIC MATERIALS SYMBOLS



TYPICAL TEST HOLE SYMBOLS

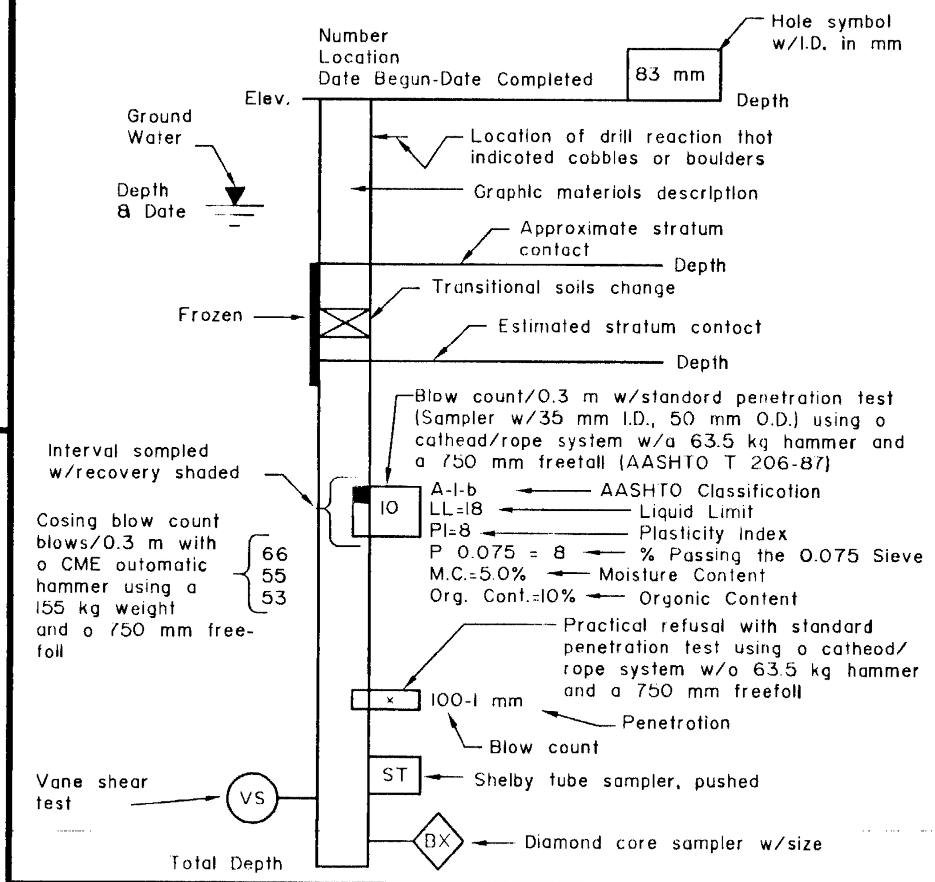


RELATIVE DENSITY AND CONSISTENCY CLASSIFICATION

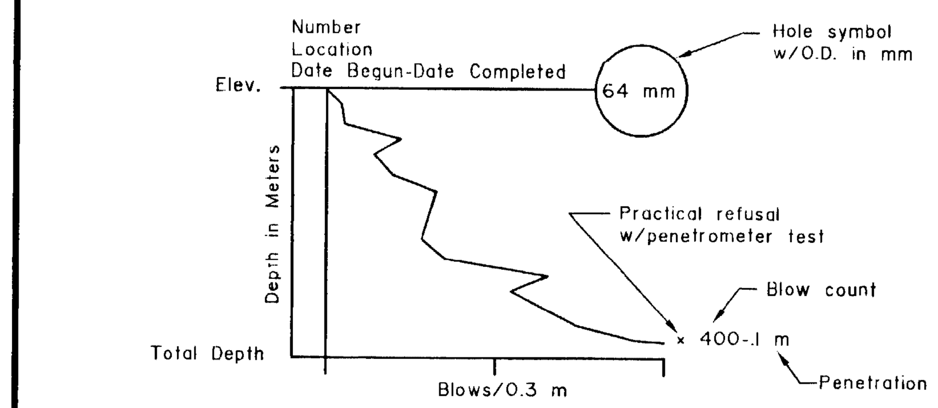
Based on Standard Penetration Test

GRANULAR		COHESIVE	
Blows/0.3 m	Rel. Density	Blows/0.3 m	Consistency
0-5	Very Loose	2	Very Soft
6-10	Loose	2-4	Soft
11-20	Firm	5-8	Medium
21-35	Compact	9-15	Stiff
36-50	Dense	16-30	Very Stiff
51-70	Very Dense	31-60	Hard
71+	V. Very Dense	61+	Very Hard

TYPICAL TEST HOLE LOG



TYPICAL PENETROMETER TEST LOG



(Penetrometer w/64 mm O.D., with a CME automatic hammer using a 155 kg weight and a 750 mm freefall)

TEST HOLE LOGS AND LOCATIONS
MONTANA CREEK PEDESTRIAN BRIDGE

State of Alaska
DEPARTMENT of TRANSPORTATION and PUBLIC FACILITIES

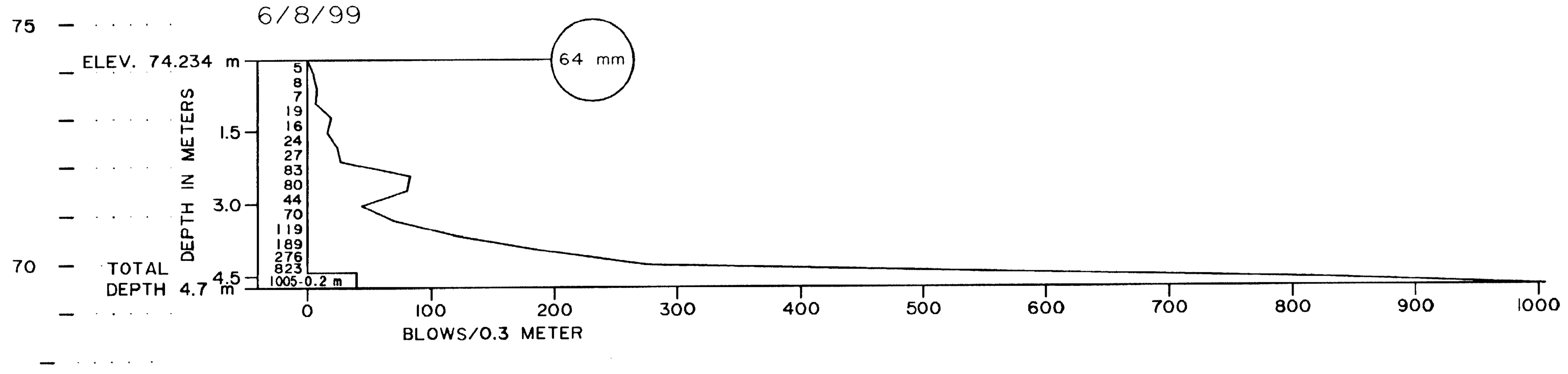
Bridge No: 1984
Drawing No: 2 OF 3

DWG NAME: C:\CAD\MATLS\MONTANA\LOGS.M VIEW 1 DATE: 8/9/99 SCALE: 1 = 10.0

STATE	PROJECT DESIGNATION	YEAR	SHEET NO.	TOTAL SHEETS
ALASKA	TEA-0A4-1(17)/53956	1999	TH3	24

PENETROMETER 99-2

STA. 542+650.507, 3.631 m RT.
6/8/99

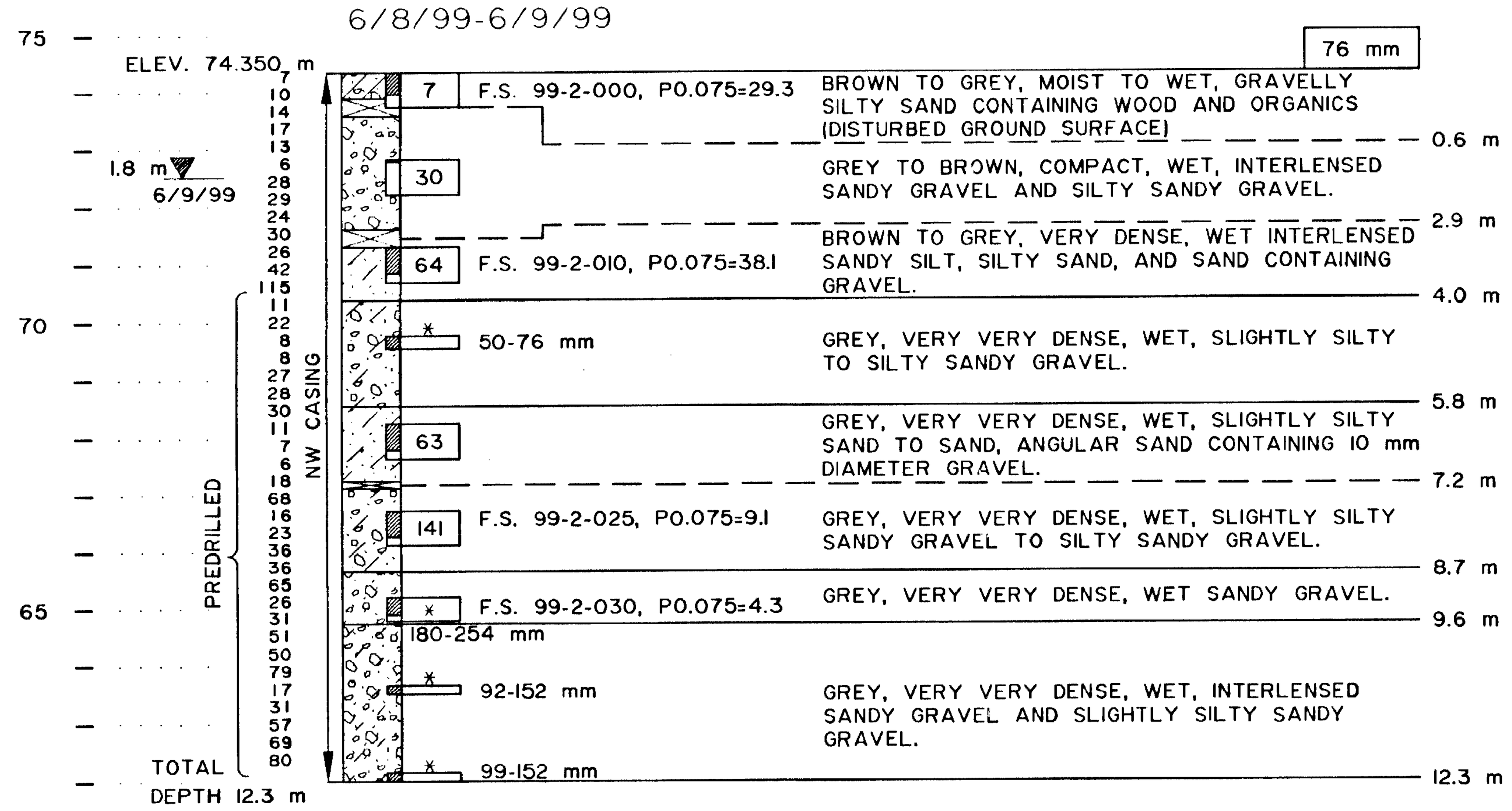


NOTE:

PENETROMETER ROD PULLOUT BREAK FORCE AFTER DRIVING TO 4.7 m IN DEPTH WAS 132, 100 NEWTONS.

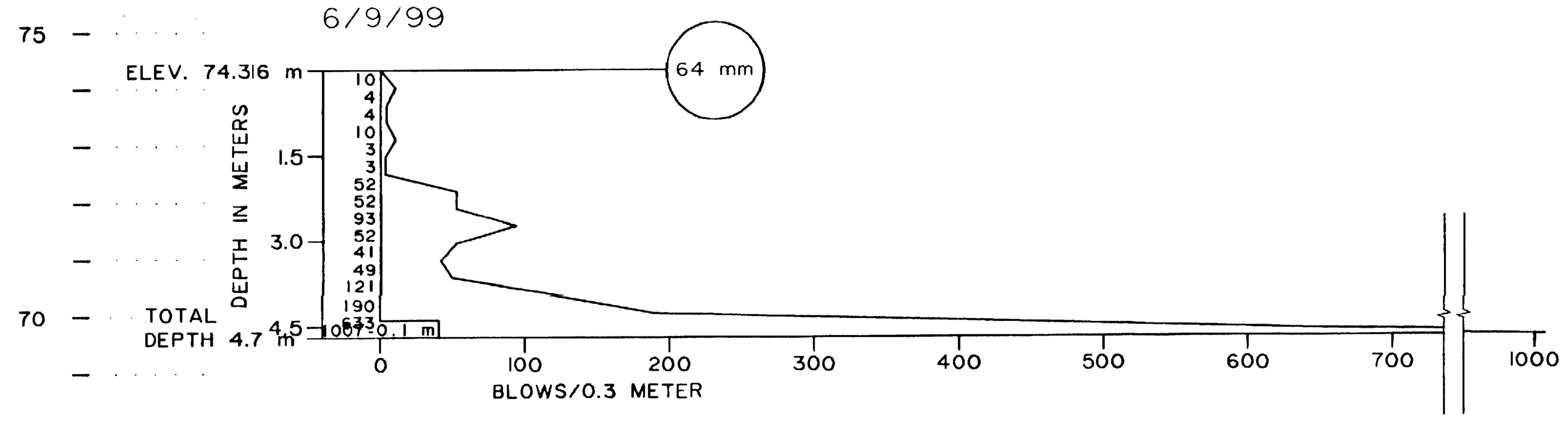
TEST HOLE 99-2

STA. 542+650.772, 0.667 m RT.
6/8/99-6/9/99



PENETROMETER 99-3

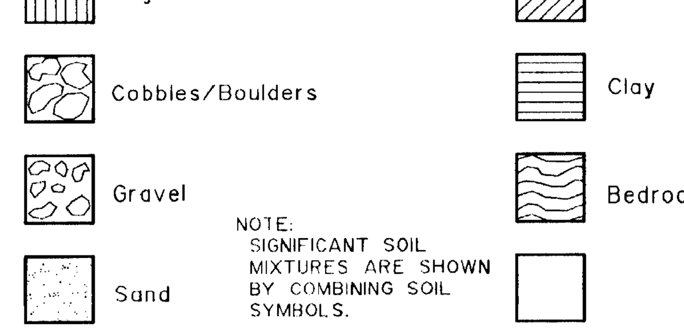
STA. 542+656.173, 3.507 m RT.
6/9/99



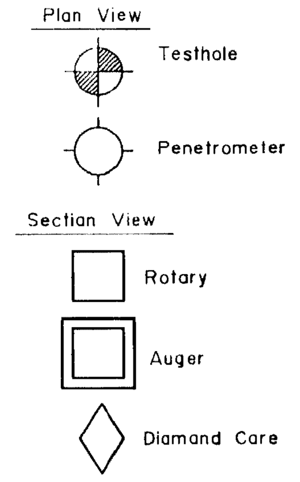
NOTE:

PENETROMETER ROD PULLOUT BREAK FORCE AFTER DRIVING TO 4.7 m IN DEPTH WAS 231,400 NEWTONS.

