TOWN OF SIMSBURY

DEPARTMENT OF PUBLIC WORKS 933 HOPMEADOW STREET
SIMSBURY, CONNECTICUT 06070 REQUEST FOR

PROPOSALS

FOR

ON-CALL PROFESSIONAL SERVICES:
SIMSBURY FLOWER BRIDGE REHABILITATION
DPW-RFP 2022-#03

The Town of Simsbury is soliciting proposals from qualified firms for professional engineering consultant services, for the design and preparation of bidding and construction documents for rehabilitation work of the Simsbury Flower Bridge. The bridge is a historic steel truss bridge structure over the Farmington river, off of Old Bridge Road and was originally built in 1892.

The bridge is used for pedestrians and is covered with plants/flowers during the summer months. The bridge does not carry vehicular traffic. The intent of the rehabilitation effort is to repair the structure to address all deterioration and to ensure that this bridge continues to function adequately for pedestrian and bicycle traffic.

Sealed Proposals will be accepted by, Amy Meriwether, Director of Finance, 933 Hopmeadow Street (Rt. 10/202), Simsbury, CT 06070 until 6/7/2022 at 10:00 a.m.

The complete RFP documents may be obtained electronically via the Town's web site at the following link: http://www.simsbury-ct.gov/finance/pages/public-bids-and-rfp. Proposal documents will not be mailed or faxed.

Each Respondent, by making their proposal, represents that they have read and understand the proposal documents. The right is reserved to reject any and all proposals not deemed to be in the best interests of the Town of Simsbury.

The right is reserved to reject any or all proposals or to waive defects in same if it be deemed in the best interest of the Town of Simsbury. The Town of Simsbury is an Equal Opportunity Employer.

Thomas J Roy, P.E.

Director of Public Works/ Town Engineer

REQUEST FOR PROPOSALS PROFESSIONAL SERVICES
SIMSBURY FLOWER BRIDGE REHABILITATION

TOWN OF SIMSBURY
Simsbury DPW-RFP 2022-#3

SCOPE OF WORK

Town of Simsbury (the "Town") is requesting proposals for professional engineering consultant services for the design and preparation of bidding and construction documents for repairs/ rehabilitation of the Simsbury Flower Bridge. The bridge is a historic built-up steel thru-truss bridge structure comprised of two Parker trusses carrying Old Drake Hill road bridge over the Farmington river and was originally built in 1892. A major rehabilitation was performed in 1977 and additional rehabilitation was performed in 1993 when the bridge was closed to vehicular traffic and converted to a pedestrian bridge. The bridge is a Historic Bridge and is listed on the National Register of Historic Places. The scope of work includes assistance with bidding and contract award, and construction administrative services.

The Intent of the Rehabilitation is too repair deteriorated steel members, replace damaged timber planking, perform bearing assembly repair, and paint the steel structure, as outlined in the rehabilitation study prepared by GMZ in 2019. A load rating report, and a rehabilitation study report were preformed and are provided as Exhibits A and B respectively. Professional engineering consultant services are intended to be comprehensive and include all aspects required to generate drawings, bid documents, technical specifications, permits, and construction cost estimates. Other Services may be required that are deemed to be in the best interest of the Town of Simsbury.

Scope of Services

The following scope of services is anticipated, but not limited to, for the project:

Phase I - Design & Construction Documents

- Project Coordination including meetings with Town representatives and preparation of minutes.
- Development of Preliminary Plans & Construction Details, to include 30%, 90% and final drawings.
- Development of Preliminary Cost Estimate.
- Development of Preliminary Specifications.
- Submission of Preliminary Plans, Estimate, and Specifications to Town for review, and scheduling a review meeting.

- Preparation of permits for Town submission as may be required.
- Development of Construction Plans and Specifications with submission to Town and scheduling a review meeting.

Phase II - Bidding Assistance

- Development of Bidding Documents.
- Notify interested bidders that might meet the Town's requirements
- Receive Bidder questions & issue Addenda as may be required.
- Review Bids & Provide recommendation of contract award to the Town.
- Prepare Construction Contract Documents for execution by Town and successful bidder.

Phase III - Construction Engineering (CE) Services

- Schedule Preconstruction Meeting with Town & Contractor.
- Review of submittals, shop drawings, certificates of compliance.
- Field observation of work in progress with reports.
- Coordination with testing agencies.
- Review and coordination of Contractor's Applications for Payment.

PROPOSAL & SELECTION

Interested firms are requested to submit three (3) copies of qualification data. The interested firm should also submit a detailed fee schedule, in a separate sealed envelope, to Amy Meriwether, Director of Finance, Town of Simsbury, 933Hopmeadow St, Simsbury, CT 06070 by 10:00 a.m. 6/7/2022 at 10:00am.

Each RFQ/RFP response / submission shall be delivered in a sealed envelope or package clearly identified as **"SIMSBURY FLOWER BRIDGE REHABILITATION, TOWN OF SIMSBURY, Simsbury DPW-RFP 2022-#03".** Fee Proposals should be submitted in a separately sealed envelope or package clearly identified as **"Fee Proposal: SIMSBURY FLOWER BRIDGE REHABILITATION, TOWN OF SIMSBURY, Simsbury DPW-RFP 2022-#03"**.

RESPONSE FORMAT

Please provide the following information:

Company Profile:

A company profile, including the firm name, business address, telephone number, year established (include former firm names and year(s) established, if applicable), type of Township, and parent company, if any

Experience:

Provide information indicative of experience on other projects (please limit to five projects) of similar complexity that document successful and reliable experience in past performance within the last seven (7) years, as is related to this proposal. Identify local governmental clients for whom similar services have been provided, including name of client, client contact person, description of services performed. Provide resumes of key staff.

Personnel:

Provide an organizational chart, short form resumes, and summary of staff qualifications. Demonstrate current capacity and current expertise in bridge work. Respondent shall document knowledge and experience of personnel in bridge engineering, bridge rehabilitation, and any relevant expertise.

Conflicts:

All Respondents must certify that neither the Respondent, nor any employee thereof, has any conflict of interest, either direct or indirect, in connection with the services sought herein, pursuant to Federal or State law. If so, state the name and address of the other contracting party and reason.

Technical Approach:

Provide a description of the Proposer's approach to the project, including implementation of the RFP Scope of Services, Estimated schedule for work completion, estimated staff hours for the various tasks, and any other relevant information. List any permitting that will be required and any alternate or innovative approaches that can be taken on this project.

References:

The respondent shall provide references for five (5) bridge rehabilitation projects of similar size performed over the past seven (7) years. Include the client name, project cost, and a brief summary of work, along with name, address, and phone number of a responsible contact person.

Capacity/Schedule:
Capacity to perform services timely for the Town is critical and could be impacted by other obligations firms may have in the general area. Provide a typical schedule outlining the numbers of staff you would assign to a project and their responsibilities.

Fee:
Include fee table divided by task, include design phase tasks, including for 30%, 90%, and final design, bidding phase tasks, and construction phase tasks. Fees should be provided as hourly not to exceed. No extra payment will be provided for mileage. Firm should include all tasks that they deem are necessary to provide the services requested in this proposal, even if they are not specifically called out in this document.

QUESTIONS:

Any questions about this project should be directed to Mr. Thomas J. Roy, PE, Director of Public Works/ Town Engineer, troy@simsbury-ct.gov, or mailed to Town of Simsbury, Public Works, 933 Hopmeadow St., Simsbury, CT 06070. To receive consideration, such questions must be received at least five (5) business days before the established submission date. No oral interpretations shall be made to any respondent as to the meaning of any of the documents. Every request for an interpretation shall be made in writing.

The Town will respond to all appropriate questions received via an addendum available to all prospective consultants. Such addenda will become part of this Request for Proposals and the resulting contract. At least three (3) days prior to the receipt of proposals, the Town will post a copy of any addenda to its website located at:
https://www.simsbury-ct.gov/finance/pages/public-bids-and-rfp
It shall be the responsibility of each prospective proposer to determine whether addenda have been issued, and if so, to download copies directly from the Town's website.

SELECTION:
The Town of Simsbury will review all proposals to determine the firm that can best meet the needs of the Town for the rehabilitation of the Simsbury Flower Bridge. This will include consideration of fee, company history, references and any other pertinent information

TAX EXEMPTIONS:
The consultant shall be aware that the Town of Simsbury is exempt from Federal Excise Taxes and Connecticut Sales and Use Taxes. Appropriate tax-exempt forms will be provided to the successful consultants(s) as part of the contract award process

INSURANCE REQUIREMENTS:

The firm must carry insurance under which the Town is named as an additional insured, as follows:

Such insurance must be by insurance companies licensed to write such insurance in Connecticut against the following risks with the following minimum amounts and minimum durations.

A. Workman's Compensation, as required by State Statute & $100,000 employers liability limit.

B. Public Liability, Bodily Injury Liability and Property Damage Liability as follows:

Injury or death of one person:	$2,000,000
Injury to more than one person in a single accident:	$1,000,000
Property damage in one accident:	$1,000,000
Property damage in all accidents:	$2,000,000

C. Automobile and Truck (Vehicular) Public Liability, Bodily Injury Liability and Property Damage Liability as follows:

Injury or death of one person:	$1,000,000
Injury to more than one person in a single accident:	$1,000,000
Property damage in one accident:	$1,000,000
Property damage in all accidents:	$1,000,000

Insurance under B, and C above must provide for a 30-day notice to the Town of cancellation/or restrictive amendment.

Insurance under B and C above must be for the whole duration of the contract and for twelve (12) months after acceptance of the project by the Town.

Subcontractors must carry A, B, and C in the same amounts as above for the duration of the project and until acceptance by the Town.

Certificates of insurance must be submitted to the Director of Public Works/ Town Engineer prior to the signing of the contract and within ten days of notification of award of contract. Should any insurance expire or be terminated during the period in which the same is required by this contract, the Director of Public Works/ Town Engineer shall be notified and such expired or terminated insurance must be replaced with new insurance and a new certificate furnished to the Director of Public Works/ Town Engineer.

Failure to provide the required insurance and certificates may, at the option of the Town, be held to be a willful and substantial breach of this contract.

W-9 FORM

The successful consultant must provide the Town of Simsbury with a completed W-9 Form prior to commencing work.

Fee Schedule:

Proposal must include an itemized fee schedule that includes prices for all phases of the project, any additional services the consultant deems necessary to complete project, and your staff classifications and their hourly rates. The hourly labor rates shall include all applicable overhead and profit. Hourly rates will only be used when the consultant is asked to preform work outside the agreed upon scope. Overtime hours will be paid at the same rate as regular time hours. All normal expenses shall be absorbed in prices, including lodging, meals, transportation, and per diem. Special costs clearly outside the anticipated scope of services, with prior approval from the Town, may be billed to the Town at cost without mark- up. Proposer may also include additional, optional positions and services.

SELECTION PROCESS

The materials submitted by the Proposers will be reviewed and ranked by Town Staff and will be based upon a Qualifications Based Selection (QBS) format.

The QBS process will incorporate without limitation the following criteria:

- Relevant project experience, including bridge design.
- Experience with bridge rehabilitation in an environmentally sensitive area.
- Experience with providing innovative solutions and alternatives.
- Experience working with government agencies that may have jurisdiction over the Project.
- Experience working with the construction process and procedures.
- Ability to comply with Project requirements.
- Experience, skill-set and demonstrated leadership of proposed Project team.
- Quality of proposal.

A short list of a single, or multiple, firms will be developed based of the qualifications and project approach, as listed above. The fee envelope(s) for the short listed firm or firms will then be opened to determine the best value for the Town.

END

Exhibit A

Simsbury Flower Bridge
Load Rating Report

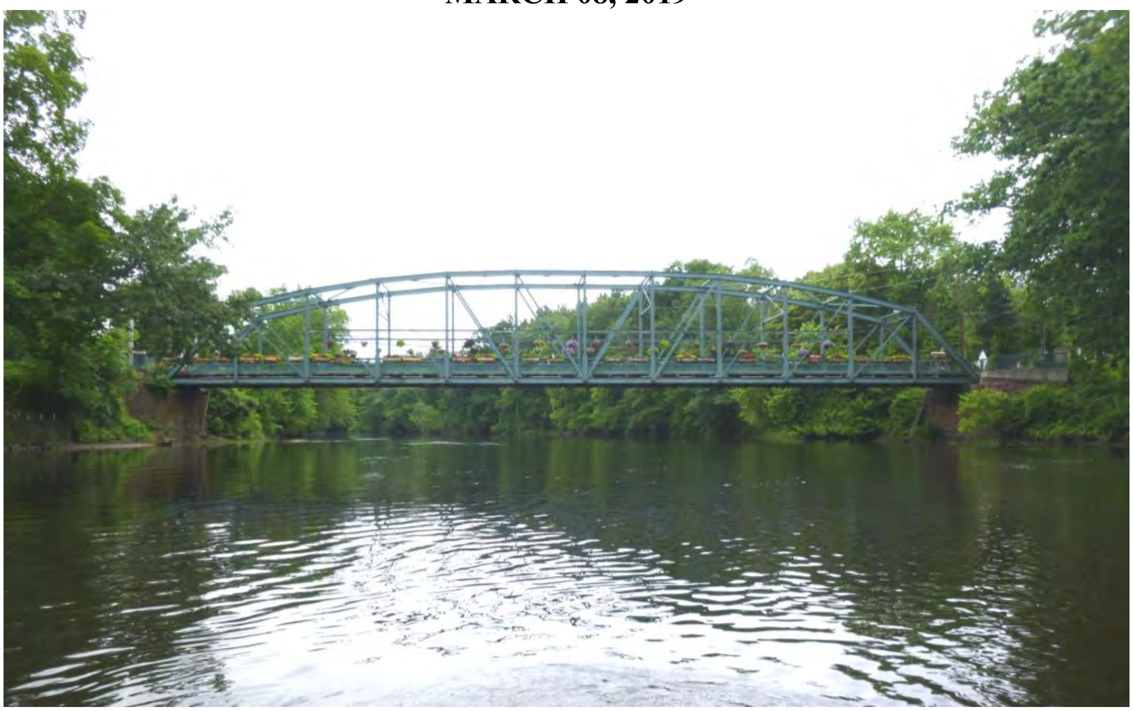
BRIDGE REHABILITATION STUDY REPORT

BRIDGE NO. 03984

OLD DRAKE HILL ROAD BRIDGE (FLOWER BRIDGE)
OVER
FARMINGTON RIVER

SIMSBURY, CONNECTICUT

MARCH 08, 2019



Prepared By:

115 GLASTONBURY BLVD.
GLASTONBURY, CT 06033

Prepared For:

TOWN OF SIMSBURY
DEPARTMENT OF ENGINEERING
SIMSBURY, CT

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EXECUTIVE SUMMARY

GM2 Associates, Inc. (GM2) has been retained by the Town of Simsbury to design and prepare a rehabilitation evaluation for Bridge No. 03984 carrying Old Drake Hill Road Bridge (Flower bridge) over Farmington River, Simsbury, Connecticut. This assignment is part of the on-call Task Based Bridge Engineering Services Contract between GM2 and Town of Simsbury.

This Rehabilitation Study Report (RSR) describes the findings of a detailed evaluation of the condition of the bridge and presents recommendations for rehabilitation to ensure its structural and functional adequacy, as well as to extend its service life. Due to the historic nature of the bridge and the unique structure type, evaluation of multiple alternates for rehabilitation was not considered.

EXISTING CONDITIONS

The bridge is a historic built-up steel thru-truss bridge structure comprised of two parker trusses carrying Old Drake Hill road bridge over the Farmington river and was originally built in 1892. A major rehabilitation was performed in 1977 and additional rehabilitation was performed in 1993 when the bridge was closed to vehicular traffic and converted to a pedestrian bridge. The bridge is a Historic Bridge and is listed on the National Register of Historic Places.

In accordance with the current bridge inspection report dated June 27th, 2017, the bridge is in a fair condition (Rated 5). The superstructure (steel truss) is in fair condition due to some deterioration of steel that exists at multiple locations. The substructures (stone masonry abutments) are in good condition (Rated 7). The deck is in satisfactory condition (Rated 6).

SCOPE OF REHABILITATION WORK

Since the bridge does not carry vehicular traffic, the intent of the rehabilitation effort is to repair the structure to address all deterioration and to ensure that this bridge continues to function adequately for pedestrian and bicycle traffic. It is not the intent of the rehabilitation to strengthen the bridge to make it compliant with current AASHTO pedestrian design loads.

All rehabilitation work will need to be performed in accordance with the ConnDOT Bridge Design Manual and AASHTO LRFD Bridge Design Specifications and keeping in mind the historic nature of the bridge. The Merritt Parkway Bridge Restoration Guide should be used as a guide to the process for rehabilitating a historic bridge.

Based upon a comprehensive review of the bridge inspection report, observations from the field visit and the load rating report for Bridge No. 03984, two rehabilitation alternatives have been evaluated as part of this RSR: Alt. 1 – Minor Rehabilitation and Alt. 2 – Major Rehabilitation. The scope of recommended rehabilitation and cost estimate for each Alternative are noted below. Scope items noted in italics are common to both alternatives.

Alt. 1 – Minor Rehabilitation

Scope of Recommended repairs:

- *Spot repairs for steel deterioration*
- *Repair damaged timber planks*
- *Replace Joint seal at abutments*
- *Perform bearing assembly repairs (requires jacking of the bridge)*
- *Spot paint structural steel*
- *Install lateral restraint at Bearings*
- *Reset brick pavers*

A minor rehabilitation will extend the service life of the bridge by 10-15 years and is estimated to cost **$ 0.5 Million**.

Alt. 2 – Major Rehabilitation

Scope of Recommended repairs:

- *Spot repairs for steel deterioration*
- *Repair damaged timber planks*
- *Replace Joint seal at abutments*
- *Perform bearing assembly repairs (requires jacking of the bridge)*
- *Abrasive Blast Clean and Paint entire bridge*
- *Install lateral restraint at Bearings*
- *Reset brick pavers*
- Remove and replace water piping system for flower pots

A major rehabilitation will extend the service life of the bridge by 25-30 years and is estimated to cost **$ 1.65 Million**.

Roadway and Drainage

Since the rehabilitation work for both the alternatives is confined to the bridge, no roadway/drainage work is anticipated to be included. No roadway/approach improvements are necessary.

Maintenance and Protection of Traffic

The bridge needs to be closed for pedestrian/ bicycle traffic for the duration of construction for both the alternatives.

Permits

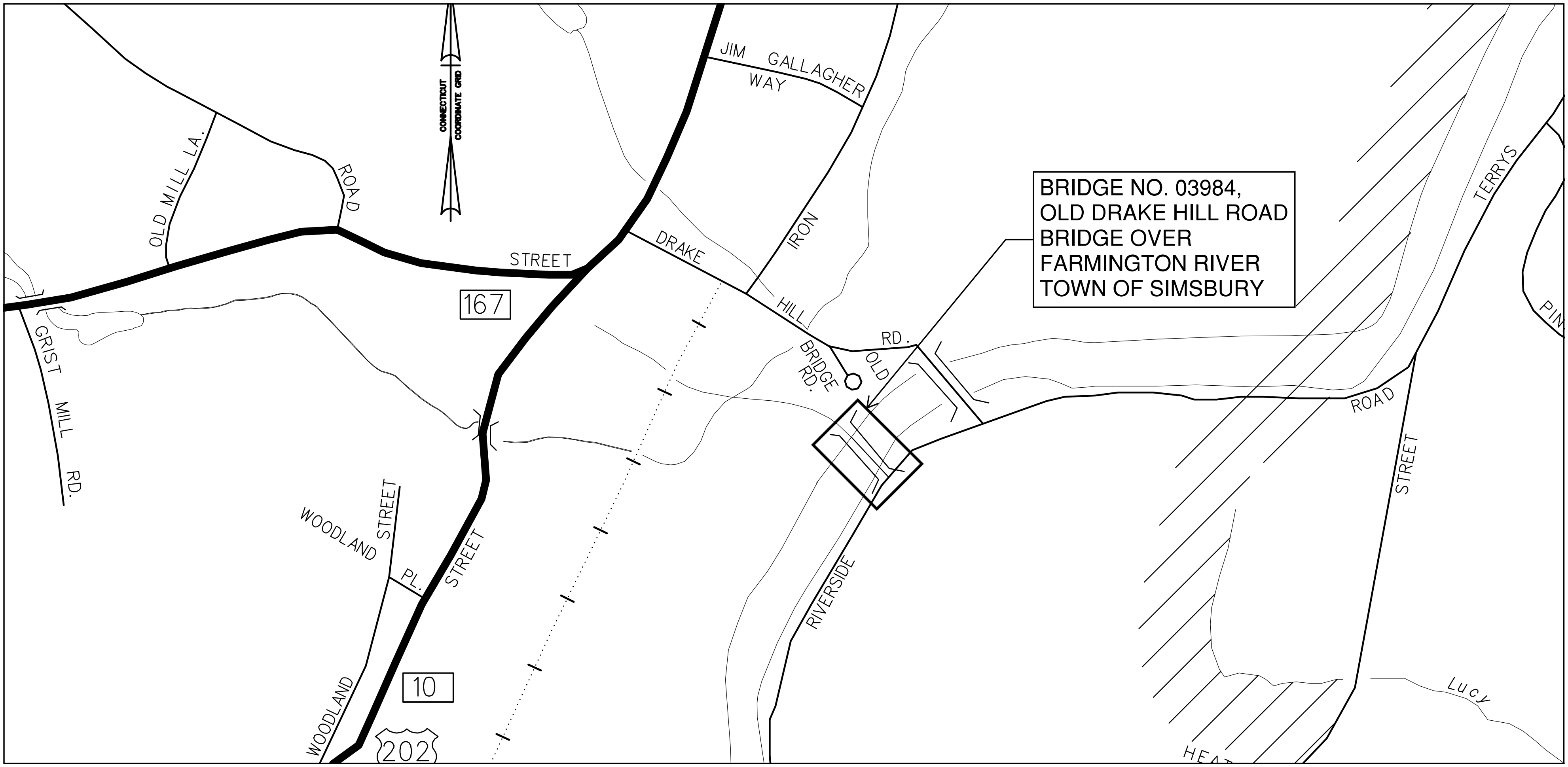
At a minimum, the following Permit Coordination will be needed.

- Inland Wetlands/Watercourses Permits
- Army Corps of Engineers/Water Quality
- DEEP Fisheries Coordination

- National Diversity Database for endangered species (NDDB)

Depending on the funding source for construction, if there is any ConnDOT involvement, the following additional permits may be necessary

- Flood Management Certification



PROJECT TITLE:

REHABILITATION OF BRIDGE NO. 03984
OLD DRAKE HILL ROAD BRIDGE (FLOWER BRIDGE)
OVER FARMINGTON RIVER
SIMSBURY, CT

GM2 ASSOCIATES, INC.
115 GLASTONBURY BLVD.
GLASTONBURY, CT 06033

DRAWING TITLE:

LOCATION PLAN

PROJECT NO.	**TBD**
SHEET NO.	**LOC**
DATE	**03/08/19**

BRIDGE DESCRIPTION

STRUCTURE OVERVIEW

The bridge is a historic built-up steel thru-truss bridge structure comprised of two parker trusses carrying old drake hill road bridge over the Farmington river and was originally built in 1892. The bridge was originally designed to carry vehicular traffic. A major rehabilitation was performed in 1977 and additional rehabilitation was performed in 1994 when the bridge was closed to vehicular traffic and converted to a pedestrian bridge. The bridge is a Historic Bridge and is listed on the National Register of Historic Places. The bridge has a total span length of 185′-0″, with an out-to-out width is 18′-6″. The bridge is decorated with planters that accommodate plantings on a seasonal basis. The bridge also has an irrigation system that is used to irrigate the planters.

Deck

The original bridge decking was replaced as part of the 1994 rehabilitation to a 3”x8” timber decking attached to a pressure treated 9”x9” nailer bolted to the floor beams.

Steel Truss

The bridge is a built-up steel thru-truss structure that is comprised of the following components:

Floor beams

Steel floor beams support the timber deck. The floor beams are connected to the bottom chord of the truss.

Truss Bottom Chord, Verticals, Top Chords & Diagonals

The truss bottom chord, verticals and diagonals are comprised of laced built-up riveted members comprised of plates and angles. Diagonal elements are Tension Tie rods. Vertical struts, diagonals are connected at the truss panel points at the top chord using the original pins. Vertical struts, diagonals are connected at the truss panel points at the bottom chord using a welded “gusset plate” type system that was a retrofit intended to bypass the connections to the pins due to severe deterioration. The top and bottom chords are connected via a pin at bearing location.

Bottom Lateral Bracings

The original bottom lateral bracing consisting of steel angle members were connected to the bottom chord. During the 1994 rehabilitation the bottom lateral bracing framing was replaced with new angles connected to the floor beams and lateral members connecting the new angles. The top lateral bracing consists of the original steel members connected to the top chord of the trusses.

Bearings

The original roller assembly bearings have been replaced with steel laminated elastomeric bearing pads during the 1994 rehabilitation.

Bridge Railing

The original bridge railings were replaced as part of the 1994 rehabilitation. The current single metal hand rail system is installed on top of the deck in front of the truss members.

Substructure

Existing substructures are stone masonry abutments and wingwalls with a concrete backwall. During the 1974 rehabilitation the top course of stone has been replaced with a concrete cap drilled and grouted into the stone masonry.

ROADWAY

The approaches to the pedestrian bridge are in line with the bridge and have bollards to restrict the traffic on the bridge to pedestrian and bicycle traffic.

TRAFFIC

Bridge No. 03984 does not carry any vehicular traffic and is open to pedestrian traffic only.

HYDRAULICS

The bridge is located over the Farmington River and is located in a designated floodway. The bridge appears to be missing in the FEMA 100 year flood profiles.

It is the GM2 understanding that this Bridge Preservation Project will not involve the replacement or the enhancement of hydraulic capacity of the structures. There will only be maintenance level repairs provided to lengthen the life span of the structure. With this defined level of scope, there is no need to perform detailed hydrologic, hydraulic or scour analyses for the structure.

GM2 will provide a hydrologic assessment and make a general evaluation of its hydraulic capacity. A hydrologic comparison will be made between FEMA FIS flows and USGS StreamStats utility flows. The more appropriate 1% recurrence interval (Q100) design storm flow rate will be made in accordance with the DOT Drainage Manual and Consulting Engineers General Memorandum 07-06.

For bridge crossings with minor scour issues, revetment design velocities may be approximated using the Continuity Equation. The FHWA Hydraulic Engineering Circular No. 23, 3rd Edition (HEC-23) will be used to establish the most appropriate mitigation measure to be used when needed.
Temporary Facilities water handling elevations shall be based on the visual/vegetative determination of Ordinary High Water (OHW). Set the top of the water handling diversion elements at one foot above OHW.

DRAINAGE

There are no drainage structures on the bridge. The decking has a gap of 1/8" between timber members allowing for drainage directly into the river.

UTILITIES

There are no utilities in the vicinity of the bridge.

R.O.W.

There does not appear to be any R.O.W. concerns in the vicinity of the bridge.

EXISTING CONDITIONS

GM2 performed an in-depth bridge safety inspection in June 2017 as part of the scope.

The information presented in this section of the Report are summarized from the 2017 bridge inspection report performed by GM2 and supplemented by observations made during the site visits. All condition ratings are as per the 2017 bridge inspection report, unless specifically noted otherwise.

DECK

The timber deck is in satisfactory condition (6). The timber deck planks show random signs of splits and checks.

BRIDGE RAILING

The metal bridge ornamental railings are in good condition (7) with isolated areas of peeling paint.

EXPANSION JOINTS

The expansion joints are in satisfactory condition (6). There is joint sealant material between the timber deck ends and concrete headers at both abutments with deteriorating joint sealant material at random locations.

STEEL TRUSS

Overall, the steel open truss is in fair condition (5). There are numerous locations that show indication of crevice corrosion due to pack rust between built up steel elements that have caused some plates to bow.

Bearing Devices

Bearing devices are in poor condition (4). Gusset plates at bearings exhibit section loss, with thick laminar rust between truss members, pin and gusset plate.

Floor beams

Floor beams are in fair condition (5). In general, floor beams exhibit section loss to the top and bottom flanges and webs.

Truss Portal

Truss Portal is in good condition (7). In general, portals exhibit peeling paint at random locations.

Truss Bracing

Truss bracings are in fair condition (5). In general, floor beams exhibit section loss to the top and bottom flanges and webs.

Miscellaneous

Rivets are in satisfactory condition (6). Isolated rivet heads have up to 50% head loss. Random rivets exhibit peeling paint and light to moderate rust.

Paint is rated as good condition (7). Less than 10% of the painted surfaces are peeling with light to moderate rust.

ABUTMENTS & WINGWALLS

Overall substructures are in good condition (7). There are random isolated stones with full height cracks. There are random voids and hairline cracks with and without efflorescence in the mortar between the stones. There is moderate to heavy growth of vegetation atop at bearings and along the wingwalls.

APPROACH CONDITION

Approach metal railings are in good condition (7). Metal rails at all four corners exhibit isolated areas of peeling paint with light to moderate rust.

Approach pavement is rated as satisfactory condition (6). Stone pavers have minor cracks between them and have isolated areas of depression in the east approach.

LOAD RATING

GM2 performed a load rating analysis for the bridge and was evaluated for a 90psf pedestrian loading and a H10 vehicle in compliance with AASHTO Guide Specification for the Design of Pedestrian Bridges. Load ratings were performed for the existing condition (including deterioration from 2017 inspection report). Refer to the load rating report for more detailed information.

The load rating was controlled by the bearing assembly pin at the northwest bearing location, which appears to have a missing plate that significantly reduces the load bearing capacity of the steel pin at the bearing. The controlling load rating factor for this pin is as follows:

- Pedestrian Design Load (90 psf) – **Rating Factor = 0.11**
    - Controlling Element = NorthWest Bearing Pin
    - This translates to a restriction of about 150 people uniformly distributed on the bridge
- H10 Vehicular Load – **Rating Factor = 0.09**
    - Controlling Element = Timber Decking
    - The bridge should be closed to vehicular traffic due to the low rating factor.

RECOMMENDATIONS FOR REHABILITATION

HYDROLOGY/HYDRAULICS

Since the project is a bridge preservation/rehabilitation project, it is assumed that enhancement of the hydraulic capacity of the structure is not necessary. There will only be maintenance level repairs provided to lengthen the life span of the structure. With this defined level of scope, there is no need to perform detailed hydrologic, hydraulic or scour analyses for the structure.
It is recommended that a hydrologic assessment and a general evaluation of its hydraulic capacity be made during final design to evaluate temporary flows during construction and specify criteria for access and containment that will not impede routine flows.

There is anecdotal evidence that the Farmington River routinely rises to the level where it is in close proximity to the bottom chord of the truss. The bridge has been in place for over a century and withstood numerous storm events. It is expected that it will continue to do so. While strengthening the bridge to withstand lateral loads from stream flow pressure may not be practical, it is recommended that restraints be added to the abutments to ensure that the bridge stays in place during storm events. This retrofit can be done relatively economically.

SCOUR

Given the scope of the project and that there are no changes in hydraulic capacity or hydraulic opening in any way, there is no need to perform a hydraulic and scour analysis for the bridge. The bridge has been in place since the 1890's and withstood many major storm events with no apparent scour.

However, an underwater inspection should be performed during the next design phase to identify if there any potential repairs or scour mitigation is required. This RSR assumes that it is not necessary.

PERMITS

The following permits are currently anticipated and require additional information which will need to be provided during the final design phase.

- Local Inland Wetlands/Watercourses Permits
- U.S. Army Corps of Engineers

Coordination will also be required with the following
- DEEP Fisheries

The necessity of a Flood Management Certification is not anticipated as there is no CTDOT oversight for the project. This will need to be evaluated during the next phase of design based on funding source for project construction.

Some temporary wetland impact and water handling may be required to provide construction access and a dry working area to perform substructure repairs.

ENVIRONMENTAL

There are some potential environmental concerns for this project that will need to be further coordinated and incorporated into the design of the rehabilitation alternative:

- Coordination with the State Historic Preservation Office (SHPO) will be necessary to ensure we can receive a Conditional No Adverse Effect determination.
- There are State and Federally listed threatened and endangered species present in the area as per the National Diversity Database (NDDB) Map. Coordination with DEEP will be necessary during final design.

PUBLIC OUTREACH

Since the bridge is a popular destination and the location for numerous events, weddings etc. during the course of the year, a robust public outreach program is recommended to ensure the public are aware of the closure of the bridge and/or access limitations that construction will entail. This coordination should start as early as possible and continue right until construction is complete.

ROADWAY

No roadway work anticipated

TRAFFIC

The bridge will need to be closed to pedestrian and bicycle traffic during construction.

HISTORIC

Bridge No. 03984 has been identified as a Historic Bridge and as such will require special considerations be followed during its rehabilitations. It is recommended that all rehabilitation work be performed in accordance with the Merritt Parkway Bridge Restoration Guide.

The following analysis and testing is recommended during final design as indicated in the restoration guide:

- Concrete Testing in order to match repairs to the historic concrete
- Paint Analysis in order to determine the original color of the paint and to assess the condition of painted metal surfaces.
- Metal Analysis to identify the original steel used and to ensure that any retrofits are electrochemically compatible with the existing steel.

GEOTECHNICAL

No geotechnical work anticipated since no changes to the substructure are being proposed.

ILLUMINATION AND UTILITIES

There are no utility or illumination related issues that will be encountered based on the rehabilitation recommendations presented in this report.

DRAINAGE

Since there is are no drainage scupper on the bridge and the timber decking and open truss system allows for self-drainage there will be no drainage related concerns to address on the bridge.

RIGHTS OF WAY

There are anticipated to be no right-of-way/property impacts based on the rehabilitation recommendations presented in this report.

STRUCTURAL

In this RSR, the load rating report was used as a guide to identify the areas of the bridge that are in need of structural repair. Only locations of the bridge that rated inadequately needs to be considered for repair. The repair should also be practical as well. Since the bridge is a historic bridge, it is recommended that the primary objective of any structural repairs be maximize the load carrying capacity and prolong the service life of the bridge.

Steel Repairs

In general, the steel elements of the bridge are not in need of strengthening except as noted below:

- Bearing pins – The most critical element is the deficient bearing support at the northwest bearing. to ensure that the load carrying capacity of the bridge can be restored. Strengthening the bearings will require jacking up of the bridge.

Based on the load rating, the welded connection between the verticals and bottom chord that was performed during the 1994 rehabilitation is undersized by a minor amount (approx. 1" of weld). It is recommended that this not be addressed during the rehabilitation since the increase in capacity of the bridge by strengthening this is connection is not necessary when considering the typical use of the bridge.

In addition, there are some routine steel repairs (not related to strengthening) that can be undertaken to ensure that the life of the rehabilitation/bridge can be maximized

- Replace deteriorated rivets
- Identify and correct perforations/contact surfaces etc. where water can "pond" and exacerbate future deterioration.
- Address Crevice corrosion (rust between the contact surfaces of two plates/built-up members) where present.
- Paint areas that have deteriorating paint. The paint system can be one of the following:
    - A 3-coat system, consisting of an epoxy mastic prime coat, an epoxy intermediate coat and a polyurethane finish coat.

- A 2-coat paint system consisting of a penetrating sealer tie coat and a polyurethane finish coat.
- A High Ratio Co-Polymerized Calcium Sulfonate (HRCSA) coating system. This will address crevice corrosion. While this product has been used in many states for similar bridges, it is a relatively new product in CT.

The 1994 rehabilitation plans indicate that a 3-coat system was used to paint the bridge after abrasive blasting the steel to white metal. The typical life span of the paint system is 25 years in less aggressive, salt containing environments (FHWA Steel Bridge Design Handbook – Corrosion Protection of Steel Bridges). The bridge is already at the end of the typical life span of the paint system and there are numerous locations of failed paint noted in the inspection report. Utilizing an approved polyurethane caulk system to prevent water infiltration into crevice connections is recommended.

Deck Expansion Joints

Replace existing joints seal at abutments

Deck

Replace damaged timber deck elements

Construction Access and Staging Area

The bridge can be accessed from the northwest and southeast approaches to facilitate construction. Rigging will be needed to access the truss over the Farmington River.

Design Criteria

The original design load for the bridge is unknown. The 1990 rehabilitation design notes the design load as 100 psf live load. However, the load rating analysis performed in Feb 2019 has indicated that the live load capacity is less than 100 psf. Since the bridge is a historic bridge, it is recommended that the primary objective of the design be to correct all critical deficiencies to the extent possible with minimal modifications to the structure with the intent of maximizing its load carrying capacity and prolonging its service life. It is not recommended that the bridge be designed to conform to current design codes as that will likely require substantial strengthening that may not even be feasible. Consequently, that was not evaluated in this RSR. The following documents should be used to perform the structural design:

- AASHTO LRFD Pedestrian Bridge Design guidelines.
- ConnDOT Bridge Design Manual (BDM)
- Merritt Parkway Bridge Restoration Guide
- AASHTO Guidelines for Historic Bridge Rehabilitation and Replacement
- AASHTO The Manual for Bridge Evaluation (MBE)

Due to the anticipated scope of the rehabilitation effort, it is not practical to design the bridge to meet all the load requirements of AASHTO LRFD. It is recommended that only the following load cases (and resulting load combinations) be evaluated during design:

- DC: Dead loads of components and attachments
- DW: Dead loads of wearing surface and utilities
- LL: Pedestrian live load

Even though Wind, Temperature, Seismic and Stream Flow Pressures are other common load cases that should ideally be evaluated based on current codes, it is likely not practical to retrofit the bridge to address any shortcomings from these load cases. The bridge was likely not designed for these load cases to begin with.

For the purposes of this RSR, cost estimates were developed for two Alternatives with the intent of providing the Town of Simsbury an economical option as well as a full fledged option to rehabilitate the bridge. The two alternatives are as follows:

- Alternative 1: Minor Rehabilitation
  - 10 to 15 year additional service life
  - Spot Painting of steel
  - Economical Option
- Alternative 2: Major Rehabilitation
  - 25 to 30 year additional service life
  - Blast clean to white metal and paint entire bridge
  - More thorough option

The scope of preservation under both these alternatives is essentially the same, except for the scope of the painting. Under the minor rehabilitation, the intent is to only do mechanical cleaning (SSPC SP-3 level) and spot painting in areas where the paint is deteriorated. Under the major rehabilitation the existing paint will be abrasive blasted to obtain a near white (SSPC-SP10 level) finish and a new paint system will be applied. Either the standard CTDOT 3 coat system can be applied or something innovative like the HRCSA coating system can be considered. The cost estimates assumes that the ConnDOT 3-coat system is being utilized.

APPENDICES

Appendix A - Photographs

Bridge No.:	03984	Feature Carried:	Old Drake Hill Rd
Town:	Simsbury, CT	Feature Crossed:	Farmington River

Photo # 1: Bridge from West approach.

Photo # 2: Bridge from East approach.

Bridge No.:	03984	**Feature Carried:**	Old Drake Hill Rd
Town:	Simsbury, CT	**Feature Crossed:**	Farmington River

Photo # 3: Upstream elevation.

Photo # 4: Downstream elevation.

Bridge No.:	03984	**Feature Carried:**	Old Drake Hill Rd
Town:	Simsbury, CT	**Feature Crossed:**	Farmington River

Photo # 5: Railing and flower pots on the bridge.

Photo # 6: Typical bearing elevation.

Appendix B - Cost Estimates

Town of Simsbury
Rehabilitation of Bridge No. 03984
Old Drake Hill Road Bridge (Flower Bridge) over Farmington River

Project No.: TBD
Date: 2/20/2019
By: DK
Checked By: JG

COST ESTIMATE FOR MINOR REHABILITATION

Rehabilitation Measures

- Spot repairs for steel deterioration
- Repair damaged timber planks
- Replace Joint seal at abutments
- Perform bearing assembly repairs (requires jacking of the bridge)
- Spot paint structural steel
- Install lateral restraint at Bearings
- Reset brick pavers

Description	Unit	Estimated Quantity	Unit Cost	Total Cost
SECTION A. - STRUCTURE ITEMS				
REMOVE AND RESET TIMBER DECKING	S.F.	550	$ 10.00	$ 5,500.00
CONSTRUCTION ACCESS	L.S.	1	$ 35,000.00	$ 35,000.00
LOCALIZED PAINT REMOVAL AND FIELD PAINTING OF STRUCTURAL STEEL	S.F.	605	$ 150.00	$ 90,750.00
ABRASIVE BLAST CLEANING AND FIELD PAINTING OF EXISTING STEEL	S.F.	300	$ 60.00	$ 18,000.00
CLASS 1 CONTAINMENT SYSTEM	L.S.	1	$ 10,000.00	$ 10,000.00
STRUCTURAL STEEL REPAIRS	CWT.	4	$ 10,000.00	$ 40,000.00
REPLACE REMOVED OR MISSING RIVETS AND BOLTS WITH HIGH STRENGTH BOLTS	EA.	50	$ 100.00	$ 5,000.00
REPAIR BEARING PIN SUPPORT ASSEMBLY	EA.	4	$ 5,000.00	$ 20,000.00
JACKING FOR BEARINGS REPAIRS	EA.	2	$ 25,000.00	$ 50,000.00
LATERAL RESTRAINTS AT BEARINGS	EA.	4	$ 2,000.00	$ 8,000.00
REPLACE JOINT SEAL	L.F.	30	$ 50.00	$ 1,500.00
REMOVE AND RESET BRICK PAVERS	S.F.	30	$ 50.00	$ 1,500.00
		A. STRUCTURE ITEMS SUBTOTAL		**$285,250**
SECTION B. - ENVIRONMENTAL COMPLIANCE ITEMS				
LEAD COMPLIANCE FOR MISCELLANEOUS EXTERIOR TASKS	L.S.	1	$ 10,000.00	$ 10,000.00
DISPOSAL OF LEAD DEBRIS	TON	3	$ 2,000.00	$ 6,000.00
		B. ENVIRONMENTAL ITEMS SUBTOTAL		**$ 10,000.00**
SECTION C. - MINOR ITEMS/UNIDENTIFIED COSTS			**% of cost**	
MINOR ITEMS	L.S.		15.0%	$ 44,287.50
SECTION D. - LUMP SUM ITEMS			**% of cost**	
MOBILIZATION	L.S.		5.0%	$ 16,976.88
		PROBABLE CONSTRUCTION COST ESTIMATE		**$ 356,514.38**
ENGINEERING PERCENTAGES (Incidentals & Contingencies)			**% of cost**	
INCIDENTALS (Construction Engineering)			20.0%	$ 71,302.88
CONTINGENCIES			10.0%	$ 35,651.44
		TOTAL COST (Base Year 2019)		**$ 463,468.69**
NUMBER OF YEARS TO CONSTRUCTION MIDPOINT		2.00		
INFLATION RATE			3.5%	
		TOTAL PROJECT COST (Year 2021)		**$ 495,911.50**
			SAY	**$0.50 Million**

Town of Simsbury
Rehabilitation of Bridge No. 03984
Old Drake Hill Road Bridge (Flower Bridge) over Farmington River

Project No.: TBD
Date: 2/20/2019
By: DK
Checked By: JG

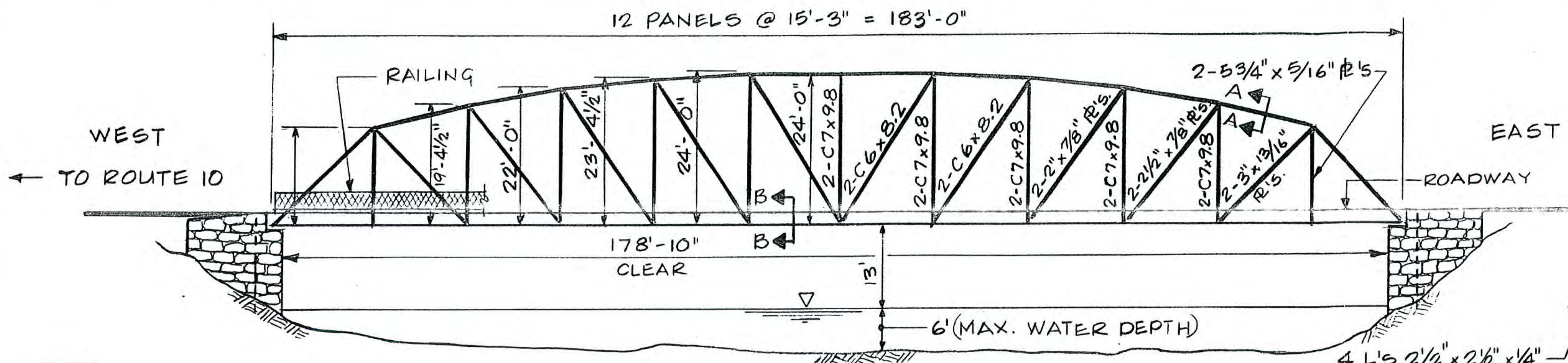
COST ESTIMATE FOR MAJOR REHABILITATION

Rehabilitation Measures

- Spot repairs for steel deterioration
- Repair damaged timber planks
- Replace Joint seal at abutments
- Perform bearing assembly repairs (requires jacking of the bridge)
- Abrasive Blast Clean and Paint entire bridge
- Install lateral restraint at Bearings
- Reset brick pavers
- Remove and replace water piping system for flower pots

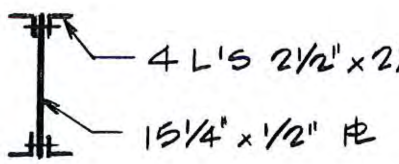
Description	Unit	Estimated Quantity	Unit Cost	Total Cost
SECTION A. - ROADWAY ITEMS				
REMOVE AND RESET TIMBER DECKING	S.F.	550	$ 10.00	$ 5,500.00
CONSTRUCTION ACCESS	L.S.	1	$ 35,000.00	$ 35,000.00
ABRASIVE BLAST CLEANING AND FIELD PAINTING OF EXISTING STEEL	S.F.	12100	$ 35.00	$ 423,500.00
CLASS 1 CONTAINMENT SYSTEM	L.S.	1	$ 350,000.00	$ 350,000.00
STRUCTURAL STEEL REPAIRS	CWT.	4	$ 10,000.00	$ 40,000.00
REPLACE REMOVED OR MISSING RIVETS AND BOLTS WITH HIGH STRENGTH BOLTS	EA.	100	$ 100.00	$ 10,000.00
REPAIR BEARING PIN SUPPORT ASSEMBLY	EA.	4	$ 5,000.00	$ 20,000.00
JACKING FOR BEARINGS REPAIRS	E.A.	2	$ 25,000.00	$ 50,000.00
LATERAL RESTRAINTS AT BEARINGS	EA.	4	$ 2,000.00	$ 8,000.00
REMOVE AND RESET WATER PIPING SYSTEM FOR FLOWER POTS	L.S.	1	$ 10,000.00	$ 10,000.00
REPLACE JOINT SEAL	L.F.	30	$ 50.00	$ 1,500.00
REMOVE AND RESET BRICK PAVERS	S.F.	30	$ 50.00	$ 1,500.00
	A. STURCTURE ITEMS SUBTOTAL			**$955,000**
SECTION B. - ENVIRONMENTAL COMPLIANCE ITEMS				
LEAD COMPLIANCE FOR ABRASIVE BLAST CLEANING	L.S.	1	$ 25,000.00	$ 25,000.00
DISPOSAL OF LEAD DEBRIS	TON	31	$ 2,000.00	$ 62,000.00
	B. ENVIRONMENTAL ITEMS SUBTOTAL			**$ 25,000.00**
SECTION C. - MINOR ITEMS/UNIDENTIFIED COSTS			**% of cost**	
MINOR ITEMS	L.S.		15.0%	$ 147,000.00
SECTION D. - LUMP SUM ITEMS			**% of cost**	
MOBILIZATION	L.S.		5.0%	$ 56,350.00
	PROBABLE CONSTRUCTION COST ESTIMATE			**$ 1,183,350.00**
ENGINEERING PERCENTAGES (Incidentals & Contingencies)			**% of cost**	
INCIDENTALS (Construction Engineering)	L.S.		20.0%	$ 236,670.00
CONTINGENCIES	L.S.		10.0%	$ 118,335.00
	TOTAL COST (Base Year 2019)			**$ 1,538,355.00**
NUMBER OF YEARS TO CONSTRUCTION MIDPOINT		2.00		
INFLATION RATE			3.5%	
	TOTAL PROJECT COST (Year 2021)			**$ 1,646,039.85**
		SAY		**$1.65 Million**

Appendix C - Existing Bridge Plans

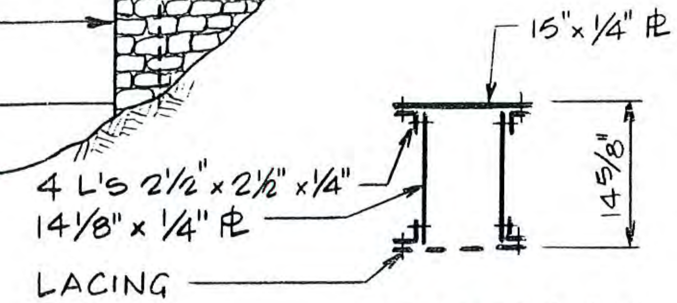


BRIDGE ELEVATION

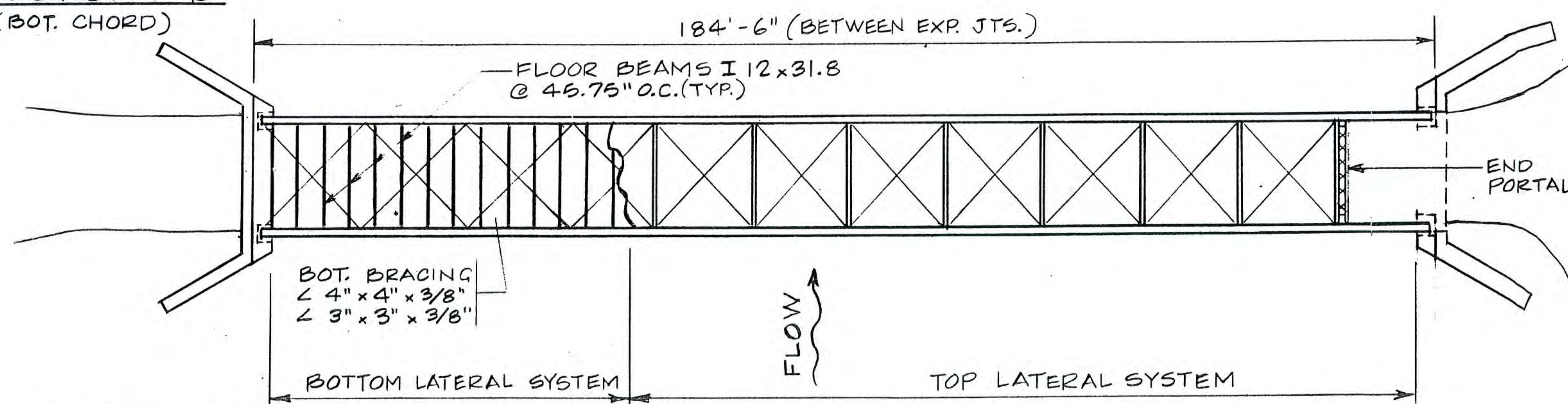
SCALE : 1" = 20'

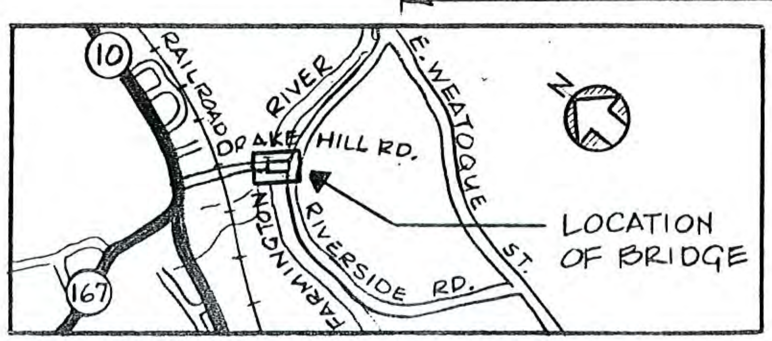


SECTION B-B
(BOT. CHORD)



SECTION A-A
(TOP CHORD)





LOCATION PLAN

BRIDGE PLAN

SCALE : 1" = 20'

DRAKE HILL ROAD
OVER
FARMINGTON RIVER
SIMSBURY, CONN.

PLAN & ELEVATION | DWG.# 1

PLANS PREPARED BY
MACCHI ENGINEERS - HFTD, CT.
JUNE 1983

TOWN OF SIMSBURY, CONNECTICUT

REHABILATATION OF
OLD BRIDGE ROAD BRIDGE OVER
THE FARMINGTON RIVER

FIRST SELECTMAN

MARY GLASSMAN

DIRECTOR OF PUBLIC WORKS

FRANK ROSSI

AUGUST 29, 1994

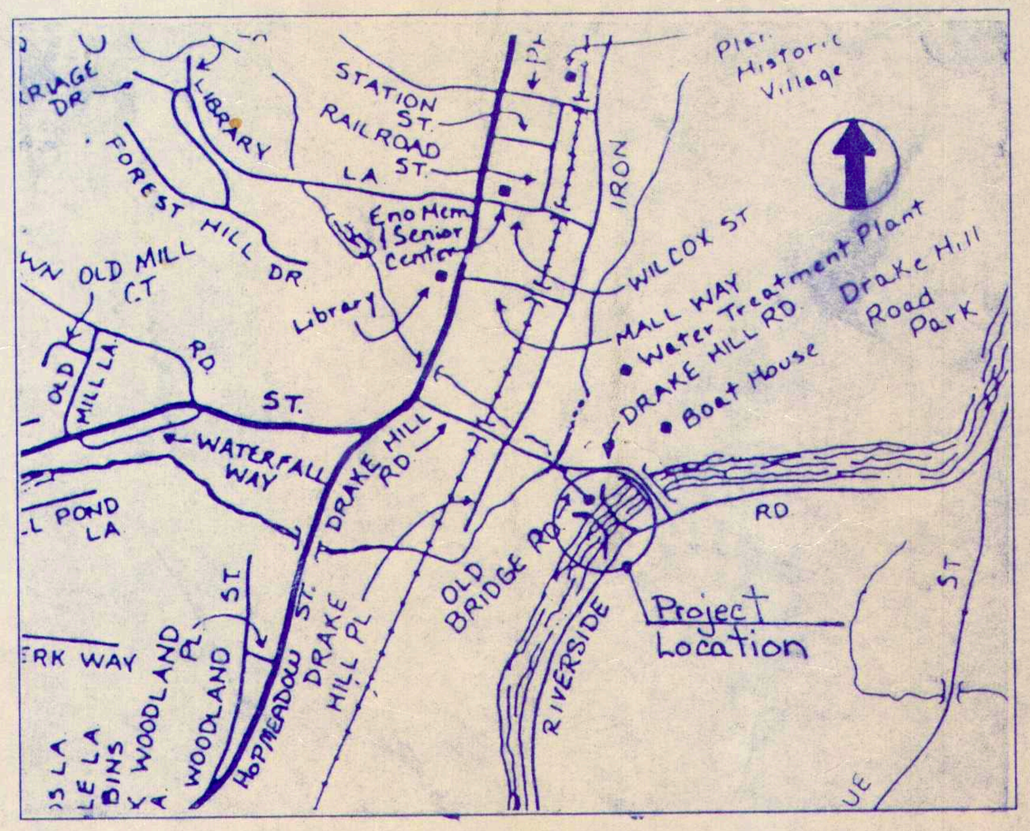
STATE PROJECT NO. 128-126

BRIDGE NO 03984

LIST OF DRAWINGS:

S-1 – GENERAL PLAN
S-2 – BRIDGE DECK AND TYPICAL DETAILS
S-3 – EXPANSION BEARING DETAILS
S-4 – PROTECTIVE FENCE DETAILS
S-5 – ABUTMENT DETAILS

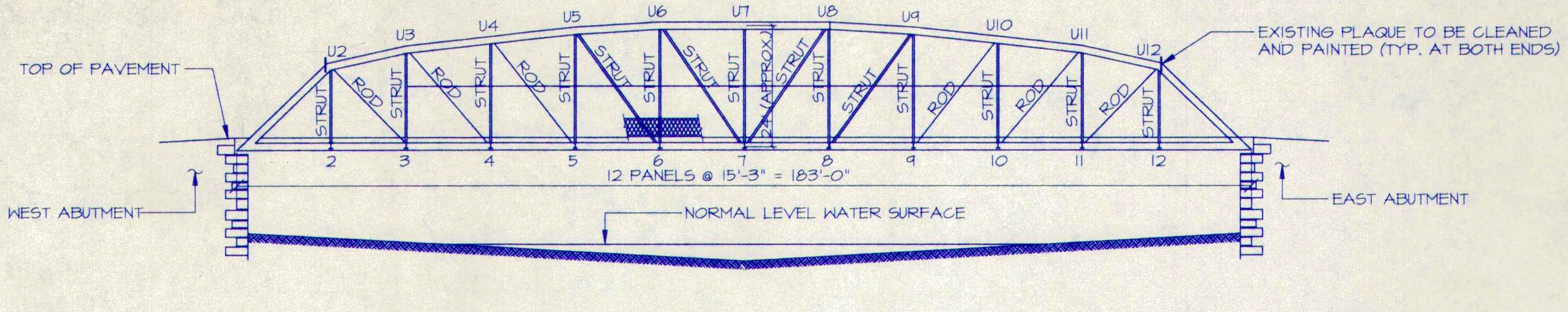
LOCATION MAP



CONSULTANTS:

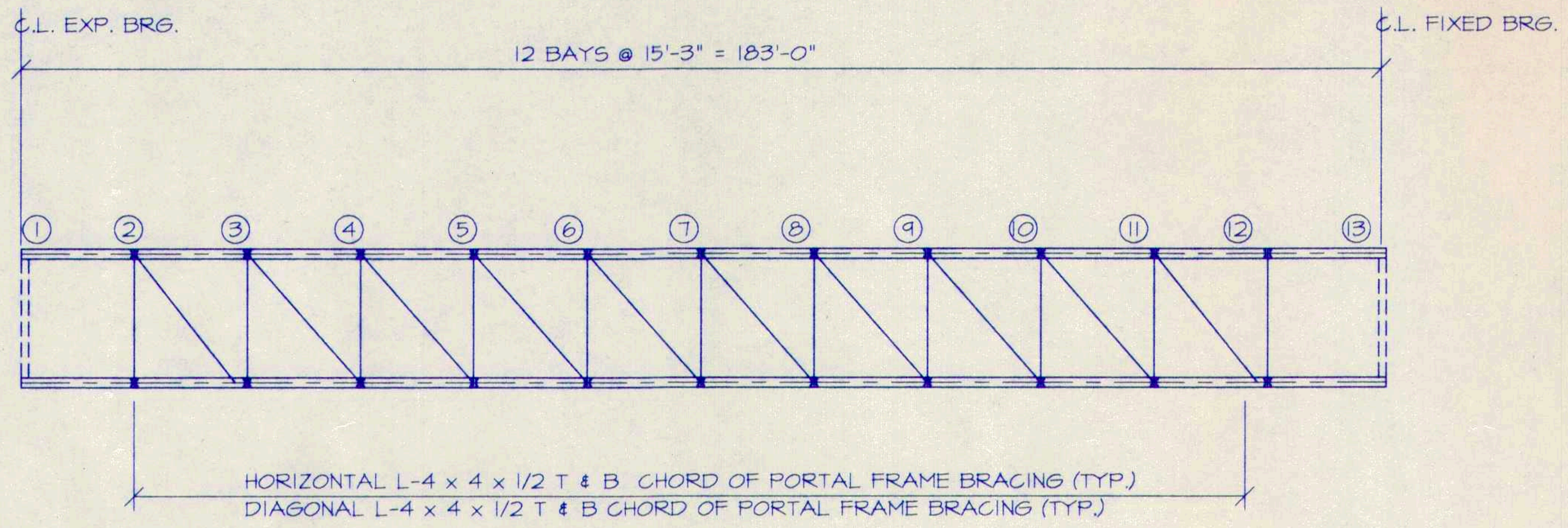
MACCHI ENGINEERS

44 GILLETT STREET, HARTFORD CT. (203) 549-6190



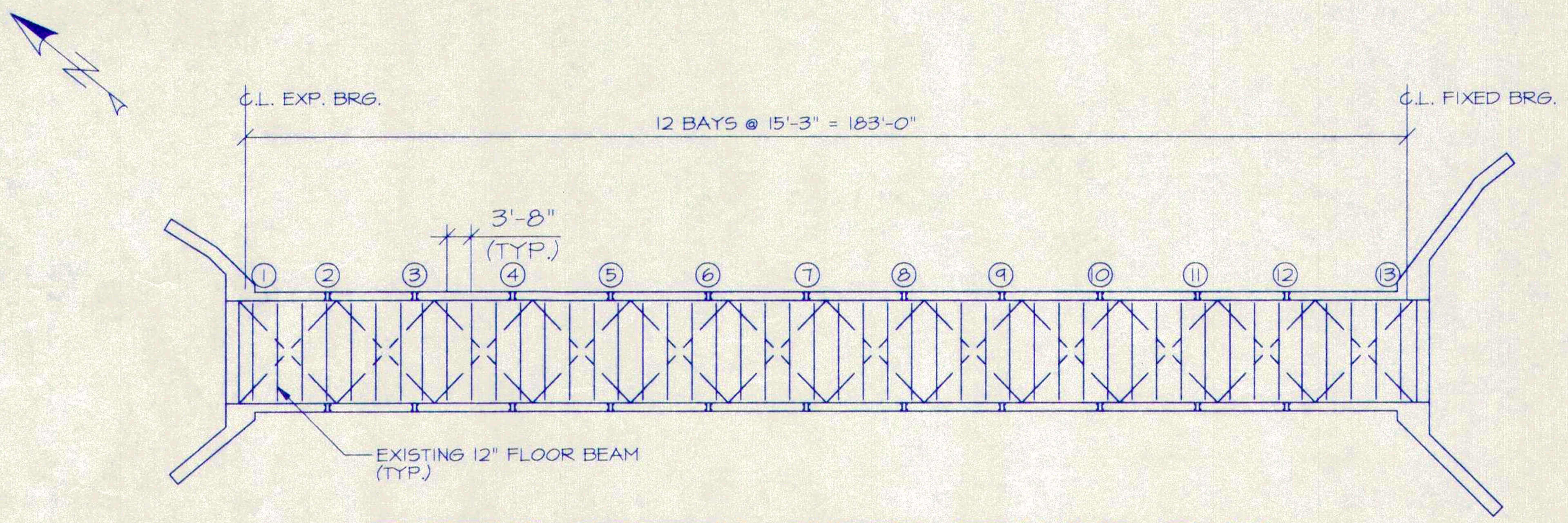
EXISTING BRIDGE ELEVATION (LOOKING UPSTREAM)

SCALE: 1" = 20'



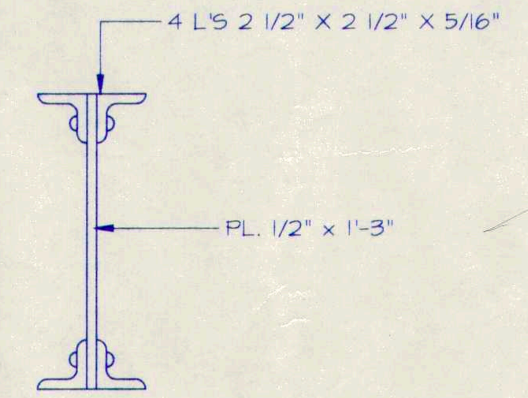
EXISTING TOP CHORD BRACING PLAN

SCALE: 1" = 20'



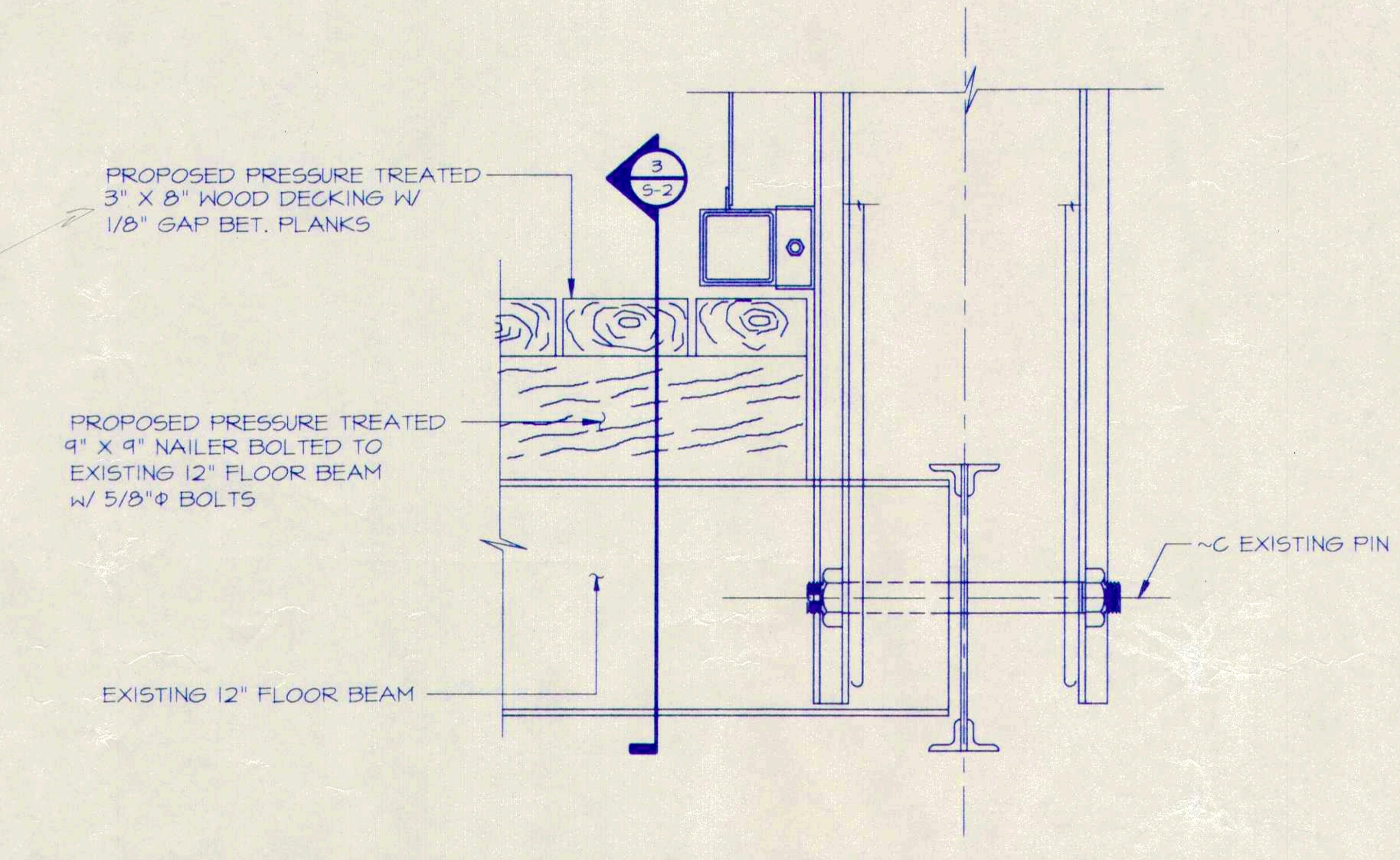
EXISTING BOTTOM CHORD FRAMING PLAN

SCALE: 1" = 20'



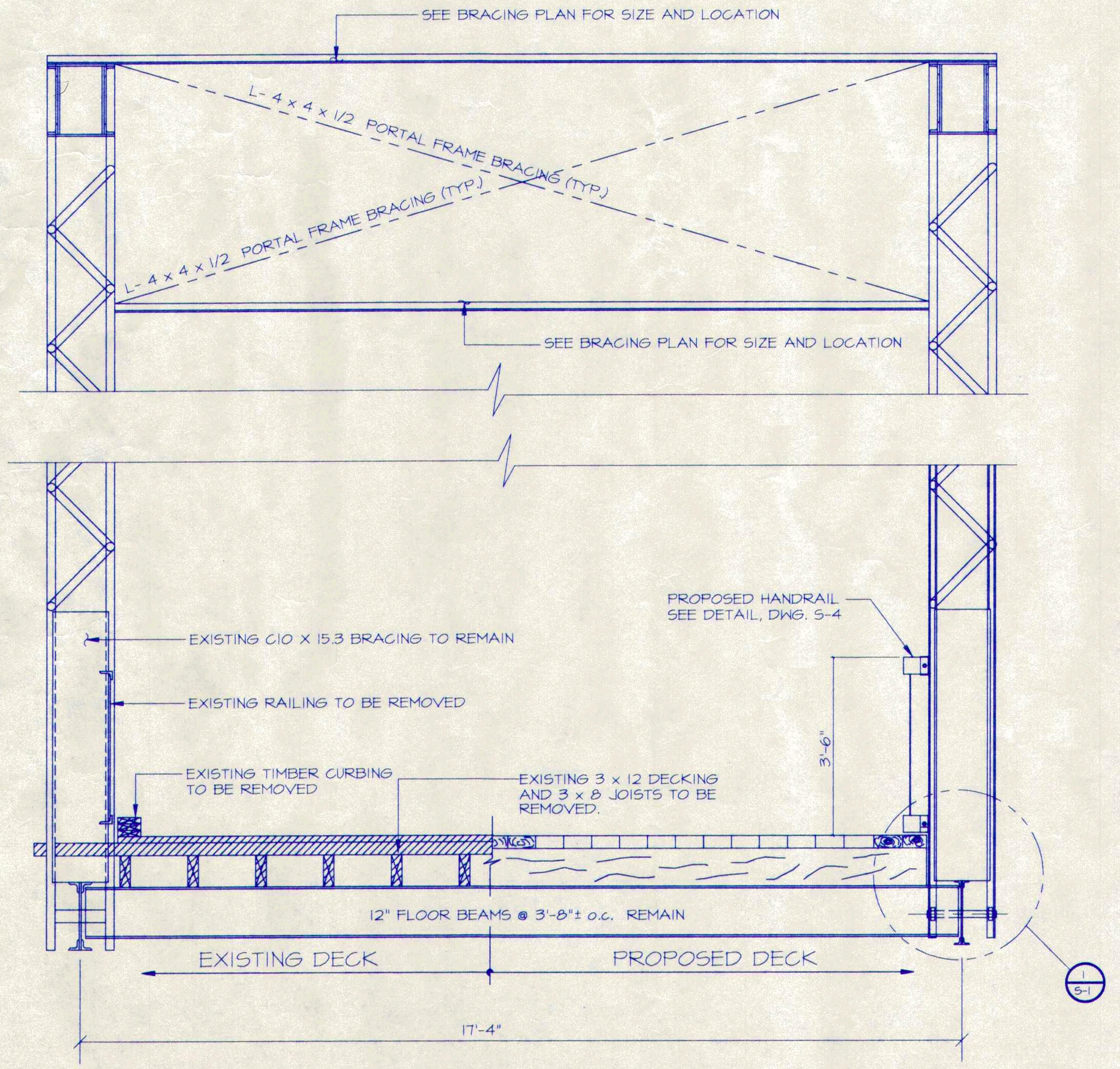
TYPICAL BOTTOM CHORD

SCALE: 1 1/2" = 1'-0"



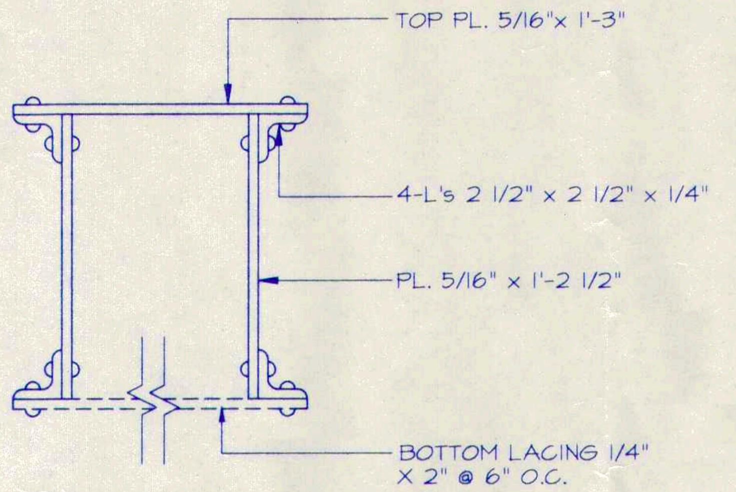
TYPICAL PROPOSED BRIDGE DECK DETAIL

SCALE: 1 1/2" = 1'-0"



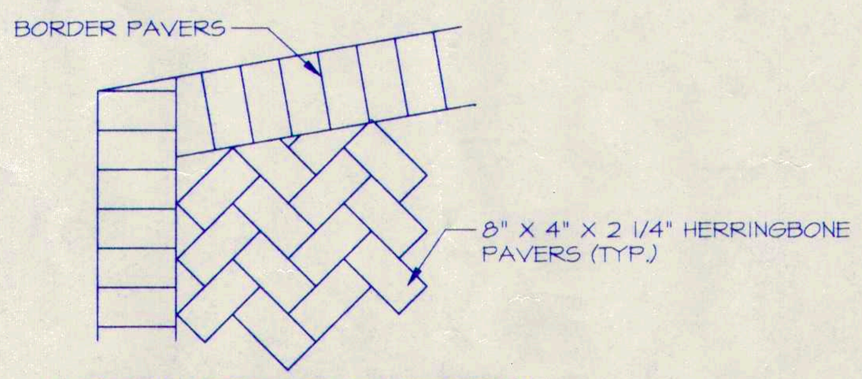
BRIDGE SECTION

SCALE: 1/2" = 1'-0"



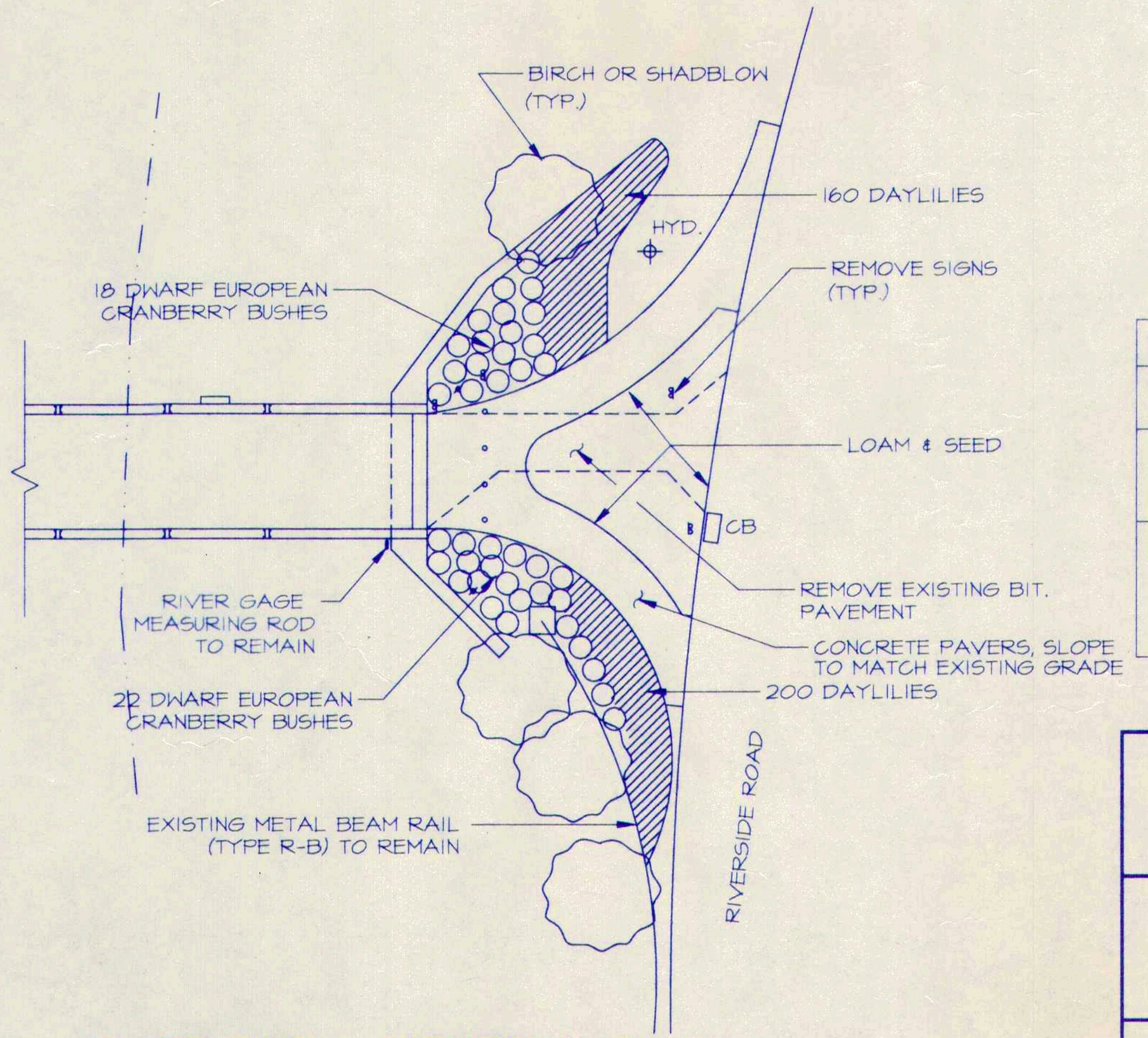
TYPICAL TOP CHORD

SCALE: 1 1/2" = 1'-0"



HERRINGBONE PATTERN BRICK PAVERS

SCALE: 3/4" = 1'-0"



EAST END - GENERAL PLAN ADD ALTERNATE BID ITEMS

SCALE: 1"=20'

EXISTING TO BE REMOVED

NOTE:
FOR SEDIMENTATION CONTROL SILT FENCE DETAIL, SEE SHEET NO. S-4.

GENERAL NOTES:

DESIGN SPECIFICATIONS:

AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES AND INTERIM SPECIFICATIONS.
BOCA 1987 W/1990 SUPPLEMENT AND CONNECTICUT AMMENDMENTS
BRIDGE WELDING CODE - ANSI/AASHTO/AWS D1.5 - 88.
CONNECTICUT DEPARTMENT OF TRANSPORTATION FORM 814 (1988), INCLUDING SUPPLEMENTS DATED JULY 1993 AND SPECIAL PROVISIONS.

LIVE LOAD: (100 P.S.F.) AFTER REHABILITATION.

EXISTING FLOOR BEAMS AND STRINGERS ARE RATED FOR TEMPORARY H-5 CONSTRUCTION LOADS.

STEEL TYPES AND ALLOWABLE DESIGN STRESSES:

ORIGINAL BRIDGE STEEL: fs = 14,000 P.S.I.

NEW STEEL:
ASTM A-36 STEEL, Fy = 36,000 P.S.I.

REINFORCING STEEL ASTM A-615, GRADE 60. (EPOXY COATED)

SPLASH GUARD PLATES, AND MISC. ITEMS:
GALVANIZED AS PER ASTM A-123

REMOVAL OF EXISTING BRIDGE DECK ITEMS AND OTHER MATERIAL:

STAGING SHALL BE PROVIDED UNDER THE BRIDGE FOR THE SAFETY OF WORKERS AND TO PREVENT MATERIALS FROM FALLING INTO FARMINGTON RIVER.

CONSTRUCTION METHODS WHICH MAY DISTORT OR DAMAGE FLOOR BEAMS OR TRUSS MEMBERS WILL NOT BE ALLOWED. SEE SPECIFICATIONS

ALL MATERIAL TO BE REMOVED AND NOT TO BE REUSED SHALL BECOME THE PROPERTY OF THE CONTRACTOR AND WILL BE REMOVED FROM THE SITE AND PROPERLY DISPOSED OF IN ACCORDANCE WITH STATE AND LOCAL ORDINANCES. AS SPECIFIED BY THE ENGINEER, MATERIAL MAY BE DISPOSED OF AT THE LANDFILL ON WOLCOT HILL ROAD. ALL OTHER MATERIAL WILL BE PROPERLY DISPOSED OF BY THE CONTRACTOR.