BASIC MATERIALS SYMBOLS



TYPICAL TEST HOLE SYMBOLS

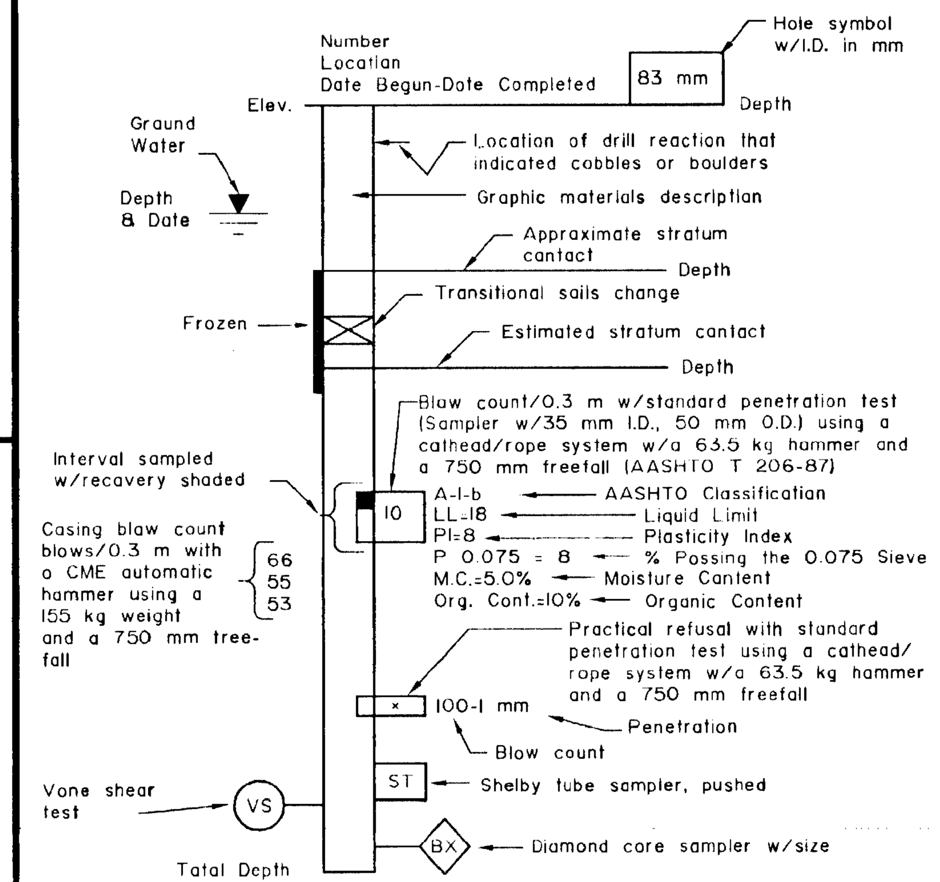


RELATIVE DENSITY AND CONSISTENCY CLASSIFICATION

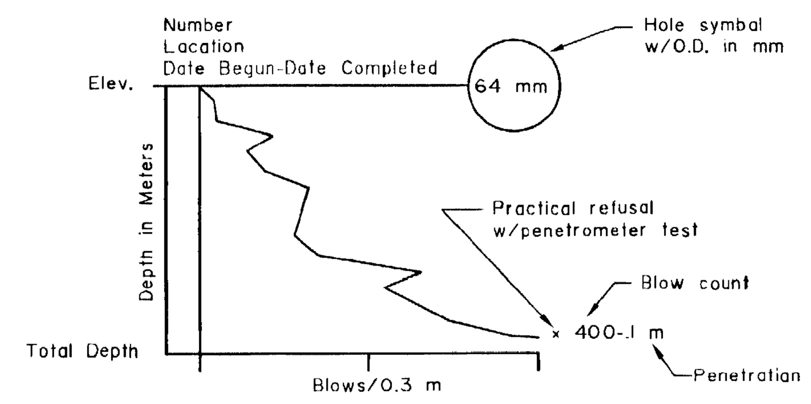
Based on Standard Penetration Test

GRANULAR		COHESIVE	
Blows/ 0.3 m	Rel. Density	Blows/ 0.3 m	Consistency
0-5	Very Loose	2	Very Soft
6-10	Loose	2-4	Soft
11-20	Firm	5-8	Medium
21-35	Compact	9-15	Stiff
36-50	Dense	16-30	Very Stiff
51-70	Very Dense	31-60	Hard
71+	V. Very Dense	61+	Very Hard

TYPICAL TEST HOLE LOG



TYPICAL PENETROMETER TEST LOG



(Penetrometer w/64 mm O.D., with a CME automatic hammer using a 155 kg weight and a 750 mm freefall)

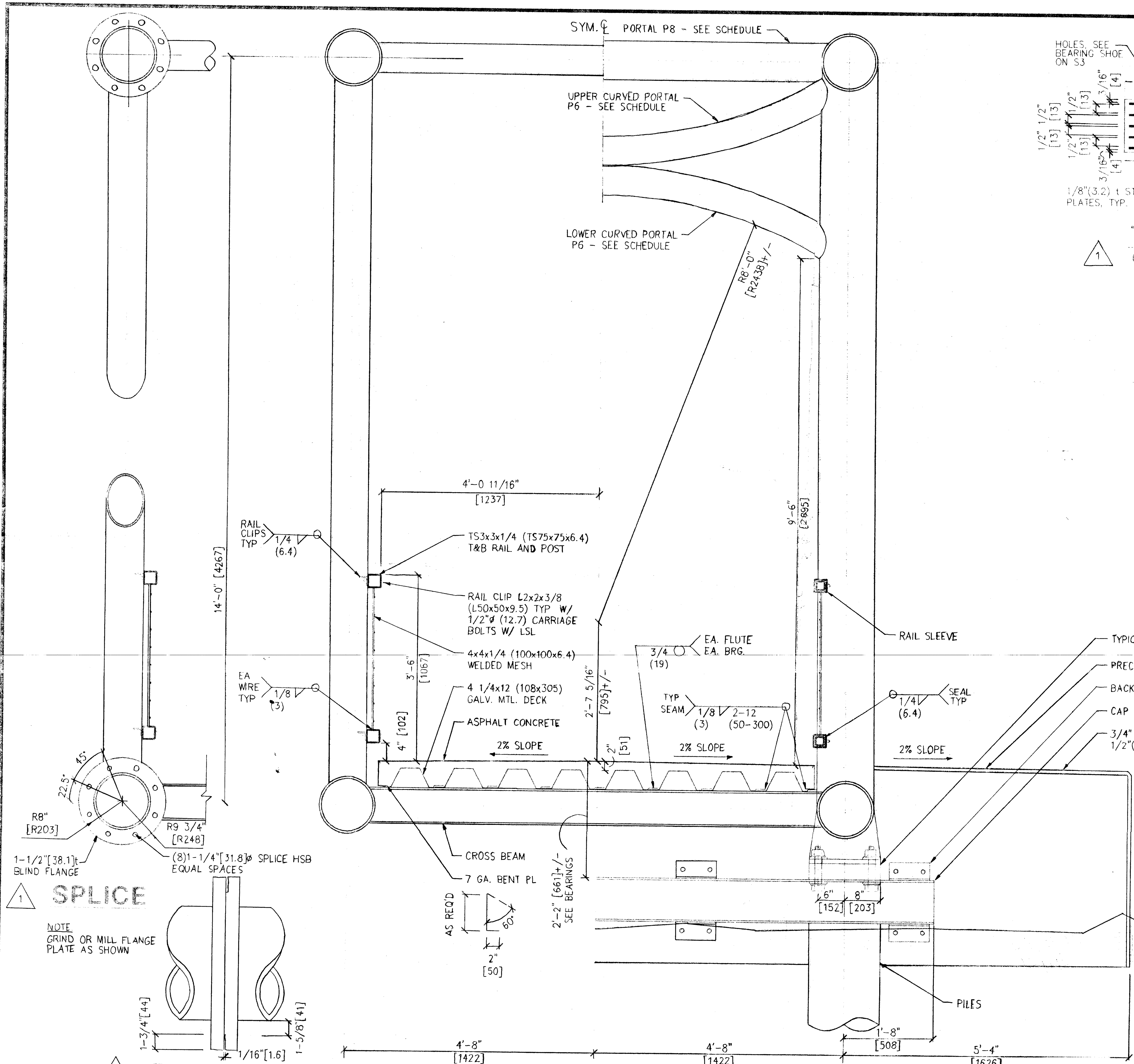
TEST HOLE LOGS AND LOCATIONS

MONTANA CREEK PEDESTRIAN BRIDGE

State of Alaska
DEPARTMENT of TRANSPORTATION and PUBLIC FACILITIES

Bridge No: 1984
Drawing No: 3 OF 3

DWG NAME: G:\CAD\MATLS\MONTANA\LOGS-M VIEW 2 | DATE: 8/9/99 | SCALE: 1 = 10.0

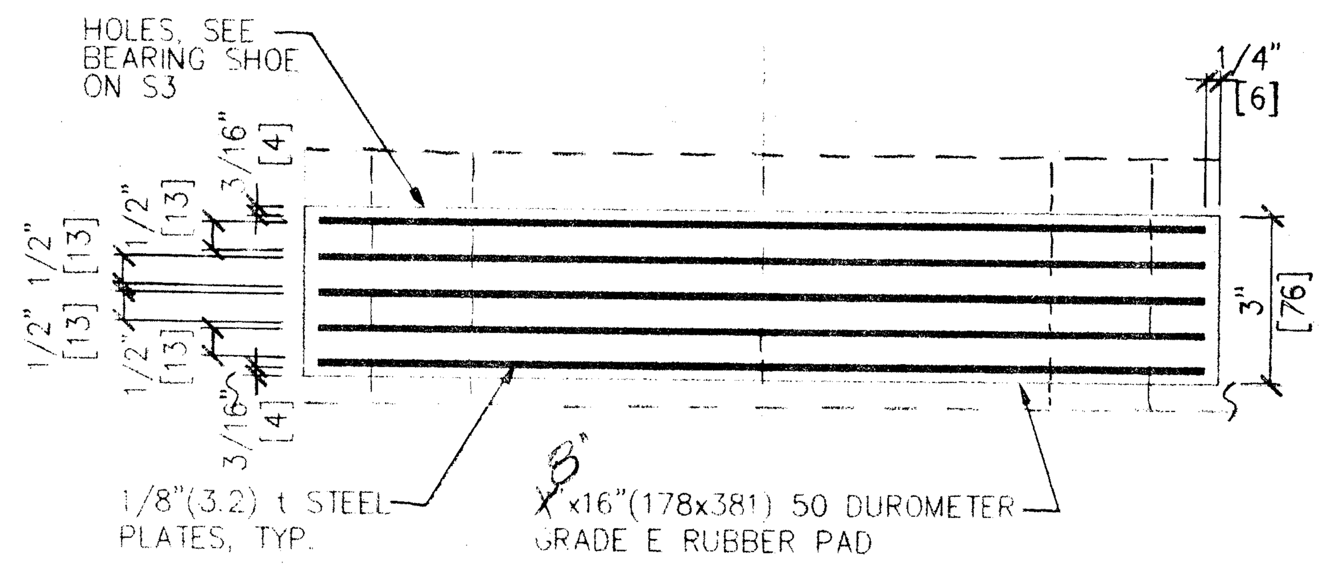


1 SPLICE

1 FLANGE DTL.

TYPICAL FRAME

PORTAL & BACKWALL [redacted]



1 TYPICAL BEARING

(PROVIDE UNDER ALL FOUR BEARING SHOES)

APPROVED AS NOTED
BRIDGE DESIGN SECTION
ALASKA DEPARTMENT OF TRANSPORTATION AND PUBLIC FACILITIES
Reviewed for conformance with basic details and dimensions of the contract plans. Accuracy of information supplied by the fabricator is not guaranteed.
By ECM Date APR 14 2000

STATE OF ALASKA 49TH D. Nottingham No. 1204-E

1 CLARIFY BEARINGS, MODIFY SPLICE CONNECTION

MONTANA CREEK PEDESTRIAN BRIDGE

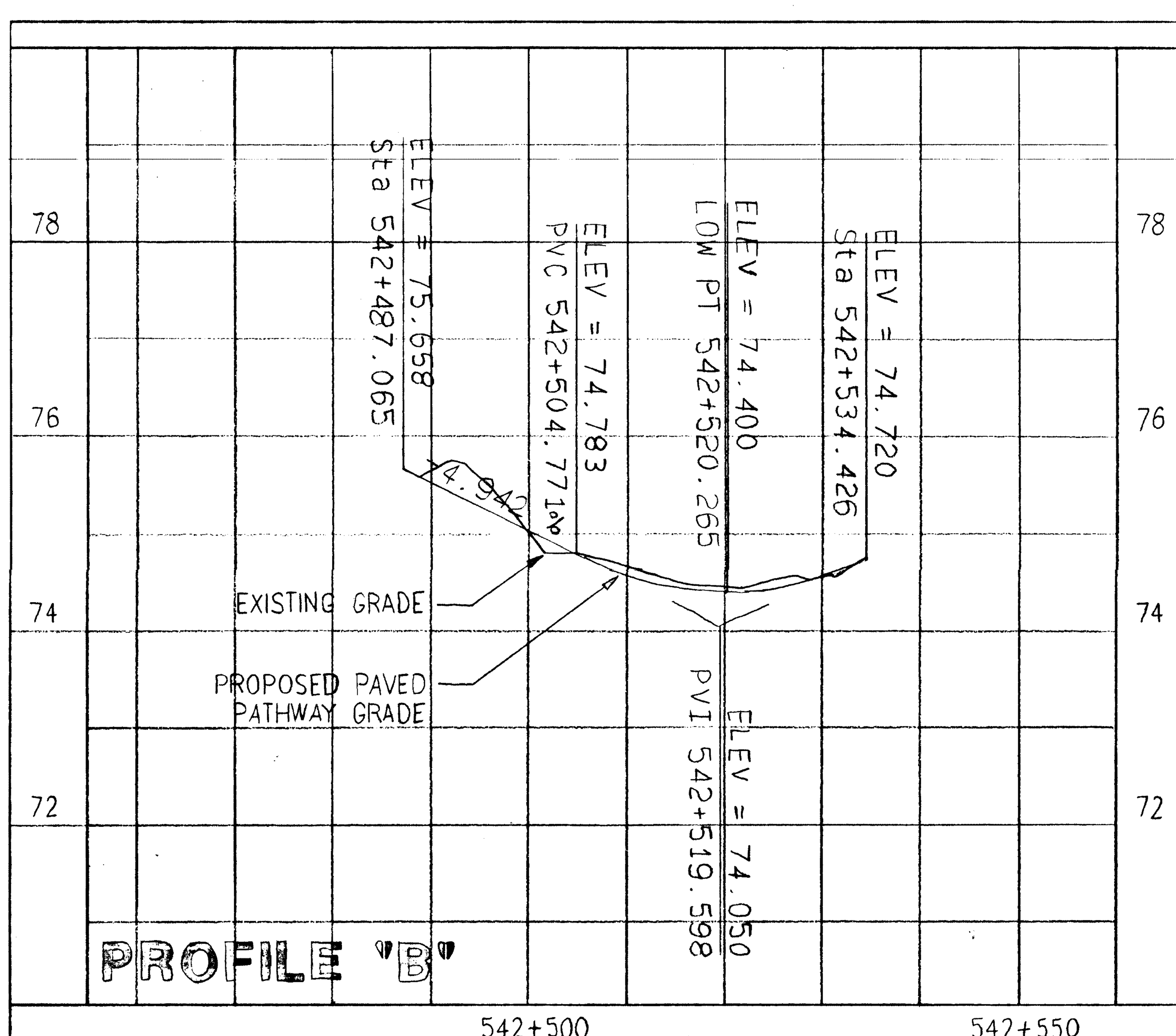
BRIDGE NO. 1394

Designed:	DN/GH
Drawn:	GH
Checked:	DN
Project No.	99110
Date:	FEB 2000
Scale:	

Peratrovich, Nottingham & Drage, Inc.
Engineering Consultants
1506 West 36th Avenue, Anchorage, Alaska 99503 (907) 561-1011 FAX (907) [illegible]

DETAILS

S4 [illegible]



STATE OF ALASKA
49TH
D. Nottingham
No. 1204-E

MONTANA CREEK PEDESTRIAN BRIDGE

Peratrovich, Nottingham & Drage, Inc.
Engineering Consultants

1506 West 36th Avenue,
Anchorage, Alaska 99503 (907) 561-1011 FAX (907) 563-4220

DN/GH
GH
DN

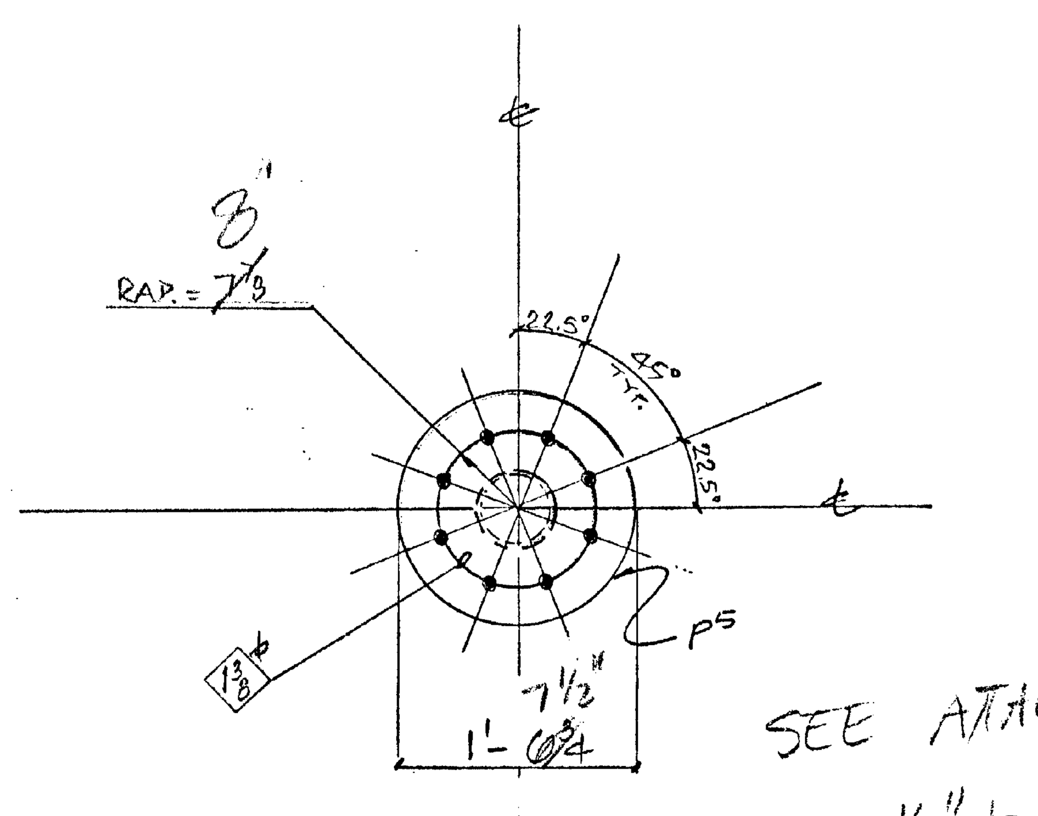
PROFILE

C2 2

MONTANA CREEK BRIDGE SHOP DRAWINGS

ONE ~ BRIDGE SECTION AS SHOWN MK 1AR

ONE ~ OPP. HAND MK 1AL



SEE ATTACHED SKETCH B

SECT. A ~ A PL 1 1/2" t

NOTE: FULL SIZE WRAP AROUND TEMPLATES OF ALL PIPE ENDS WILL BE FURNISHED TO THE SHOP BEFORE FABRICATION

JOB NO. 99110 DATE: 5/3/00
PERATROVICH, NOTTINGHAM & DRAGE, INC.
☐ NO EXCEPTIONS TAKEN
☐ REVISE AND RESUBMIT
☑ REVISE AS NOTED
Checking is only for general conformance with the design concept of the project and compliance with the information given in the contract documents. Contractor is responsible for dimensions and quantities to be confirmed and correlated at the job site, for information that pertains solely to the fabrication processes or to the techniques of construction, and for the coordination of the work of all trades.
CHECKED BY

SEE DWG. #2 FOR BILL OF MATERIALS

WORK THIS DWG. WITH 2 THRU. 6

MAT'L SPEC.	SEE PAGE 2
OPEN HOLES	EXCEPT AS NOTED
END & EDGE DIST.	EXCEPT AS NOTED
PAINT	METALLIZED 6 MILS
SHIP TO	
ERECTION BY:	

KEISER STEEL FABRICATOR INC.
22620 85TH. PL., S. KENT, WA 98031 (206) 852-1910

12:58:22 PM DATE: Apr 11 2000 | APPROVED: | DRAWN BY MKE | CHK'D BY

ELEVATION

MONTANA CREEK BRIDGE

ANCHORAGE, ALASKA

CONTRACTOR	JOB NO.	DRAWING NO.	REV.
WEST COAST	00-10	1	0

STRUCTURAL STEEL

ALL STEEL MATERIALS, FABRICATION, CONSTRUCTION, AND ERECTION SHALL CONFORM TO ADOT STANDARD 504, "STEEL STRUCTURES", AND 716, "STRUCTURAL STEEL". WELDING AND FABRICATIONS SHALL CONFORM TO THE 2000 AWS D1.1.

305 mmϕx12.7mm (12"ϕx0.500t") DIAMETER PIPE - Fy=275 MPa (40 KSI), Fu= 415 MPa (60 KSI), $E_{2"}$ =15%
ALL OTHER PIPE - Fy=240 MPa (35 KSI), Fu= 415 MPa (60 KSI), $E_{2"}$ =15%
ALL PIPE SHALL HAVE CHEMISTRY SUITABLE FOR WELDING, PIPE MAY BE ASTM A53 GRADE B, A106 GRADE C, A252 GRADE 3, API 5L GRADE B OR X42, OR ENGINEER APROVED EQUAL, MEETING ADDITIONAL MINIMUM REQUIREMENTS AS INDICATED. SPIRAL WELDED PIPE SHALL NOT BE PERMITTED.
TUBING SHALL BE ASTM A500 GRADE B.
SPLICE AND BEARING PLATES SHALL BE A572 OR OTHER SUITABLE GRADE 345 MPa (50 KSI).
STEEL CAP BEAMS SHALL CONFORM TO ASTM A709 OR A572, GR. 345 MPa (50 ksi).
MISCELLANEOUS STEEL PLATES, ANGLES AND CHANNELS SHALL BE ASTM A36.

METAL BRIDGE DECKING SHALL BE PRE-GALVANIZED ASTM A607 340 MPa (50 KSI) OR ASTM A570 310 MPa (45 KSI) MIN., PREGALVANIZED PER ASTM A924, PROVIDE MINIMUM 12.7mm (1/2") DIAMETER WELD HOLES AT EACH FLUTE, AT EACH SUPPORT. PROVIDE SIDE SEAM WELDS AT LAPS AND BOUNDARIES AS INDICATED ON THE PLANS. LOCATE FULL STRENGTH SPLICES AT SUPPORTS. REPAIR GALVANIZING AT WELDS AS INDICATED BELOW FOR FIELD REPAIR. PROVIDE FOLLOWING MINIMUM DECK PROPERTIES:

DECK TYPE	THICKNESS	S cm^3/M (IN^3/FT)	I cm^4/M (IN^4/FT)
108x305 (4.25"x12")	4.5mm (7 Ga - 0.179")	233 (4.34)	1407 (10.34)

DURING CONSTRUCTION DO NOT LOAD INDIVIDUAL DECK SPANS GREATER THAN 9.1 MT (20-KIPS) WITHOUT PRIOR APROVAL OF THE ENGINEER.

WELDING FILLERS SHALL BE AS FOLLOWS: SMA - E7018-1 OR E8018, FCAW - E71T8-Ni 1% or Ni 2%, OR ENGINEER APPROVED EQUAL. PROVIDE MINIMUM PREHEAT FOR FIELD WELDS OF 40 DEG. C (100 DEG. F).

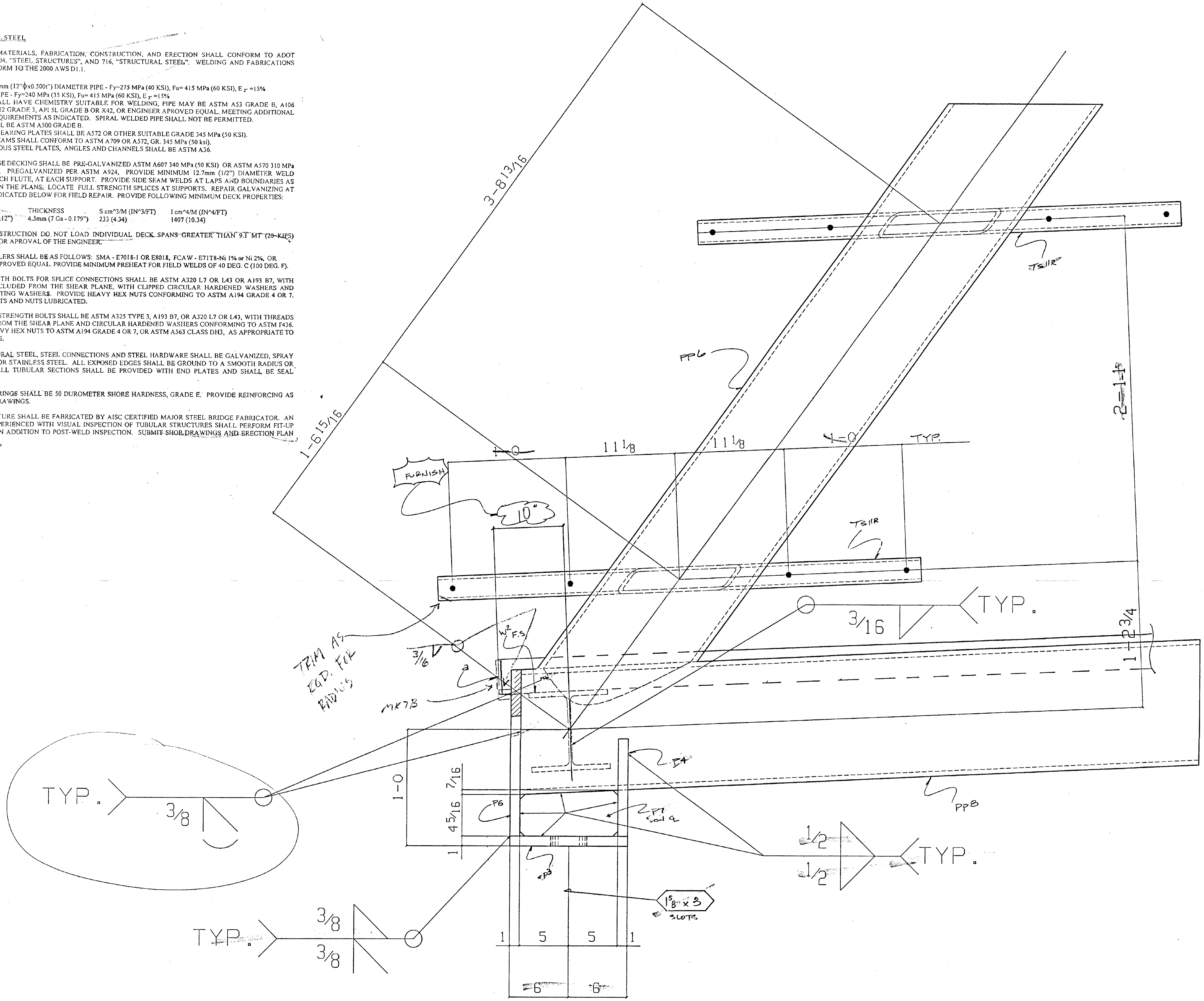
HIGH STRENGTH BOLTS FOR SPLICE CONNECTIONS SHALL BE ASTM A320 L7 OR L43 OR A193 B7, WITH THREADS EXCLUDED FROM THE SHEAR PLANE, WITH CLIPPED CIRCULAR HARDENED WASHERS AND LOAD INDICATING WASHERS. PROVIDE HEAVY HEX NUTS CONFORMING TO ASTM A194 GRADE 4 OR 7. PROVIDE BOLTS AND NUTS LUBRICATED.

OTHER HIGH STRENGTH BOLTS SHALL BE ASTM A325 TYPE 3, A193 B7, OR A320 L7 OR L43, WITH THREADS EXCLUDED FROM THE SHEAR PLANE AND CIRCULAR HARDENED WASHERS CONFORMING TO ASTM F436. PROVIDE HEAVY HEX NUTS TO ASTM A194 GRADE 4 OR 7, OR ASTM A563 CLASS DH3, AS APPROPRIATE TO MATCH BOLTS.

ALL STRUCTURAL STEEL, STEEL CONNECTIONS AND STEEL HARDWARE SHALL BE GALVANIZED, SPRAY METALIZED, OR STAINLESS STEEL. ALL EXPOSED EDGES SHALL BE GROUND TO A SMOOTH RADIUS OR CONTOUR. ALL TUBULAR SECTIONS SHALL BE PROVIDED WITH END PLATES AND SHALL BE SEAL WELDED.

RUBBER BEARINGS SHALL BE 50 DUROMETER SHORE HARDNESS, GRADE E. PROVIDE REINFORCING AS SHOWN ON DRAWINGS.

SUPERSTRUCTURE SHALL BE FABRICATED BY AISC CERTIFIED MAJOR STEEL BRIDGE FABRICATOR. AN AWS CWI EXPERIENCED WITH VISUAL INSPECTION OF TUBULAR STRUCTURES SHALL PERFORM FIT-UP INSPECTION IN ADDITION TO POST-WELD INSPECTION. SUBMIT SHOP DRAWINGS AND ERECTION PLAN FOR REVIEW.