REPAIRING AND POINTING OF MASONRY WALLS:

SEE SPECIFICATIONS.

EROSION CONTROL

SEDIMENTATION CONTROL SILT FENCE SHALL BE PLACED AT THE TOE OF SLOPES AND AT OTHER LOCATIONS AS REQUIRED TO PREVENT EROSION INTO THE FARMINGTON RIVER.

BRIDGE PAINTING:

ALL STEEL SURFACES, EXCEPT NEW GALVANIZED ITEMS, SHALL BE ABRASIVE BLAST CLEANED IN ACCORDANCE WITH SSPC-SP-10 PRIOR TO THE APPLICATION OF A THREE COAT PAINT SYSTEM AS FOLLOWS:

PRIMER COAT: . . . . ORGANIC ZINC RICH PRIMER
INTERMEDIATE COAT: . . . . EPOXY MASTIC
TOPCOAT: . . . . HIGH BUILD ALIPHATIC URETHANE
COLOR OF TOP COAT: . . . . GREEN
(FED. STD. COLOR NO. 34058)

LOAMING AND SEEDING:

AREAS DISTURBED DURING THIS CONSTRUCTION SHALL BE LOAMED AND SEEDED AS PER THE SPECIFICATIONS.

STRUCTURAL DIMENSIONS:

ALL DIMENSIONS AND ANGLES SHOWN ON THE PLANS ARE BASED ON LIMITED FIELD INVESTIGATION. THE CONTRACTOR SHALL BE RESPONSIBLE FOR FIELD VERIFICATION OF ALL DIMENSIONS AND ANGLES.

CONCRETE:

CLASS 'A' CONCRETE SHALL BE USED FOR ALL WORK ON THE ABUTMENTS AND WINGWALLS. ALLOWABLE DESIGN STRENGTH SHALL BE BASED ON Fc' = 3000 psi.

TIMBER

SEE SPECIFICATIONS.

LANDSCAPE SCHEDULE

(ADD ALTERNATE ITEMS)

BOTANICAL NAME	COMMON NAME	QUANTITY	SIZE AREA
VIBURNAM OPULUS NANUM	DWARF EUROPEAN CRANBERRY BUSH	40	15"-18"SPD.
AMBLANCHIER CANADENSIS OR BETULA PLATYPHYLLA JAPONICA	SHADBLOW / WHITESPIRE BIRCH	4	10'-12' HT. MULTI STEM
HEMEROCALLIS FLAVA HEMEROCALLIC HYPERION HEMEROCALLIS STELLA D'ORO HEMEROCALLIS HALLIS PINK	DAYLILIES	90 90 90 90 (QT. CONT.)	12" O.C.

GENERAL PLAN

REHABILATATION OF OLD BRIDGE ROAD BRIDGE
SIMSBURY, CONNECTICUT

Macchi Engineers
44 Gillett Street
Hartford, CT 06105
Phone: (203) 549-6190

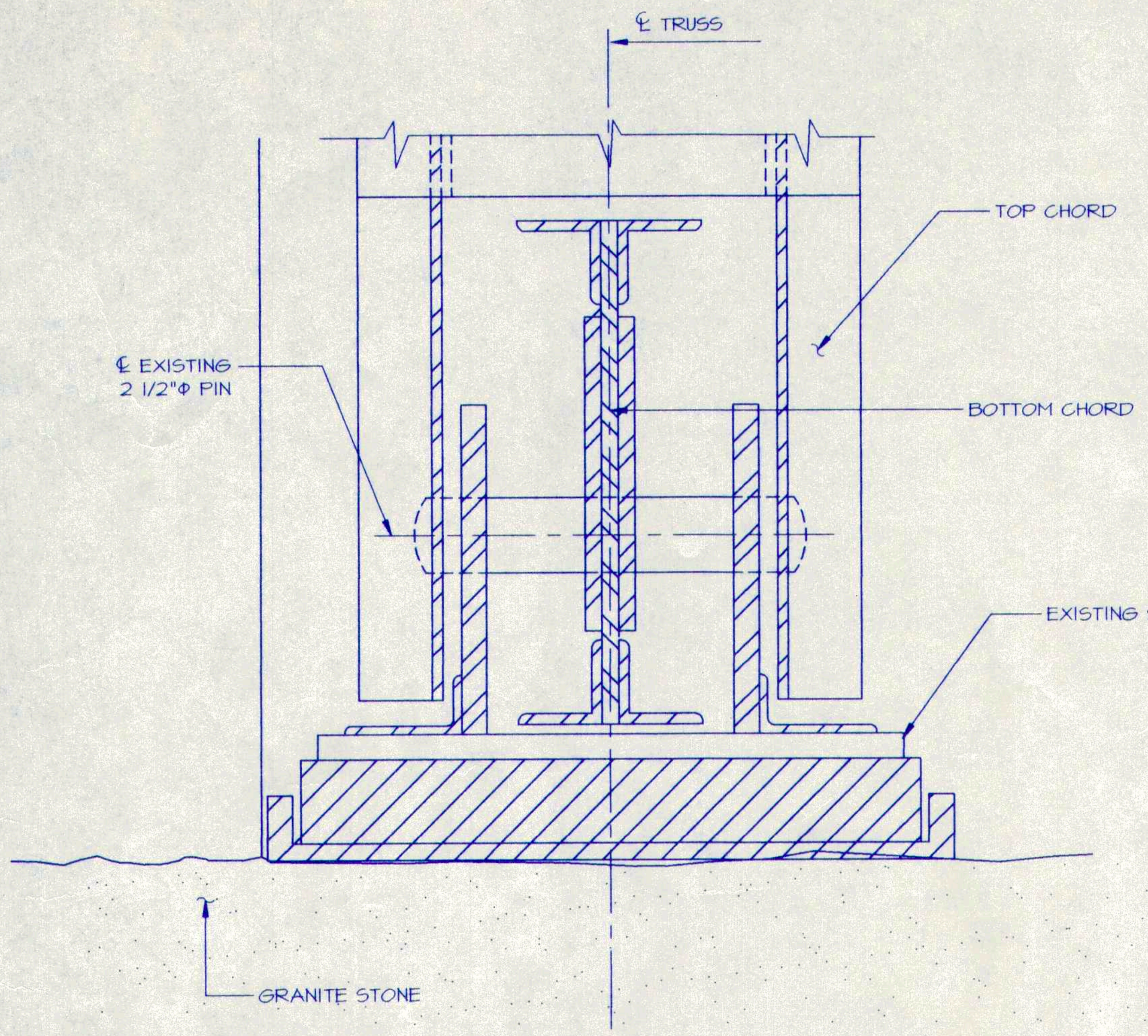
DRAWN BY:	J.Z.
APPROVED BY:	J.B.
DATE:	8/23/94
SCALE:	AS NOTED
Drawing No.	S-1

BRIDGE DECK AND TYPICAL DETAILS

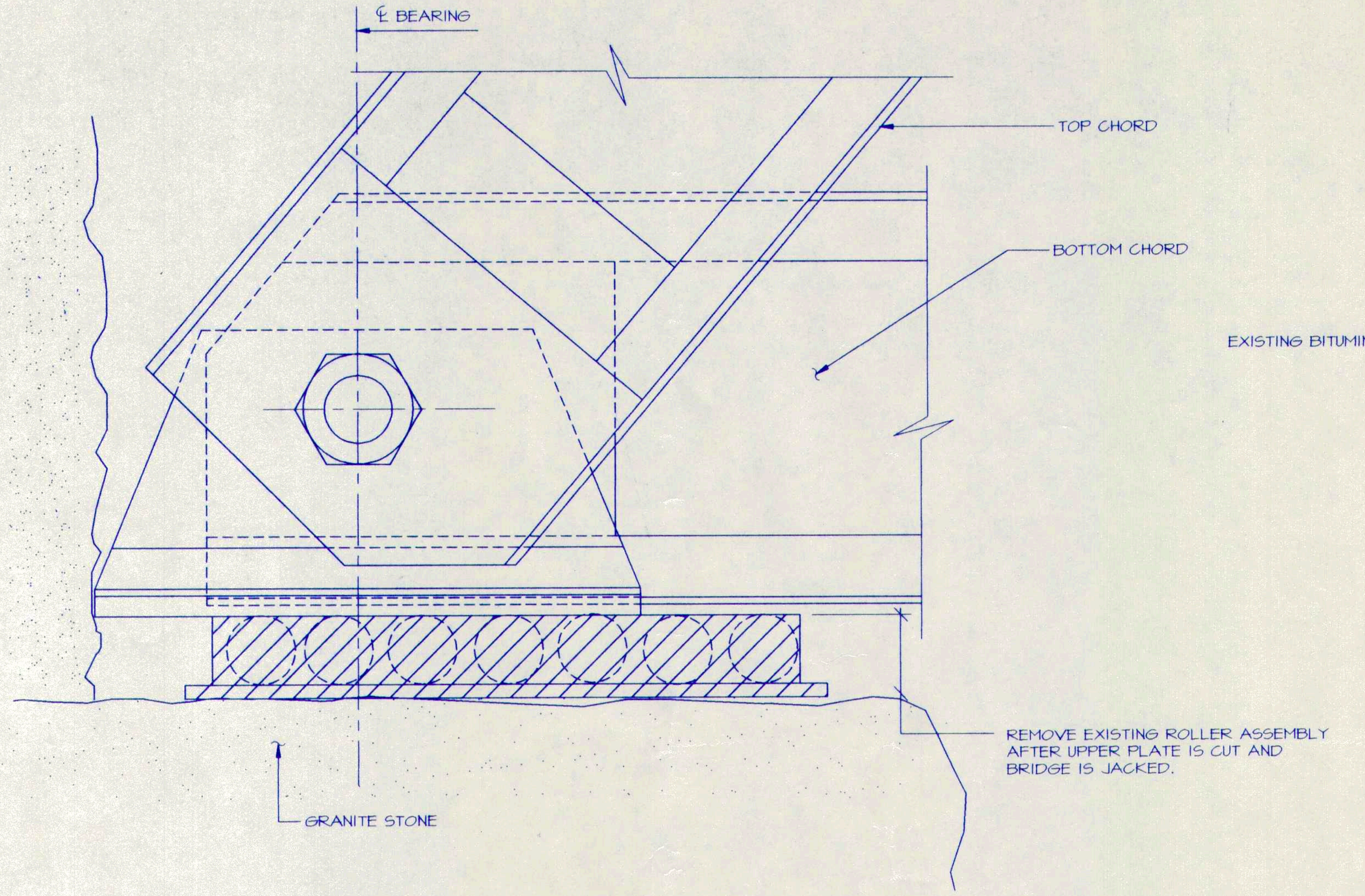
REHABILATATION OF OLD BRIDGE ROAD BRIDGE SIMSBURY, CONNECTICUT

Macchi Engineers
44 Gillett Street
Hartford, CT 06105
Phone: (203) 549-6190

DRAWN BY:	J.Z.
APPROVED BY:	J.B.
DATE:	8/01/94
SCALE:	AS NOTED
Drawing No.	S-2



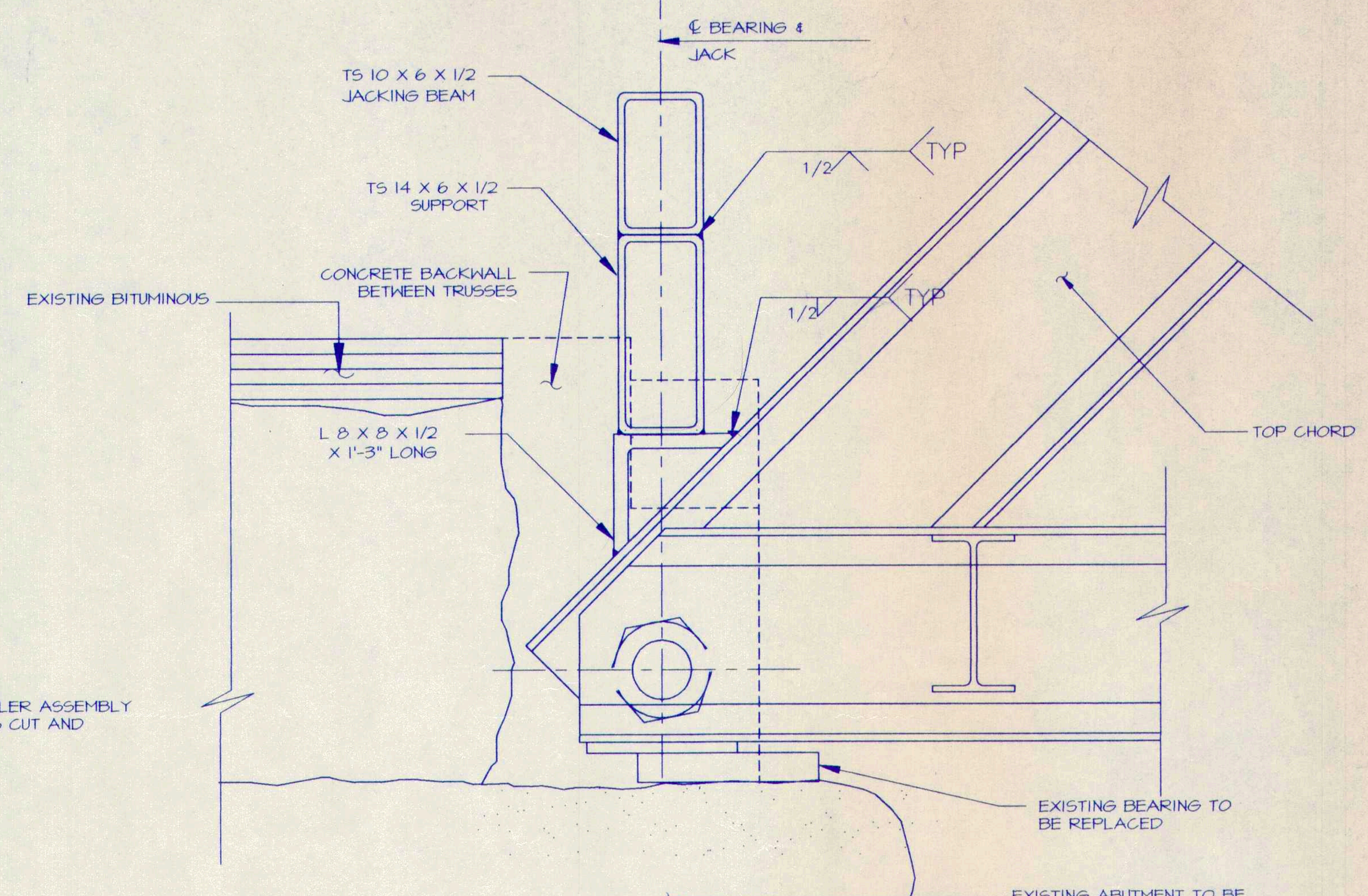
SECTION



ELEVATION

EXISTING EXPANSION BEARING

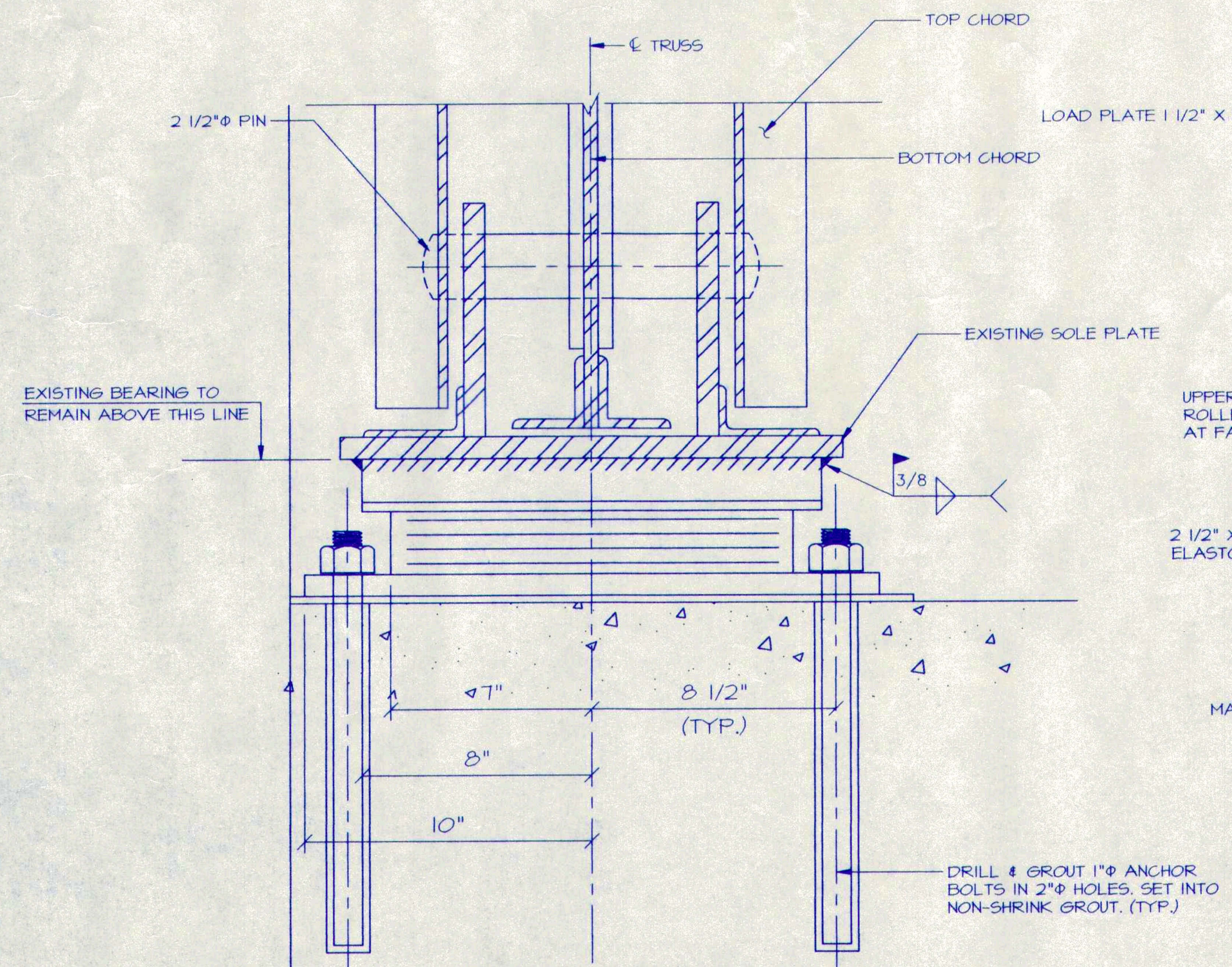
SCALE: 3" = 1'-0"



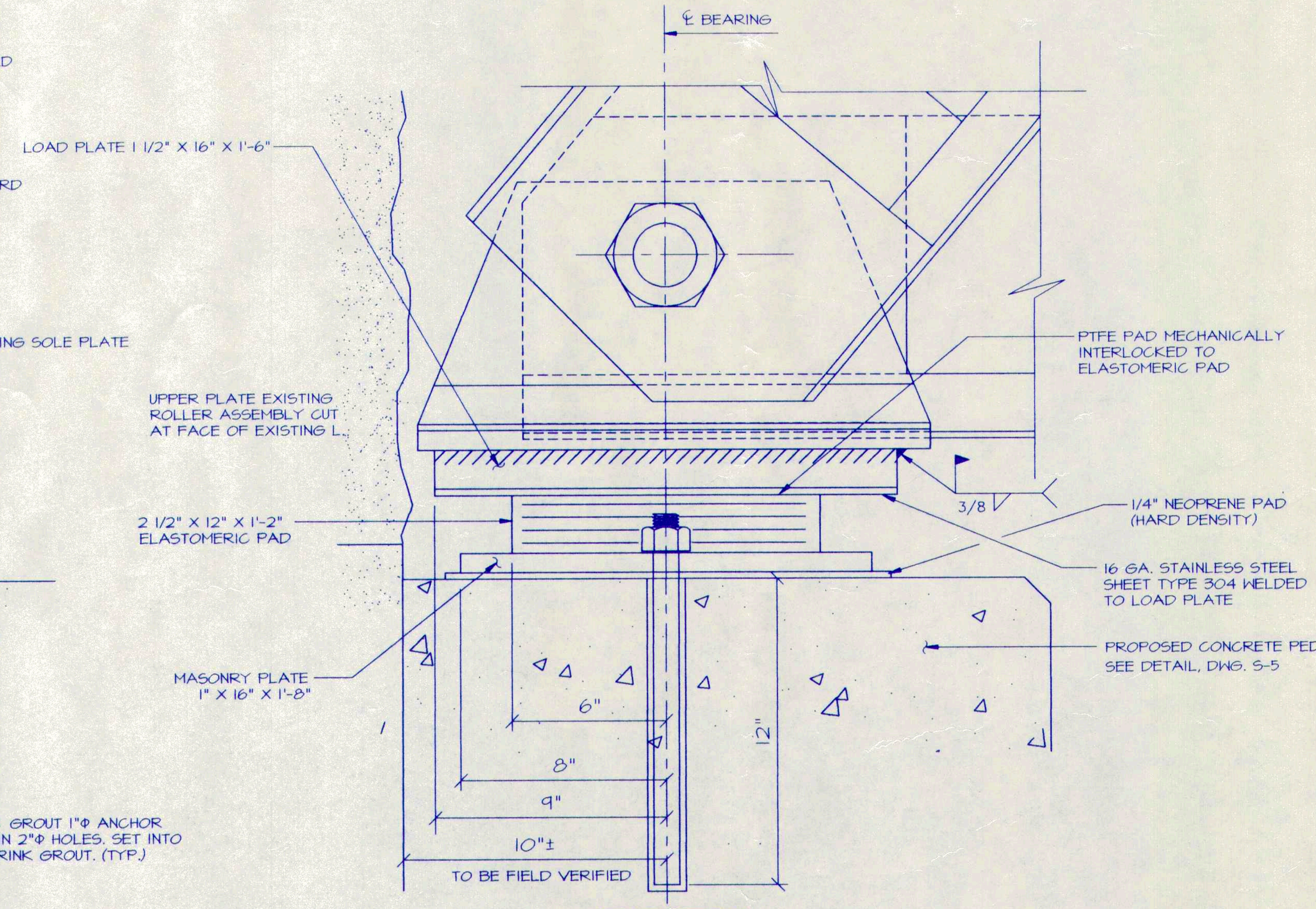
JACKING BEAM • WEST BEARING

SCALE: 1 1/2" = 1'-0"

NOTE:
JACKING FORCE EQUALS APPROXIMATELY 20 TONS



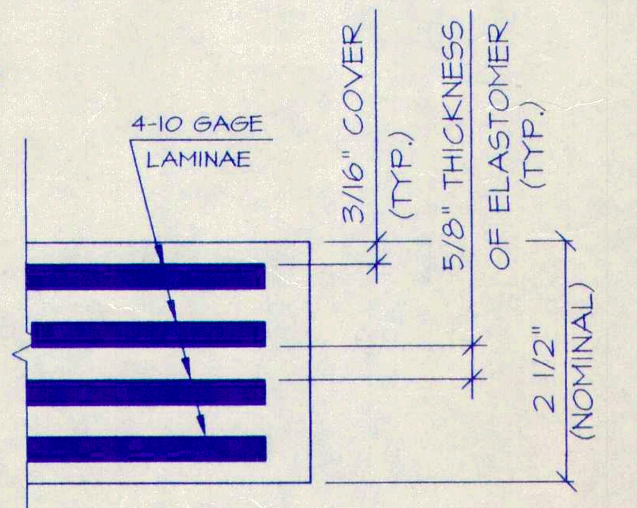
SECTION



ELEVATION

PROPOSED EXPANSION BEARING

SCALE: 3" = 1'-0"



ELASTOMERIC PAD DETAIL

N.T.S.

SEQUENCE OF CONSTRUCTION

1. REMOVE TIMBER DECKING AND JOISTS. REMOVE WINGWALL RAIL AT WEST END OF BRIDGE
2. WELD JACKING TUBE TO TRUSS, PLACE PLATE ON WEST ABUTMENT BACKWALL AND JACK WEST END OF BRIDGE, JACKING BOTH TRUSSES SIMULTANEOUSLY. SEE "JACKING EXISTING TRUSSES" IN SPECIAL PROVISIONS.
3. REMOVE EXISTING ROLLER ASSEMBLY
4. REMOVE TOP COURSE OF STONE MASONRY UNDER BEARING AND REPLACE WITH CONCRETE
5. LOWER TRUSS ONTO NEW BEARINGS AND REMOVE JACKING BEAM
6. PLACE TIMBER DECKING

BEARING NOTES:

1. THE STEEL FOR NEW BEARING ASSEMBLIES SHALL CONFORM TO ASTM A-709, GR. 50W. THE STEEL LAMINAE FOR ELASTOMERIC PADS SHALL CONFORM TO ASTM I-36, OR AN APPROVED EQUAL.
2. THE ELASTOMER BEARING PADS SHALL CONFORM TO ASTM D-4014, TYPE CR, GR.3. IT SHALL HAVE A SHORE 'A' DUROMETER HARDNESS OF 60, AND A SHEAR MODULUS WITHIN THE RANGE OF 170 psi TO 200 psi.
3. THE EXPANSION BEARINGS SHALL BE INSTALLED WHEN THE AMBIENT TEMPERATURE IS IN THE RANGE OF 40°F TO 80°F, AND HAS BEEN IN THIS RANGE FOR AT LEAST TWO HOURS.

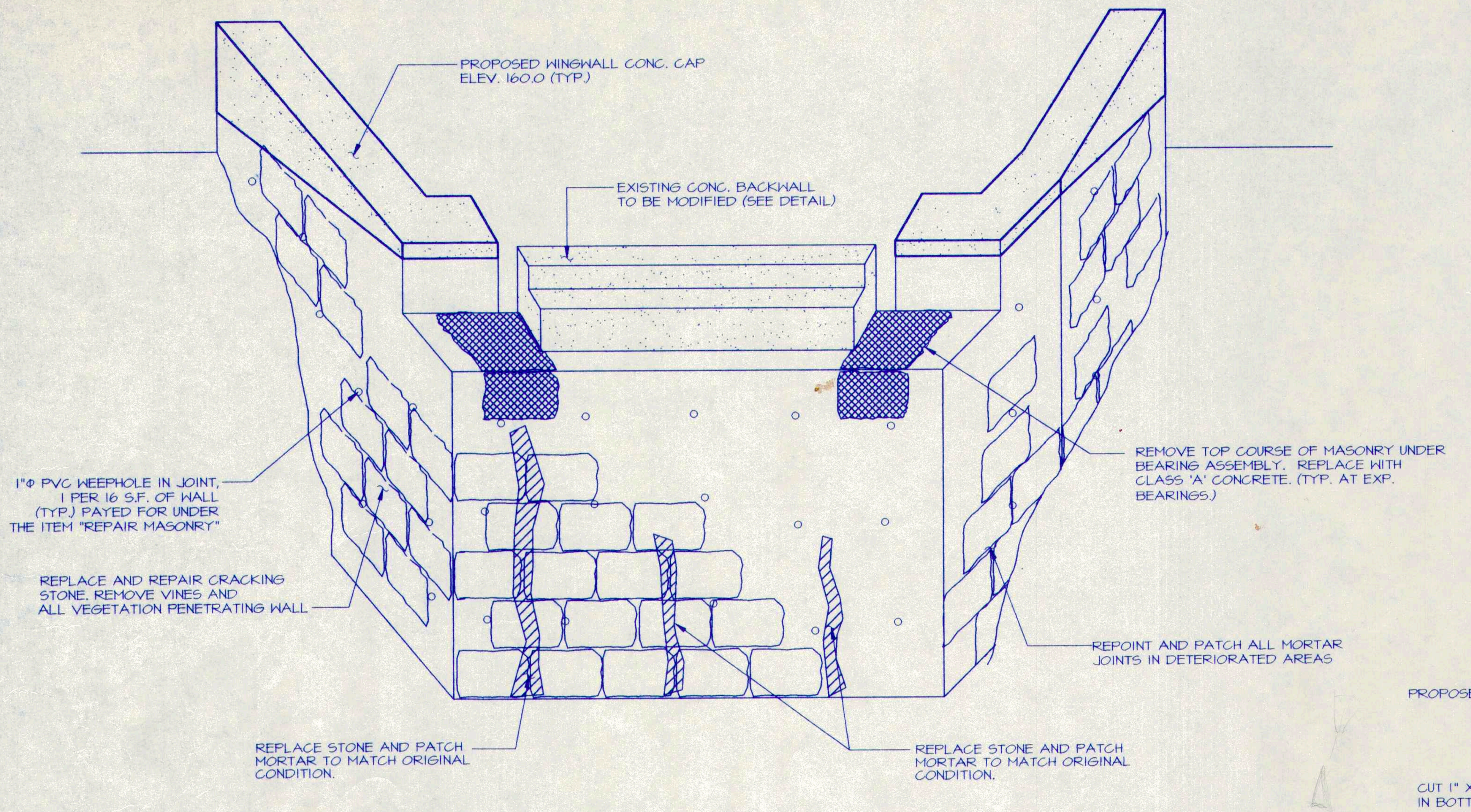
EXPANSION BEARING DETAILS

REHABILATATION OF OLD BRIDGE ROAD BRIDGE
SIMSBURY, CONNECTICUT

Macchi Engineers
44 Gillett Street
Hartford, CT 06105
Phone: (203) 549-6190

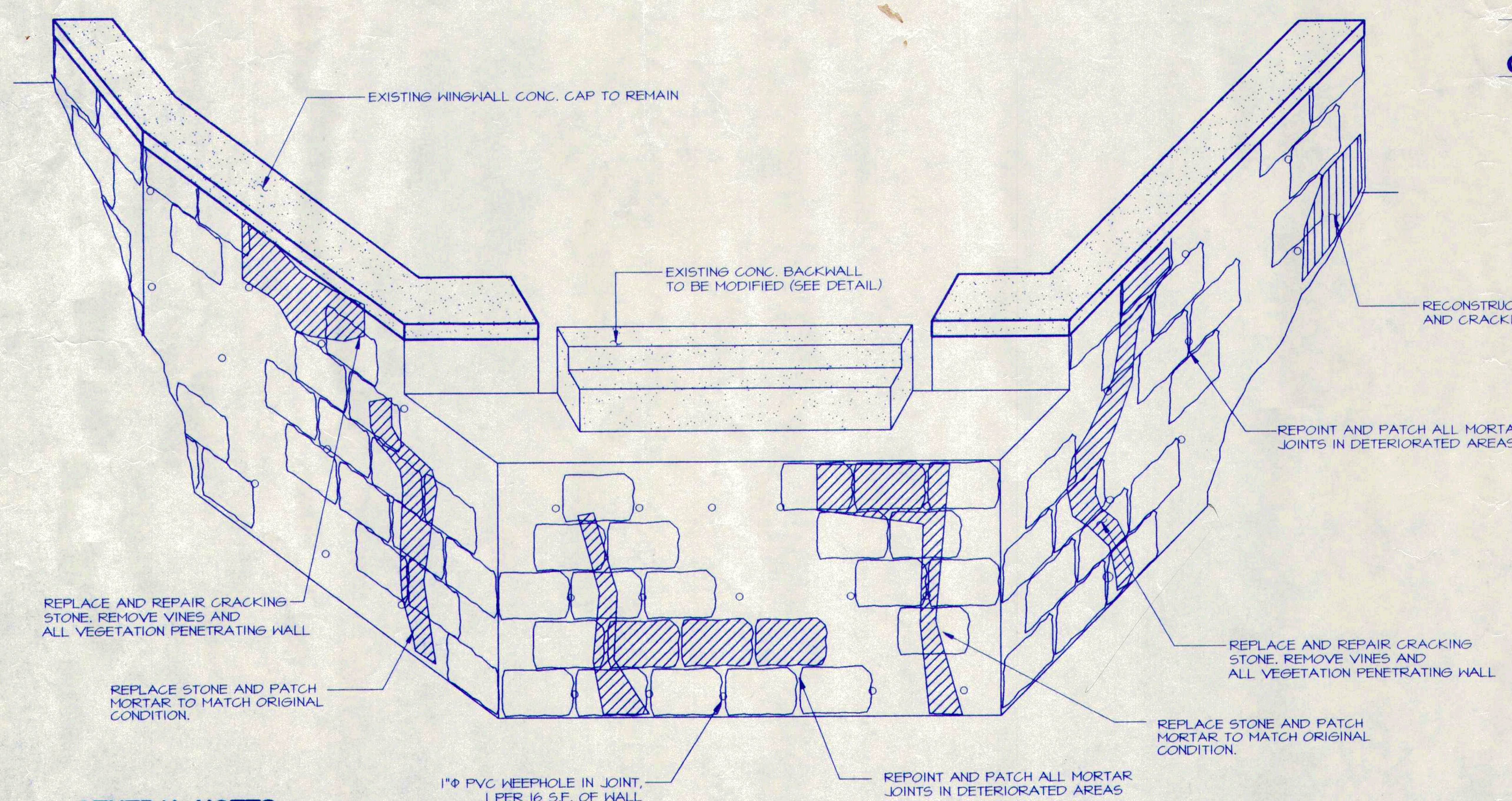
DRAWN BY: J.Z.	Drawing No. S-3
APPROVED BY: J.B.	
DATE: 8/01/94	
SCALE: AS NOTED	

NEW COVER ℄ 1/2" X 1'-3 3/4" TO MATCH HEIGHT OF INTERMEDIATE COVER PLATES
EAST END
BRIDGE RAIL ELEVATION
SCALE: 1" = 1'-0"
EXISTING C7
NEW COVER PL. 1/2" X 8" FROM BOTTOM OF EXISTING C7 TO TOP OF EXISTING C10 X 15.3. (W/HOLE FOR PIN)
NEW 1/2" CAP PL.
PL. 1/4" X 1" @ 5" o.c. INFILL WELDED TO L-1 1/2" X 1 1/2" X 1/4"
EXISTING C10 X 15.3
42"
L 1 1/2" X 1 1/2" X 1/4" FRAME WELDED TO T.S. 4 X 4 X 1/4 TYP. T & B
TOP OF PROPOSED WOOD DECK
1"-CLR.
EXISTING 12" FLOOR BEAM (TYP.)
BOTTOM CHORD OF TRUSS
PANEL POINTS 3 THRU 11
(PANEL POINTS 2 & 12 SIMILAR - SEE SECTION 1A/S-4)
1/2" CAP PL. @ EA. C7
T.S. 4 X 4 X 1/4 RAIL (CONT.) TYP. T & B
L-1 1/2 X 1 1/2 X 1/4 TYP. T & B
1/4" (CONT.)
NEW 1/2" COVER PL. @ EA. C7 (TYP.)
PROPOSED 3" X 12" WOOD DECK
CUT HOLE IN PLATE TO CLEAR PIN
℄ EXISTING PIN
EXISTING 12" FLOOR BEAM
℄ EXISTING C10 X 15.3
SECTION
SCALE: 1" = 1'-0"
EXISTING PLATE 5/16" X 1'-3"
EXISTING BOTTOM CHORD
1/2" CAP PLATE, COPE PLATE TO FIT TIGHT AGAINST ANGLES AND WEB
FULL HEIGHT OF PLATE
TS 2 X 2 X 1/4 RAILING W/CAP PL. 3/8"X 5" X 7"
NEW 1/2" X 1'-3 3/4" COVER PLATE
TS 4 X 4 X 1/4 RAILING W/CAP PL. 3/8" X 5" X 7" BOLTED TO PL. 1/2" X 2 1/4" X 5"
℄ L. HANDRAIL
SECTION
SCALE: 1 1/2" = 1'-0"
EXISTING C7
EXISTING C10 X 15.3
NEW 1/2" X 8" COVER PL.
EXISTING PIECE OF W16 X 45
1/2" CAP PLATE COPE PLATE TO FIT TIGHT AGAINST WEB AND FLANGES OF C7
T.S. 4 X 4 X 1/4 RAILING W/ CAP PL. 3/8" X 5" X 0'-7" BOLTED TO PL. 1/2" X 3" X 0'-5" W/ 1/2"Φ BOLTS
12 1/2" (TYP.)
FULL HEIGHT OF PLATE
SECTION
SCALE: 1 1/2" = 1'-0"
EXISTING PIECE OF W16 X 45
1/2" CAP PL. COPE PL. TO FIT TIGHT AGAINST TUBE.
NEW TS 4 X 4 X 1/4
T.S. 4 X 4 X 1/4 RAILING W/ CAP PL. 3/8" X 5" X 0'-7" BOLTED TO PL. 1/2" X 3" X 0'-5" W/ 1/2"Φ BOLTS
FULL HEIGHT OF TUBE
SECTION
SCALE: 1 1/2" = 1'-0"
(AT PANEL POINTS 2 & 12)
DECORATIVE POST TO MATCH EXISTING POSTS
TS 2 X 2 X 1/4 (TYP. T & B)
PL. 1/4" X 1" @ 5" O.C. INFIL WELDED TO L-1 1/2" X 1 1/2" X 1/4"
L 1 1/2 X 1 1/2" X 1/4" FRAME WELDED TO TS 2 X 2 X 1/4 (TYP. T & B)
TOP OF CONCRETE WINGWALL CAP
DRILL AND GROUT POST IN CONCRETE WINGWALL CAP
WINGWALL RAIL DETAIL
SCALE: 1" = 1'-0"
2" X 3/8" STEEL BRACKET SUPPORT
℄ 3/8" CARRIAGE BOLT - NON-RETRIEVABLE TYPE, THREAD INTO 4 X 4 TUBE
8" X 8" MIN. OPENING
FLOWER BOX TO BE SUPPLIED BY TOWN
T.S. 4 X 4 X 1/4 RAIL (CONT.)
NOTE: COST TO BE INCLUDED IN THE PRICE FOR "STRUCTURAL STEEL (LOW ALLOY)"
PLANTER SUPPORT
(2 PER PLANTER) N.T.S.
4'-0"±
TOE OF SLOPE
FILTER FABRIC
POST OR OTHER SUITABLE MOUNTING
FLOW
EXISTING GROUND
ANCHOR FILTER FABRIC IN GROUND
SECTION
VARIES • 10'-0" MAX.
FILTER FABRIC
6"-BURIED FABRIC
ELEVATION
NOTE:
• POST SPACING AND EMBEDMENT VARIES BASED ON THE MANUFACTURERS REQUIREMENTS.
SILT FENCE
(LIGHT DUTY)
PROTECTIVE FENCE DETAILS
REHABILATATION OF OLD BRIDGE ROAD BRIDGE
SIMSBURY, CONNECTICUT
Macchi Engineers
44 Gillett Street
Hartford, CT 06105
Phone: (203) 549-6190
DRAWN BY: J.Z.
APPROVED BY: J.B.
DATE: 8/01/94
SCALE: AS NOTED
Drawing No. S-4



WEST ABUTMENT AND WINGWALLS

SCALE: 1/4" = 1'-0"