BILL OF MATERIAL

No. TO SHIP	SHIP MARK	PC. Mark	SECTION	LENGTH FT.	LENGTH IN	Remarks	Steel Grade	ZONE
								1
2	1A		BRIDGE SECTION					
4		p2	PL1x15				A36	
4		pp20	PIPE 8 STD	16	11 7/16		A53	
4		pp10	PIPE 8 STD	16	11 7/16		A53	
4		pp9	PIPE 8 STD	16	11 9/16		A53	
4		pp21	PIPE 8 STD	16	11 9/16		A53	
4		pp7	PIPE 8 STD	16	11 1/2		A53	
4		p3	PL1x12	1	4		A572-50	
4		p4	PL1x10	1	4		A572-50	
4		p6	PL1x16	1	5 1/16		A572-50	
4		p7	PL1x4 3/4	0	10		A572-50	
4		pp8	PIPE 12 XSTG	67	0 5/16		A53	
4		pp11	PIPE 12 XSTG	55	9 13/16		A53	
8		p5	RPL1 1/2x18 3/4	0	10		A572-50	
4		pp24	PIPE 6 STD	8	11 11/16		A53	
2		pp13	PIPE 8 STD	8	7 1/4		A53	
4		pp6	PIPE 12 XSTG	18	1 5/8		A53	
2		pp22	PIPE 6 STD	13	6 1/4		A53	
2		pp23	PIPE 6 STD	13	6 3/4		A53	
6		pp2	PIPE 6 STD	13	6 13/16		A53	
2		w2	W8x35	8	6 3/8		A36	
12		w3	W8x15	8	6 1/4		A36	
2/2		ts11 R/L	TS2 1/2x2 1/2x5/16	4	1 5/16		A500A	
20/20		a3R	L2x2x3/8	0	8		A36	
20/20		a2R	L2x2x3/8	0	8		A36	
2		a	4 1/4 x 12 x 7 GA DECKING	550	0	LIN FT.		
2		b'	R 76A x 8'4"	140	0	}		
			7 GA					

MAT'L SPEC.	
OPEN HOLES	EXCEPT AS NOTED
END & EDGE DIST.	EXCEPT AS NOTED
PAINT	METALLIZED 6 MILS
SHIP TO	
ERECTION BY:	

KEISER STEEL FABRICATOR INC.
22620 85TH. PL., S. KENT, WA 98031 (206) 852-1910

11:48:20 AM	APPROVED:	DRAWN BY MKE
DATE: Apr 11 2000		CHK'D BY

ELEVATION

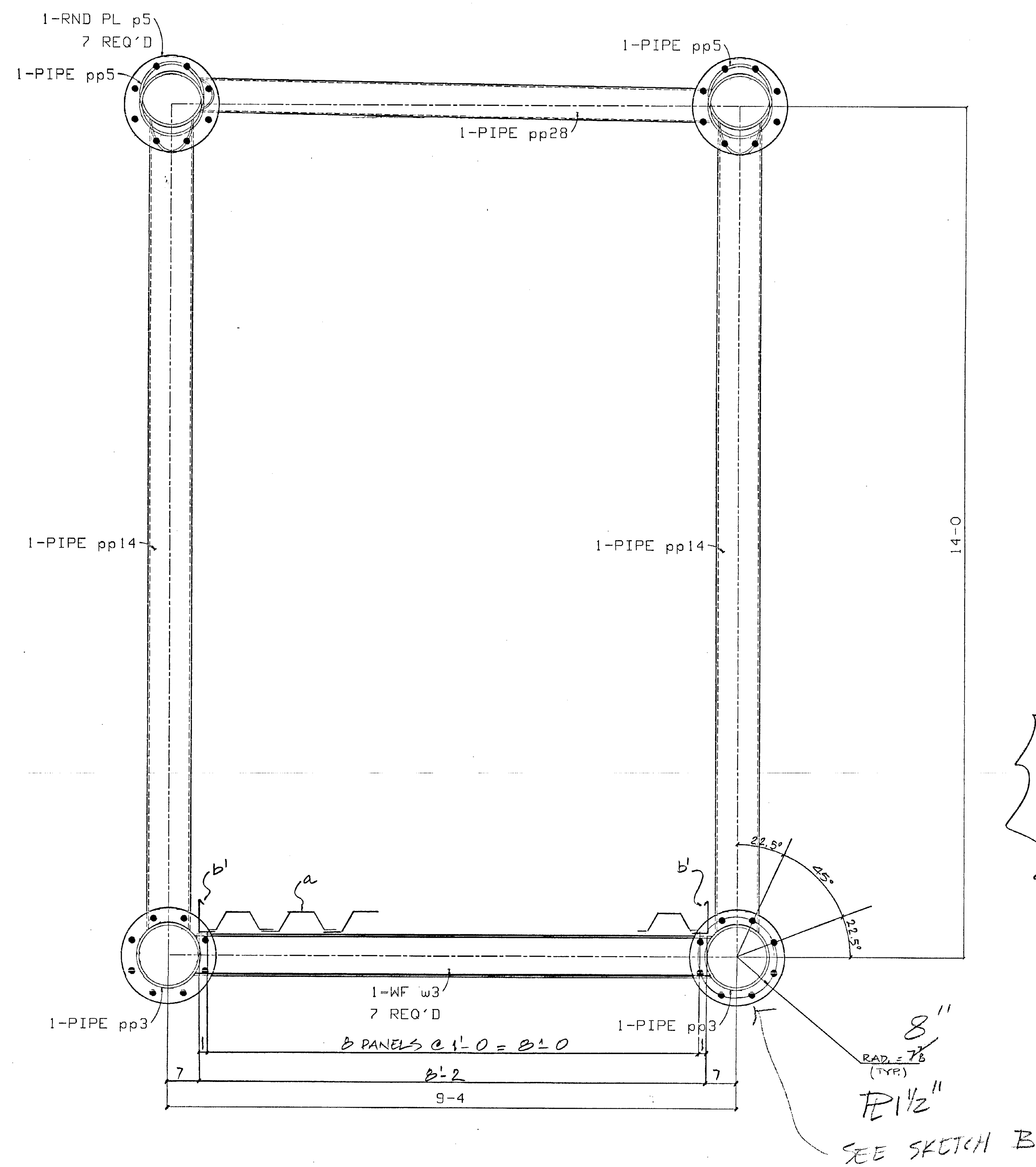
MONTANA CREEK BRIDGE

ANCHORAGE, ALASKA

CONTRACTOR	JOB NO.	DRAWING NO.	REV.
WEST CONST	00-10	2	0

MKE
987
REVISIONS

DO NOT SCALE



END VIEW

BILL OF MATERIAL

No. TO SHIP	SHIP MARK	PC. Mark	SECTION	LENGTH FT.	LENGTH IN	Remarks	Steel Grade	ZONE
ONE	12		MISC					1
2		pp15	PIPE 8 STD	16	11½		A53	
2		pp16	PIPE 8 STD	16	11 7/16		A53	
2		pp17	PIPE 8 STD	16	11 7/16		A53	
2		pp18	PIPE 8 STD	16	11½		A53	
2		pp19	PIPE 8 STD	16	11 7/16		A53	
7		p5	RPL1½x18¾	0	10		A572-50	
2		pp5	PIPE 12 XSTG	66	9 9/16		A53	
2		pp3	PIPE 12 XSTG	66	3 7/8		A53	
2		pp14	PIPE 8 STD	16	11 7/16		A53	
1		p11	RPL1½x18¾	0	10		A572-50	
4		pp2	PIPE 6 STD	13	6 13/16		A53	
1		pp28	PIPE 6 STD	13	6 13/16		A53	
7		w3	W8x15	8	6¼		A36	
1		pp4	PIPE 6 STD	13	6¾		A53	
1		a	4¼x12x7GA DECKING			620	LN. FT	
2		b'						
16		a3R	L2x2x3/8	0	8		A36	
8		a3L		0	8			
8		a2R		0	8			
16		a2L		0	8			
1			FIELD BOLTS					
		64	B1¼	0	5	SEE NOTE		

SHOP NOTE:

BOLTS SHALL BE ASTM A193 B7, OR A320 L7 OR L43, WITH THREADS EXCLUDED FROM THE SHEAR PLANE AND WITH LOAD INDICATING WASHERS, AND CIRCULAR HARDENED WASHERS CONFORMING TO ASTM F436, AND HEAVY HEX NUTS PER ASTM A563 CLASS DH, OR A194 GRADE 2H.

ALL STEEL HARDWARE SHALL BE HOT-DIP GALVANIZED, SPRAY METALIZED, OR STAINLESS STEEL. ALL EXPOSED EDGES SHALL BE GROUND TO A SMOOTH RADIUS OR CONTOUR. ALL TUBULAR SECTIONS SHALL BE PROVIDED WITH END PLATES AND SHALL BE SEAL WELDED.

MAT'L SPEC. SEE PAGE 2
OPEN HOLES EXCEPT AS NOTED
END & EDGE DIST. EXCEPT AS NOTED
PAINT METALLIZED 6MILS
SHIP TO 10
ERECTION BY:

KEISER STEEL FABRICATOR INC.
22620 85TH. PL., S. KENT, WA 98031 (206) 852-1910

04:32:28 PM | APPROVED: | DRAWN BY MKE
DATE: Apr 11 2000 | CHK'D BY

ELEVATION

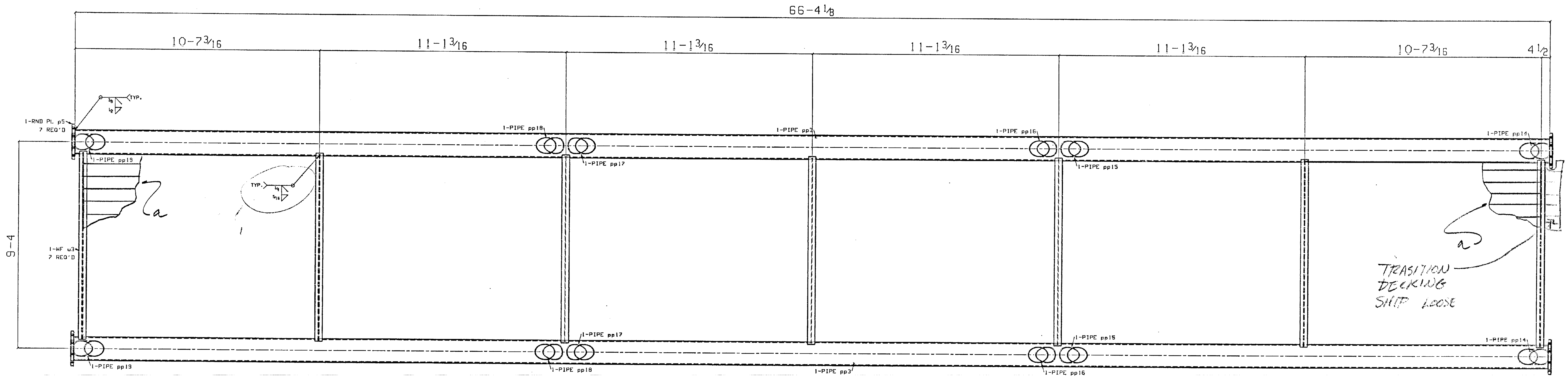
MONTANA CREEK BRIDGE

ANCHORAGE, ALASKA

CONTRACTOR: WEST CONST | JOB NO. 00-10 | DRAWING NO. 12 | REV. 0

MKE
987
REVISIONS

DO NOT SCALE



BOTTOM VIEW

MAT'L SPEC. SEE PAGE 2
OPEN HOLES EXCEPT AS NOTED
END & EDGE DIST. EXCEPT AS NOTED
PAINT METALLIZED 6 MILS
SHIP TO
ERECTION BY:

KEISER STEEL FABRICATOR INC.
22620 85TH. PL., S. KENT, WA 98031 (206) 852-1910

03:48:23 PM APPROVED: DRAWN BY MKE
DATE: Apr 11 2000 CHK'D BY

ELEVATION
MONTANA CREEK BRIDGE
ANCHORAGE, ALASKA

CONTRACTOR WEST CONST | JOB NO. 00-10 | DRAWING NO. 11 | REV. 0

MKE
987
REVISIONS

DO NOT SCALE

ONE ~ BRIDGE SECTION ~ MK10A

MAT'L SPEC.	SEE PAGE 2
OPEN HOLES	EXCEPT AS NOTED
END & EDGE DIST.	EXCEPT AS NOTED
PAINT	METALLIZED 6 MILS
SHIP TO	
ERECTION BY:	

KEISER STEEL FABRICATOR INC.
22620 85TH. PL., S. KENT, WA 98031 (206) 852-1910

03:48:25 PM	APPROVED:	DRAWN BY MKE
DATE: Apr 11 2000		CHK'D BY

ELEVATION

MONTANA CREEK BRIDGE

ANCHORAGE, ALASKA

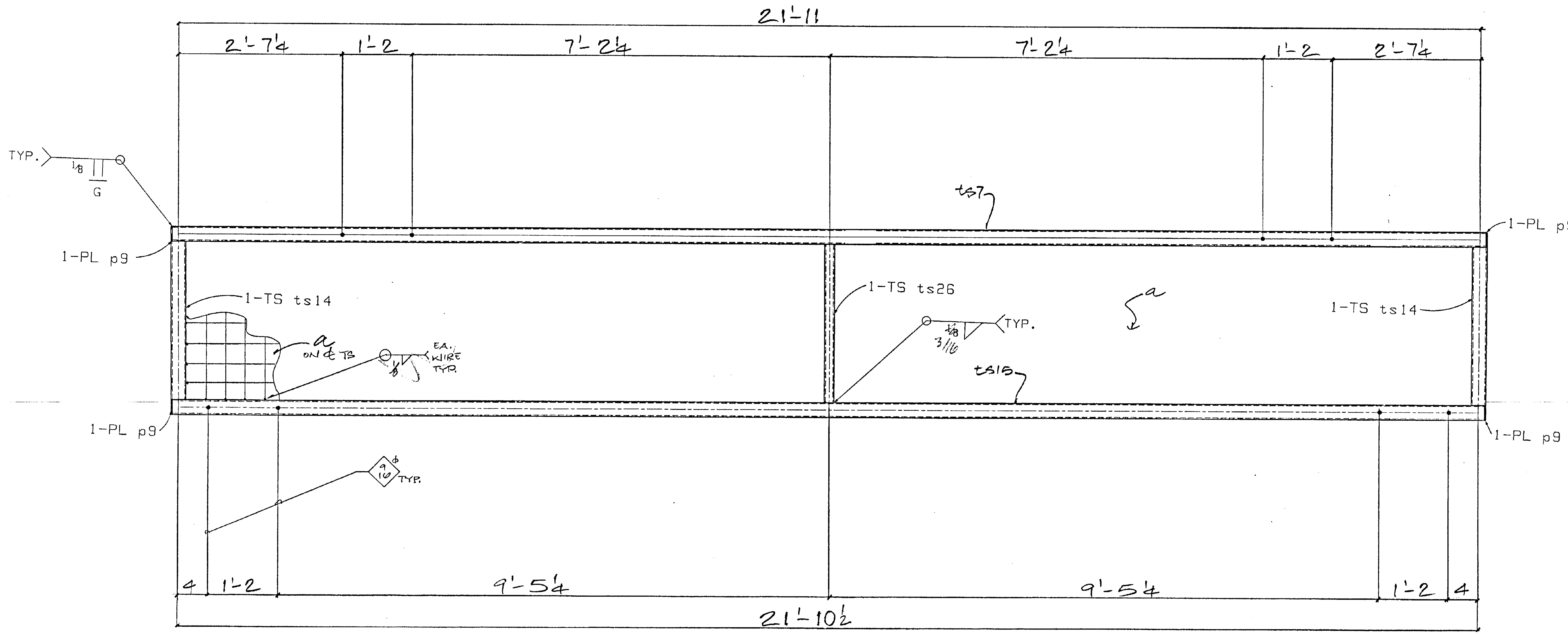
CONTRACTOR	JOB NO.	DRAWING NO.	REV.
WEST CONST	00-10	10	0

MKE 987 REVISIONS

DO NOT SCALE

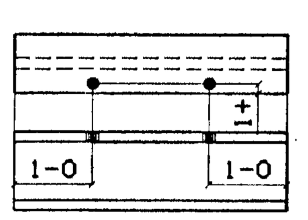
BILL OF MATERIAL

No. TO SHIP	SHIP MARK	PC. Mark	SECTION	LENGTH FT.	LENGTH IN	Remarks	Steel Grade	ZONE
14	9A		MISC					1
14		ts7	TS3x3x¼	22	1		A500A	
14		ts14	TS3x3x¼	2	8		A500A	
14		ts15	TS3x3x¼	22	1		A500A	
14		p9	PL¼x3	0	3		A36	
14		ts26	TS2x2x¼	2	8		A500A	
28		a	WIRE 32x MESH	10	8¼	SEE NOTE		
1			FIELD BOLTS					
	112		⅝"ϕ CARRIAGE	0	3½			



14 ~ HANDRAILS ~ MK9A

SHOP NOTE:
4x4x¼ WELDED MESH



SAFETY CABLE HOLES
TYP. ALL BM'S OVER 8'-0"

MAT'L SPEC. SEE PAGE 2
OPEN HOLES EXCEPT AS NOTED
END & EDGE DIST. EXCEPT AS NOTED
PAINT METALLIZED 6 MILS
SHIP TO
ERECTION BY:

KEISER STEEL FABRICATOR INC.
22620 85TH. PL., S. KENT, WA 98031 (206) 852-1910

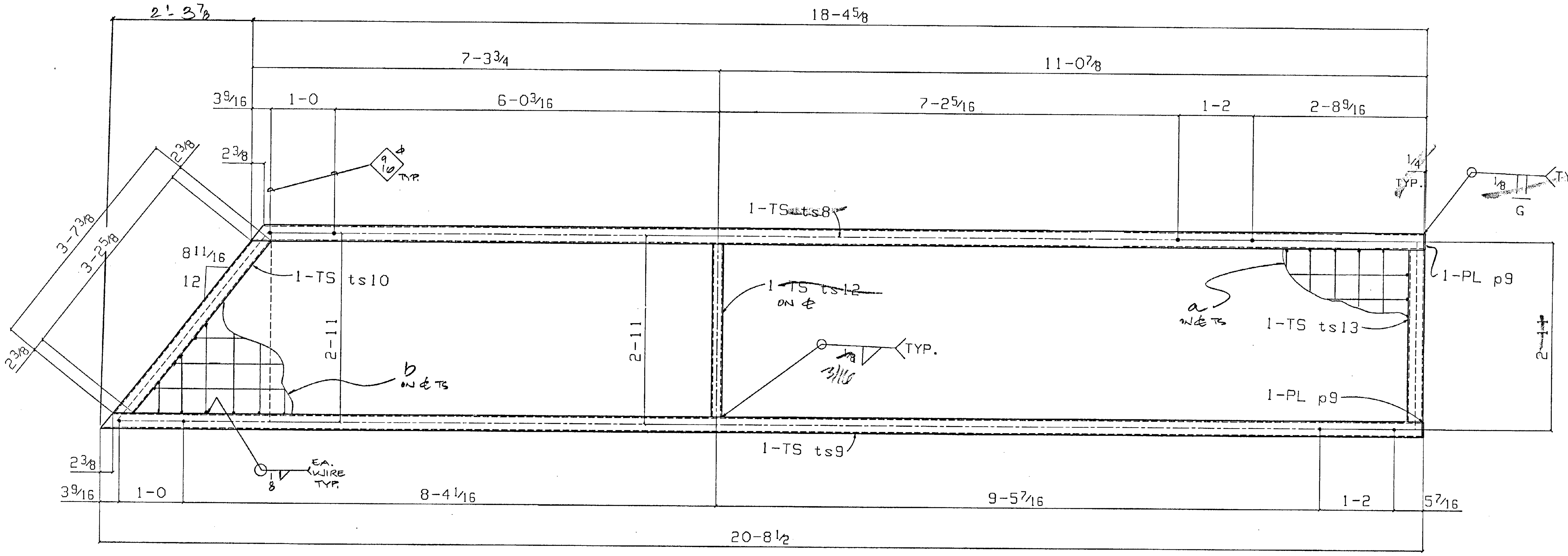
01:24:01 PM
DATE: Apr 05 2000
APPROVED:
DRAWN BY MKE
CHK'D BY

ELEVATION
MONTANA CREEK BRIDGE
ANCHORAGE, ALASKA

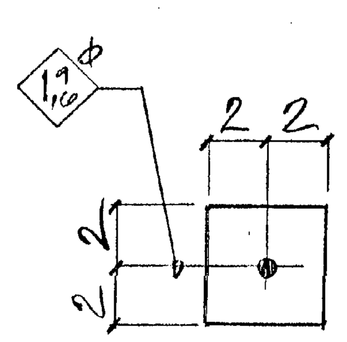
CONTRACTOR WEST CONST
JOB NO. 00-10
DRAWING NO. 9
REV.

MKE
987
REVISIONS

DO NOT SCALE

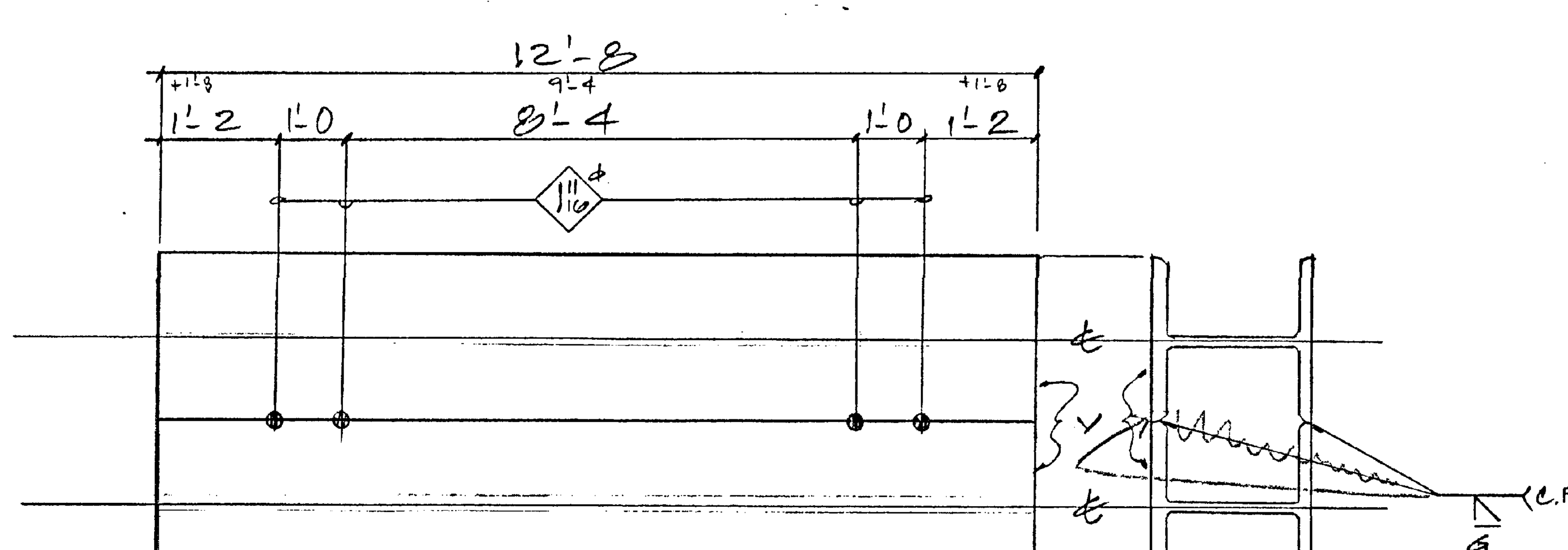


4 ~ HANDRAILS ~ MK8A



16 ~ ℞ 1/4 x 4 x 0'-4 ~ MK8D

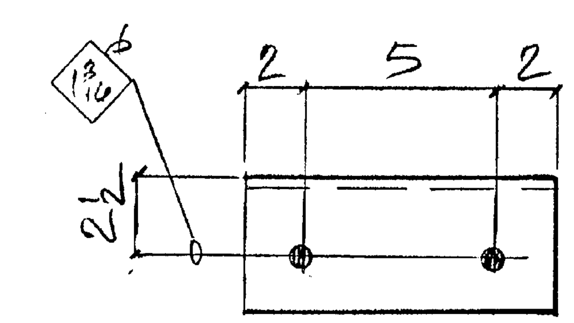
WASHER ℞



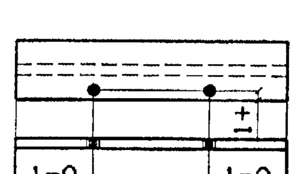
2 BEARING SEAT MK8B

SHOP NOTE:

4 x 4 x 1/4 WELDED MESH



16 ~ L4 x 4 x 1/2 x 0'-9 ~ MK8C



SAFETY CABLE HOLES
TYP. ALL 8M'S OVER 8'-0"

BILL OF MATERIAL

No. TO SHIP	SHIP MARK	PC. Mark	SECTION	LENGTH FT.	IN	Remarks	Steel Grade	ZONE
4	8A		MISC					1
4		ts9	TS3x3x1/4	20	8 1/4		A500A	
4		ts10	TS3x3x1/4	3	7 3/8		A500A	
8		p9	PL1/4x3	0	3		A36	
4		ts8	TS3x3x1/4	18	4 3/8		A500A	
4		ts13	TS3x3x1/4	2	8		A500A	
4		ts12	TS2x2x1/4	2	8 1/16		A500A	
4		a	32 x WIRE MESH	10	8 7/8	SEE NOTE		
4		b	32	9	5			
2	8B		BEARING SEAT					
4		1	W10 x 49	12	8			
1			FIELD BOLTS					
	32		B 1/2 ⌀ CARRIAGE	0	4 1/2			
	8		B 1 1/2	0	7 1/2			
	32		B1	1	0			
	32		P1 ⌀	0	10	SLEEVE		
16	8C		L4x4x1/2	0	9			A36
16	8D		℞ 1/4 x 4	0	4			A36

MAT'L SPEC.	SEE PAGE 2
OPEN HOLES	EXCEPT AS NOTED
END & EDGE DIST.	EXCEPT AS NOTED
PAINT	METALLIZED 6 MILS
SHIP TO	
ERECTION BY:	