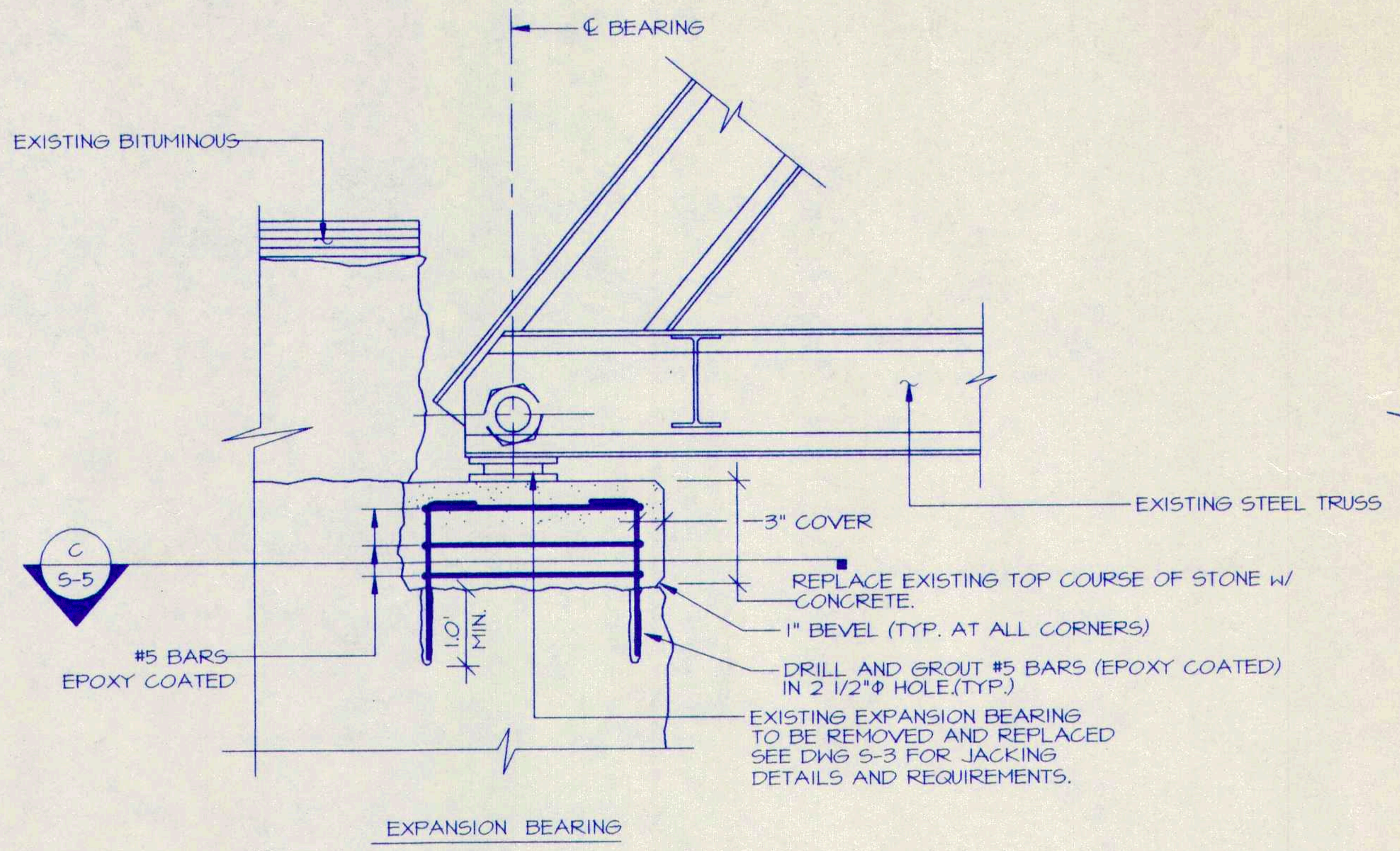


GENERAL NOTES:

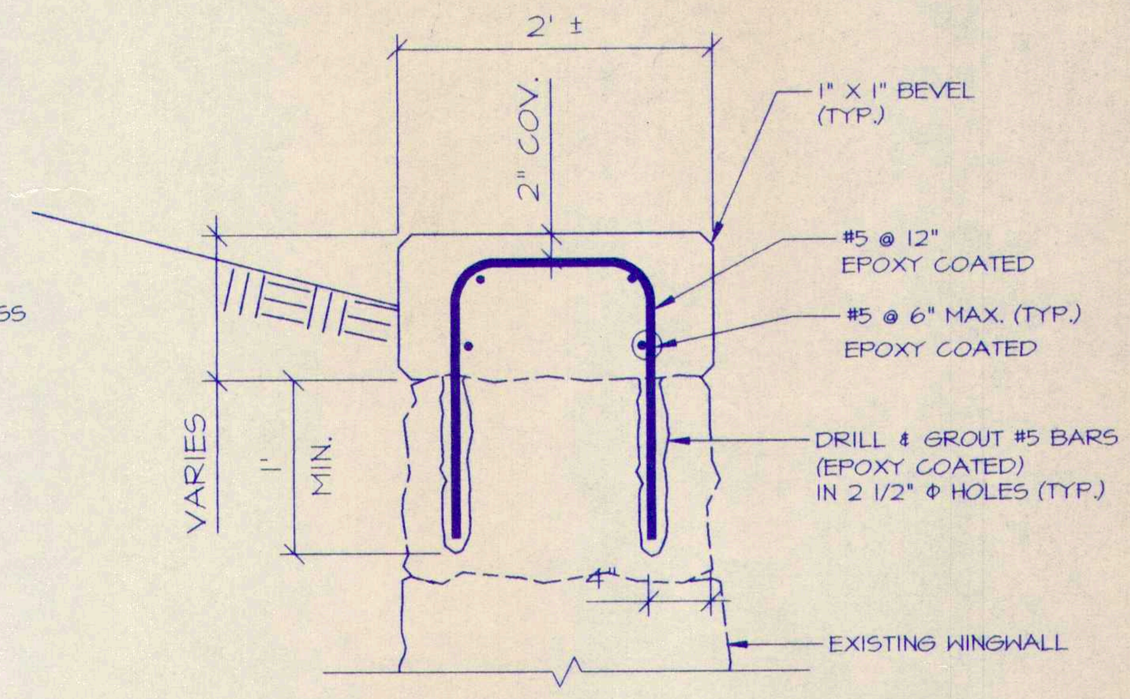
- APPROXIMATE AREAS OF REPAIR
- EXISTING MASONRY TO BE REMOVED

EAST ABUTMENT AND WINGWALLS

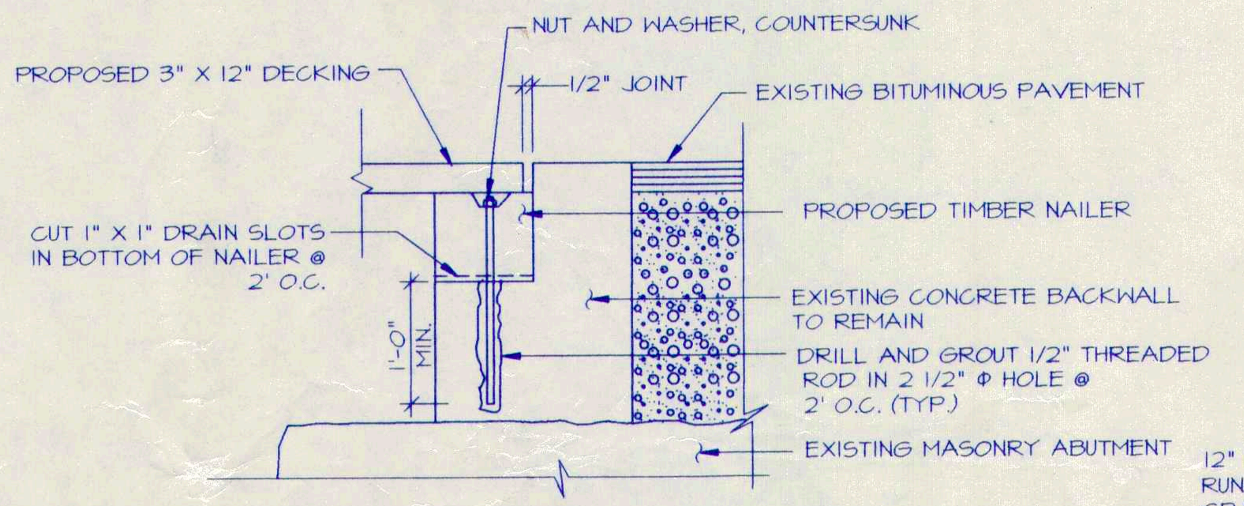
SCALE: 1/4" = 1'-0"



SECTION N.T.S. A S-5

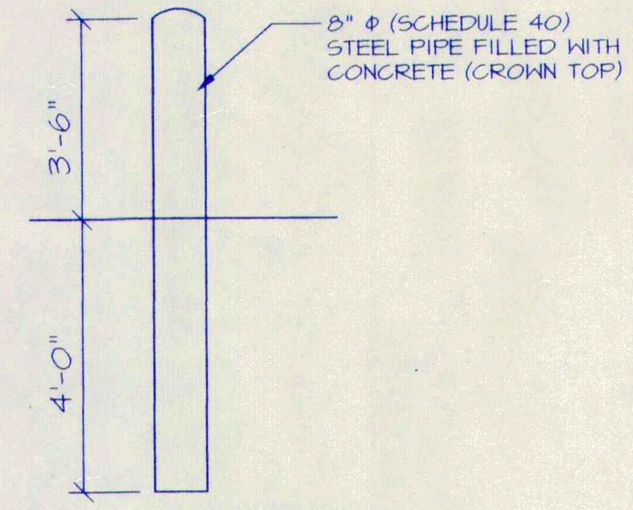


SECTION SCALE: 1"=1'-0" B S-5



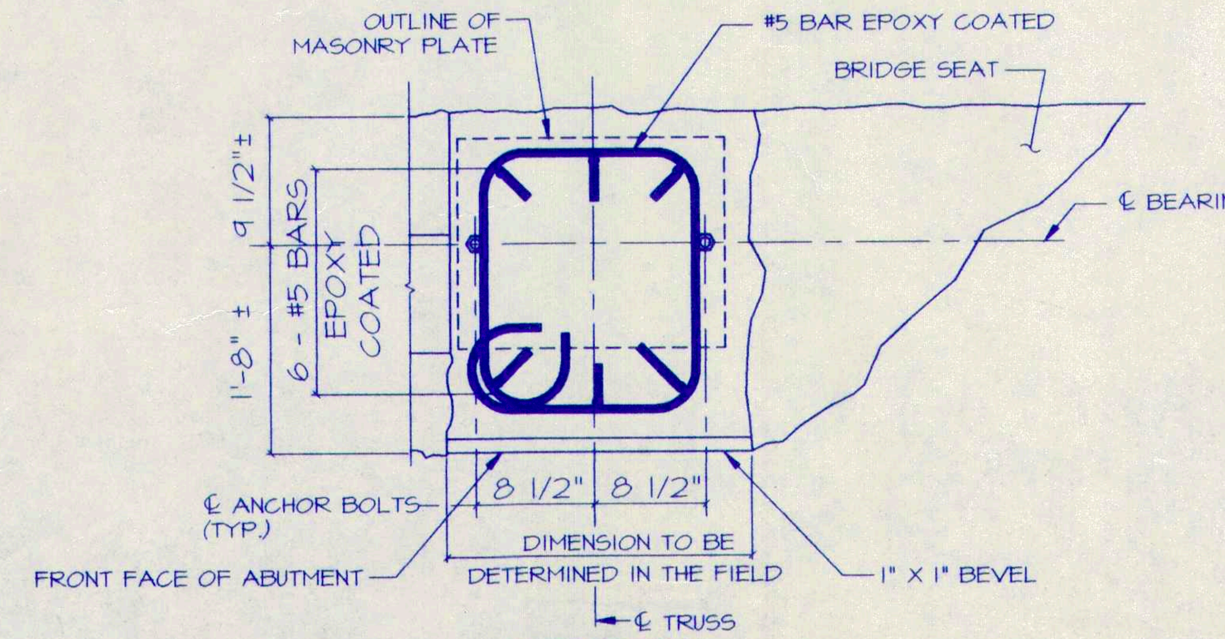
CONCRETE BACKWALL MODIFICATIONS

SCALE: 3/4"=1'-0"
(TYPICAL AT BOTH ABUTMENTS)

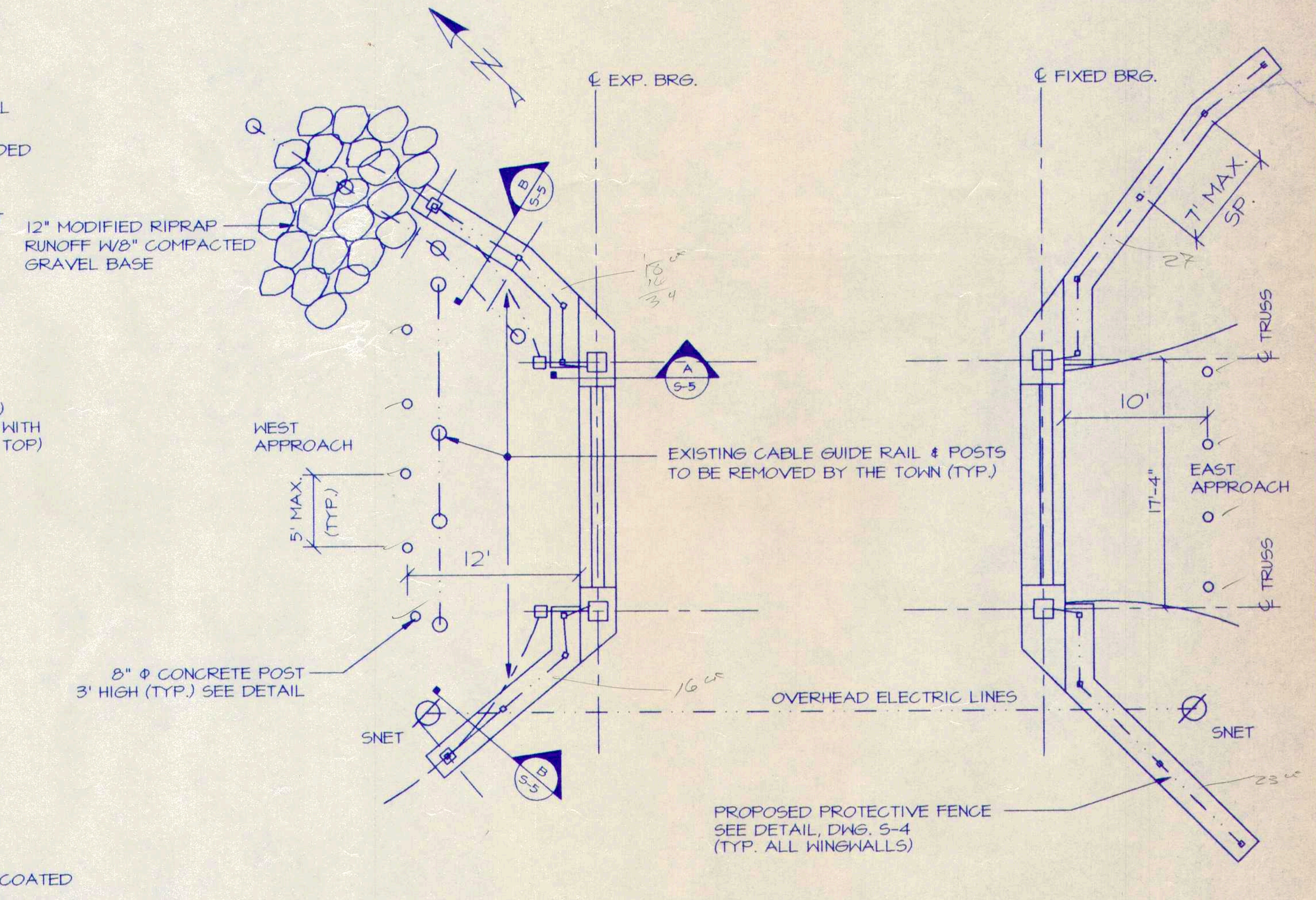


CONCRETE POST DETAIL

SCALE: 1/2"=1'-0"



SECTION SCALE: 1" = 1'-0" C S-5



ABUTMENT PLAN

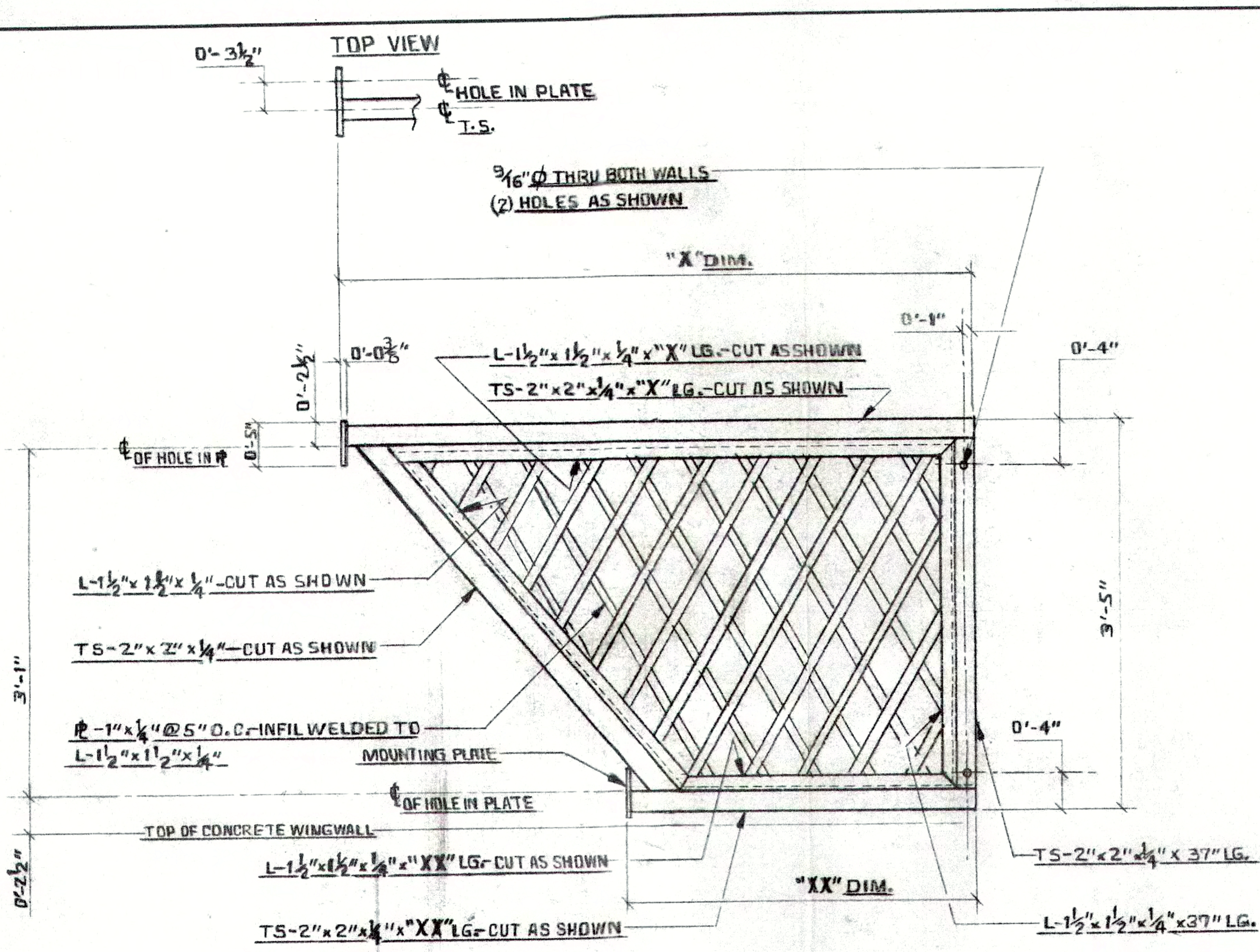
SCALE: 1/8" = 1'-0"

ABUTMENT DETAILS

REHABILATATION OF OLD BRIDGE ROAD BRIDGE
SIMSBURY, CONNECTICUT

Macchi Engineers
44 Gillett Street
Hartford, CT 06105
Phone: (203) 549-6190

DRAWN BY:	J.Z.
APPROVED BY:	J.B.
DATE:	8/01/94
SCALE:	AS NOTED
Drawing No.	S-5

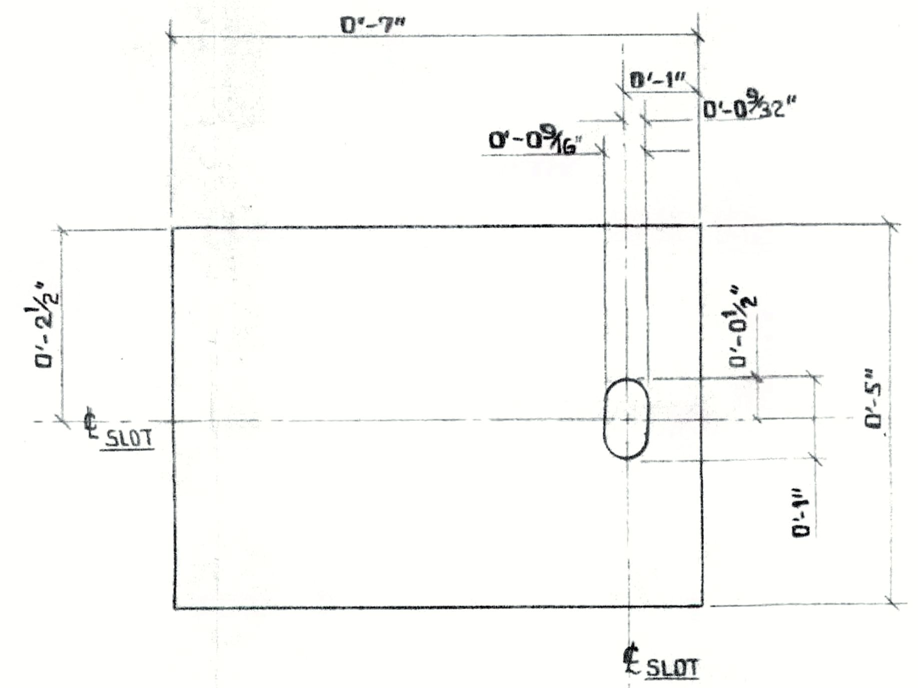


IDENT. NO.	X	XX
WRS-4	5'-10⅝"	3'-0¾"
WRS-11	5'-5½"	2'-7"

*

LEFT ANGLED WINGWALL RAIL SECTION

SCALE: 1"=1'-0"



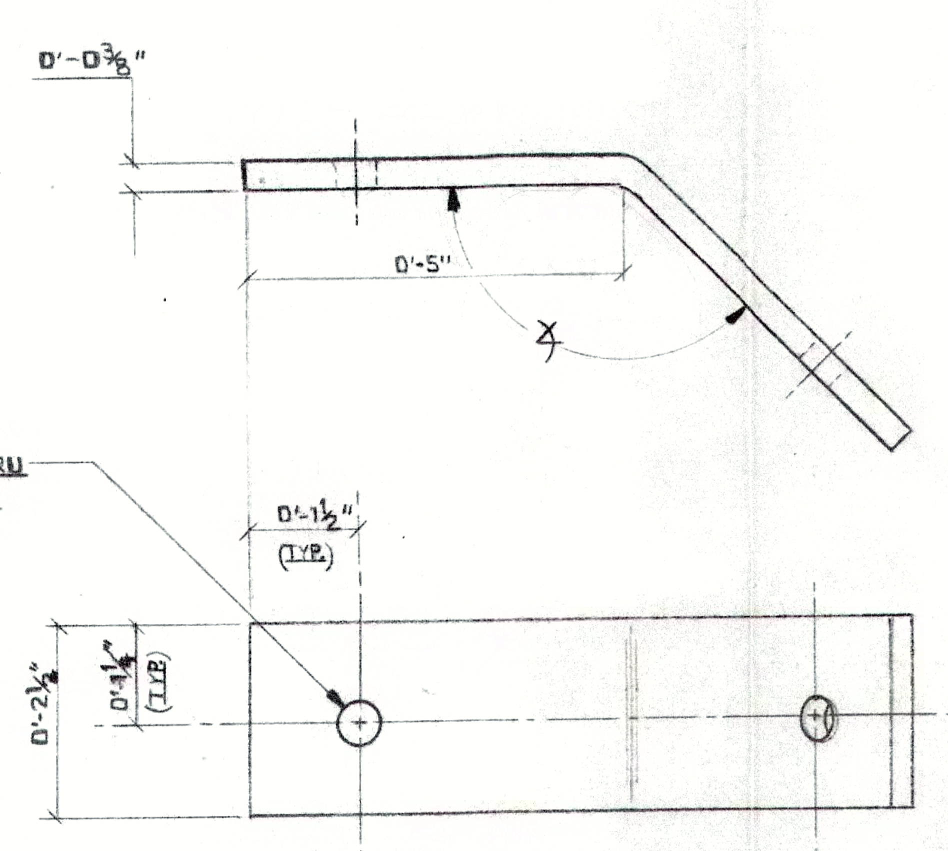
MOUNTING PLATE

SCALE: HALF

MATERIAL: 7"x5"x⅜" THK. PLATE

QTY. REQ'D.: 8 PCS.

* G.C. to Verify

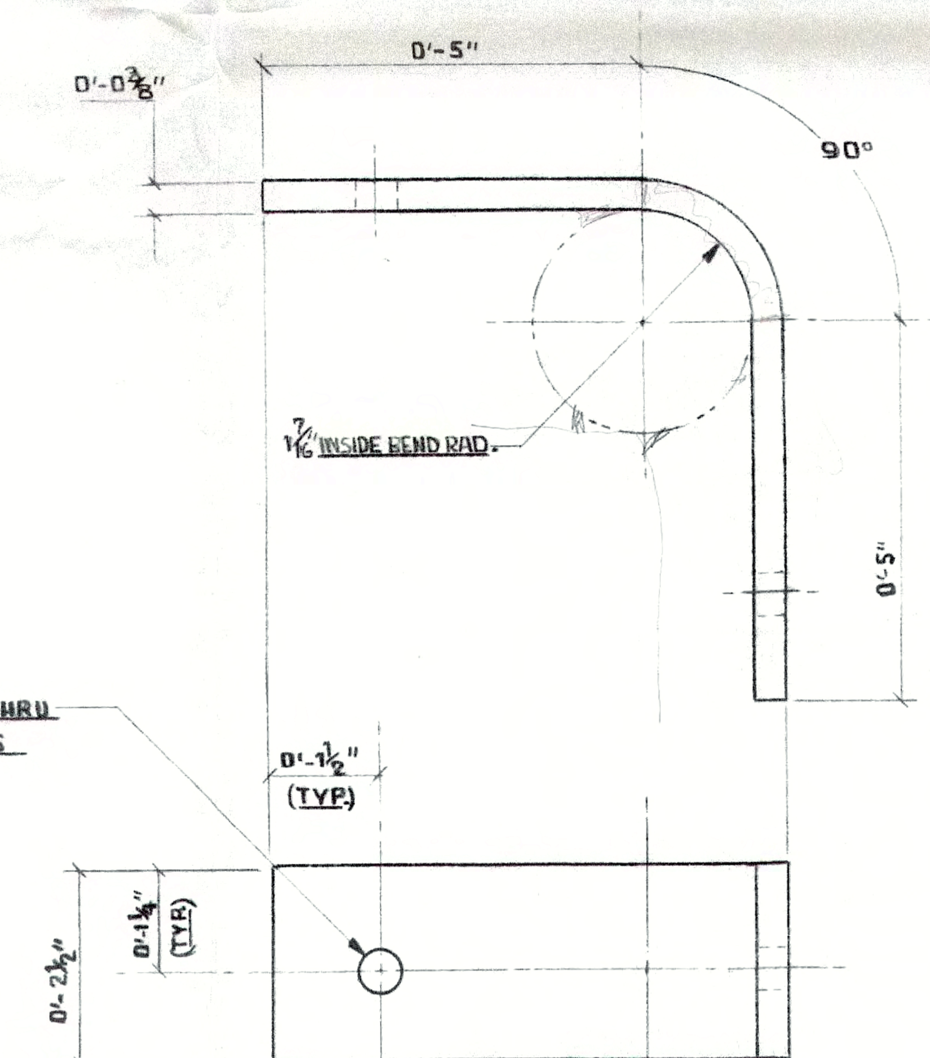


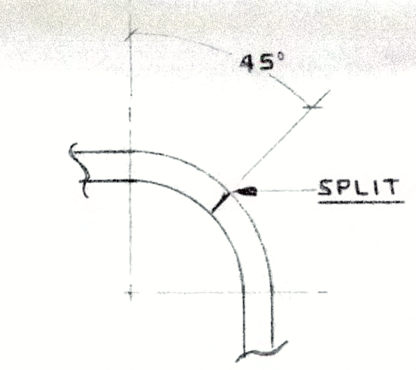
DRILLED PLATE

SCALE: HALF

MATERIAL: 2½" WIDE x ⅜" THK. x 10" LG. PLATE

ASSEMB. W/ POST NO.	QTY. REQ'D.	∡
P2	2	135°
P5	2	124°
P6	2	168¼°
P9	2	133°
P12	2	111°
ALL ENDS	8	FLAT





ASSEMB. W/ POST NO.	QTY. REQ'D.
P1	1 PC.
P7	PER POST
P10	(SPLIT AS
P13	SHOWN)

ASSEMB. W/ POST NO.	QTY. REQ'D.
P3	2
P4	2
P8	2
P11	2

CORNER PLATE

SCALE: HALF

MATERIAL: 2½" WIDE ⅜" THK. x 12.56" LG. PLATE

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MACCHI ENGINEERS
DATE: 9/28/95 BY: DCH

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SYM	SHT	QTY	DESCRIPTION	MATERIAL	UNIT WT.	TOTAL WT.

AISC AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC. FOUNDED 1921
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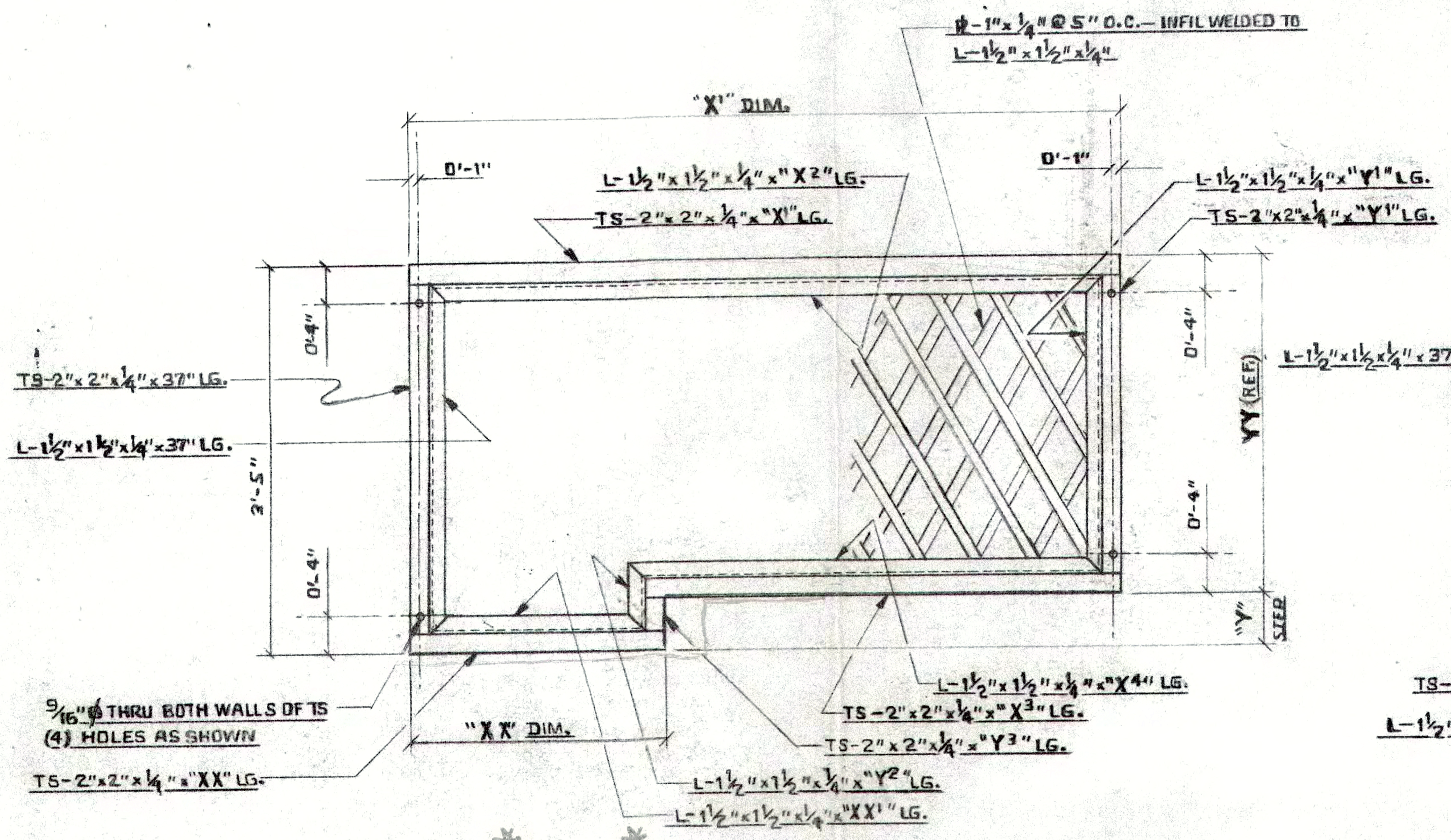
PRINT RECORD		
DATE	DRWG	ACTION
DATE	REVISIONS	

SOUTHINGTON METAL FABRICATING COMPANY
P.O. BOX 371 PLANTSVILLE, CONNECTICUT 06479

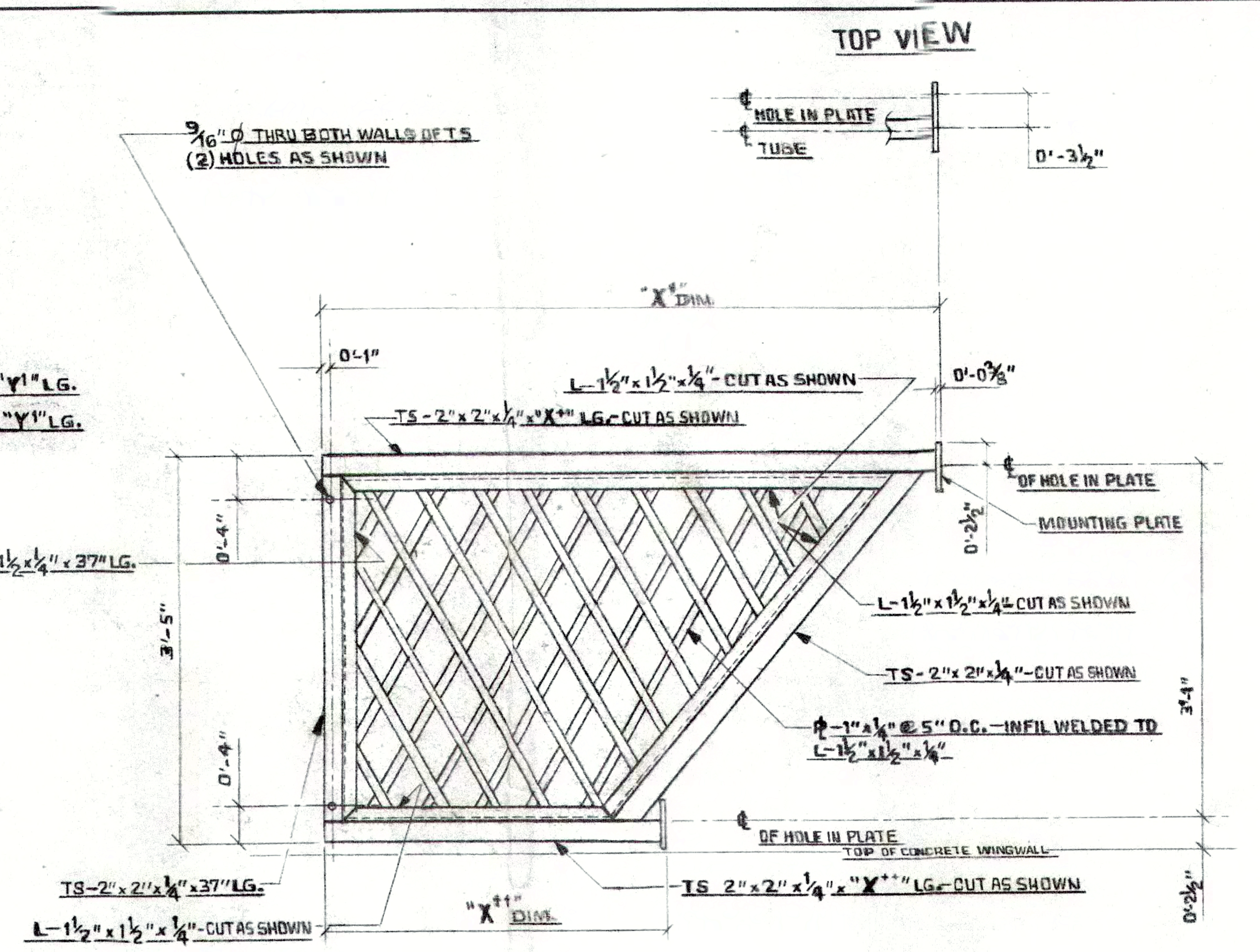
TITLE OLD BRIDGE ROAD - SIMSBURY, CT - BRIDGE RE-HAB. WINGWALL RAIL SECTION ASSEMBLY & DETAILS

SCALE AS NOTED	CHECKED BY	DRAWN BY
DATE 09-15-95		M.G. LEMKE
CONTRACTOR	ROTHA CONSTRUCTION	
PROJ. NUMBER SIMSBURY	JOB NUMBER 4999	DRWG. NUMBER M-6



IDENT. NO.	"X¹" DIM.	X²	X³	X⁴	XX DIM.	XX¹	"Y"	YY	Y¹	Y²	Y³
WRS-2	5'-5¾"	5'-1¾"	1'-11¼"	1'-9¼"	3'-8½"	3'-4½"	3"	3'-5"	2'-10"	3"	1"
WRS-5	4'-11¾"	4-7¾"	4'-11¼"	4'-9¾"	0'-2½"	—	6"	3'-5"	2'-7"	6"	4"
WRS-9	5'-4¼"	5'-0¼"	4'-10¾"	4'-7¾"	0'-8½"	0'-4½"	9½"	3'-5"	2'-6½"	6½"	4½"
WRS-12	5'-1⅛"	4'-9⅛"	1'-8⅝" 1'-7⅝"	1'-6⅝"	3'-7½"	3'-3½"	3"	3'-5"	2'-10"	3"	1"

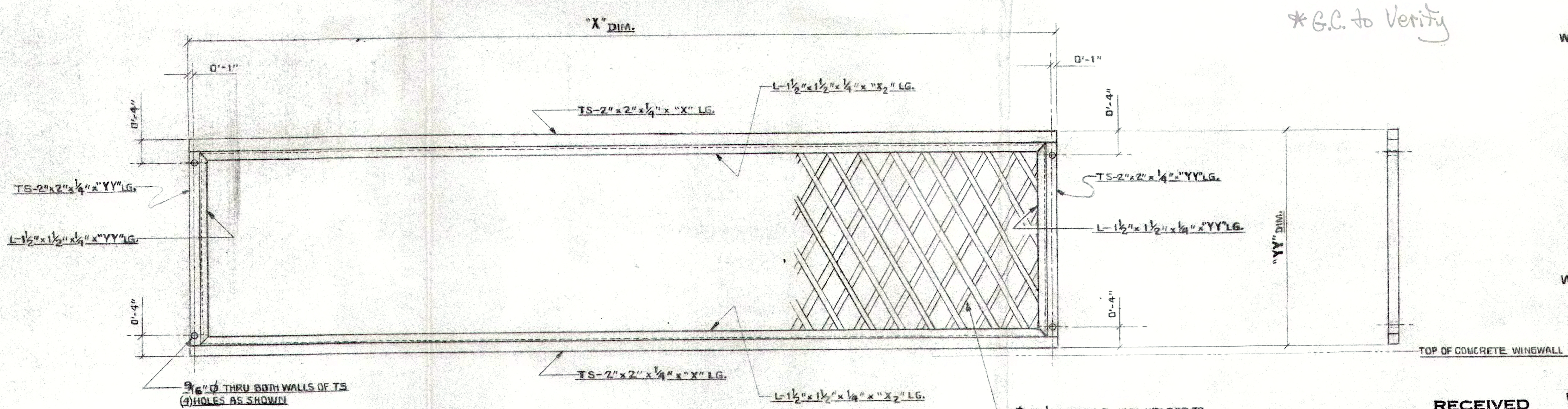
STEPPED WINGWALL RAIL SECTION
SCALE: 1" = 1'-0"



IDENT. NO.	X⁺	X⁺⁺
WRS-3	5'-6½"	2'-7¾"
WRS-8	5'-7¼"	2'-8⅜"

RIGHT ANGLED WINGWALL RAIL SECTION
SCALE: 1" = 1'-0"

*G.C. to Verify



IDENT. NO.	"X" DIM.	"X₂"	"YY"
WRS-1	14'-0"	13'-8"	3'-2"
WRS-6	15'-0"	14'-8"	2'-11"
WRS-7	5'-2"	4'-10"	2'-11"
WRS-10	15'-8½"	15'-4½"	2'-10½"
WRS-13	13'-6½"	13'-2½"	3'-2"

3'-5"

STRAIGHT WINGWALL RAIL SECTION
SCALE: 1" = 1'-0"

See Revised Section on Attached Sheet 2/4.