KEISER STEEL FABRICATOR INC.
22620 85TH. PL., S. KENT, WA 98031 (206) 852-1910

02:28:55 PM	APPROVED:	DRAWN BY MKE
DATE: Apr 05 2000		CHK'D BY

ELEVATION

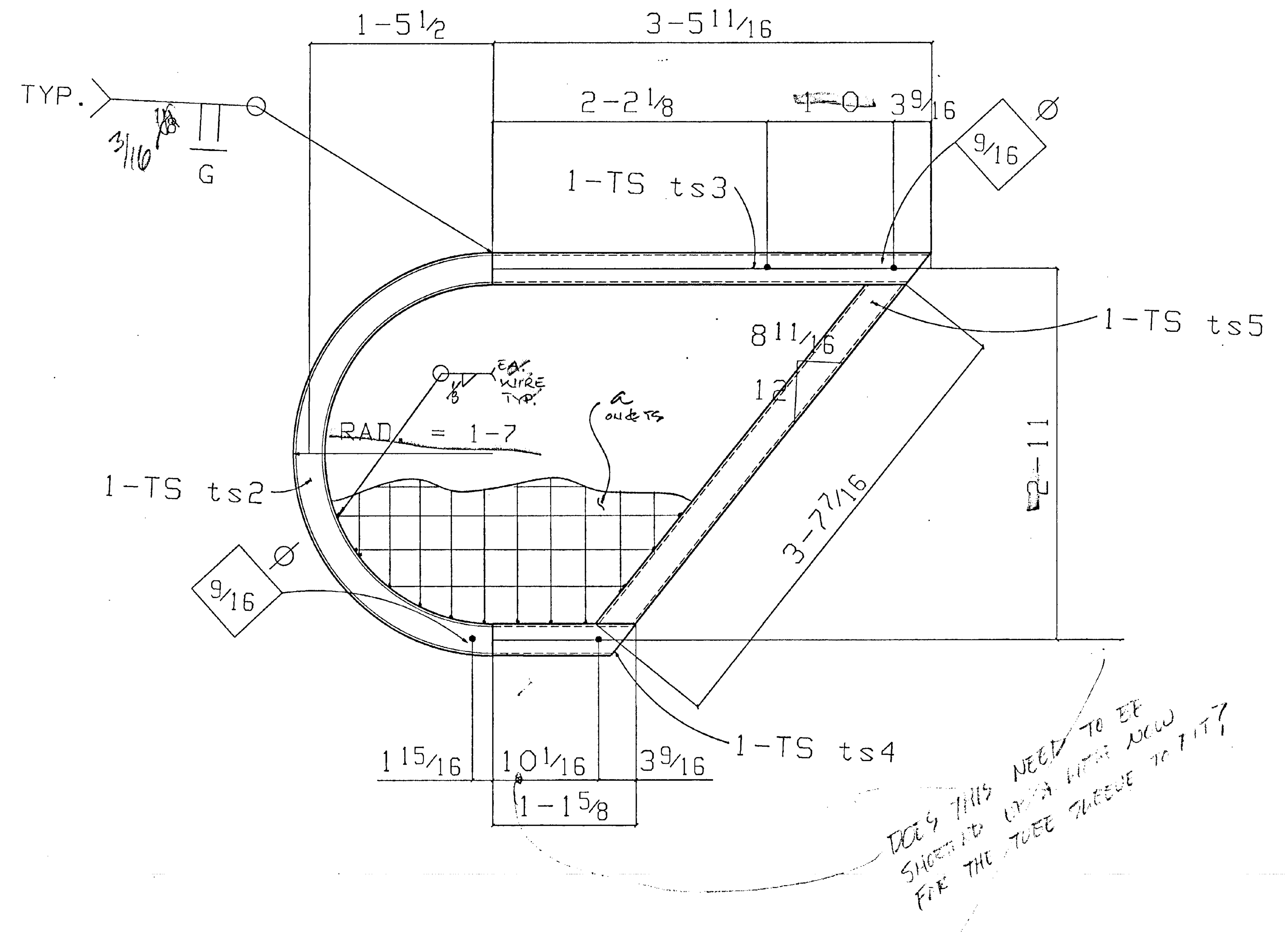
MONTANA CREEK BRIDGE

ANCHORAGE, ALASKA

CONTRACTOR	JOB NO.	DRAWING NO.	REV.
WEST CONST	00-10	8	1

MKE
987
REVISIONS

DO NOT SCALE

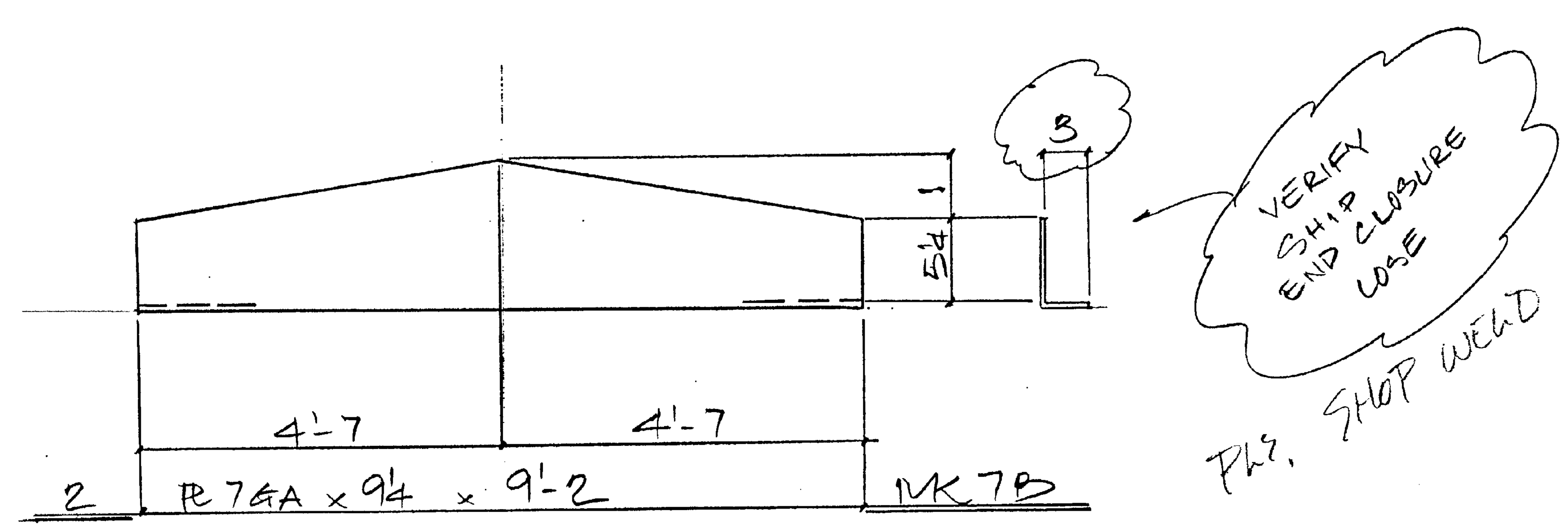


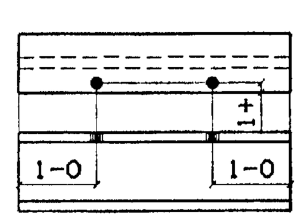
4 ~ HANDRAILS ~ MK7A

SHOP NOTE:

4x4x¼ WELDED MESH

TRIM TO FIT.





SAFETY CABLE HOLES
TYP. ALL BM'S OVER 8'-0"

BILL OF MATERIAL

No. TO SHIP	SHIP MARK	PC. Mark	SECTION	LENGTH FT.	LENGTH IN	Remarks	Steel Grade	ZONE
4	7A							1
4		ts2	TS3x3x¼	4	7		A500A	
4		ts4	TS3x3x¼	1	1⅝		A500A	
4		ts3	TS3x3x¼	3	5 11/16		A500A	
4		ts5	TS3x3x¼	3	7 7/16		A500A	
4		a	WIRE 32x MESH	4	8	SEE NOTE		
2	7B		PL 7GA x 9¼	9	2	BEND		
16	7C		4¼ x 12 x 7GA	1	0	DECKING		
1			FIELD BOLTS					
	16		½ϕ CARRIAGE	0	3½			

MAT'L SPEC. SEE PAGE 2

OPEN HOLES EXCEPT AS NOTED

END & EDGE DIST. EXCEPT AS NOTED

PAINT METALLIZED 6MILS

SHIP TO

ERECTION BY:

KEISER STEEL FABRICATOR INC.
22620 85TH. PL., S. KENT, WA 98031 (206) 852-1910

02:28:54 PM	APPROVED:	DRAWN BY MKE
DATE: Apr 05 2000		CHK'D BY

ELEVATION

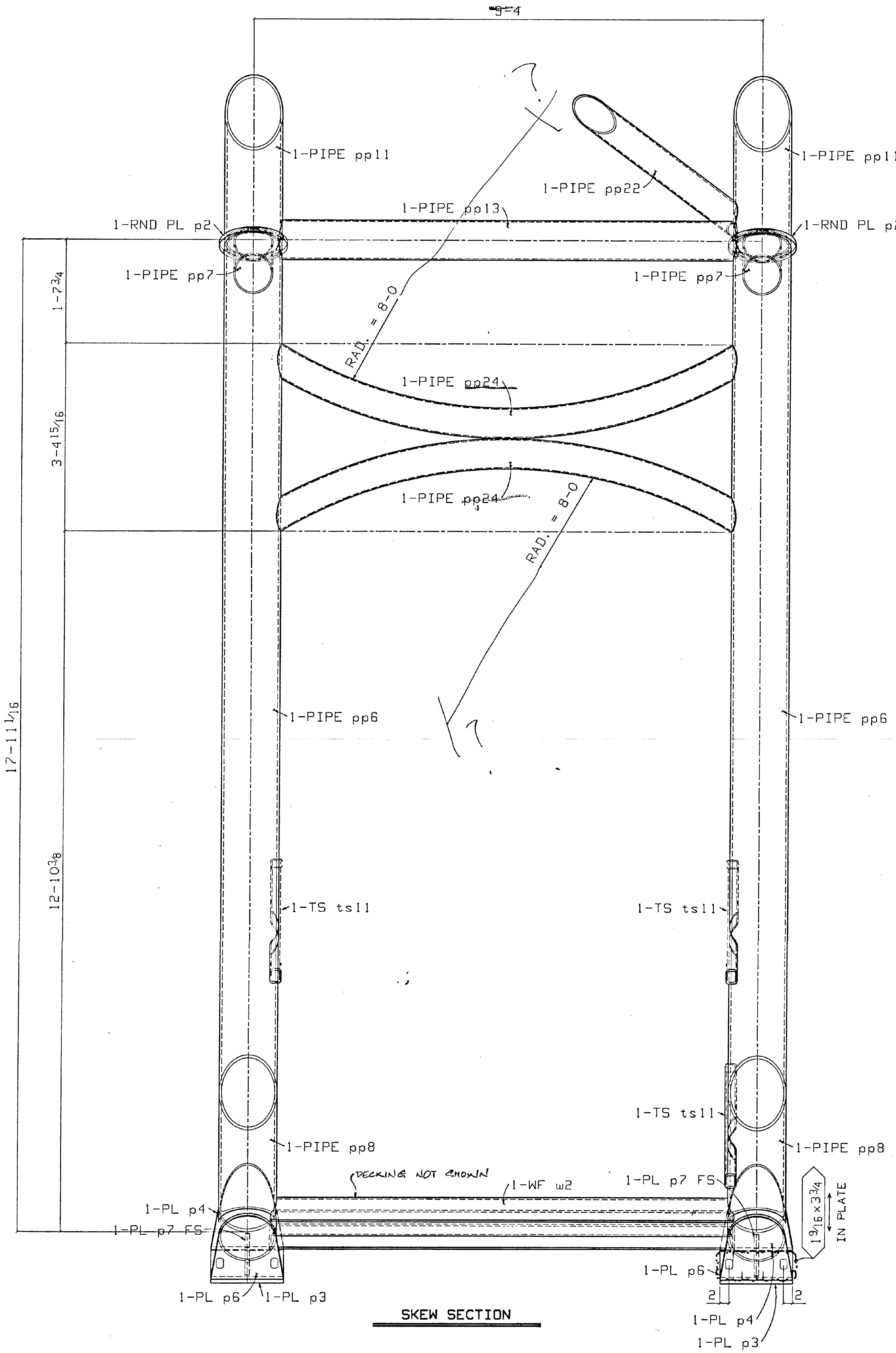
MONTANA CREEK BRIDGE

ANCHORAGE, ALASKA

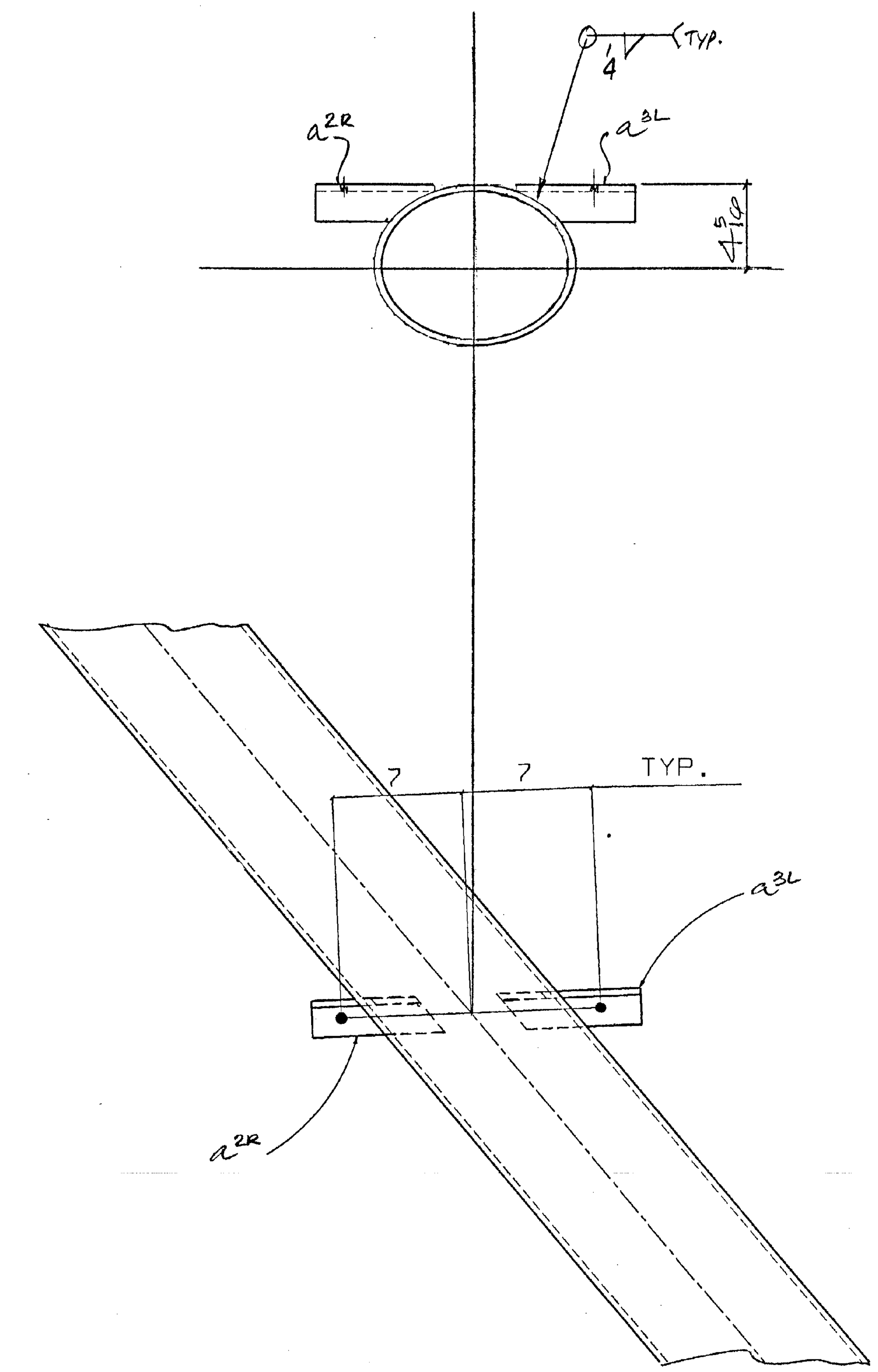
CONTRACTOR	JOB NO.	DRAWING NO.	REV.
WEST CONST	00-10	7	

MKE
987
REVISIONS

DO NOT SCALE



SKEW SECTION

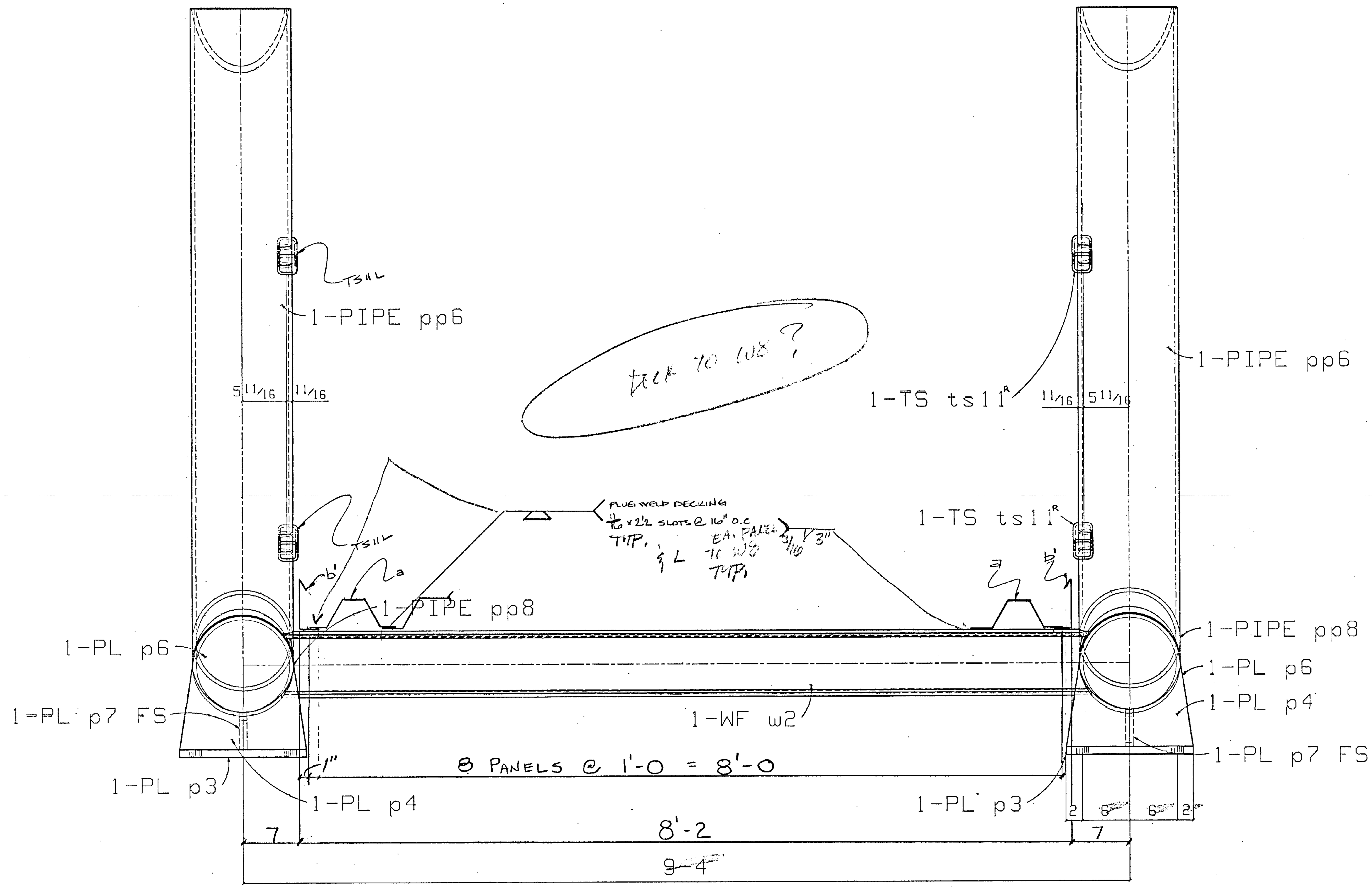


DET. B.