	SYM	SHT	QTY	DESCRIPTION	MATERI
WRS-13			A/R	℄-1"x¼"	
			2	L-1½"x1½"x¼"x3'-2" LG.	
			2	L-1½"x1½"x¼"x13'-2½" LG.	
			2	TS-2"x2"x¼"x3'-2" LG.	
			2	TS-2"x2"x¼"x13'-6½" LG.	
WRS-10			A/R	℄-1"x¼"	
			2	L-1½"x1½"x¼"x2'-10½" LG.	
			2	L-1½"x1½"x¼"x15'-4½" LG.	
			2	TS-2"x2"x¼x2'-10½" LG.	
			2	TS-2"x2"x¼"x15'-8½" LG.	
WRS-7			A/R	℄-1"x¼"	
			2	L-1½"x1½"x¼"x2'-11" LG.	
			2	L-1½"x1½"x¼"x4'-10" LG.	
			2	TS-2"x2"x¼"x2'-11" LG.	
			2	TS-2"x2"x¼"x5'-2" LG.	
WRS-6			A/R	℄-1"x¼"	
			2	L-1½"x1½"x¼"x2	
			2	L-1½"x1½"x¼"x14'-8" LG.	
			2	TS-2"x2"x¼"x2'-11" LG.	
			2	TS-2"x2"x¼"x15'-0" LG.	
WRS-1			A/R	℄-1"x¼"	
			2	L-1½"x1½"x¼"x3'-2" LG.	
			2	L-1½"x1½"x¼"x13'-8" LG.	
			2	TS-2"x2"x¼"x3'-2" LG.	
			2	TS-2"x2"x¼"x14'-0" LG.	

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REJECTED – SEE REMARKS
MACCHI ENGINEERS
DATE: 9/28/95 BY: DCH

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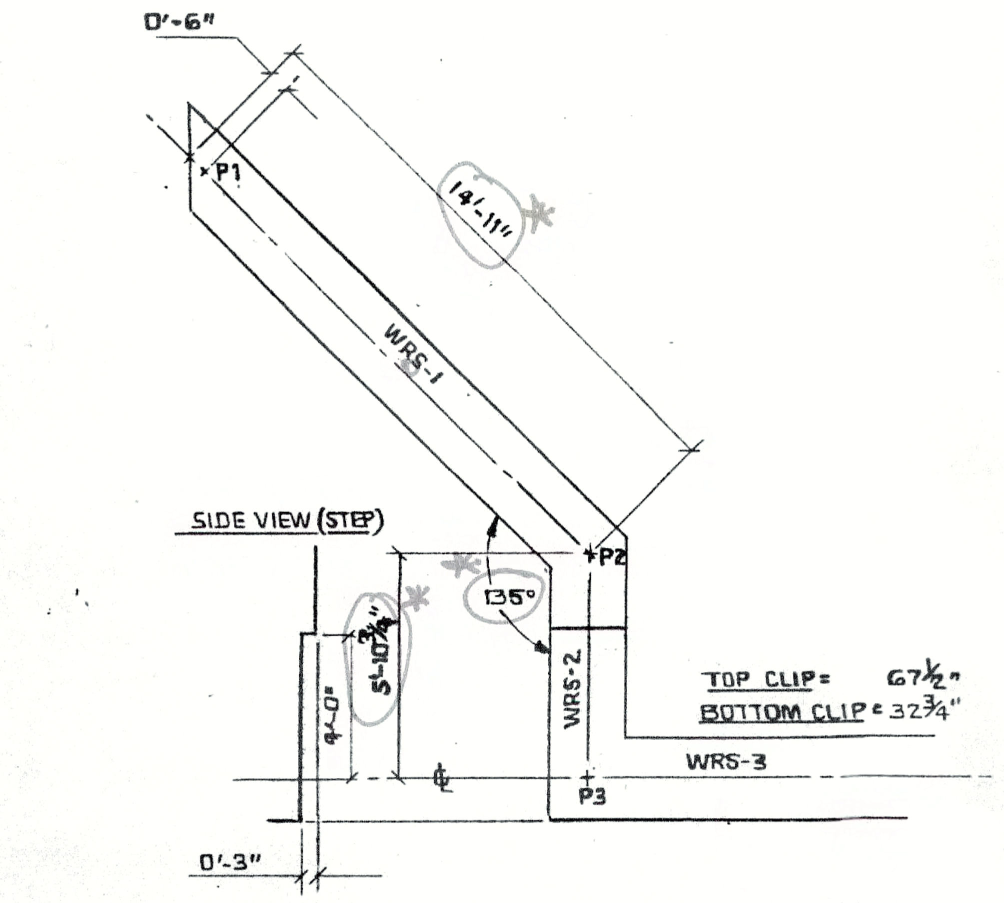
DATE	DRWG.	ACTION
DATE	REVISIONS	

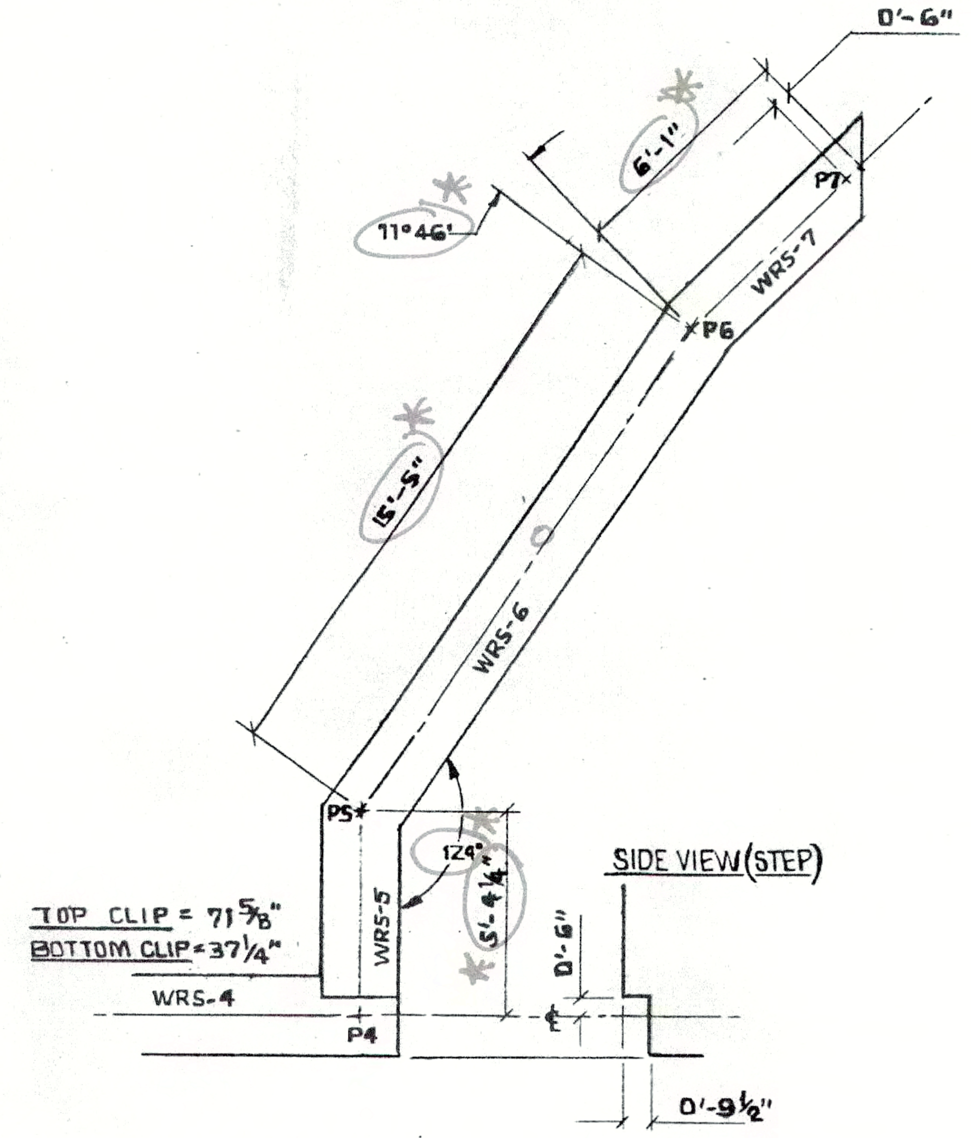
SOUTHINGTON ME
COM
P.O. BOX 371 PLANTSV

TITLE OLD BRIDGE ROAD-SIMSBURY,C
WINGWALL RAIL SECTION ASSEMBLI
SCALE AS NOTED
CHECKED BY
DATE 09-13-95
CONTRACTOR
ROTHA CONTRACTING
PROJ. NUMBER
JOB NU
49

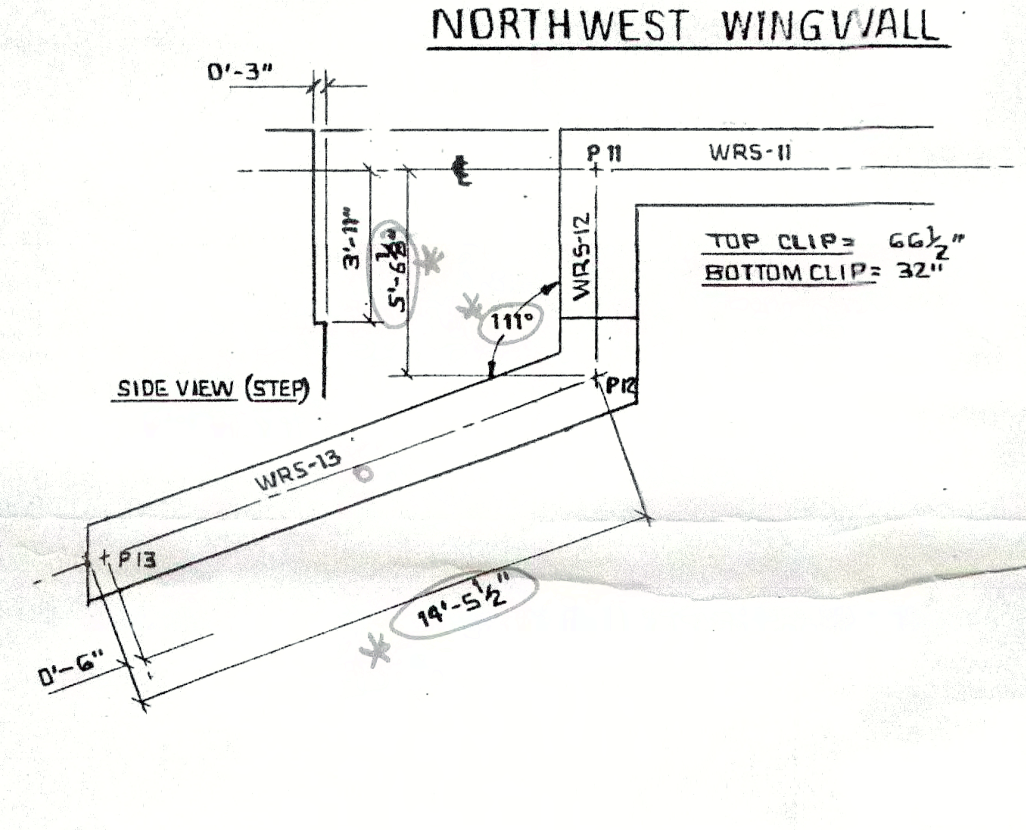
NOTE:
1. "P" = POST + NUMBER



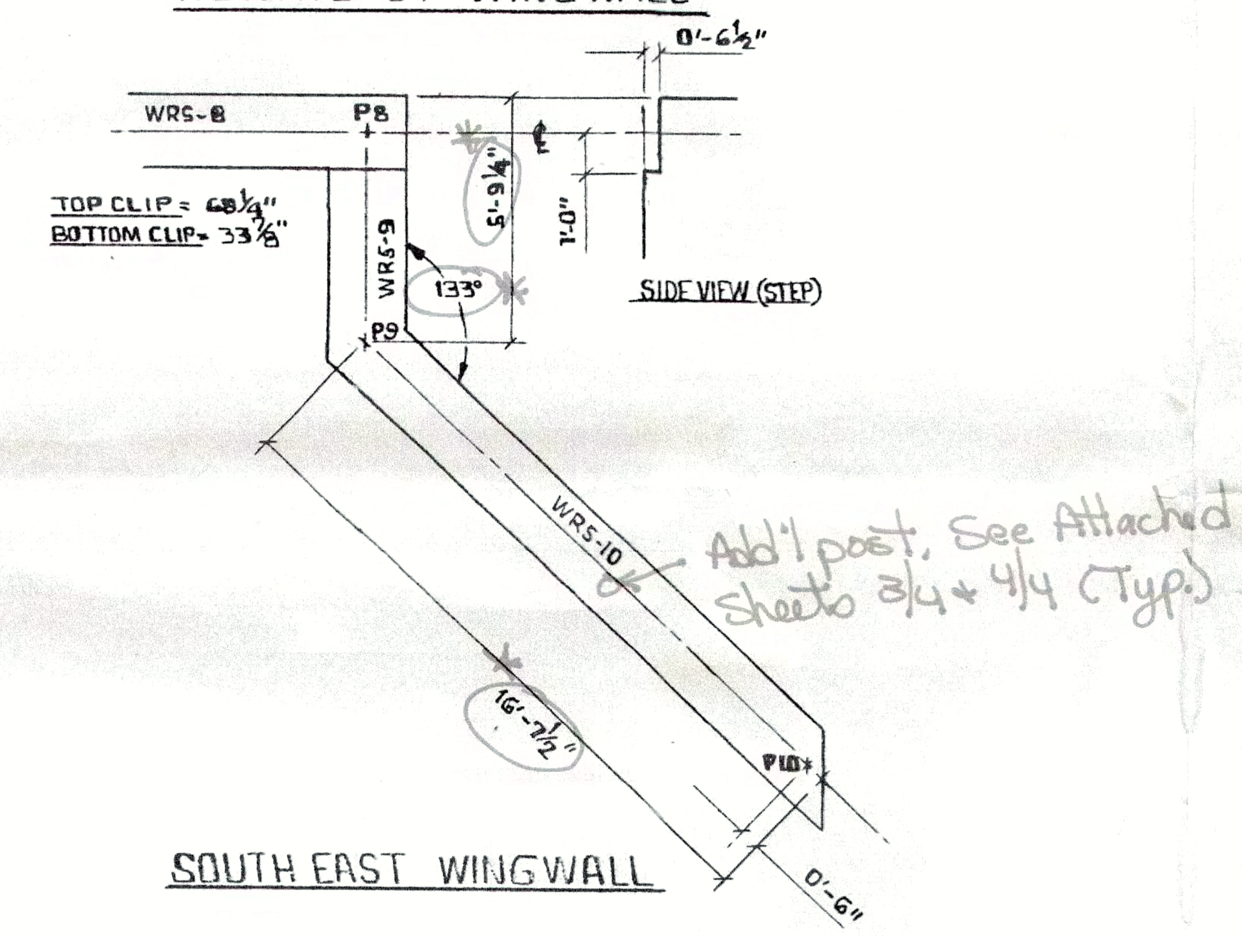
NORTHWEST WINGWALL



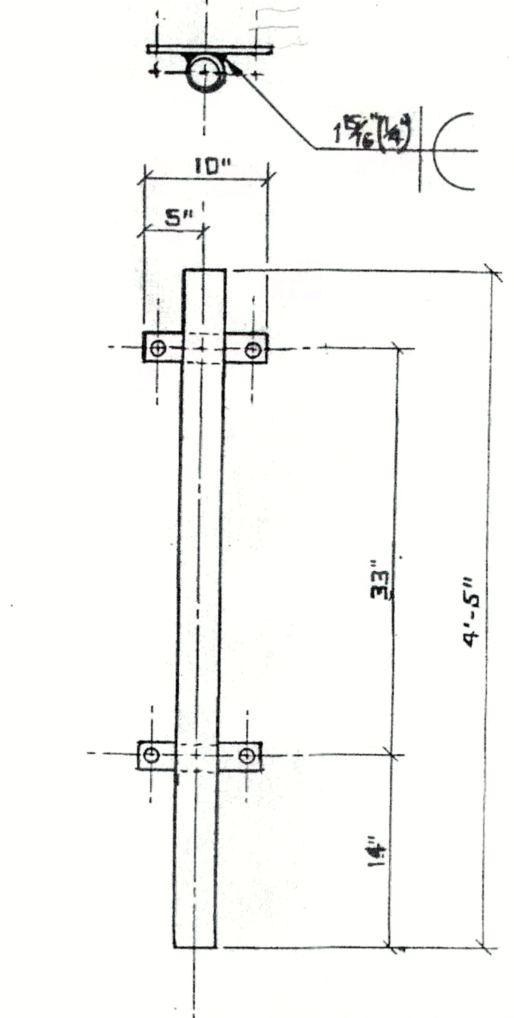
NORTHEAST WINGWALL



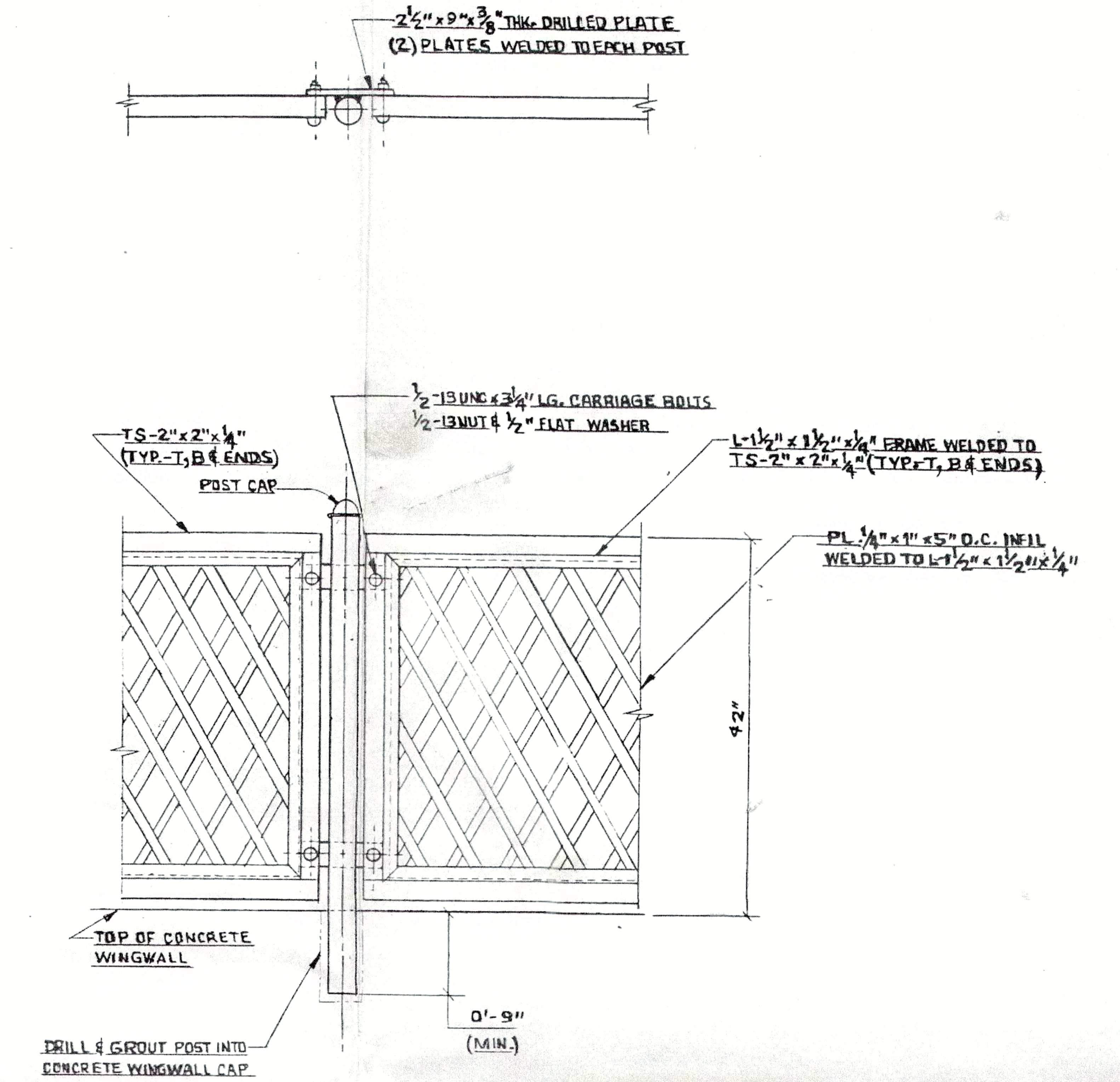
SOUTHWEST WINGWALL



SOUTH EAST WINGWALL



POST ASSEMBLIES



Pg 38-39

*G.C. to verify

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MACCHI ENGINEERS
DATE: 9/28/95 BY: DKH

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MACCHI ENGINEERS

THIS SUBMITTAL HAS BEEN REVIEWED AND APPROVED, AND TO THE BEST OF OUR KNOWLEDGE IS IN CONFORMANCE WITH THE REQUIREMENTS OF THE CONTRACT DOCUMENTS.
SUBMITTAL # ___
SPEC SECT ___
DATE 9-26-95
ROTHA CONTRACTING COMPANY, INC.
BY ___

AMERICAN INSTITUTE OF STEEL CONSTRUCTION INC. AISC FOUNDED 1921
ACTIVE MEMBER

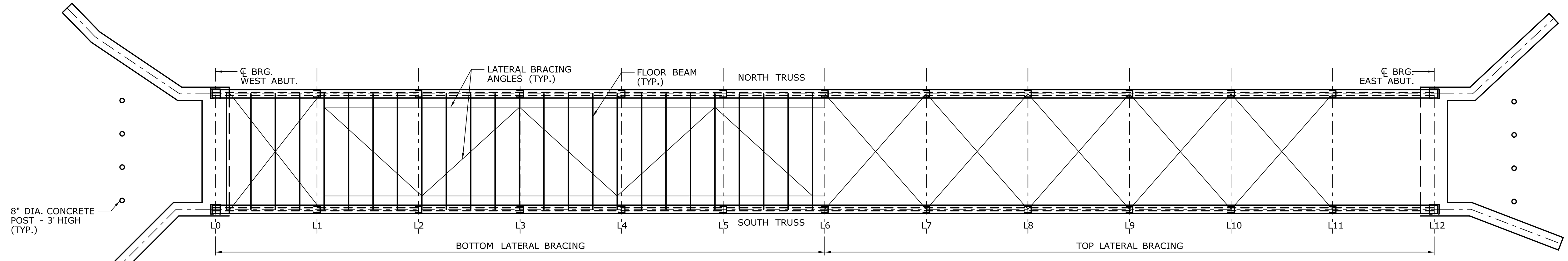
PRINT RECORD		
DATE	DRWG	ACTION
DATE	REVISIONS	

SOUTHINGTON METAL FABRICATING COMPANY
P.O. BOX 371 PLANTSVILLE, CONNECTICUT 06479

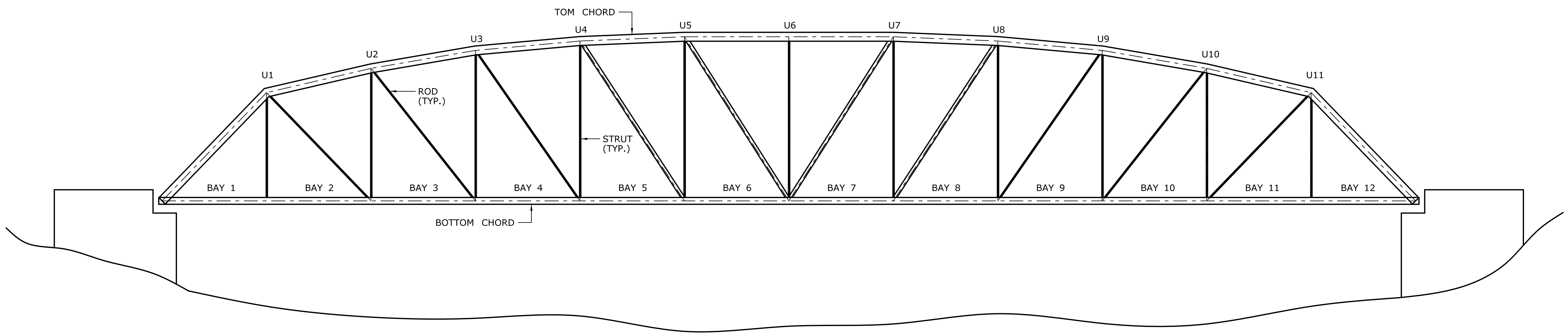
TITLE OLD BRIDGE ROAD - SIMSBURY, CT - BRIDGE REHAB. WINGWALL LAYOUT

SCALE ⅛" = 1'-0"	CHECKED BY	DRAWN BY M. G. LEMKE
DATE 09-01-95		
CONTRACTOR QUAKER CORP.		
PROJ. NUMBER	JOB NUMBER 4999	DRWG. NUMBER M4

Appendix D - Steel Repair Locations



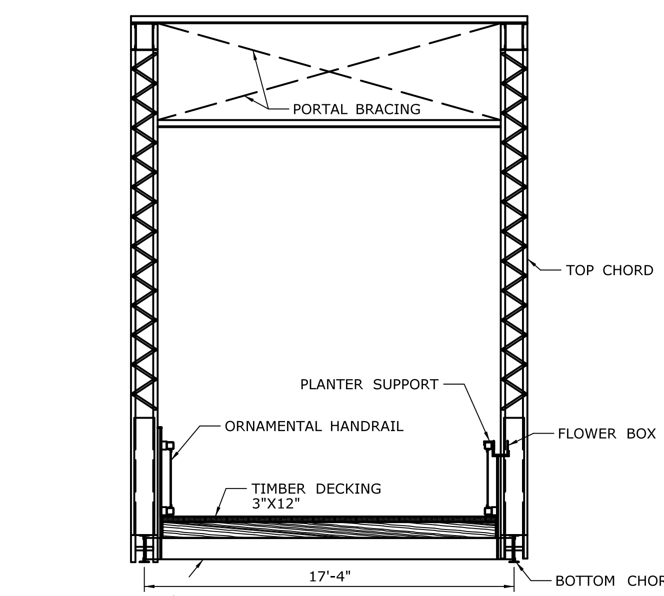
GENERAL PLAN



ELEVATION

SCOPE OF WORK:

1. REPAIR DAMAGED TIMBER DECK.
2. PERFORM STEEL REPAIRS.
3. REPAIR ORNAMENTAL BRIDGE RAILING ON THE BRIDGE AND APPROACHES.
4. SPOT PAINT STRUCTURAL STEEL/ ABRASIVE BLAST CLEANING AND PAINTING OF ENTIRE STRUCTURE.
5. JACK THE BRIDGE AND REPAIR BEARING ASSEMBLIES AND RESTORE TO ORIGINAL CONDITION.
6. ABRASIVE BLAST CLEAN AND PAINT BEARING ASSEMBLIES.
7. INSTALL LATERAL RESTRAINTS AT BEARINGS.
8. REMOVE AND REINSTALL PLANTER SUPPORTS & PIPING SYSTEM.
9. REPLACE JOINT SEAL AT ABUTMENT JOINTS.



TYPICAL SECTION

PROJECT TITLE:

REHABILITATION OF BRIDGE NO. 03984
OLD DRAKE HILL ROAD BRIDGE (FLOWER BRIDGE)
OVER FARMINGTON RIVER
SIMSBURY, CT

GM2 ASSOCIATES, INC.
115 GLASTONBURY BLVD.
GLASTONBURY, CT 06033

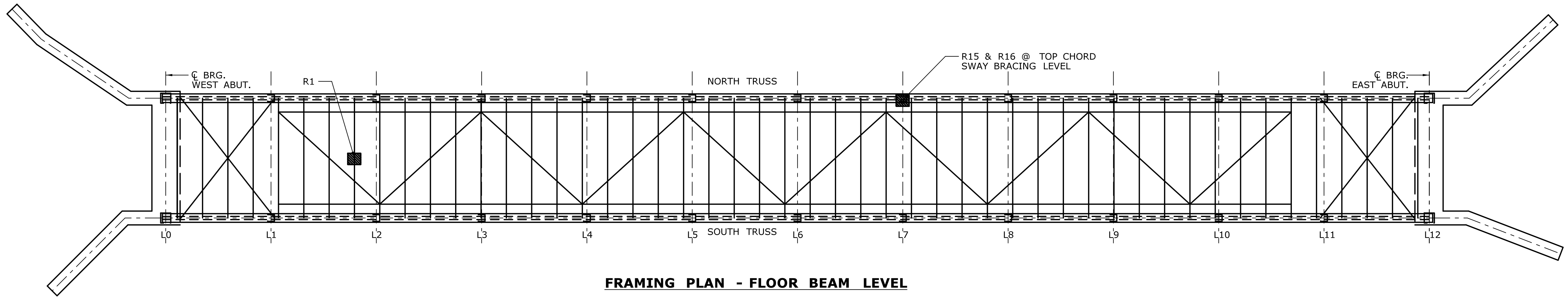
DRAWING TITLE:

GENERAL PLAN & ELEVATION

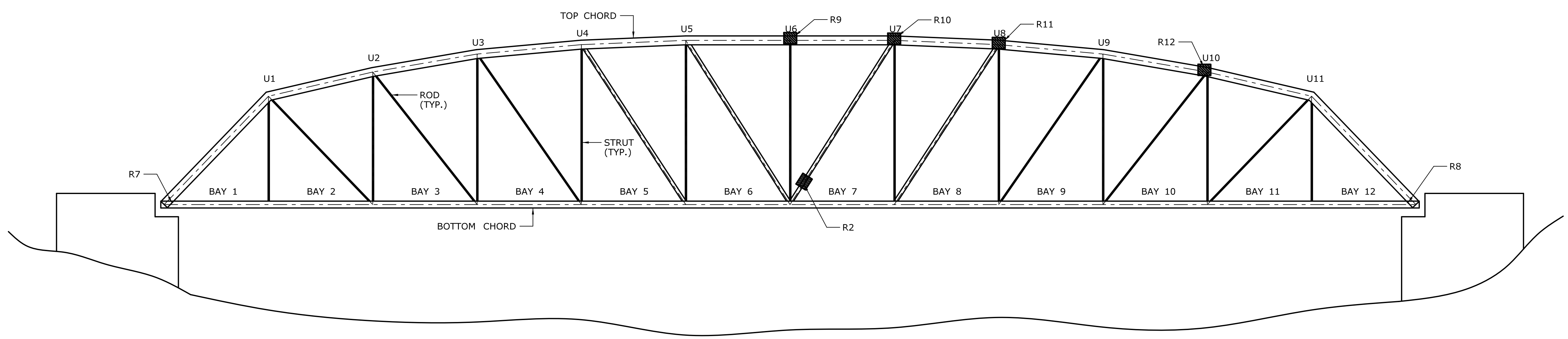
PROJECT NO. **TBD**

SHEET NO. **RSR-D01**

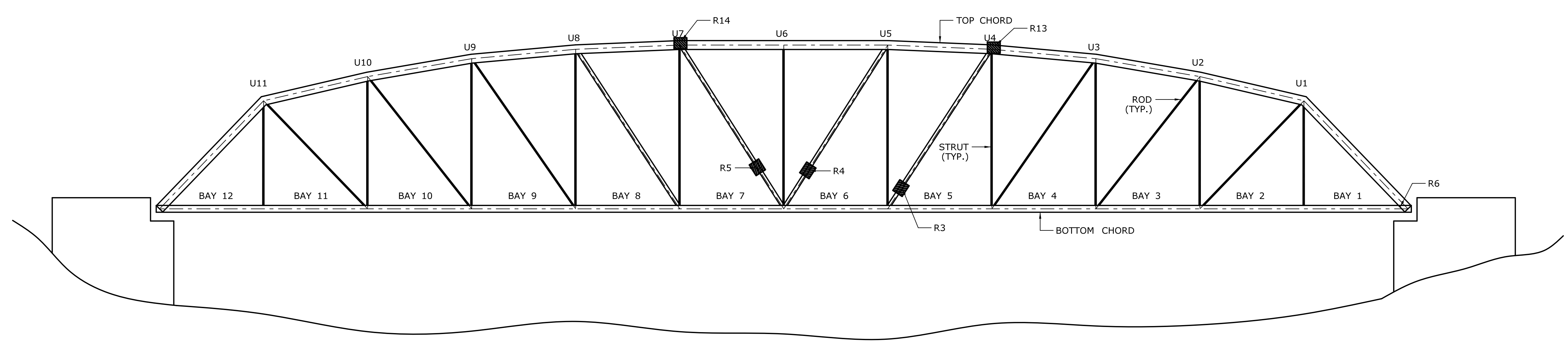
DATE **03/08/19**



FRAMING PLAN - FLOOR BEAM LEVEL



SOUTH TRUSS SOUTH ELEVATION



NORTH TRUSS NORTH ELEVATION

PROJECT TITLE:

REHABILITATION OF BRIDGE NO. 03984
OLD DRAKE HILL ROAD BRIDGE (FLOWER BRIDGE)
OVER FARMINGTON RIVER
SIMSBURY, CT

SIMSBURY CONNECTICUT 1670

GM2 ASSOCIATES

GM2 ASSOCIATES, INC.
115 GLASTONBURY BLVD.
GLASTONBURY, CT 06033

DRAWING TITLE:

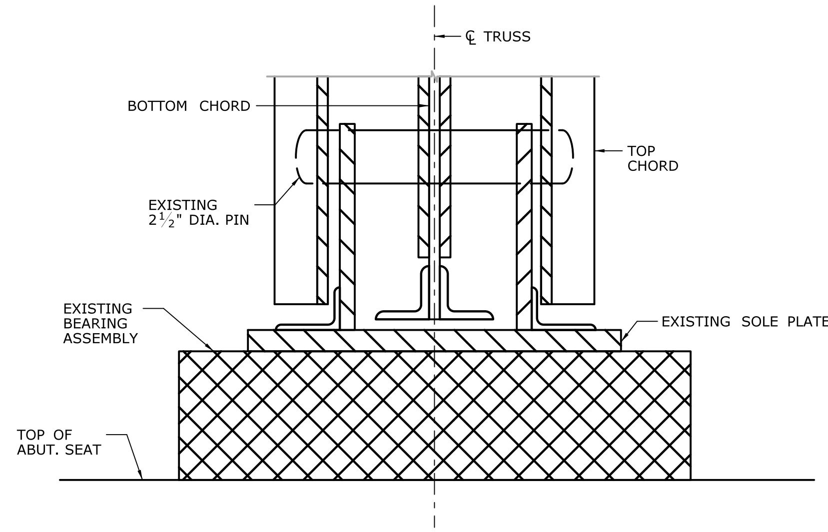
STEEL REPAIR LOCATIONS

PROJECT NO. **TBD**

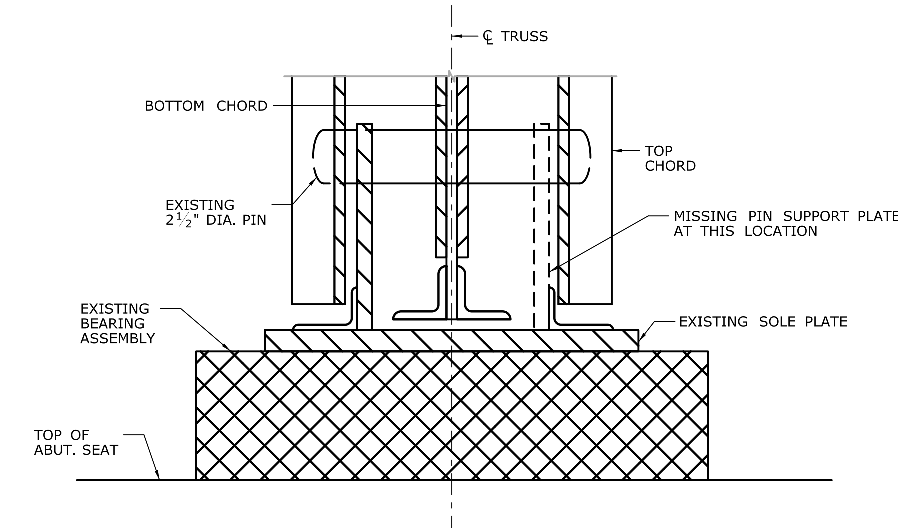
SHEET NO. **RSR-D02**

DATE **03/08/19**

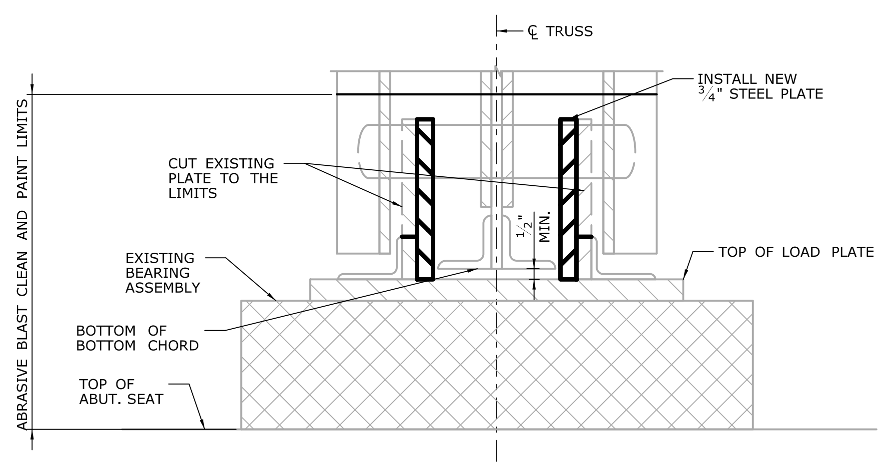
Appendix E – Conceptual Repair Details



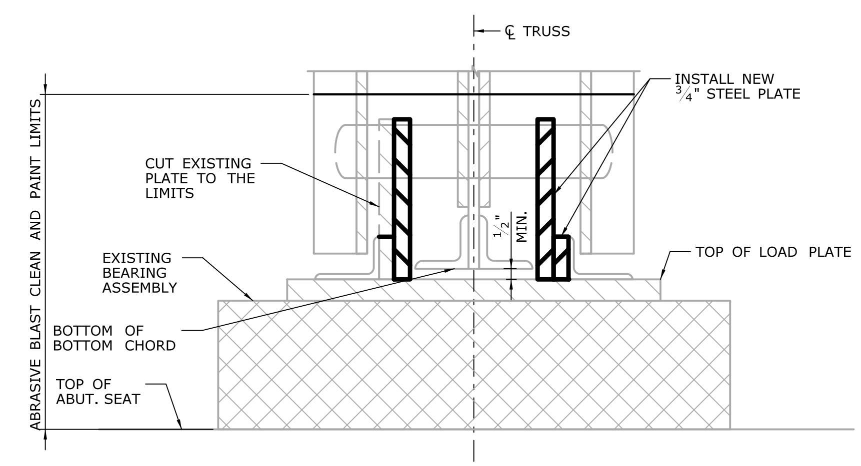
EXISITNG DETAIL AT BEARINGS



EXISITNG DETAIL AT NORTHWEST BEARING



PROPOSED DETAIL AT EXPANSION BEARING



PROPOSED DETAIL AT EXPANSION BEARING

PROJECT TITLE:

REHABILITATION OF BRIDGE NO. 03984
OLD DRAKE HILL ROAD BRIDGE (FLOWER BRIDGE)
OVER FARMINGTON RIVER
SIMSBURY, CT

SIMSBURY CONNECTICUT 1670

GM2 ASSOCIATES

GM2 ASSOCIATES, INC.
115 GLASTONBURY BLVD.
GLASTONBURY, CT 06033

DRAWING TITLE:

BEARING REPAIR SCHEMATIC

PROJECT NO.
TBD

SHEET NO.
RSR-D03

DATE
03/08/19

Appendix F - Bridge Inspection Report

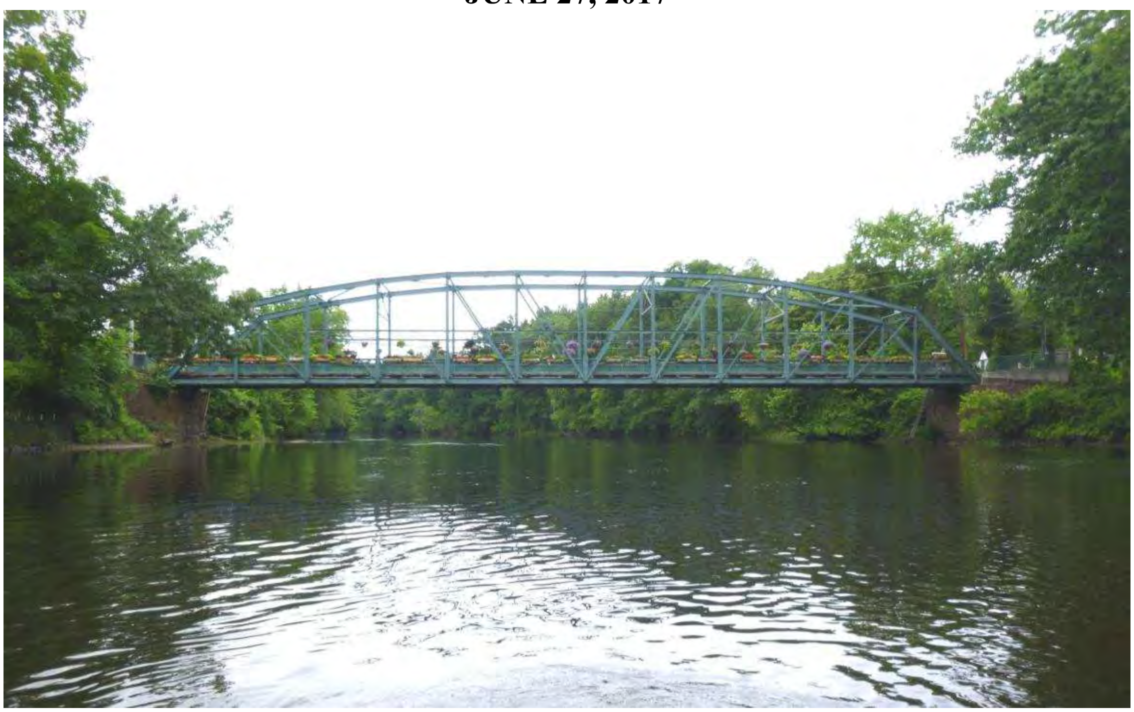
BRIDGE SAFETY INSPECTION

BRIDGE NO. 03984

OLD DRAKE HILL ROAD BRIDGE (FLOWER BRIDGE)
OVER
FARMINGTON RIVER

SIMSBURY, CONNECTICUT

JUNE 27, 2017



Digitally signed by Faisal Aziz
DN: E=faziz@gm2inc.com, CN=Faisal Aziz, O="GM2 Associates, Inc.", L=Glastonbury, S=CT, C=US
Contact Info: 8606591416 x132
Date: 2017.08.09 14:25:16-04'00'

Prepared By:

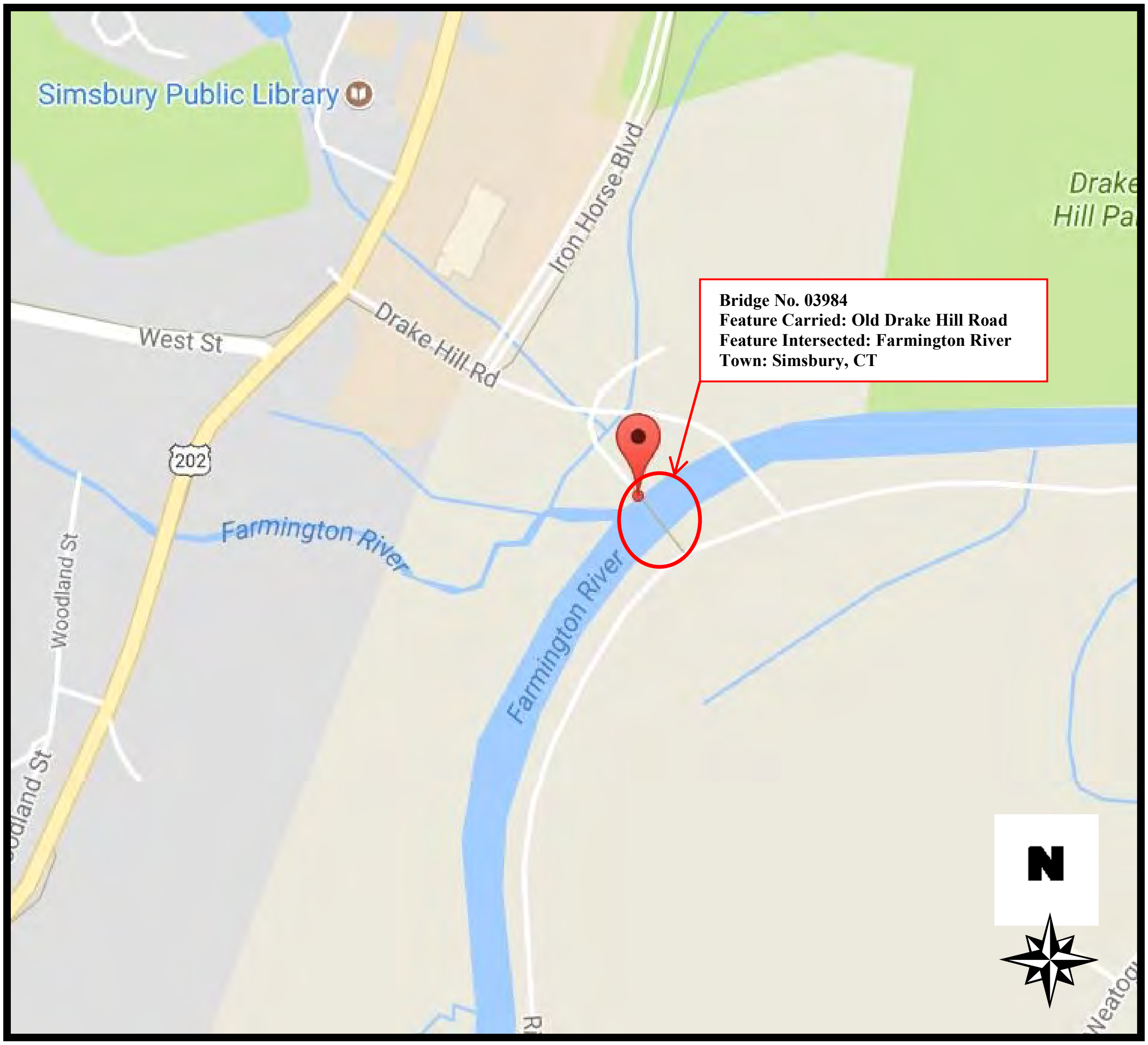
GM2 ASSOCIATES

115 GLASTONBURY BLVD.
GLASTONBURY, CT 06033

Prepared For:

TOWN OF SIMSBURY
DEPARTMENT OF ENGINEERING
SIMSBURY, CT

LOCATION MAP



TOWN OF SIMSBURY, CT
DEPARTMENT OF ENGINEERING

INSPECTION REPORT TRANSMITTAL FORM

STRUCTURE NO. 03984 TOWN SIMSBURY

Inspectors AKC, BJS, SR Date 06/27/2017

TABLE OF CONTENTS

EXECUTIVE SUMMARY

Bridge No. 03984 carries Old Drake Hill Road Bridge (Flower Bridge) over Farmington River in Simsbury, Connecticut. The overall length of the structure is 183 feet and curb to curb width is 15 feet. This steel thru-truss bridge structure is comprised of two Parker trusses and was built in 1892, with structural repairs performed in 1977, and further rehabilitated in 1993 for pedestrian traffic. The repairs and rehabilitation encompassed weldment of the gusset plates atop the bottom chord member, addition of channel sections to the truss vertical members, steel plates weldment to the truss diagonal members and gusset plates along with new timber deck planks installation. Currently, the bridge is closed to any vehicular traffic; and is open to carry pedestrian and bicycle traffic only.

During this in-depth inspection, completed in June 2017, the footbridge was found to be in “fair” condition. Also, all accessible truss pins were checked for deficiencies, utilizing Ultrasonic Testing (UT), and found to be in “acceptable” condition.

The structure is listed on the National Register of Historic Places in Connecticut; signifying it being a vital asset to the community, and dictating the need to preserve its historic character.

The deficiencies found on the bridge are as follows:

Deck: (Rated – 6 "Satisfactory")

No major deficiencies.

Superstructure: (Rated – 5 "Fair")

1. The vertical gusset plates at the truss bearings exhibit section loss down to 1/8" remaining with rust holes up to 1" x 1/4". In addition, the expansion bearing for the north truss at West Abutment is missing a vertical gusset plate.
2. The truss bottom chords exhibit section loss down to 1/16" remaining with edge rust holes, primarily in the bottom interior angles. The maximum resulting section loss in the bottom chord is approximately 5% (critical zone).
3. There are areas of pack rust up to 1/2" thick between the truss elements at random locations.
4. The channel web of truss vertical members exhibit areas of painted over pitting up to 1/16" deep with up to 1/2" x 3/8" rust holes (less than 5% section loss).
5. Channel webs of truss diagonal members exhibit random rust holes up to 4" diameter, primarily around the bolted tie-rod attachment between the channels (up to 16% section loss in the diagonal member; and 32% section loss in the channels).
6. Isolated locations in the sway bracing exhibit section loss up to full width x 6" long x down to knife edge remaining with up to 1" wide x 1/2" long rust holes.

Note: A condition assessment of the superstructure, in compliance with CTDOT Bridge Inspection Manual and National Bridge Inspection Standards, warrants an overall condition rating of “4 – Poor” or lower. However, a “5 – Fair” condition rating has been assigned due to the structure’s classification as a pedestrian facility only (no vehicular traffic permitted).

<u>Substructure:</u> (Rated – 7 "Good")

No major deficiencies.

<u>Channel and Channel Protection:</u> (Rated – 6 "Satisfactory")

No major deficiencies.

<u>Recommendations:</u>

Based on the extent of deterioration observed on the superstructure steel during this footbridge safety inspection, performed in June 2017, a reanalysis of the structure is recommended to ascertain its safe load capacity and evaluate feasibility of its possible reopening to any vehicular traffic, including the maintenance vehicles.

GM2 also recommends programming this footbridge for rehabilitation, including zone painting, to preserve its historic character and maximize its useful service life.

STATE OF CONNECTICUT
DEPARTMENT OF TRANSPORTATION
BRIDGE SAFETY & EVALUATION

STRUCTURE EVALUATION

SHEET 1 OF 2 FORM BRI-19 REV 10/00

SHEET 1 OF 2 (INSP. REPORT)

Bridge Number: 03984

Inspected By: AMIT KC, BRIAN SWANSON & SAIPAVAN RALLABHANDHI

Sufficiency Rating:
Previous Inspection Date: 05/01/1988

BS&E Received ☐ Data Entry By:
Copies Made ☐ Data Entry Date:

90) Inspection Date: 0 6 2 7 1 7 | Inspection Team: | 91) Frequency: | Class:
Indepth Insp: | Deck Survey: | Access: 5 0 | Flagman:

RED FLAG

CRITICAL FEATURE INSPECTIONS

	Type	Frequency	Team	Date
Fracture:				
Uwater:				
Special:				

IDENTIFICATION

Bridge Name: OLD DRAKE HILL FLOWER BRIDGE
Town Name: SIMSBURY — Town Code: 128
5) Inventory Route:
- A) Record Type: 1 — D) Route Number: 00000
- B) Signing Prefix: 8 — E) Directional Suffix: 0
- C) Level of Service: 0

6) Feature Intersected: FARMINGTON RIVER
7) Facility Carried: OLD DRAKE HILL ROAD BRIDGE
9) Location: 0.3 M E OF ROUTE 202
11) Milepoint: 0000000 Miles
16) Latitude: 41 deg 52 min 08 sec — deg min . sec
17) Longitude: -72 deg 47 min 59.8 sec — deg min . sec
98) Border Bridge:
- A) State Code: — B) Percent Responsibility: %
- C) Border Town Name:

99) Border Bridge Structure No:

STRUCTURE TYPE AND MATERIAL

43) Structure Type, Main:
- A) Material: 3 — B) Design Type: 10

44) Structure Type, Approach:
- A) Material: 0 — B) Design Type: 00

45) Number of Spans, Main Unit: 001
46) Number of Approach Spans: 0000
107) Deck Structure Type: 8
108) Wearing Surface/Protective System:
- A) Type of Wearing Surface: 0
- B) Type of Membrane: 0
- C) Type of Deck Protection: 0

AGE AND SERVICE

27) Year Built: 1892 — 106) Year Reconstructed: 1993
42) Type of Service:
- A) On: 3 — B) Under: 5

28) Number of Lanes:
- A) On: 01 — B) Under: 00

29) Average Daily Traffic: N/A — Half ADT?: N/A
109) Percent Truck: N/A %
30) Year of ADT: N/A
19) Bypass, Detour Length: N/A miles

GEOMETRIC DATA

48) Length of Max Span: 183 ft
49) Structure Length: 185 ft
50) Curb or Sidewalk Widths:
- A) Left: 000 ft — B) Right: 000 ft

51) Brg Rdwy width,curb-curb: 15.2 ft
52) Deck Width, Out-Out: 18.5 ft
32) Approach Roadway Width: N/A ft
33) Bridge Median: 0
Deck Area: 3422 sqft
34) Skew Angle: 0 deg
35) Structure Flared: 0
10)Inv. Rte. Min. Vert Clearance: 20 ft 0 in
47) Log Inv. Rte. Total Horiz Clr.: 15.2 ft
47) RLog Inv. Rte. Total Horiz. Clr.: 0 ft
53) Min Vert Clearance Over Bridge: ft in
54) Min Vert Under Clearance: N Ref 0000 ft in Ref
55) Min Lat Under Clearance on Right: N Ref 0000 ft Ref
56) Min Lat Under Clearance on Left: 000 ft

BRIDGE COMMENTS

ITEM 29 - PEDESTRIAN BRIDGE

STRUCTURE EVALUATION

SHEET 2 OF 2 FORM BRI-19 REV 10/00

SHEET 2 OF 2 (INSP. REPORT)

Bridge Number	03984
NBIS Length	
Town Name	SIMSBURY
Facility Carried	OLD DRAKE HILL ROAD BRIDGE
Feature Crossed	FARMINGTON RIVER

Inspected By: AMIT KC, BRIAN SWANSON & SAIPAVAN RALLABHANDHI

CLASSIFICATION

112) NBIS Bridge Length	
104) Highway System	
26) Functional Class	
100) Defense Highway	
101) Parallel Structure	
102) Direction of Traffic	
103) Temporary Structure	
110) Designated National Network	
20) Toll	
21) Maintain	03
22) Owner	03
Report Class	
37) Historical Significance	1

WATERWAY

DrainageBasinCode
38) Navigation Control
39) Navigation Vert Clr.
40) Navigation Horiz Clr.
116) Vert-Lift Brg Nav Min
111) Pier Abutment Protection

PROPOSED IMPROVEMENTS

75A) Type of Work Proposed
75B) Work Done By
76) Length of Struct. Improvement ft ft
94) Bridge Improvement Cost $
95) Roadway Improvement Cost $
96) Total Project Cost $
97) Year of Improvement Cost Est.
114) Future ADT
115) Year Future ADT
List No. Project No. Advertised

POSTED SIGNS & UTILITIES

Other Posted Signs 1			
Other Posted Signs 2			
Actual P.L. Single Unit Truck	tons	Actual P.L. 4Axle Truck	tons
Rec. P.L. Single Unit Truck	tons	Rec. P.L. 4Axle Truck	tons
Actual P.L. Semi-TrailerTruck	tons	Actual P.L. 3S2 Truck	tons
Rec. P.L. Semi-TrailerTruck	tons	Rec. P.L. 3S2 Truck	tons
Rec. P.L. All Vehicles	tons	Actual P.L. All Vehicles	tons
Posted Vert Clearance On Bridge	ft in		ft In
Posted Vert UnderClearance	ft in		ft In
Posted Speed Limit	mph		mph
Utility			

LOAD RATING AND POSTING

31) Design Load		Evaluation Code	
63) Operating Rating Type		Year of Evaluation	
64) Operating Rating		70) Bridge Posting	
65) Inventory Rating Type		41) Structure Status	K
66) Inventory Rating			

CONDITION

	Rating	By
58) Deck	6	AKC
59) Superstructure	5	AKC
60) Substructure	7	AKC
61) Channel & Chan. Protection	6	AKC
62) Culverts	N	AKC

APPRAISALS

	Rating	By
67) Structure Evaluation	5	FA
68) Deck Geometry	N	AKC
69) Under Clear Vert & Horiz	N	AKC
71) Waterway Adequacy	7	AKC
72) Approach Rdwy Alignment	N	AKC
113) Scour Critical		

Items 58 Thru 72 Checked By:

36) Traffic Safety Features:

A) Bridge Railings	N
B) Transitions	N
C) Approach Guardrail	N
D) Approach Guardrail End	N

OTHER FEATURES

Fence Required		Barrel Ladder	
Fence Present		Stand Pipes	
Fence Height	ft . ft	Cat Walks	
Fence Type		Movable Inspection System	
Fence Material		Loose Concrete Checked?	
Fence Top Type			

INSPECTION COMMENTS

Proposed Next Indepth Insp Year
Senior
Supervisor

REVIEWED BY: Faisal Aziz, P.E. Date 08/09/2017

Town of Simsbury
Bridge Inspection Report BRI-18

Bridge No. 03984 **Inspection Date: 06/27/17**

Inspection Type: In-depth	Previous Inspection Date: 1988
Inspection Performed By: AKC, BJS, SR	Feature Carried: Old Drake Hill Road Bridge
Town: Simsbury	Feature Crossed: Farmington River
Year Built: 1892	Main Material: Steel
Year Rehab:1993	Main Design: Parker Through Truss

58. DECK: Overall Rating: 6

	Rating	
Overlay	N	
Deck Str.-Condition	6	The top side of timber deck planks exhibit the following deficiencies: • Random deck planks with splits and checks open up to 1/2". • Random deck plank ends have sections which are broken and lifted up by up to 1/4" high. • Random deck planks with vertical misalignment up to 1/8" high and an isolated location with 1/2" high. • Random deck planks with gaps of up to 1/2" between the segments. • Isolated 7" x 7" x 1" deep area of timber rot with exposed screws near midspan. The underside of timber deck exhibits the following deficiencies: • Random deck planks with longitudinal splits and checks open up to 1/16". • Timber ties atop the floorbeams with longitudinal checks up to 1/16" wide. (See Sketch No. 2 and Photo Nos. 7 - 10)
Curbs	N	
Median	N	
Sidewalks	N	
Parapet	N	
Railing	7	There are metal bridge ornamental railings along both fascia of the bridge, which exhibit isolated areas of peeling paint with light to moderate rust. There are wooden plantation beds for flower pots attached to the outer face of the bridge railings with S-shaped brackets. There are also watering pipes along the railings for irrigating the flower beds. (See Sketch No. 2 and Photo Nos. 11 - 12)
Paint	7	Less than 5% of the painted railing surfaces are peeling with light to moderate rust.
Fence	N	
Drains	N	
Lighting Standard	N	
Utility Type/Size	7	There is an irrigation system in place for the flower beds. A horizontal channel section has been attached to the vertical members of both trusses to accommodate the flower bed irrigation system, which exhibit isolated areas of peeling paint.

Town of Simsbury
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Bridge No. 03984 **Inspection Date: 06/27/17**

	Rating	
		(See Photo Nos. 11 - 12)
Construction Joints	N	
Expansion Joints	6	There is joint sealant material between the timber deck ends and concrete headers at both abutments, which exhibits the following deficiencies: • Deteriorating joint sealant material at random locations. • Minor accumulation of sand along the joints. (See Sketch No. 2 and Photo No. 13)

Approach Condition: Overall Rating: 6

	Rating	
Approach Slab	N	
Relief Joints	N	
Approach Guide Rail	7	There are metal railings at each corner of the bridge which exhibit isolated areas of peeling paint with light to moderate rust. (See Photo No. 14)
Approach Pavement	6	There are stone pavers in both approaches with the following deficiencies: • Minor cracks between the stone pavers. • Isolated depressed area up to 1' long x full width x 1" deep in the east approach. (See Sketch No. 2 and Photo No. 15)
Approach Embankment	N	

Traffic Safety Features:

	Rating	
Bridge Railings	N	Pedestrian bridge.
Transitions	N	Pedestrian bridge.
Approach Guardrails	N	Pedestrian bridge.
Approach Guardrail Ends	N	Pedestrian bridge.

59. Superstructure: Overall Rating: 5

	Rating	
Bearing Devices	4	There are expansion bearings at West Abutment with the following deficiencies: • Vertical gusset plates at the bearings exhibit heavy rust with section loss up to 2" high x 1/16" deep along the bottom. • The bearing for North Truss at West Abutment is missing a vertical gusset plate between the pin and truss members. • Pack rust up to 1/4" thick between the truss members, pin and vertical gusset plate. • Light to moderate accumulation of pack rust and timber debris atop the bearing plates.

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Bridge No. 03984 **Inspection Date: 06/27/17**

		There are fixed bearings at East Abutment with the following deficiencies: • Vertical gusset plate at the bearing exhibit section loss up to 11" long x full height x down to 1/8" remaining with rust holes up to 1" wide x 1/4" high. • Isolated locations with pack rust up to 1/4" thick between the truss members, pin and gusset plate. • Bearing for the North Truss is undermined for 9" long x 1" deep due to spall in the abutment stone, resulting in less than 5% loss of bearing area. • Light to moderate accumulation of pack rust and timber debris atop the bearing plates. (See Sketch Nos. 37 - 39 and Photo Nos. 16 - 18)
Stringers	N	
Girders	N	
Floor Beams	5	There are steel floorbeams (S12 x 31.8), which exhibit the following deficiencies: • Top flanges with up to full length x full width x down to 1/4" remaining section loss and isolated location with 3" long x 3/4" wide rust hole (less critical areas). • Floorbeam webs with up to 6" long x 2" high x 1/16" deep section loss along the bottom at isolated locations (original web thickness = 9/16"). • Bottom flanges with up to full length x full width x 1/16" deep painted over pitting. • Clip angles at the floorbeam bottom chord truss connection exhibit peeling paint with light to moderate rust. (See Sketch Nos. 3 - 10 and Photo Nos. 19 - 20)
Trusses-General	5	The steel superstructure is comprised of two Parker through trusses. The connections at the nodes along the bottom chord has been retrofitted in the past to address severe section losses in the diagonal strut and rod members, and bottom web and flanges of vertical strut members. The bottom chords consist of a built-up rivetted section, which exhibits the following deficiencies: • Areas of peeling paint with moderate to heavy rust, primarily at the interior truss nodes. • Areas of pitting up to 40" long x full width x down to 1/16" remaining, with up to 3" long x 1/4" wide rust holes in the interior bottom angle. The maximum resulting section loss in bottom chord area is approximately 5% (critical zone). • The bottom chord splice connections exhibit pack rust up to 1/2" thick between the bottom/top splice plates and bottom chord angles resulting in the sections bending up/down up to 1/2". The vertical members (2- C7 x 9.8) exhibit the following deficiencies: • Areas of painted over pitting up to 1/16" deep with up to 1/2" x 3/8" rust holes in

Town of Simsbury
Bridge Inspection Report BRI-18

Bridge No. 03984 **Inspection Date: 06/27/17**

		the channel web. • Vertical members at the lower nodes with severe section loss (up to 100%) in the channel webs and flanges (a previously noted condition). Connections have been previously retrofitted. There are diagonal strut members with channel sections (2- C6 x 8.2) between U4-L4 to U8-L8, which exhibit the following deficiencies: • Areas of severe section loss at the lower nodes (up to 100%) in the channel webs (a previously noted condition). Connections have been previously retrofitted. • Channel webs with areas of painted over pitting up to 1/16" deep. Random rust holes in the channel web up to 4" diameter, primarily around the bolted tie-rod attachments between the channels (up to 16% section loss in diagonal member; 32% of the channels). Additional plates have been welded previously at some severely deteriorated locations. There are diagonal eye bar/rod members between U1-L1 to U4-L4 and U8-L8 to U11-L11, which exhibit the following deficiencies: • Areas of severe corrosion at the lower nodes (up to 100%), primarily around the pins (a previously noted condition). Connections have been previously retrofitted. The top chord consists of built-up rivetted section, which exhibits the following deficiencies: • Random areas of peeling paint with light to moderate rust. • Upper truss nodes with pack rust up to 1/2" thick between the top connection plate and top angles of top chord resulting in the sections bending up/down up to 1/4". • Upper truss nodes with pack rust up to 1/4" thick between the connection plate and top chord members. • Upper truss nodes with top angles with up to 11" long x full width x down to knife edge remaining section loss with up to 3-1/2" long x full width rust holes in horizontal legs. • Upper truss nodes with bottom angles of top chords with 9" long x full width x knife edge remaining section loss with 7" long x 1-1/4" wide rust holes in horizontal legs. • Upper chord pins with up to 1/4" thick pack rust/gap between the chord member web and pin. • Random locations in upper chord members with bird nests at the nodes. (See Sketch Nos. 11 - 62 and Photo Nos. 21 - 34)
Trusses-Portal	7	There are steel portals at L1-U1 & L11-U11 chords, with the following deficiency: • Random areas of peeling paint with light rust. (See Sketch No. 65)

Town of Simsbury
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Bridge No. 03984 **Inspection Date: 06/27/17**

Trusses-Bracing	5	The bottom lateral and diagonal bracing between the floorbeams exhibit the following deficiencies: • Random areas of peeling paint with light to moderate rust. • Isolated bolts are loose/flush with the nuts. • Isolated locations with missing bolts. • Gusset plates with peeling paint and light to moderate rust. The strut and sway bracing exhibits the following deficiencies: • Random areas of peeling paint with light to moderate rust. • Isolated locations in the top strut angle with up to 12" long x full width x down to knife edge remaining section loss with 1" wide x 1/2" long hole in the horizontal leg. • Isolated locations in the diagonal bracing member with up to full width x 6" long x 1/8" deep section loss with up to 1" diameter rust holes. • Isolated locations with gaps up to 3/8" between the diagonal, and top and bottom members of the lateral bracing system. (See Sketch Nos. 3 - 10 & 63 - 64 and Photo Nos. 9 - 10 & 35 - 39)
Paint	7	Less than 10% of the painted surfaces are peeling with light to moderate rust. See items above entitled "Bearing Devices", "Floor Beams", "Trusses-General", "Trusses-Portal" and "Trusses-Bracing".
Rust	4	See items above entitled "Bearing Devices", "Floor Beams", "Trusses-General", "Trusses-Portal" and "Trusses-Bracing".
Machinery Mov. Span	N	
Rivets and Bolts	6	The rivets in the structure exhibit the following deficiencies: • Random rivets with peeling paint and light to moderate rust. • Isolated rivet heads with up to 50% head loss. See item above entitled "Trusses-Bracing". (See Sketch No. 3 - 62 and Photo Nos. 18, 24 - 25, & 37 - 38)
Welds and Cracks	6	There are repair welds in the structure, which exhibit the following deficiencies: • A 2-1/2" long horizontal crack between the top chord and strut at node U1 north side of South Truss (non-critical zone). • Sloppy welds in the repair plates attached to the diagonal truss element. (See Sketch No. 40 and Photo No. 39)
Timber Decay	N	
Concrete Cracking	N	
Collision Damage	N	
Member Alignment	7	Diagonal member, L8-U9 at South Truss is slightly bent.

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Bridge No. 03984 **Inspection Date: 06/27/17**

		(See Sketch Nos. 11 – 12).
Deflect. Under Load	N	(N) Normal; (E) Excessive. Note: Bridge does not carry any vehicular traffic. Open for pedestrian traffic only.
Vibr. Under Load	N	(N) Normal; (E) Excessive. Note: Bridge does not carry any vehicular traffic. Open pedestrian traffic only.
Stand Pipes	N	
Barrel Ladders	N	

60. Substructure: Overall Rating: 7

	Rating	
Abutments-Stem	7	There are stone masonry abutment stems, which exhibit the following deficiencies: • Isolated stones with full height cracks open up to 1/16". • East Abutment Stem with isolated 18" long x 9" high x 6" deep spall in stone under the bearing for the North Truss which undermines the bearing up to 9" long x 1" deep. • Isolated stone in East Abutment with full height crack open up to 1/16" and 7" high x 2" wide x 2" deep chipped off. • Random voids in the joint mortar between the stones along the base of stem. • Hairline cracks with and without efflorescence in the mortar between the stones. • Heavy growth of vegetation atop the abutment seats at the bearings. (See Sketch Nos. 66 - 67 and Photo Nos. 40 - 42)
Abutments-Backwall	7	The top of backwalls are exposed along top of the timber deck interface. The west abutment backwall top has cracks up to 1' long x 1/2" wide. (See Sketch No. 2)
Abutments-Footings	N	Not visible.
Abutments-Settlement	8	None observed.
Abutments-Wingwalls	7	There are stone masonry wingwalls with concrete caps, which exhibit the following deficiencies: • Isolated stones with horizontal hairline cracks with efflorescence. • Random hairline cracks in the mortar between the stones. • Moderate to heavy growth of vegetation along the wingwalls. (See Sketch Nos. 68 - 69 and Photo Nos. 43 - 44)
Piers/Bents-Caps	N	
Piers/Bents-Pile Bent	N	
Piers/Bents-Columns	N	
Piers/Bents-Footings	N	
Piers/Bents-	N	

Town of Simsbury
Bridge Inspection Report BRI-18

Bridge No. 03984 **Inspection Date: 06/27/17**

Settlement		
Erosion-Scour	8	Erosion: Rated - '8'. Scour: Rated - '8'.
Concrete Crack-Spall	N	
Steel Corrosion	N	
Paint	N	
Timber Decay	N	
Collision Damage	N	
Debris	7	Light accumulation of timber debris atop the abutment seats.

61. Channel and Channel Protection **Overall Rating: 6**

	Rating	
Channel Scour	8	The channel bottom consists of sand with small to medium size stones. (See Sketch No. 70 - 71 and Photo Nos. 45 - 48)
Embankment Erosion	6	Areas of erosion along the embankments up to 3' high x 3' deep with exposed tree roots. (See Sketch No. 70 and Photo Nos. 47 - 48)
Debris	N	
Vegetation	6	Heavy growth of vegetation along the channel embankments, some of which is overhanging the channel. Light to moderate growth of vegetation in the channel. (See Sketch No. 70 and Photo Nos. 45 - 48)
Channel Change	8	The channel flow is perpendicular to the bridge.
Fender System	N	
Spur Dikes & Jetties	N	
Rip Rap	7	Small to medium size riprap is in place along the embankment.

62. Culvert & Retaining Wall: **Overall Rating: N**

	Rating	
Barrel	N	
Concrete	N	
Steel	N	
Timber	N	
Headwall	N	
Cutoff Wall	N	
Debris	N	
Retaining Wall System	N	
Footing	N	

Town of Simsbury
Bridge Inspection Report BRI-18

Bridge No. 03984 **Inspection Date: 06/27/17**

Load Posting: N

Miscellaneous:

Minimum Vertical Under Clearance:	The structure spans over a waterway.
Posted Clearance Under Bridge:	
Posted Clearance on Bridge:	
Advanced Warning:	
Speed Limit:	
Character of Traffic:	Pedestrian bridge.

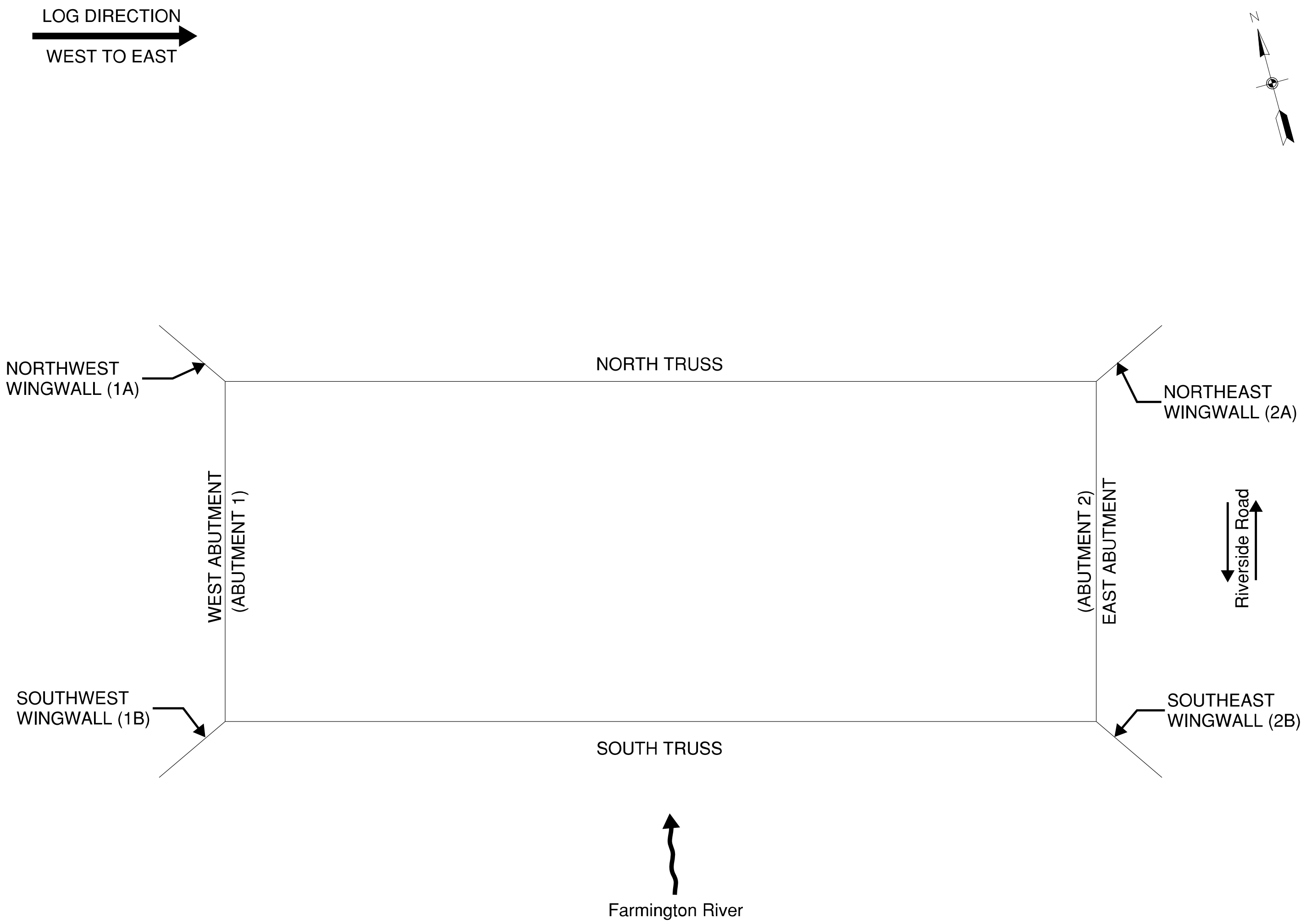
Additional Notes:

Additional Comments:

- Bridge is logged from west to east.
- Farmington River flows from south to north.
- Bridge was inspected using a rigging platform and an extension ladder.
- A safety boat was present during the inspection.

Inspectors' Signatures:	1) (AMIT KC)	Date: 08/09/2017
	2) (SAIPAVAN RALLABHANDI)	Date: 08/09/2017
	3)	Date:
	4)	Date:
P.E. Signature:	(FAISAL AZIZ)	Date: 8/9/17
P.E. #:	29339	
Reviewed by:	Town of Simsbury	Date:

CREW: AKC, BJS, SR (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984

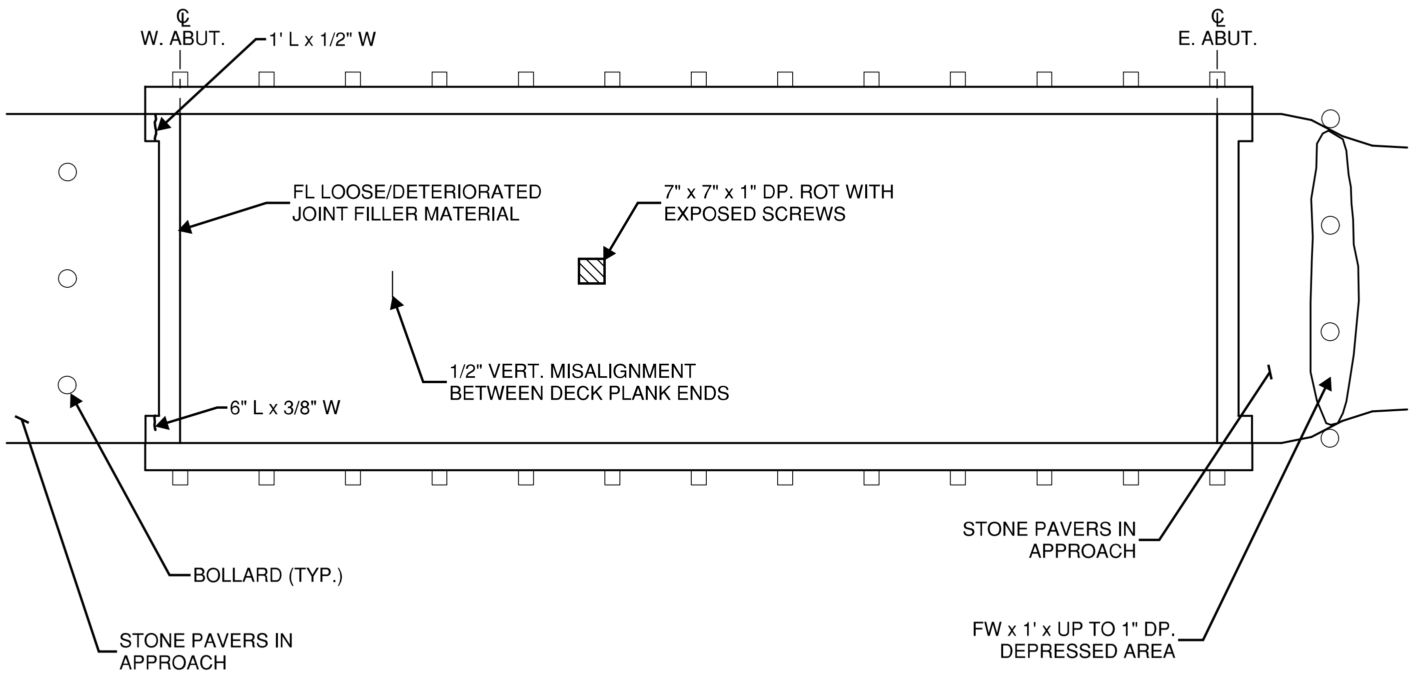


KEY PLAN
(N.T.S.)

(SKETCH 1)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- Top of timber deck planks with random splits and checks open up to 1/2"; random deck plank ends have sections which are broken and lifted up to 1/4" high.
- Timber deck planks are vertically misaligned up to 1/8"; gaps up to 1/2" between the planks.
- Joint sealant material is deteriorating at random locations.
- Metal bridge ornamental railing with random areas of peeling/chipped paint with light rust along the base.

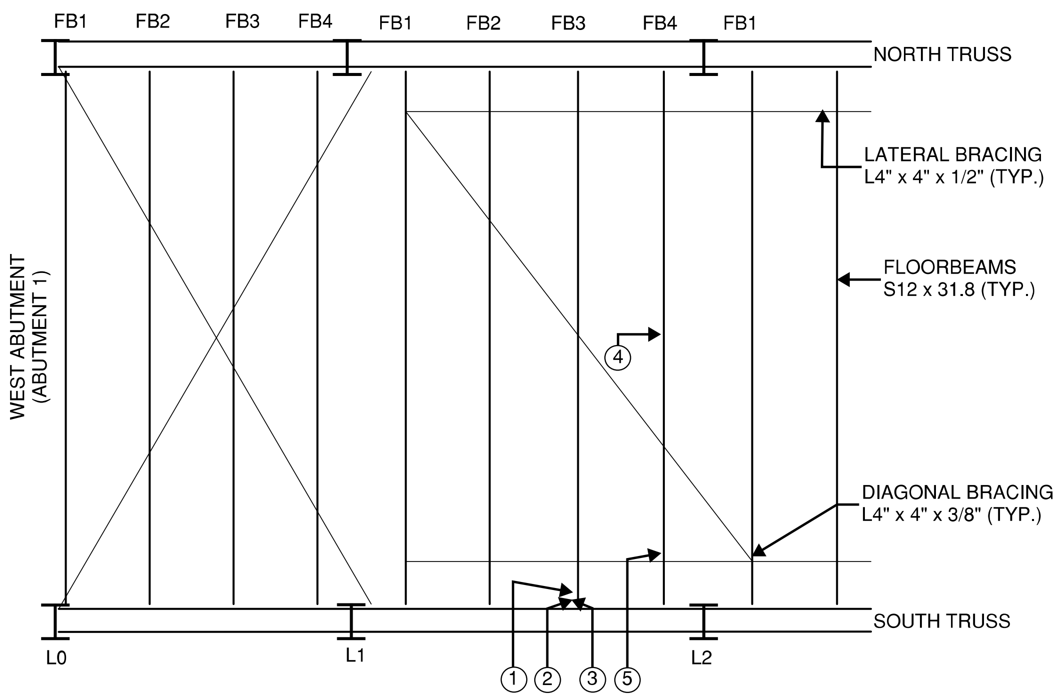
TOP OF DECK

(N.T.S.)

(SKETCH 2)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984



DETERIORATION NOTES:

- See "DETERIORATION NOTES - UNDERSIDE OF DECK AND FRAMING (L0 TO L2)" sheet.

GENERAL NOTES:

- See "GENERAL NOTES - UNDERSIDE OF DECK AND FRAMING" sheet.

UNDERSIDE OF DECK AND FRAMING (L0 TO L2)

(N.T.S.)

(SKETCH 3)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984

DETERIORATION NOTES - UNDERSIDE OF DECK & FRAMING (L0 TO L2):

① Floorbeam top flange with 6" long x 1" wide x down to 3/16" remaining.

② Floorbeam web bottom with 6" long x 3" high x 1/16" deep pitting on west side at the truss connection.

③ Floorbeam web bottom with 6" long x 2" high x 1/16" deep section loss on east side at the truss connection.

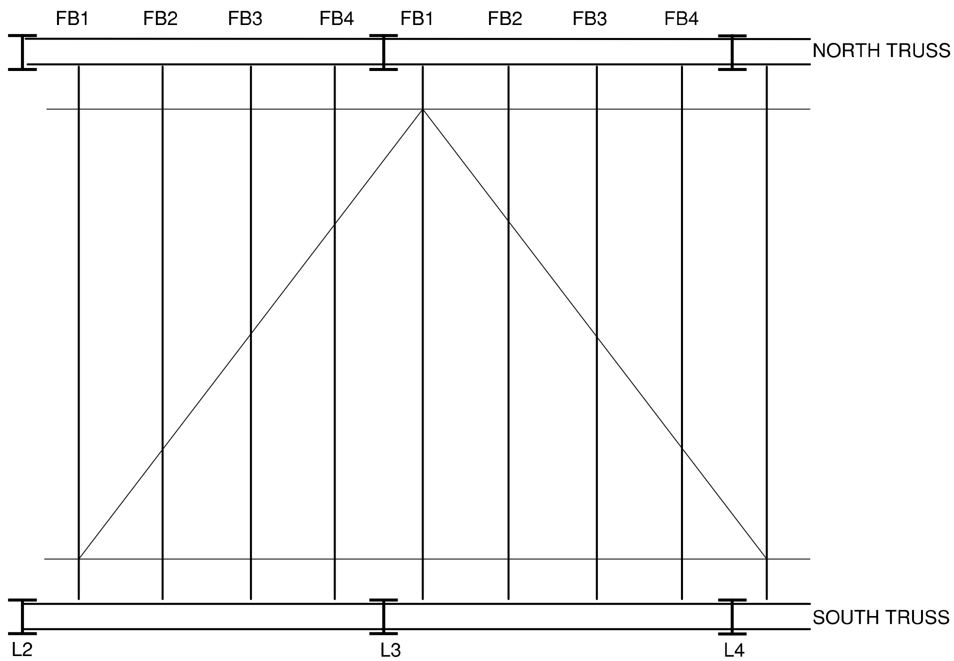
④ Floorbeam top flange with full length x full width x down to 1/4" remaining and bottom flange with full length x full width x 1/16" deep pitting.

⑤ Floorbeam top flange with 4' long x 2.5" x down to knife edge remaining section loss, starting at 10" from South Truss, with a 3" long x 3/4" wide rust hole at 1'± from the longitudinal bracing.

(SKETCH 4)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- See "GENERAL NOTES - UNDERSIDE OF DECK AND FRAMING" sheet.

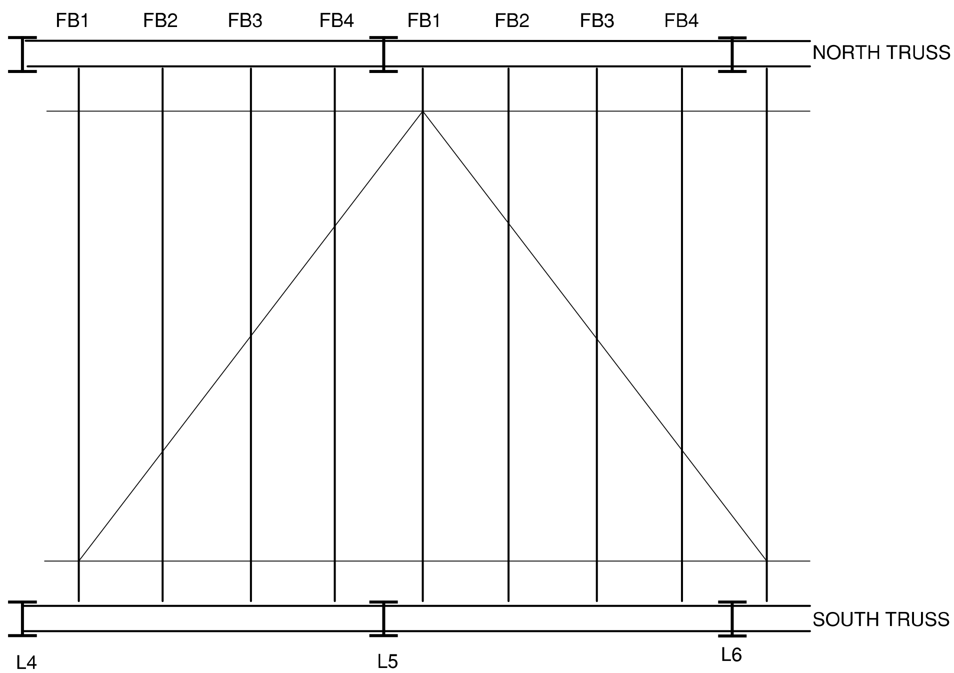
UNDERSIDE OF DECK AND FRAMING (L2 TO L4)

(N.T.S.)

(SKETCH 5)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- See "GENERAL NOTES - UNDERSIDE OF DECK AND FRAMING" sheet.

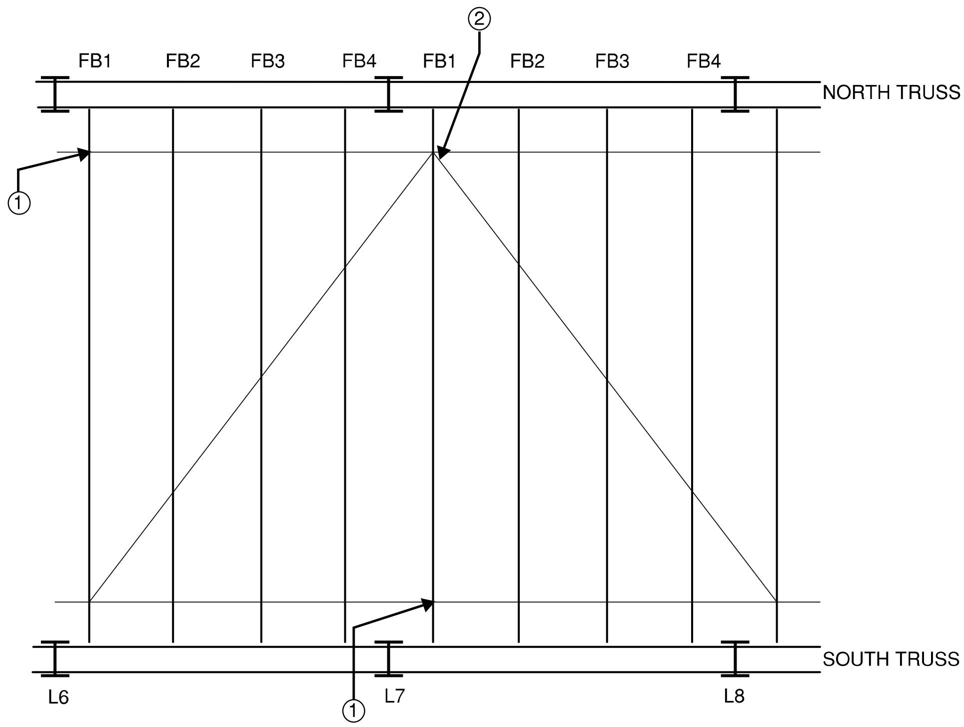
UNDERSIDE OF DECK AND FRAMING (L4 TO L6)

(N.T.S.)

(SKETCH 6)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984



DETERIORATION NOTES - UNDERSIDE OF DECK & FRAMING (L6 TO L8):

① (2) missing bolts between the bottom flange of floorbeam and longitudinal bracing.

② (1) loose bolt at horizontal gusset plate for lateral bracing; random short bolts.

GENERAL NOTES:

- See "GENERAL NOTES - UNDERSIDE OF DECK AND FRAMING" sheet.

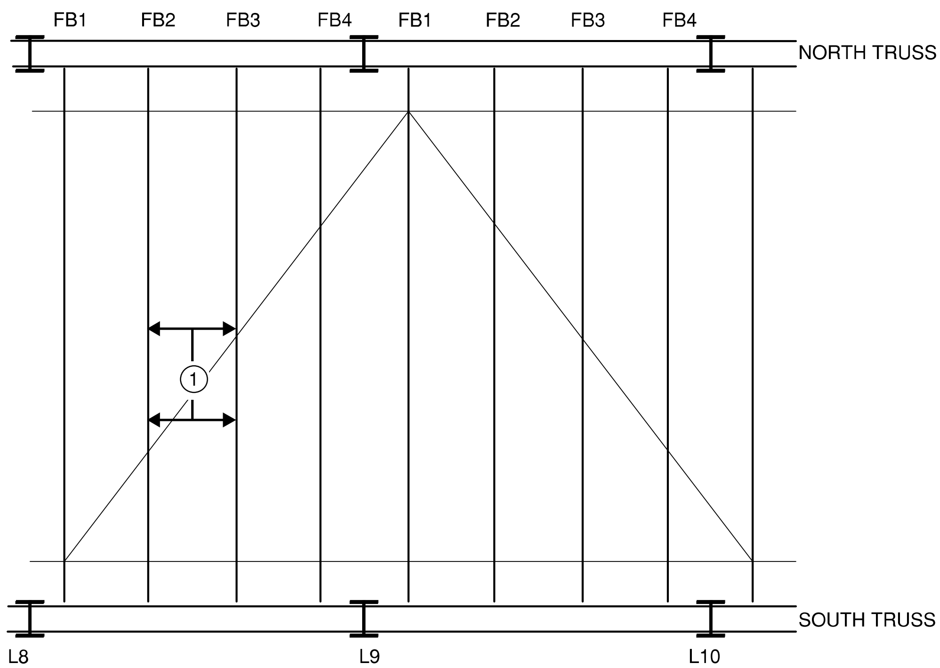
UNDERSIDE OF DECK AND FRAMING (L6 TO L8)

(N.T.S.)

(SKETCH 7)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984



DETERIORATION NOTES:

1 Floorbeam bottom flanges with 1" diameter drilled holes.

GENERAL NOTES:

- See "GENERAL NOTES - UNDERSIDE OF DECK AND FRAMING" sheet.

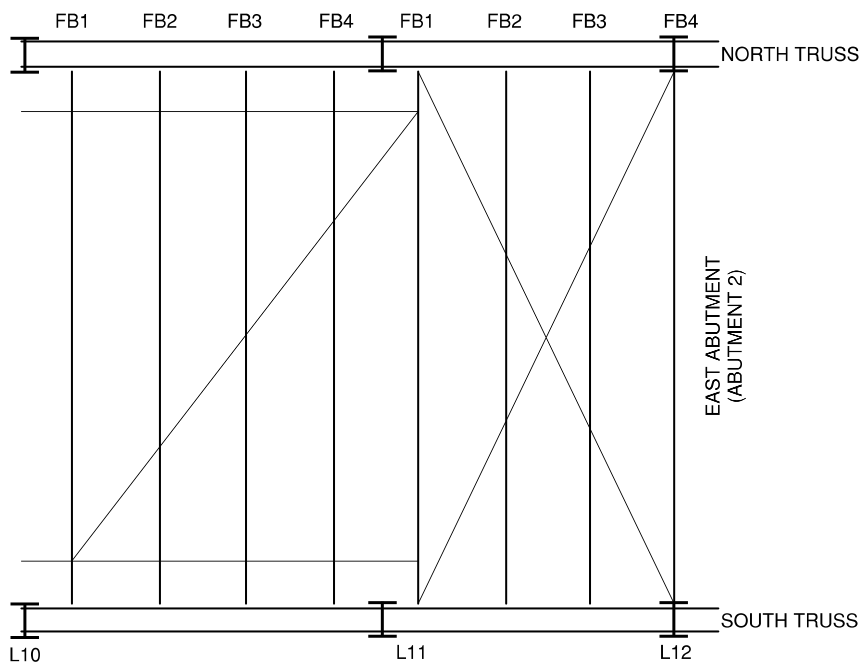
UNDERSIDE OF DECK AND FRAMING (L8 TO L10)

(N.T.S.)

(SKETCH 8)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- See "GENERAL NOTES - UNDERSIDE OF DECK AND FRAMING" sheet.

UNDERSIDE OF DECK AND FRAMING (L10 TO L12)

(N.T.S.)

(SKETCH 9)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984

GENERAL NOTES - UNDERSIDE OF DECK & FRAMING:

- Timber deck planks with random longitudinal checks open up to 1/16".

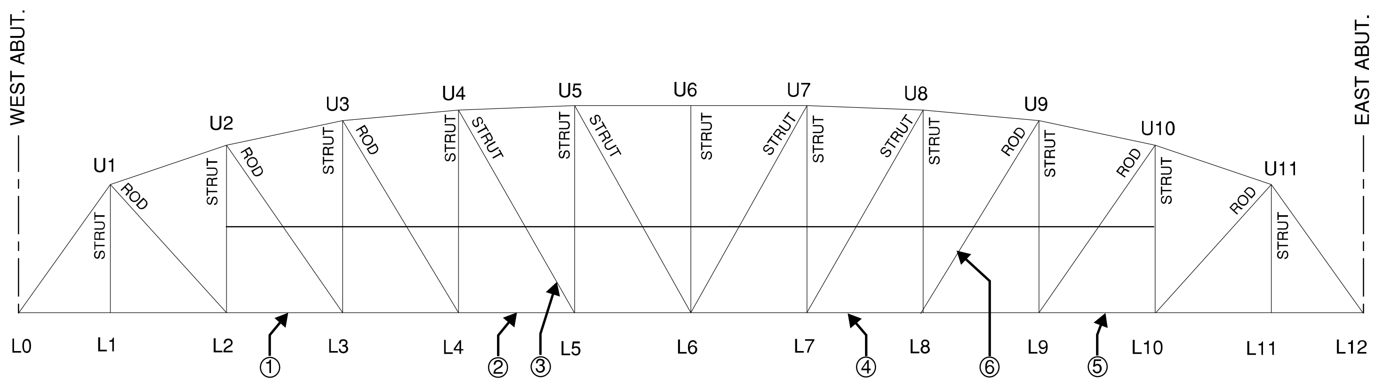
- Timber ties atop the floorbeams with longitudinal checks open up to 1/16".

- Clip angles between the bottom chord web and floorbeams with peeling paint and light rust.

(SKETCH 10)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



DETERIORATION NOTES:

- See "DETERIORATION NOTES - SOUTH TRUSS - SOUTH ELEVATION" sheet.

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.
- See "REPAIR PLATES" sheet for gusset plate dimensions.

SOUTH TRUSS - SOUTH ELEVATON

(N.T.S.)

(SKETCH 11)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

DETERIORATION NOTES - SOUTH TRUSS - SOUTH ELEVATION:

① Bottom chord splice connection between L2 & L3, splice plate bending out due to up to 1/2" thick pack rust.

② Bottom chord splice connection between L4 & L5, splice plate bending out due to up to 1/4" thick pack rust.

③ L5-U4 diagonal member with full width x 3/4" high x down to knife edge remaining with perforations up to 1" long x 1/2" wide. ✳

④ Bottom chord splice connection between L3 & L4, splice plate bending out due to up to 1/8" thick pack rust; missing rivet in the vertical leg of top angle.

⑤ Bottom chord splice connection between L9 & L10, splice plate bending out due to up to 1/4" thick pack rust.

⑥ Outside strut of the diagonal member L8-U9 is slightly bent..

NOTE:

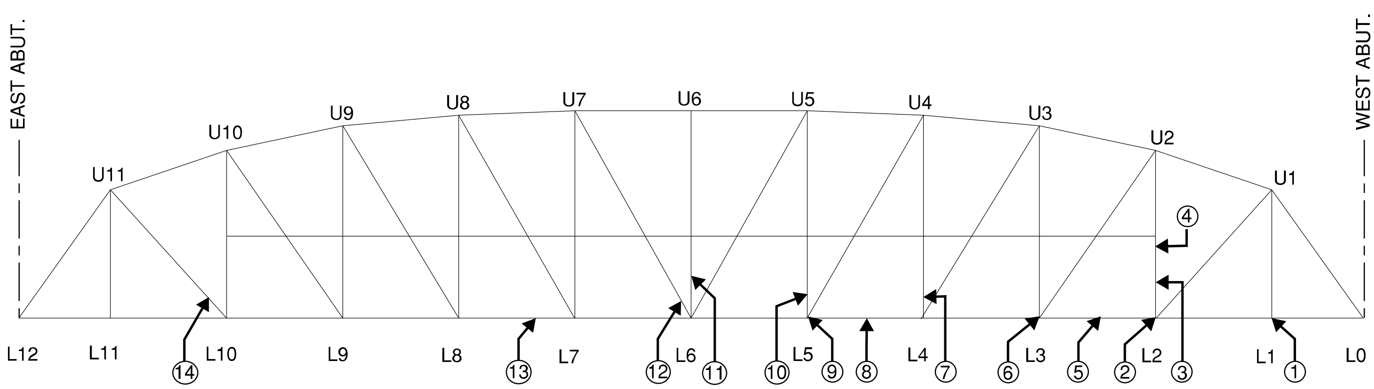
✳ Retrofit assembly in place to address the section losses.

- See "REPAIR PLATES" sheet for retrofit gusset plate dimensions.

(SKETCH 12)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2) | DATE: 6/27/2017 | BRIDGE NO.: 03984



DETERIORATION NOTES:

- See "DETERIORATION NOTES - SOUTH TRUSS - NORTH ELEVATION" sheet.

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.
- See "REPAIR PLATES" sheet for gusset plate dimensions.

SOUTH TRUSS - NORTH ELEVATON

(N.T.S.)

(SKETCH 13)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984

DETERIORATION NOTES - SOUTH TRUSS - NORTH ELEVATION:

① Bottom chord, interior bottom angle at L1 with 4' long x full width x down to 1/8" remaining pitting and a 3-1/4" long x1/4" wide rust hole in the horizontal leg (near L1); also at the same location, 4' long x full height x 3/16" deep pitting in vertical leg (approx. 5% section loss in overall chord area).

② Bottom chord, interior angle horizontal leg at L2 with 40" long x full width x down to 1/16" remaining (approx. 5% section loss in overall chord area) with a 3" long x 1/4" wide rust hole at the edge.

③ Vertical member U2-L2 channel web with 1/2" high x 1/4" wide rust hole at the welded repair channel. ✳

④ Vertical member U2-L2 with 1/16" diameter hole in the weld.

⑤ Bottom chord splice connection between L2 & L3, bottom splice plate bent down full width x 3/16" over 9" long due to pack rust; bottom angle horizontal leg with 2" long x 2" wide rust hole; web splice plate with 6" long x 2" high x 1/8" deep section loss at bottom; one rivet head at the bottom with heavy rust and 25% head loss.

⑥ Bottom interior angle, horizontal leg at L3 with 4' long x full width x down to 1/8" remaining (less than 5% loss in overall area).

⑦ Vertical member U4-L4 with a 3" x 1" x 1/8" deep section loss in flange with gap between the vertical member and welded repair channel. ✳

⑧ Bottom chord splice connection between L4 & L5, bottom splice plate is bent down 1/2" over 9" long due to pack rust.

⑨ L5-U4 diagonal member channel web with 1" high x full width x down to knife edge remaining section loss with random perforations.✳

⑩ Vertical member U5-L5 with 2" x 1/2" x 1/8" deep section loss in flange with gap between vertical member and welded channel. ✳

⑪ Vertical member U6-L6 with 2" x 1/2" x 1/8" deep section loss in flange with gap between vertical member and welded channel on both sides.✳

⑫ L6-U7 diagonal member with 6" x 6" x 1/16" deep pitting at the pin connection.✳

⑬ Bottom chord splice connection between L7 & L8, web splice plate bent for 6" long x 3/16" due to pack rust;

⑭ L10-U11 diagonal member with 1" diameter x 1/8" deep section loss at the pin connection. ✳

NOTE:

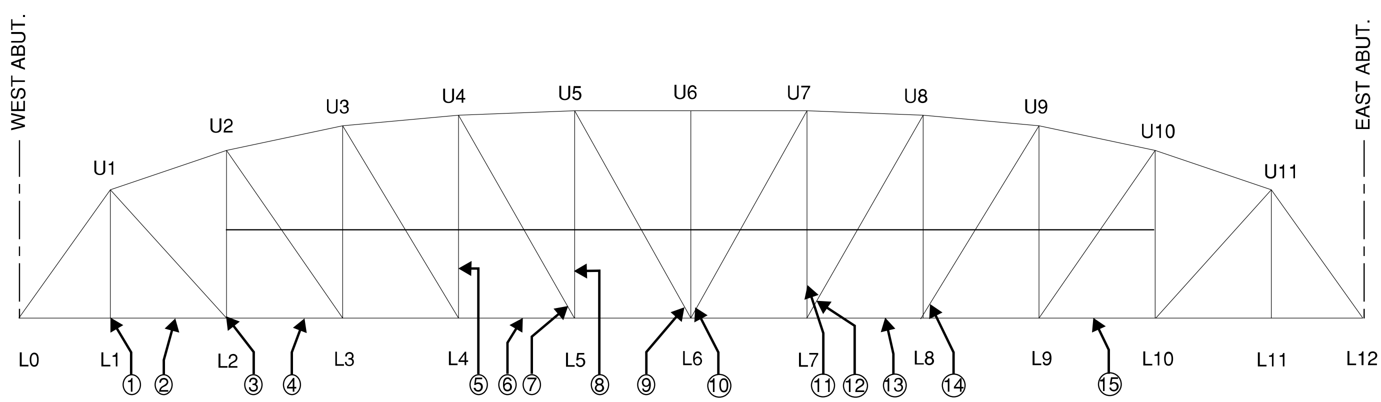
✳ Retrofit assembly in place to address the section losses.

- See "REPAIR PLATES" sheet for retrofit gusset plate dimensions.

(SKETCH 14)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2) | DATE: 6/27/2017 | BRIDGE NO.: 03984



DETERIORATION NOTES:
- See "DETERIORATION NOTES - NORTH TRUSS - SOUTH ELEVATION" sheet.

GENERAL NOTES:
- See "GENERAL NOTES - BOTTOM CHORD" sheet.
- See "REPAIR PLATES" sheet for gusset plate dimensions.

NORTH TRUSS - SOUTH ELEVATON

(N.T.S.)

(SKETCH 15)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984

DETERIORATION NOTES - NORTH TRUSS - SOUTH ELEVATION:

① Bottom chord interior bottom angle at L1 with 3' long x full width x 1/8" remaining pitting (painted over) in horizontal leg (1.5' on each side of L1) and 3' long x full height x 3/16" deep pitting in the vertical leg (approx. 5% section loss in overall chord area).

② Bottom chord interior angle of L1-L2 horizontal leg member with 3' long x full width x 1/8" deep pitting in the vertical and horizontal legs (approx. 5% section loss in overall chord area).

③ Bottom chord interior angle horizontal leg at L2 with 4' long x full width x down to 1/8" remaining (under L2) with a 3" long x 1" wide rust hole (approx. 5% section loss in overall chord area).

④ Bottom chord splice connection between L2 & L3, Splice plate with 1' long x 1" high x up to 1/8" deep section loss (one rivet with 50% head loss); bottom angle horizontal leg with 4" long x full width x up to 1/4" deep section loss and bent out 1/4" due to pack rust; up to 1/2" gap between the top splice plate and top angles of bottom chord due to pack rust.

⑤ Vertical member U4-L4 with full width x 1" high x 1/8" deep section loss in flange with gap between the vertical member and welded repair channel. ✳

⑥ Bottom chord splice connection between L4 & L5, bottom splice plate is bent down 1/4" over 6" long due to pack rust.

⑦ L5-U4 diagonal member channel web with 2" long x 1" high x down to knife edge remaining with a 3/8" diameter rust hole & 4" diameter rust hole and 1/8" thick pack rust between the connection plate and channel web. ✳

⑧ Vertical member U5-L5 with 1-1/2" high x 1/2" wide x 3/16" deep section loss with gap between the vertical member and welded repair channel. ✳

⑨ L6-U5 diagonal member channel web with full width x 3/4" high x 1/4"± deep section loss above the rivet head plate. ✳

⑩ L6-U7 diagonal member with full width x 1" high x down to knife edge remaining with 3" long x 1" high rust hole. ✳

⑪ Vertical member U7-L7 with 1" high x 1/2" wide x 1/8" deep section loss with gap between the vertical member and welded repair channel. ✳

⑫ L7-U8 diagonal member with full width x 2" high x down to knife edge remaining and random perforations. ✳

⑬ Bottom chord splice connection between L7 & L8, web splice plate bent for 6" long x 1/2" due to pack rust;

⑭ L8-U9 diagonal member with 1" high x 1" wide x 1/8" deep section loss with gap between the vertical member and welded repair channel. ✳

⑮ Bottom chord splice connection between L9 & L10 with pack rust up to 1/2" thick between the splice plates and bottom chord angles; 3" long x 2" high x 3/16" deep section loss in the web splice plate along the bottom.

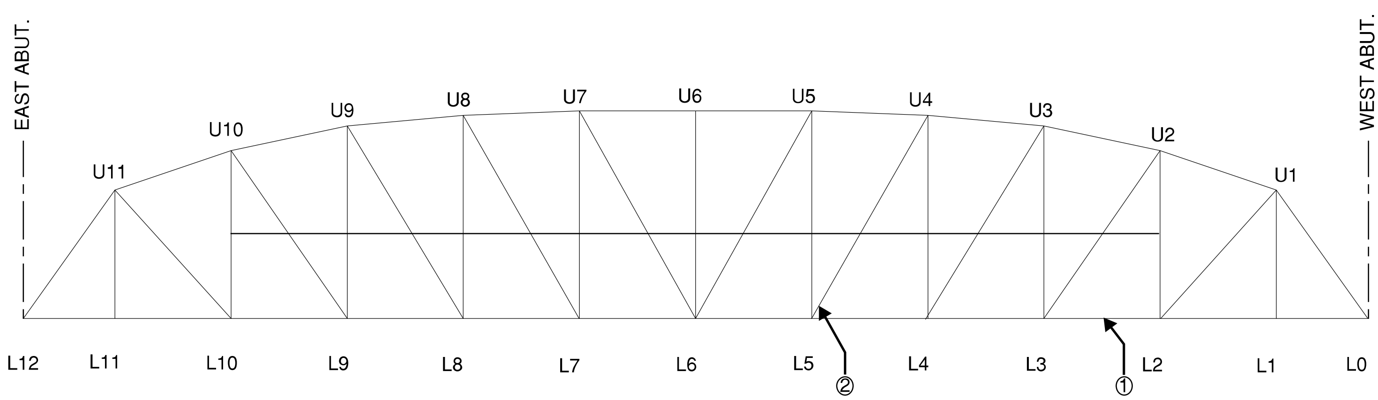
NOTE:

✳ Retrofit assembly in place to address the section losses (See "REPAIR PLATES" sheet for retrofit gusset plate dimensions).

(SKETCH 16)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



DETERIORATION NOTES:

- See "DETERIORATION NOTES - NORTH TRUSS - NORTH ELEVATION" sheet.

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.
- See "REPAIR PLATES" sheet for gusset plate dimensions.

NORTH TRUSS - NORTH ELEVATON

(N.T.S.)

(SKETCH 17)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

DETERIORATION NOTES - NORTH TRUSS - NORTH ELEVATION:

① Bottom chord splice connection between L2 & L3, web splice plate bent up to 1/2" due to pack rust for 6"± long at east edge.

② L5-U4 diagonal member with 1/8" thick pack rust and full width x 1" high x 3/16" deep section loss in channel web.✳

NOTE:

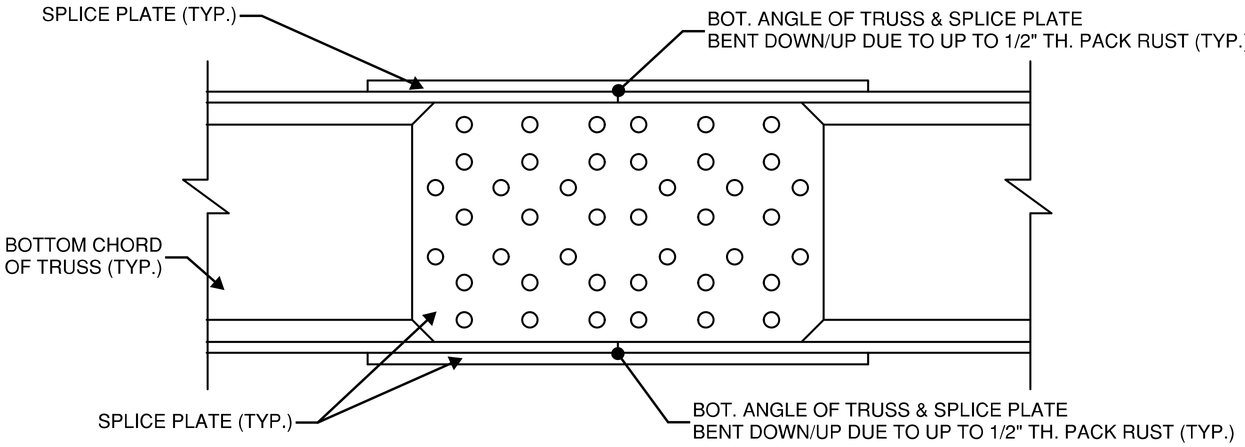
✳ Retrofit assembly in place to address the section losses.

- See "REPAIR PLATES" sheet for retrofit gusset plate dimensions.

(SKETCH 18)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



BOTTOM CHORD SPLICE CONNECTION (TYP.) - SOUTH ELEVATION

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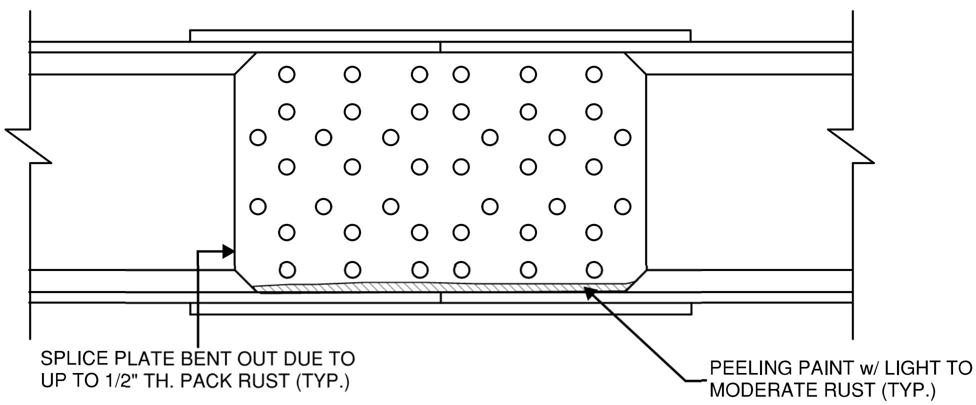


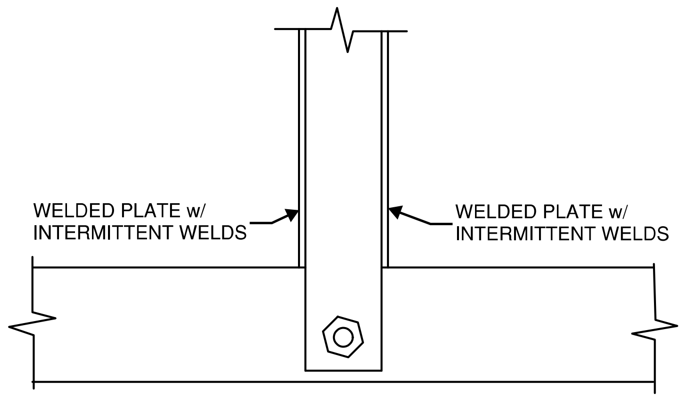
BOTTOM CHORD SPLICE CONNECTION (TYP.) - NORTH ELEVATION

(N.T.S.)

(SKETCH 19)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L1 SOUTH TRUSS SOUTH ELEVATION

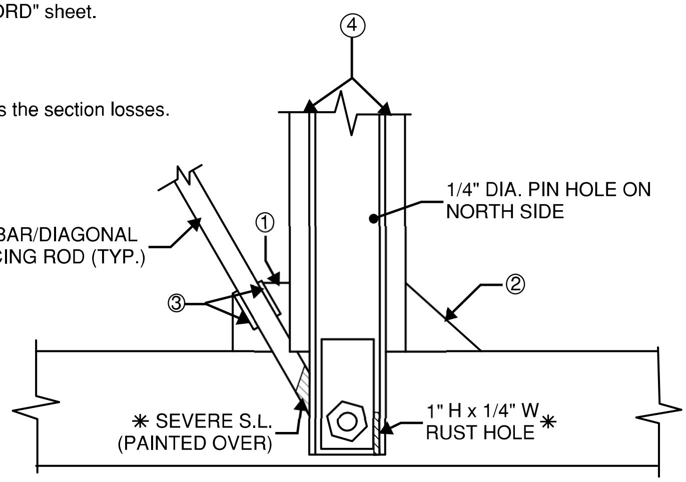
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L2 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 20)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

ORIGINAL TRUSS MEMBER DETAILS:

- Bottom chord is comprised of (1) - 15-1/4" x 1/2" plate & (4) - 2-1/2" x 2-1/2" x 5/16" angles.

- Upper chord is comprised of (1) - 15" x 5/16" plate, (2) - 14-1/2" x 5/16" plate & (4) - 2-1/2" x 2-1/2" x 1/4" angles.

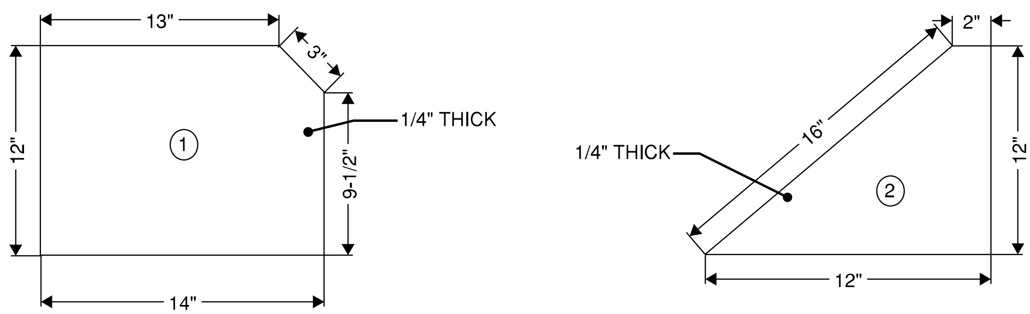
- Vertical members at L2-L10 are comprised of (2) - C7 x 9.8 channels and L1, L11 are comprised of (2) - 2-5 3/4" x 5/16" plates.

- Diagonal members L5-U4, L6-U5, L6-U7, L7-U8 are comprised of (2) - C6 x 8.2 channels.

- Diagonal members L3-U2, L4-U3, L8-U9, L9-U10 are comprised of (2) - 2" x 7/8" plates.

- Diagonal members L2-U1, L10-U11 are comprised of (2) - 3" x 13/16" plates.

- See "ADDITIONAL BACK-UP MATERIAL".



NOTE:

- The repair gusset plates are welded atop the bottom chord and channel bracing of vertical members at chords L2 - L10.

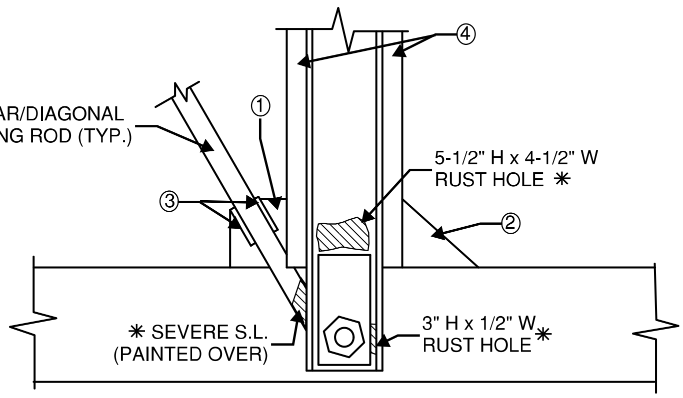
REPAIR PLATES

(N.T.S.)