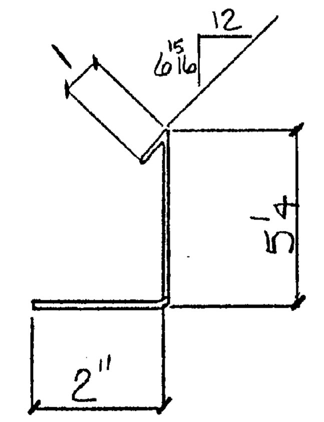
MAT'L SPEC. SEE PAGE 2	
OPEN HOLES	EXCEPT AS NOTED
END & EDGE DIST.	EXCEPT AS NOTED
PAINT METALLIZED 6 MILS	
SHIP TO	
ERECTION BY:	
KEISER STEEL FABRICATOR INC. 22620 85TH. PL., S. KENT, WA 98031 (206) 852-1910	
11:53:54 AM DATE: Apr 13 2000	APPROVED: DRAWN BY MKE CHK'D BY
ELEVATION	
MONTANA CREEK BRIDGE	
ANCHORAGE, ALASKA	
CONTRACTOR WEST COAST	JOB NO. 00-10 · DRAWING NO. 6 · REV. 0

MKE 987 REVISIONS

DO NOT SCALE



END VIEW



PROFILE P.C b'

MAT'L SPEC. SEE PAGE 2	
OPEN HOLES	EXCEPT AS NOTED
END & EDGE DIST.	EXCEPT AS NOTED
PAINT METALLIZED 6 MILS	
SHIP TO	
ERECTION BY:	

KEISER STEEL FABRICATOR INC.
22620 85TH. PL., S. KENT, WA 98031 (206) 852-1910

01:51:12 PM	APPROVED:	DRAWN BY MKE
DATE: Apr 11 2000		CHK'D BY

ELEVATION

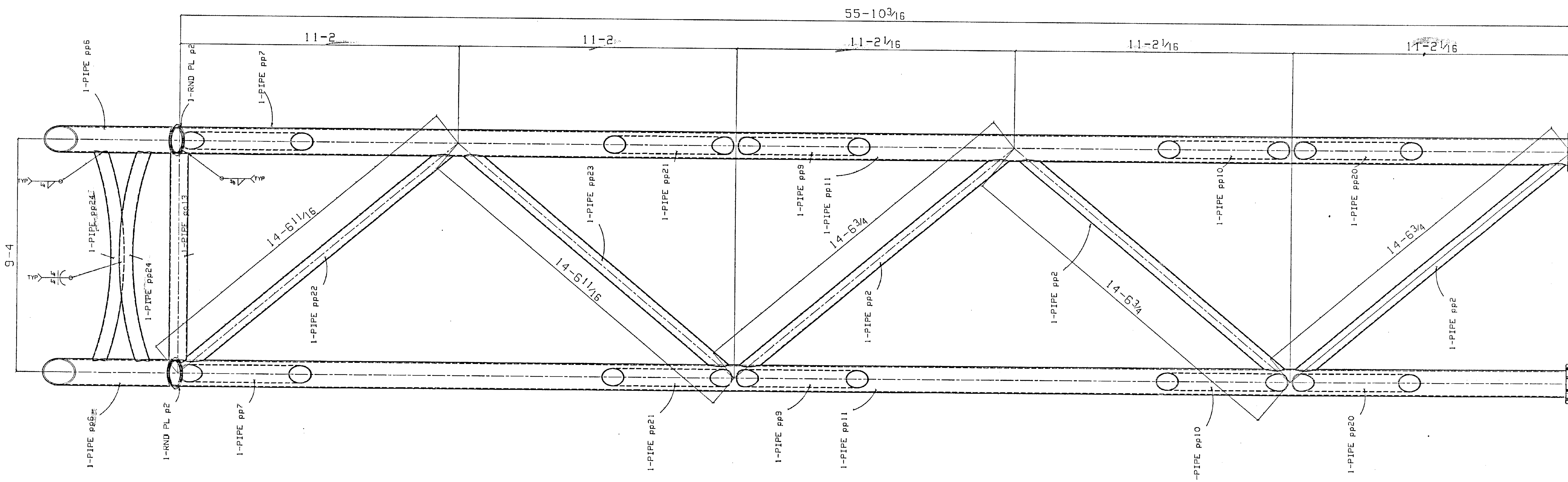
MONTANA CREEK BRIDGE

ANCHORAGE, ALASKA

CONTRACTOR	JOB NO.	DRAWING NO.	REV.
WEST CONST	00-10	5	0

MKE
987
REVISIONS

DO NOT SCALE



TOP CHORD PLAN

MAT'L SPEC.	SEE PAGE 2
OPEN HOLES	EXCEPT AS NOTED
END & EDGE DIST.	EXCEPT AS NOTED
PAINT	METALLIZED 6 MILS
SHIP TO	
ERECTION BY:	

KEISER STEEL FABRICATOR INC.
22620 85TH. PL., S. KENT, WA 98031 (206) 852-1910

02:29:00 PM DATE: Apr 11 2000	APPROVED:	DRAWN BY MKE CHK'D BY

- ELEVATION -

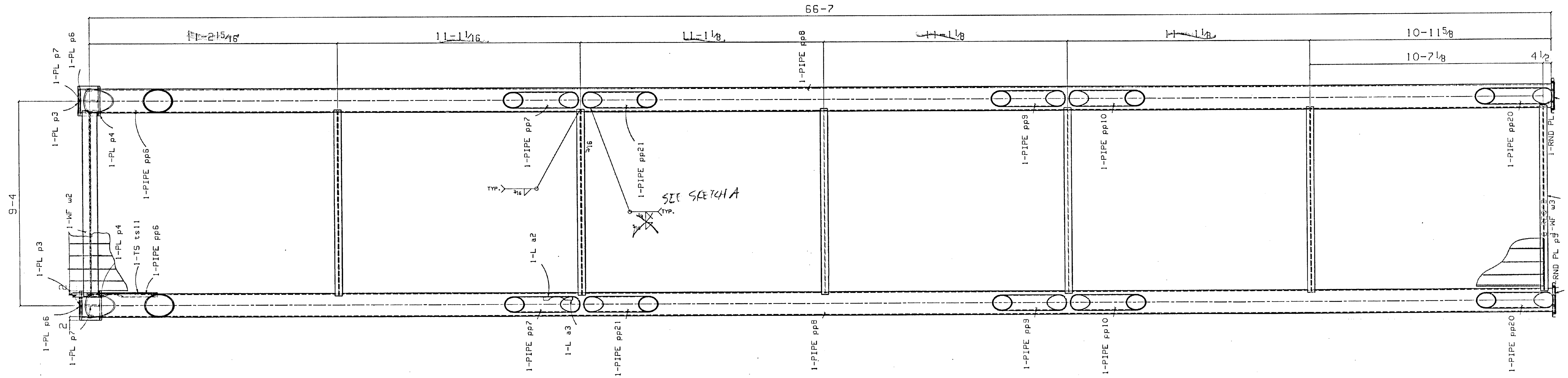
MONTANA CREEK BRIDGE

ANCHORAGE, ALASKA

CONTRACTOR	JOB NO.	DRAWING NO.	REV
WEST CONST	00-10	4	0

MKE 987 REVISION

DO NOT SCALE



BOTTOM CHORD PLAN

MAT'L SPEC. SEE PAGE 2	
OPEN HOLES	EXCEPT AS NOTED
END & EDGE DIST.	EXCEPT AS NOTED
PAINT METALLIZED 6 MILS	
SHIP TO	
ERECTION BY:	

KEISER STEEL FABRICATOR INC.
22620 85TH. PL., S. KENT, WA 98031 (206) 852-1910

02:18:11 PM DATE Apr 11 2000	APPROVED:	DRAWN BY MKE / CHK'D BY -

ELEVATION

MONTANA CREEK BRIDGE

ANCHORAGE, ALASKA

CONTRACTOR	JOB NO.	DRAWING NO.	REV.
WEST CONST	00-10	3	0

MKE 987 REVISION

DO NOT SCALE

Routine Inspection Report

Date: 6/16/2018

Br No 6012 | MONTANA CR PEDESTRIAN

Rain 50 F

INSPECTOR: Sara Manning **ASSISTANT:** Mary McRae

Initials SEM Initials MM

N.E.: Anchorage — From

Left / Right

F.E.: Fairbanks — To

Flow: ☐ (down) ☑ (up)

North Direction: ☑ (east)

Apprch Rdwy Width (ft)	8.20
Est Camber (inch)	4.00
Bridge Length (ft)	200
Bridge Width O-O (ft)	9.3
CDS Route	170000
CDS Mile Point	61.1

Left

Type BRIDGE RAIL	Ht (in)	Curb Height (in)	Transition Type	Ht (in)	Approach Type/Post	Leading End Treat
Pedestrian Rail	41.50	0.00		0.00		

Right

Type BRIDGE RAIL	Ht (in)	Curb Height (in)	Transition Type	Ht (in)	Approach Type/Post	Leading End Treat
Pedestrian Rail	41.50	0.00		0.00		

SIGNS

Near End

Lt. Rt.

- ☐ ☐ Object Markers
- ☐ Restricted Width
- ☐ Vertical Clear
- ☐ Load Limit
- ☐ Name Place

Load Limit Wording or Other Signs:

Far End

Lt. Rt.

- ☐ ☐ Object Markers
- ☐ Restricted Width
- ☐ Vertical Clear
- ☐ Load Limit
- ☐ Name Place

Load Limit Wording or Other Signs:

Deck Material	Thk(Inches)
	0.00

Location Measured: Can't Measure

Wear Surface	Thk (Inches)
	0.00

Wear Location: Can't Measure

Drive Condition:

Locality: Talkeetna

Route: Pedestrian Path

Utilities

Kind	Size	Location	Owner
Box		Mid -span DS	Unknown
Stream gage		NE bank beneath bridge	USGS
Gaging station		NE DS	USGS

MONTANA CREEK BRIDGE INSPECTION REPORT

Abbreviations: FE=Far End, NE=Near End, DS= Downstream, US=Upstream, LT=Left, RT=Right, P=Pier, FB=Floorbeam, G=Girder, S=Span

Bridge No. 6012, MONTANA CR PEDESTRIAN

Work Candidates

Inspected on: 06/16/2018

Priority	*Description*	*Quantity*	*Work Needed*
Medium	Approach Roadway (EA)	2	Fill eroded shoulders and level bump at each end.

Bridge No. 6012, MONTANA CR PEDESTRIAN

Element Inspection

Inspected on: *06/16/2018*

Element	*Description*	*Observations*
30	Steel Deck Corrugated/Orthotropic/Etc.	Steel stay-in-place corrugated deck filled with asphalt. Minor rust spots on soffit. Transverse crack at 1/4, 2/3 and 3/4 span.
> 510	*Wearing Surfaces*	
> 515	*Steel Protective Coating*	
120	Steel Truss	Rust stains bottom chord and diagonal at first panel point NE DS and NE US. Several other areas of spot rust, US concentrated at NE.
> 515	*Steel Protective Coating*	
215	Reinforced Concrete Abutment	None.
231	Steel Pier Cap	None.
310	Elastomeric Bearing	Debris on bearings, typical.
330	Metal Bridge Railing	Rust staining at USGS gage box connection, DS. Minor galvanizing failure with surface rust on top portion of panel ends, multiple locations.
> 515	*Steel Protective Coating*	
606	Approach Fill Erosion Smart Flag	Shoulder eroded and asphalt spalling at each side, NE. 1 inch bump at approach to bridge, each end. NE DS filter fabric exposed.

Br No 6012	**MONTANA CR PEDESTRIAN**	**Date:** 6/16/2018	ID: 04 Even 2018 6012
INSPECTOR: Sara Manning	**ASSISTANT:** Mary McRae	**Weather** Rain	**Temperature** 50 F

HYDRAULICS REPORT

Inspection To Mudline At All Piers and Abuts? *Yes*

Apparent HW *No*

AHW Comments

Bank Erosion *No*

Erosion Comments

Drift Comments Branches along stream bank

Stream Bottom Material: Sand, Cobble, Gravel

Activities None

Drift Light

Riprap Condition Good

Other Hydraulic Comments For soundings, see BN 215

SOUNDINGS

Measured At Surface

Location Upstream

Abbreviations: FE=Far End, NE=Near End, DS= Downstream, US=Upstream, LT=Left, RT=Right, P=Pier, FB=Floorbeam, G=Girder, S=Span

Structure Inventory and Appraisal Sheet (English Units)

Bridge Key: 6012	Agency ID: 6012	SR: -2.0	SD/FO: NA

IDENTIFICATION

State 1:	02 Alaska	Struc Number 8:	6012
Facility Carried 7:	PEDESTRIAN ROUTE	Location 9:	MILE POINT 61.6
Rte.(On/Under)5A:	Route On Structure	Rte Signing Prefix 5B:	!
Level of Service 5C:	0	Rte. Number 5D:	00000
Directional Suffix 5E:	0 N/A (NBI)	% Responsibility:	
SHD District 2:	01 Central	County Code 3:	Matanuska Susitna
Place Code 4:	Willow	Mile Post 11:	61.587 mi
Feature Intersected 6:	MONTANA CR PEDESTRIAN		
Latitude 16:	62d 06' 15.9 "	Longitude 17:	150d 03' 34.4 "
Border Bridge Code 98:	Unknown (P)		
Border Bridge No. 99:	NA		

STRUCTURE TYPE AND MATERIALS

Number of Approach Spans 46: 0 — Number of Spans Main Unit 45: 1

Main Span 43A/B:

3 Steel	10 Truss - Thru

Appr Span 44A/B:

0 Other	0
Deck Type 107:	6
Wearing Surface 108A:	6 Bituminous
Membrane 108B:	0 None
Deck Protection 108C:	_

AGE AND SERVICE

Year Built 27:	2001	Year Reconstructed 106:	
Type of Service on 42A:	3 Pedestrian-bicycle		
Type of Service under 42B:	5 Waterway		
Lanes on 28A:	0	Lanes under 28B: 0	Detour Length 19: 1 mi
ADT 29:	0	Truck ADT 109: %	Year of ADT 30: 2017

GEOMETRIC DATA

Length Max Span 48:	200 ft	Structure Length 49:	200 ft
Curb/Sdwlk Width L 50A:	0.0 ft	Curb/Sidewalk Width R 50B:	0.0 ft
Width Curb to Curb 51:	8.2 ft	Width Out to Out 52:	9.3 ft
Approach Roadway Width 32: (w/ shoulders)	10 ft	Median 33:	0 No median
Deck Area:	1,862.2 sq ft		
Skew 34:	0	Structure Flared 35:	0 No flare
Vertical Clearance 10:	9.50 ft	Horizontal Clearance 47:	8.00 ft
Minimum Vertical Clearance Over Bridge 53:	9.6 ft		
Minimum Vertical Underclearance 54A:	N Feature not hwy or RR		
Minimum Vertical Underclearance 54B:	0.0 ft		
Minimum Lateral Underclearance 55A:	N Feature not hwy or RR		
Minimum Lateral Underclearance R 55A:	0.0 ft		
Minimum Lateral Underclearance L 56:	0.0 ft		

INSPECTION

Frequency 91:	48 months	Inspection Date 90:	06/16/2018	Next Inspection:	06/16/2022
FC Freq. 92A:	48 months	FC Insp. Date 93A:	01/01/1901	Next FC Inspection:	01/01/1901
UW Freq. 92B:	NA	UW Insp. Date 93B:	NA	Next UW Inspection:	NA
SI Freq. 92C:	NA	SI Date 93C:	NA	Next SI:	NA

CLASSIFICATION

Defense Highway 100:	0 Not a STRAHNET hwy	Parallel Structure 101:	No ‖ bridge exists
Traffic Direction 102:	0	Temporary Structure 103:	Unknown (NBI)
Highway System 104:	0 Not on NHS	NBIS Length 112:	N
Toll Facility 20:	3 On free road	Functional Class 26:	-2
Natl. Network 110:	0 Not on truck network	Historical Sig. 37:	5 Not eligible for NRHP
Owner 22:	State Highway Agency		
Custodian 21:	State Highway Agency		

CONDITION

Deck 58: 8 Very Good	Super 59: 8 Very Good	Sub 60:	8 Very Good
Channel/Ch. Protection 61:	8 Protected	Culvert 62:	N N/A (NBI)

LOAD RATING AND POSTING

Inventory Method 65:	5	Operating Method 63:	5
Inventory Rating 66:	HS 0	Operating Rating 64:	HS 0
Design Load 31:	7	Posting 70:	5 At/Above legal loads
Posting Status 41:	A Open, no restriction		

APPRAISAL

Bridge Rail 36A:	N	Approach Rail 36C:	N
Transition 36B:	N	Approach Rail Ends 36D:	N
Str Evaluation 67:	N Not applicable (NBI)	Deck Geometry 68:	N Not applicable (NBI)
Underclearance, Vertical and Horizontal 69:		N Not applicable (NBI)	
Waterway Adequacy 71:	8 Equal Desirable	Approach Alignment 72:	!
Scour Critical 113:	6		

PROPOSED IMPROVEMENTS

Bridge Cost 94:	$-1	Type of Work 75:	Unknown (P)
Roadway Cost 95:	$-1	Length of Improvement 76:	- 1.0 ft
Total Cost 96:	$-1	Future ADT 114:	0
Year of Cost Estimate 97:	Unknown	Year of Future ADT 115:	2035

NAVIGATION DATA

Navigation Control 38:	0 Permit Not Required		
Vertical Clearance 39:	0.0 ft	Horizontal Clearance 40:	0.0 ft
Pier Protection 111:	1 Not required	Lift Bridge Vertical Clearance 116:	- 1.0 ft

Bridge No. **6012** Br. Name **Montana Cr Pedestrian** Date **06/16/18**
Inspector **Sara Manning / Mary McRae** Frame **1**
Ahead at bridge File **P6160110.JPG**

Bridge No. **6012** Br. Name **Montana Cr Pedestrian** Date **06/16/18**
Inspector **Sara Manning / Mary McRae** Frame **2**
Back at bridge File **P6160123.JPG**

Bridge No. **6012** Br. Name **Montana Cr Pedestrian** Date **06/16/18**
Inspector **Sara Manning / Mary McRae** Frame **3**
Looking US File **P6160112.JPG**

Bridge No. **6012** Br. Name **Montana Cr Pedestrian** Date **06/16/18**
Inspector **Sara Manning / Mary McRae** Frame **4**
Looking DS File **P6160115.JPG**

Bridge No. **6012** Br. Name **Montana Cr Pedestrian** Date **06/16/18**
Inspector **Sara Manning / Mary McRae** Frame **5**
US elevation File **P6160161.JPG**

Bridge No. **6012** Br. Name **Montana Cr Pedestrian** Date **06/16/18**
Inspector **Sara Manning / Mary McRae** Frame **6**
US elevation File **P6160160.JPG**

Bridge No. **6012** Br. Name **Montana Cr Pedestrian** Date **06/16/18**
Inspector **Sara Manning / Mary McRae** Frame **7**
NE abutment File **P6160152.JPG**

Bridge No. **6012** Br. Name **Montana Cr Pedestrian** Date **06/16/18**
Inspector **Sara Manning / Mary McRae** Frame **8**
FE abutment File **P6160126.JPG**

Bridge No.	**6012**	Br. Name	**Montana Cr Pedestrian**	Date	**06/16/18**
Inspector		**Sara Manning / Mary McRae**		Frame	**9**
	NE portal			File	**P6160144.JPG**

Bridge No.	**6012**	Br. Name	**Montana Cr Pedestrian**	Date	**06/16/18**
Inspector		**Sara Manning / Mary McRae**		Frame	**10**
	FE portal			File	**P6160137.JPG**

Bridge No.	**6012**	Br. Name	**Montana Cr Pedestrian**	Date	**06/16/18**
Inspector		**Sara Manning / Mary McRae**		Frame	**11**
	Typical debris at exterior bearing (NE DS shown)			File	**P6160134.JPG**

Bridge No.	**6012**	Br. Name	**Montana Cr Pedestrian**	Date	**06/16/18**
Inspector		**Sara Manning / Mary McRae**		Frame	**12**
	Bump and spalls at NE joint			File	**P6160145.JPG**

Bridge No.	**6012**	Br. Name	**Montana Cr Pedestrian**	Date	**06/16/18**
Inspector		**Sara Manning / Mary McRae**		Frame	**13**
	Wearing surface			File	**P6160142.JPG**

Bridge No.	**6012**	Br. Name	**Montana Cr Pedestrian**	Date	**06/16/18**
Inspector		**Sara Manning / Mary McRae**		Frame	**14**
	Rust on bottom chord typical NE			File	**P6160151.JPG**

Bridge No.	**6012**	Br. Name	**Montana Cr Pedestrian**	Date	**06/16/18**
Inspector		**Sara Manning / Mary McRae**		Frame	**15**
	Typical rail panel			File	**P6160143.JPG**

Bridge No.	**6012**	Br. Name	**Montana Cr Pedestrian**	Date	**06/16/18**
Inspector		**Sara Manning / Mary McRae**		Frame	**16**
	Typical surface rust at rail panel ends			File	**P6160139.JPG**

Bridge No. **6012** Br. Name **Montana Cr Pedestrian** Date **06/16/18**
Inspector **Sara Manning / Mary McRae** Frame **17**
Typical bearing (NE US shown) File **P6160153.JPG**

Bridge No. **6012** Br. Name **Montana Cr Pedestrian** Date **06/16/18**
Inspector **Sara Manning / Mary McRae** Frame **18**
Spalls, exposed geotextile, erosion NE DS File **P6160146.JPG**

Bridge No. **6012** Br. Name **Montana Cr Pedestrian** Date **06/16/18**
Inspector **Sara Manning / Mary McRae** Frame **19**
Spalled asphalt, erosion gully NE US File **P6160162.JPG**

Bridge No. **6012** Br. Name **Montana Cr Pedestrian** Date **06/16/18**
Inspector **Sara Manning / Mary McRae** Frame **20**
Bump at FE joint File **P6160136.JPG**

Appendix F – Cost Estimates

The following section contains construction cost estimates in 2021 dollars for the various bridge types and their respective alignments.

Winner Creek Trail Bridge at Glacier Creek Feasibility Study

LOWER TRAIL CROSSING - 84-FOOT STEEL TRUSS BRIDGE

ITEM NO.	WORK DESCRIPTION (UNIT PRICE IN WORDS)	PAY UNIT	ESTIMATED QUANTITY	UNIT BID PRICE	TOTAL BID PRICE
A-1	Storm Water Pollution Prevention Plan (Type 3)	per LS	All Req'd	$ 15,000.00	$15,000.00
A-2	Trail Rehabilitation (ATV Access)	per LS	All Req'd	$ 50,000.00	$50,000.00
A-3	Earth Stripping and Stockpiling	per LS	All Req'd	$ 11,600.00	$11,600.00
A-4	Tree and Stump Removal	per LS	All Req'd	$ 49,900.00	$49,900.00
A-5	Drilling and Blasting	per LS	All Req'd	$ 360,000.00	$360,000.00
A-6	Hand Rail	per LS	All Req'd	$ 158,600.00	$158,600.00
A-7	Concrete Bridge Abutment	per LS	All Req'd	$ 17,000.00	$17,000.00
A-8	84' Steel Truss Bridge	per LS	All Req'd	$ 204,100.00	$204,100.00
A-9	Mobilization/Demobilization (Sky Crane Helicopter)	per LS	All Req'd	$ 500,000.00	$500,000.00
A-10	Set Bridge per Lift (Sky Crane Helicopter)	per EA	2	$ 4,100.00	$8,200.00
A-11	Material & Equipment Support (K-Max Helicopter)	per HR	12	$ 7,000.00	$84,000.00
A-12	Demobilization Equipment (K-Max Helicopter)	per HR	3	$ 7,000.00	$21,000.00
				Construction Total:	**$ 1,479,400.00**

Winner Creek Trail Bridge at Glacier Creek Feasibility Study

UPPER TRAIL CROSSING NO. 1 - 180-FOOT STEEL TRUSS BRIDGE

ITEM NO.	WORK DESCRIPTION (UNIT PRICE IN WORDS)	PAY UNIT	ESTIMATED QUANTITY	UNIT BID PRICE	TOTAL BID PRICE
A-1	Storm Water Pollution Prevention Plan (Type 3)	per LS	All Req'd	$ 5,000.00	$5,000.00
A-2	Trail Rehabilitation (ATV Access)	per LS	All Req'd	$ 50,000.00	$50,000.00
A-3	Concrete Bridge Abutment	per LS	All Req'd	$ 10,200.00	$10,200.00
A-4	180' Steel Truss Bridge	per LS	All Req'd	$ 645,700.00	$645,700.00
A-5	Mobilization/Demobilization (Sky Crane Helicopter)	per LS	All Req'd	$ 500,000.00	$500,000.00
A-6	Set Bridge (per lift)(Sky Crane Helicopter)	per EA	7	$ 4,100.00	$28,700.00
A-7	Material & Equipment Support (K-Max Helicopter)	per HR	8	$ 7,000.00	$56,000.00
A-8	Demobilization Equipment (K-Max Helicopter)	per HR	3	$ 7,000.00	$21,000.00
				Construction Total:	**$ 1,316,600.00**

Winner Creek Trail Bridge at Glacier Creek Feasibility Study

UPPER TRAIL CROSSING NO. 2 - 180-FOOT SUSPENSION BRIDGE

ITEM NO.	WORK DESCRIPTION (UNIT PRICE IN WORDS)	PAY UNIT	ESTIMATED QUANTITY	UNIT BID PRICE	TOTAL BID PRICE
A-1	Storm Water Pollution Prevention Plan (Type 3)	per LS	All Req'd	$ 5,000.00	$5,000.00
A-2	Trail Rehabilitation (ATV Access)	per LS	All Req'd	$ 50,000.00	$50,000.00
A-3	Concrete Bridge Abutment	per LS	All Req'd	$ 20,300.00	$20,300.00
A-4	Cable Suspension Bridge	per LS	All Req'd	$ 219,000.00	$219,000.00
A-5	Mobilization/Demobilization (Sky Crane Helicopter)	per LS	All Req'd	$ -	$0.00
A-6	Set Bridge (per hour)(K-Max Helicopter)	per HR	3	$ 7,000.00	$21,000.00
A-7	Material & Equipment Support (K-Max Helicopter)	per HR	14	$ 7,000.00	$98,000.00
A-8	Demobilization Equipment (K-Max Helicopter)	per HR	3	$ 7,000.00	$21,000.00
			Construction Total:		$ 434,300.00

Winner Creek Trail Bridge at Glacier Creek Feasibility Study

UPPER TRAIL CROSSING NO. 3 - RELOCATED BRIDGE

ITEM NO.	WORK DESCRIPTION (UNIT PRICE IN WORDS)	PAY UNIT	ESTIMATED QUANTITY	UNIT BID PRICE	TOTAL BID PRICE
A-1	Storm Water Pollution Prevention Plan (Type 3)	per LS	All Req'd	$ 5,000.00	$5,000.00
A-2	Trail Rehabilitation (ATV Access)	per LS	All Req'd	$ 50,000.00	$50,000.00
A-3	Concrete Bridge Abutment	per LS	All Req'd	$ 27,100.00	$27,100.00
A-4	Reconfigure Bridge Sections (20,000 lbs Max)	per LS	All Req'd	$ 40,700.00	$40,700.00
A-5	Paint Bridge Structure	per LS	All Req'd	$ 20,600.00	$20,600.00
A-6	Decking	per LS	All Req'd	$ 99,000.00	$99,000.00
A-7	Relocate Bridge (Labor & Equipment Only)	per LS	0	$ 308,580.00	$308,580.00
A-8	Mobilization/Demobilization (Sky Crane Helicopter)	per LS	All Req'd	$ 500,000.00	$500,000.00
A-9	Set Bridge (per lift)(Sky Crane Helicopter)	per EA	8	$ 4,100.00	$32,800.00
A-10	Material & Equipment Support (K-Max Helicopter)	per HR	18	$ 7,000.00	$126,000.00
A-11	Demobilization Equipment (K-Max Helicopter)	per HR	3	$ 7,000.00	$21,000.00
			Construction Total:		**$1,230,780.00**