(SKETCH 21)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L3 SOUTH TRUSS SOUTH ELEVATION

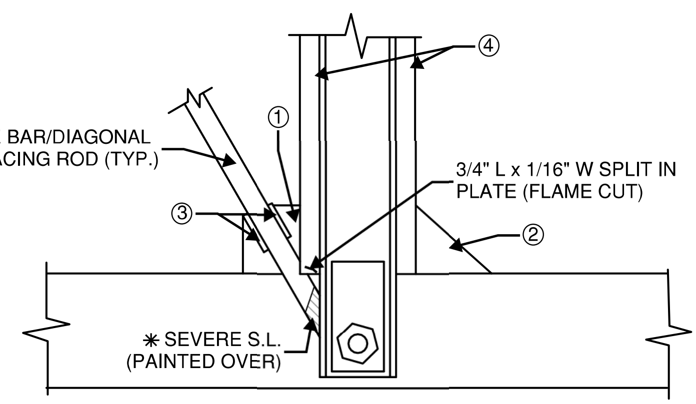
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L4 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

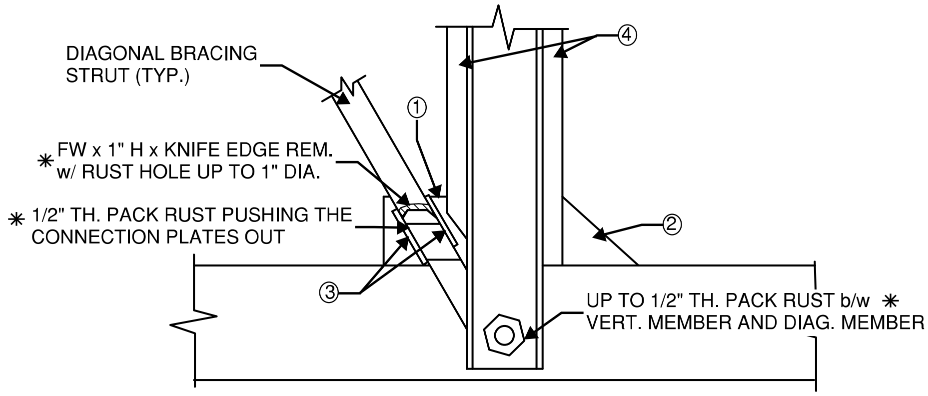
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 22)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L5 SOUTH TRUSS SOUTH ELEVATION

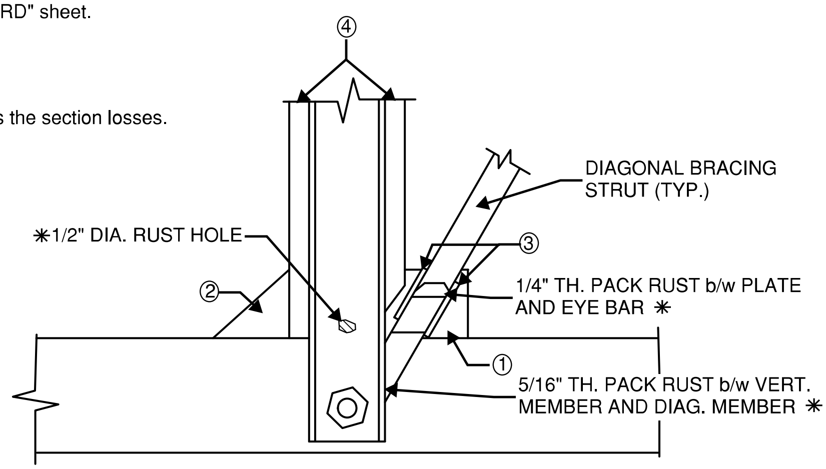
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L7 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

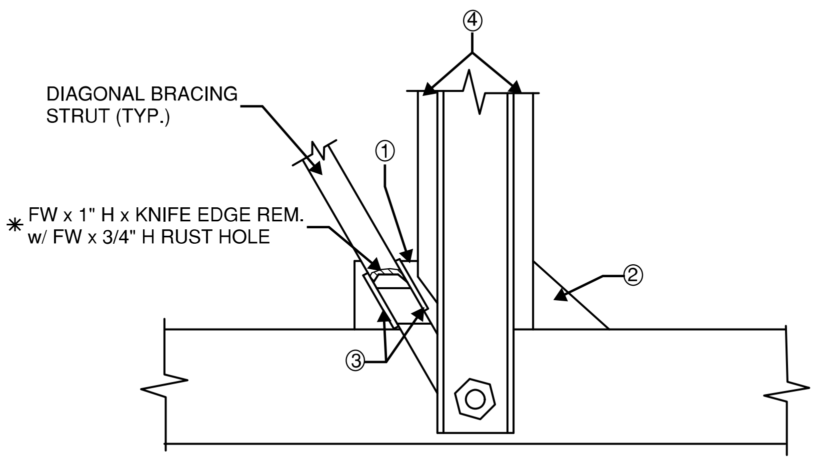
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 23)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L7 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

* Retrofit assembly in place to address the section losses.

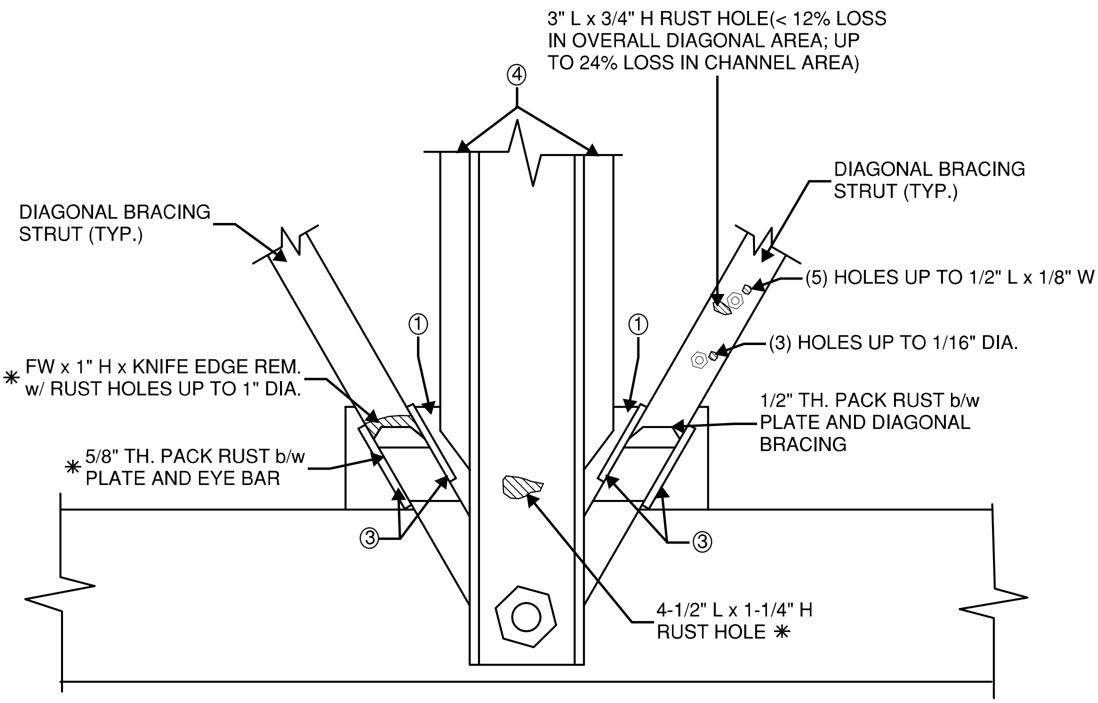
REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)
-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate
-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 24)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L6 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

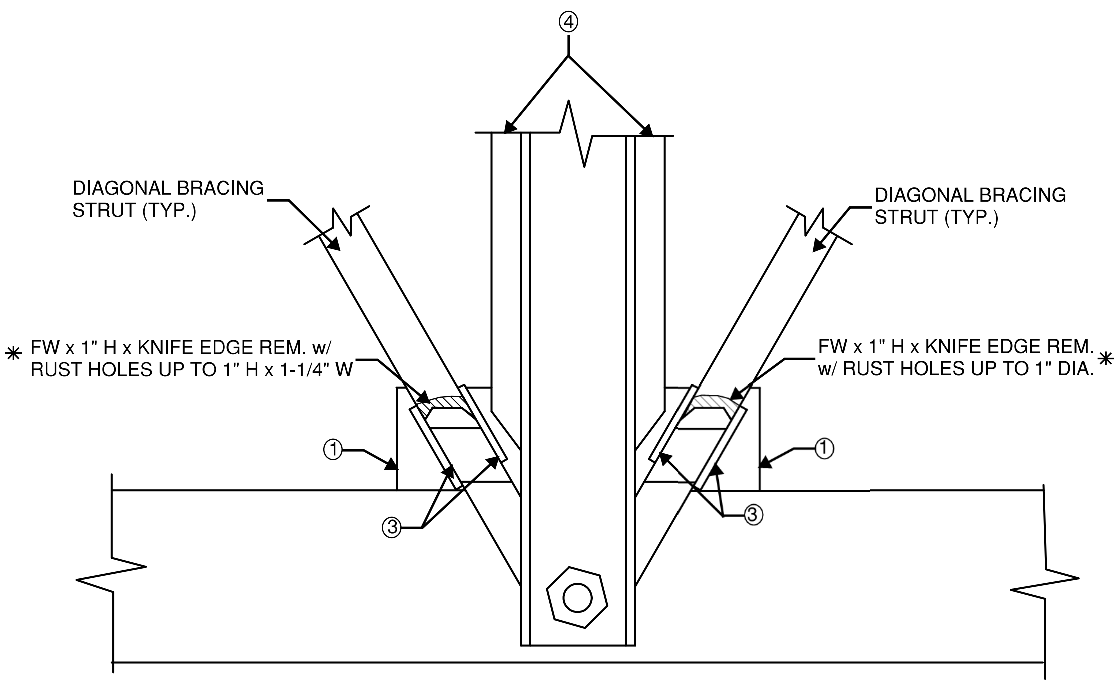
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 25)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L6 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

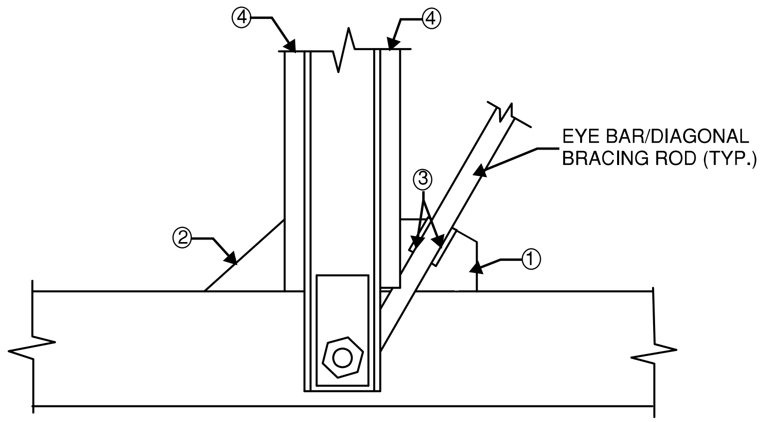
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 26)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L8 SOUTH TRUSS SOUTH ELEVATION

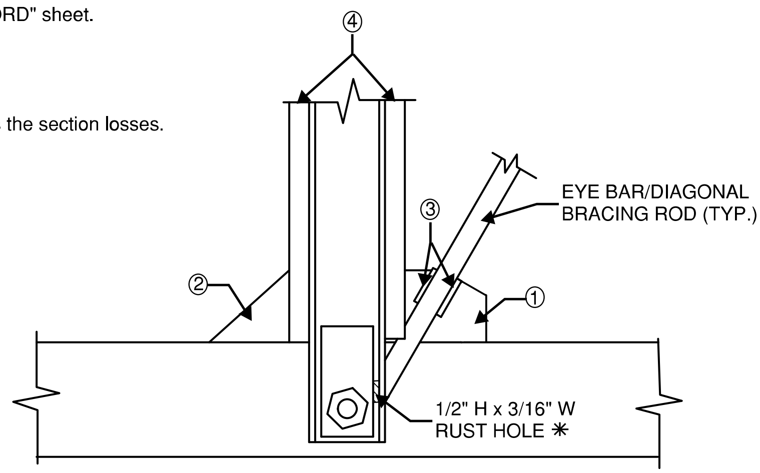
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L9 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

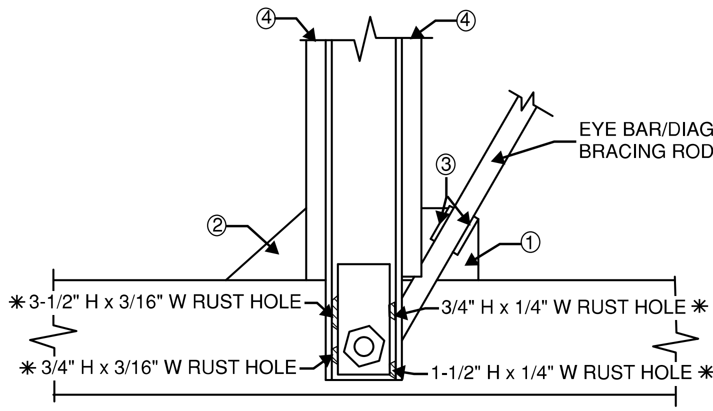
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 27)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L10 SOUTH TRUSS SOUTH ELEVATION

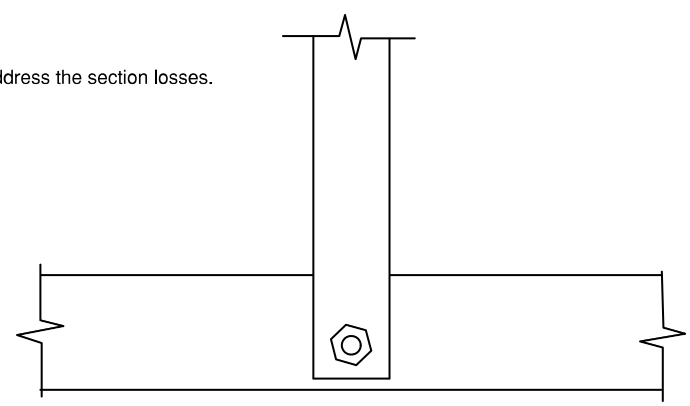
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L11 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 28)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

GENERAL NOTES - BOTTOM CHORD:

- Random areas of peeling paint with moderate to heavy rust.

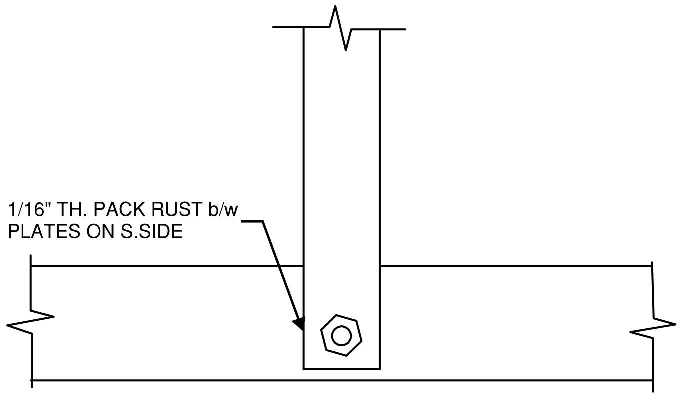
- Severe section loss in the vertical chords and diagonal members were addressed by retrofit gusset plates. The retrofit gusset plates were welded to the bottom chord, diagonal members and vertical chords and painted over during rehabilitation.

- Pack rust up to 1/2" thick between the connection plates and truss members at the pin connections, bottom chord splice connections and diagonal member - truss element connections.

(SKETCH 29)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L1 NORTH TRUSS NORTH ELEVATION

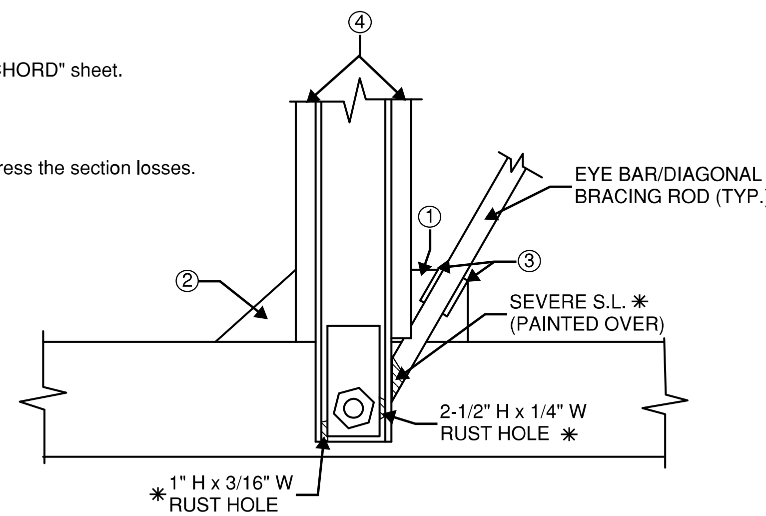
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L2 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

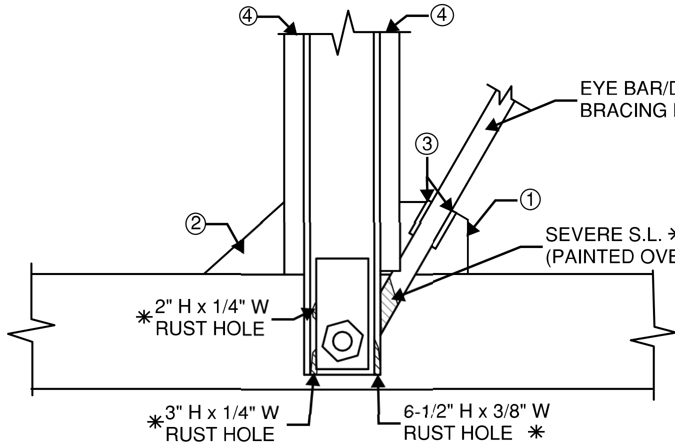
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 30)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L3 NORTH TRUSS NORTH ELEVATION

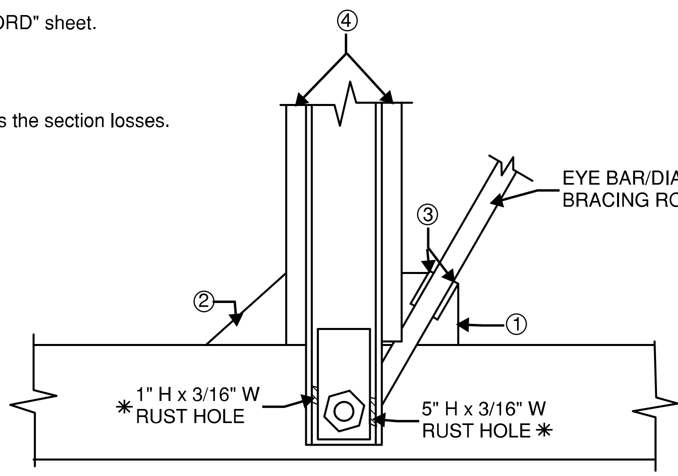
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L4 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

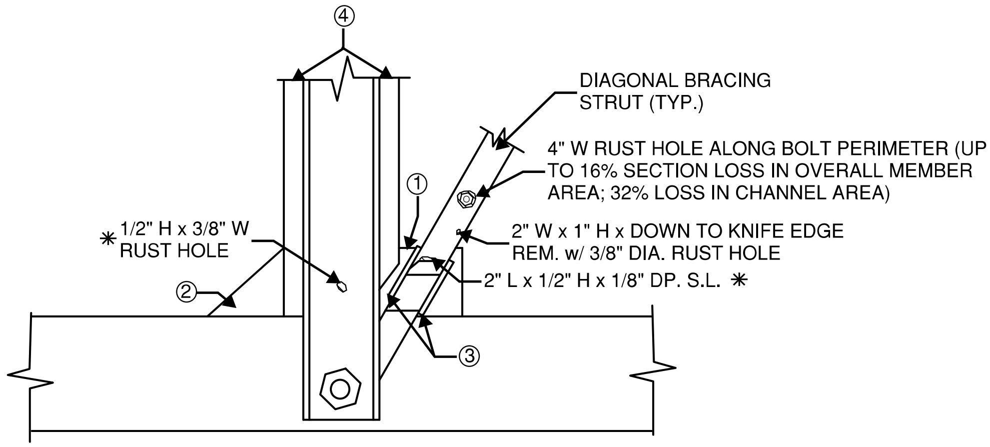
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 31)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L5 NORTH TRUSS NORTH ELEVATION

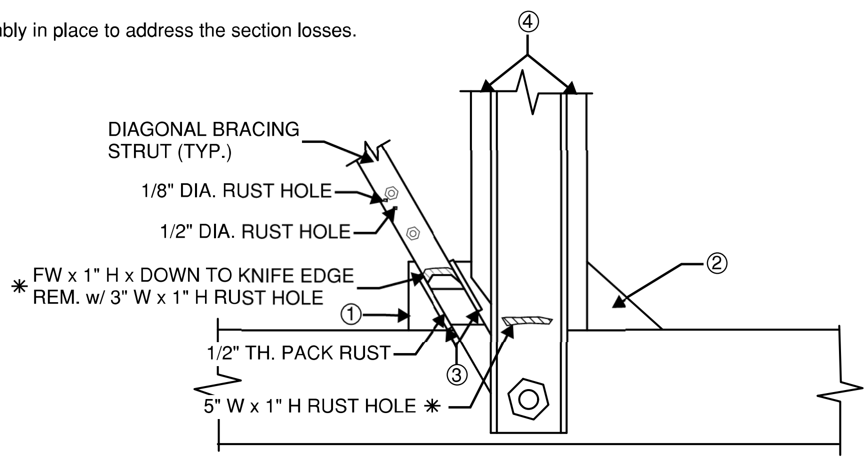
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L7 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

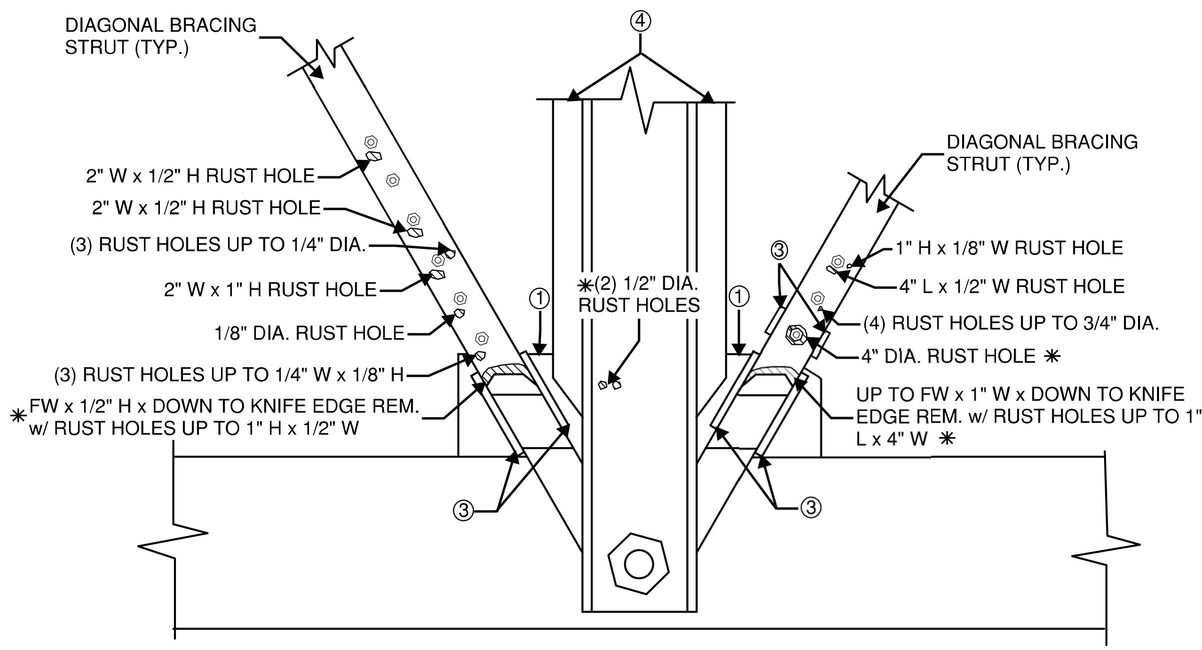
REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)
-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate
-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 32)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L6 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

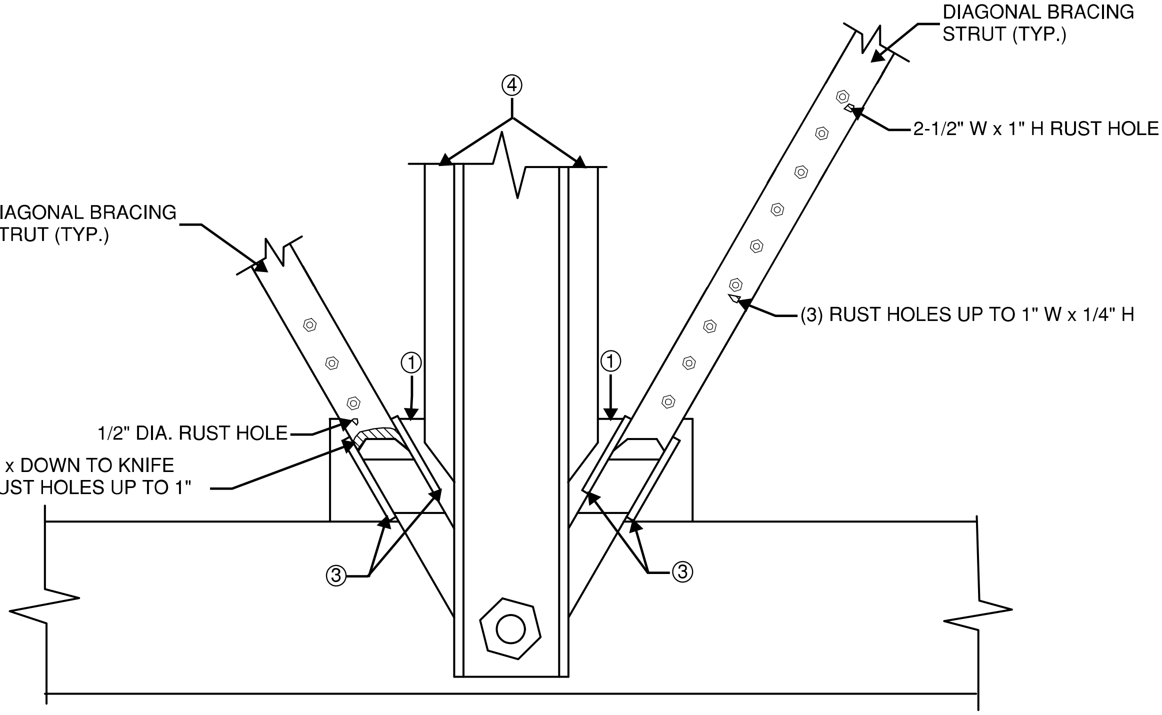
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 33)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L6 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

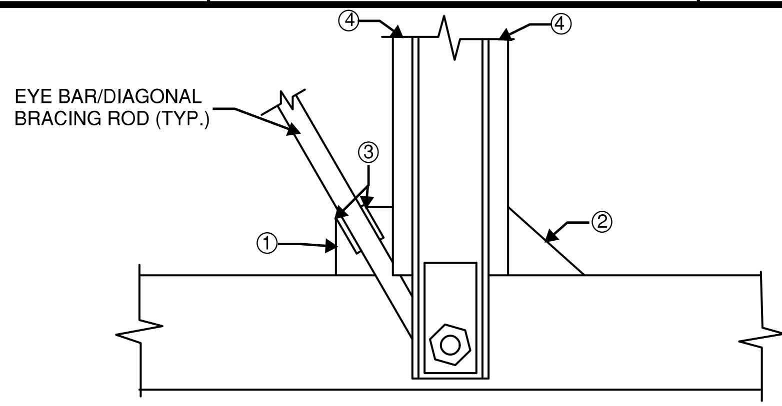
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 34)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L8 NORTH TRUSS NORTH ELEVATION

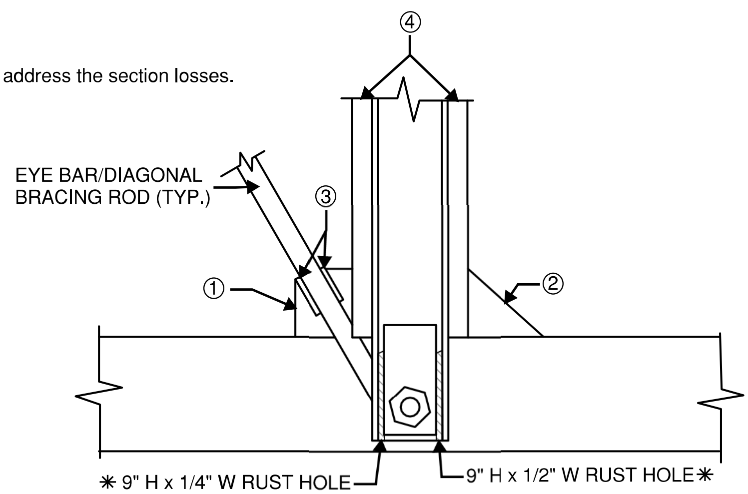
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L9 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

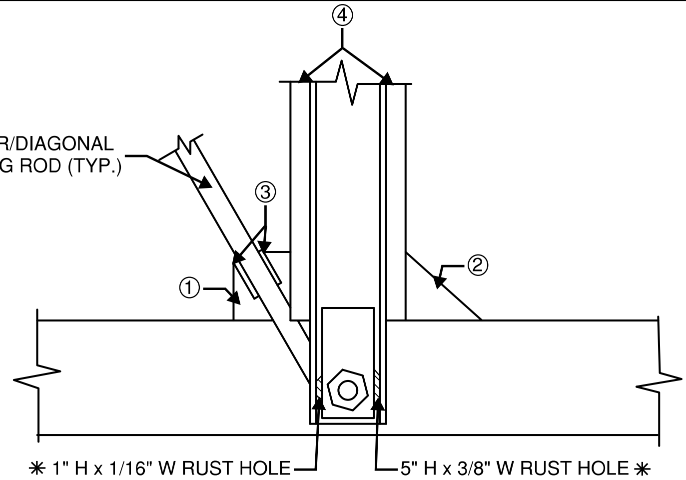
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 35)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2) | DATE: 6/28/2017 | BRIDGE NO.: 03984



L10 NORTH TRUSS NORTH ELEVATION

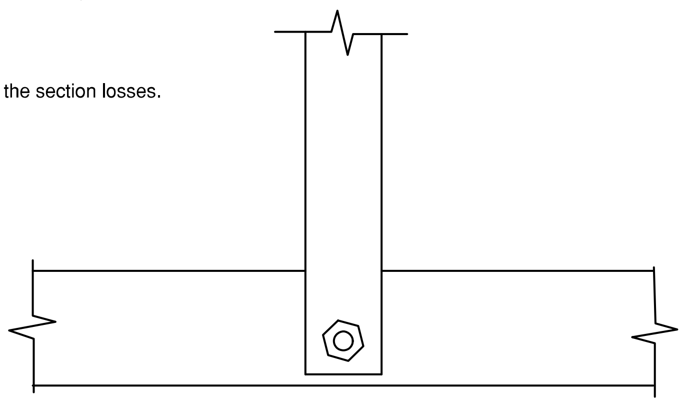
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L11 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

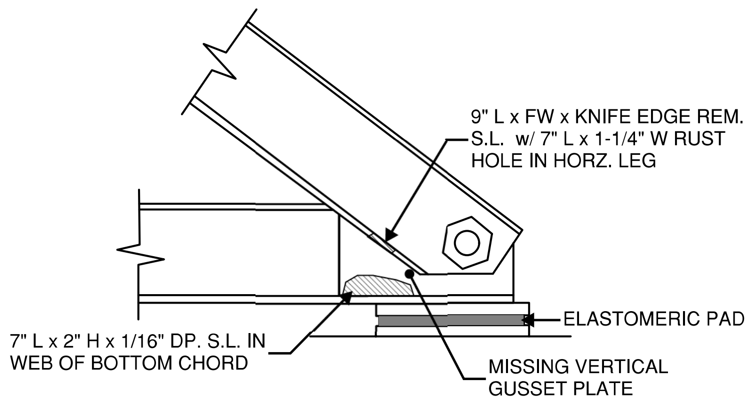
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 36)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

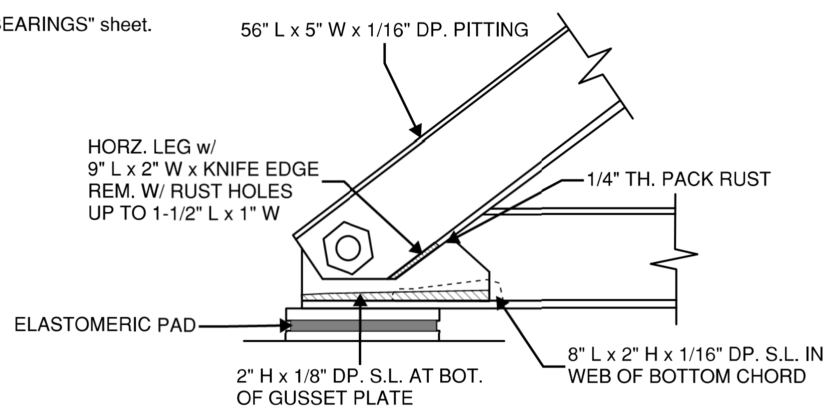


EXPANSION BEARING, L0 NORTH TRUSS AT WEST ABUTMENT

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BEARINGS" sheet.



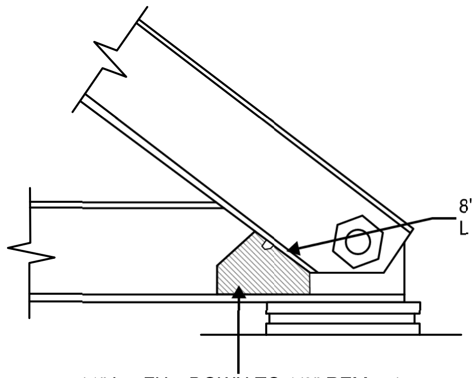
EXPANSION BEARING, L0 SOUTH TRUSS AT WEST ABUTMENT

(N.T.S.)

(SKETCH 37)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

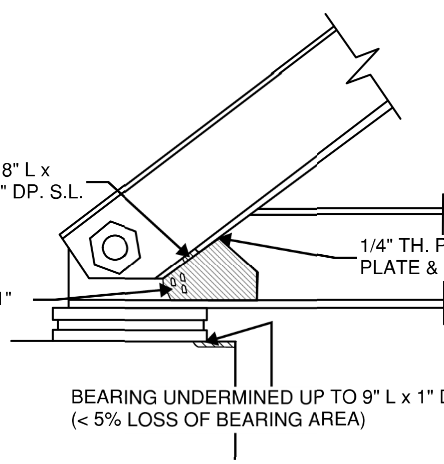


FIXED BEARING, L12 SOUTH TRUSS AT EAST ABUTMENT

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BEARINGS" sheet.



FIXED BEARING, L12 NORTH TRUSS AT EAST ABUTMENT

(N.T.S.)

(SKETCH 38)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

GENERAL NOTES - BEARINGS:

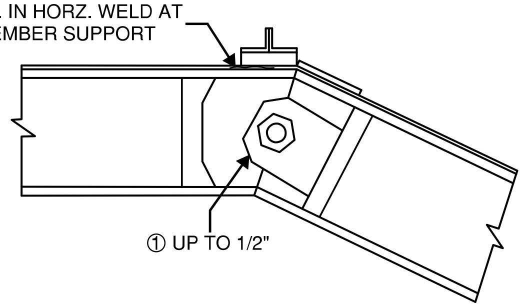
- Moderate to heavy accumulation of pack rust and timber debris atop the bearing plates.

- Areas of peeling paint with light to moderate rust.

(SKETCH 39)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

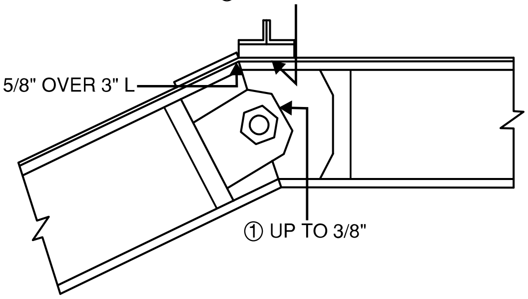
CREW: BJS, SR (GM2) | DATE: 6/29/2017 | BRIDGE NO.: 03984



U1 SOUTH TRUSS NORTH ELEVATION
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U1 SOUTH TRUSS SOUTH ELEVATION
(N.T.S.)

DETERIORATION LEGEND:

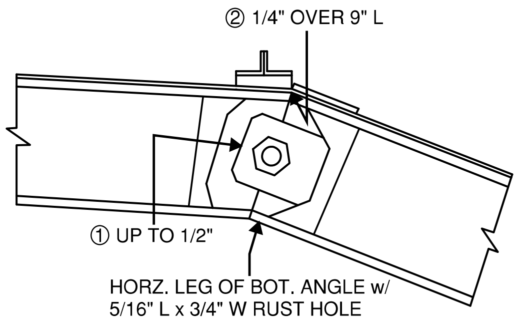
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 40)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

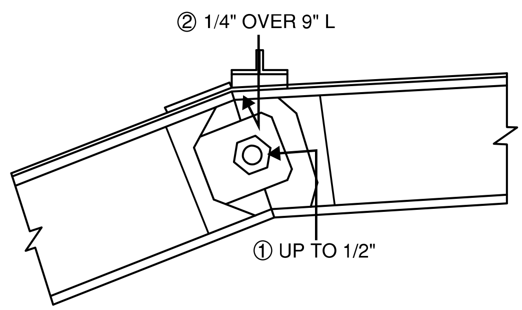


U2 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U2 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

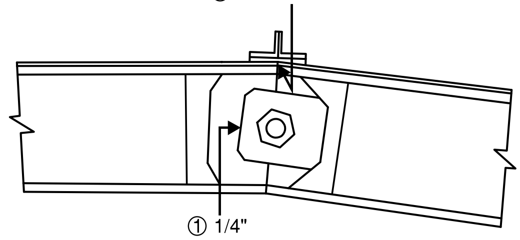
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 41)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

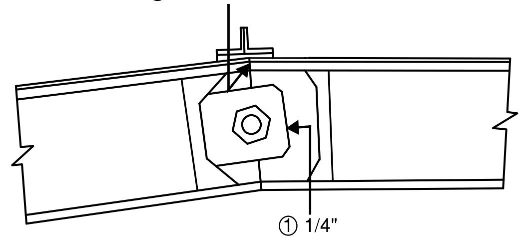


U3 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U3 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

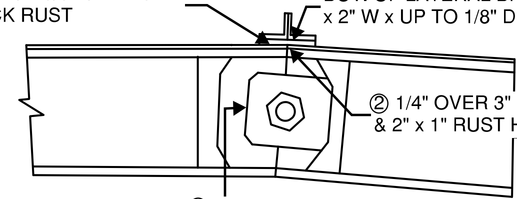
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 42)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

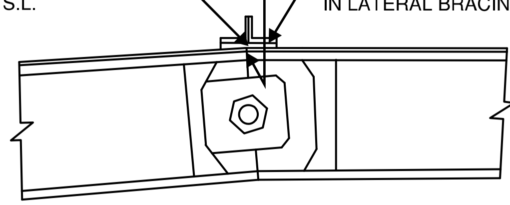


U4 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U4 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

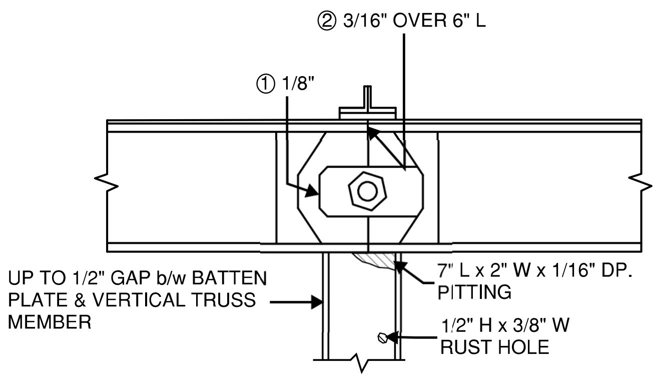
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 43)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

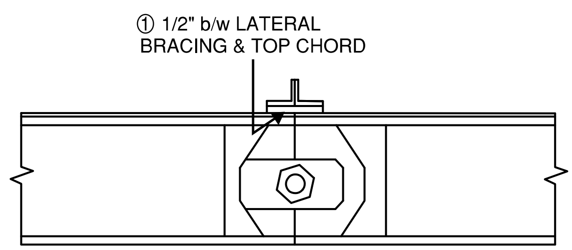


U5 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U5 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

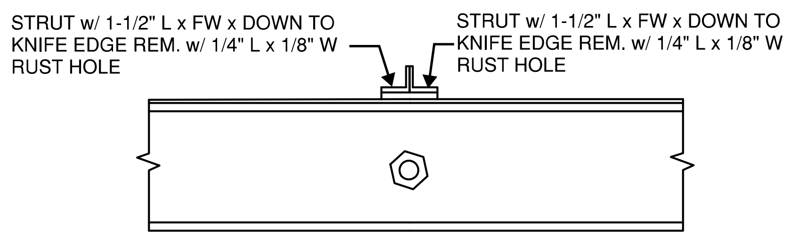
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 44)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

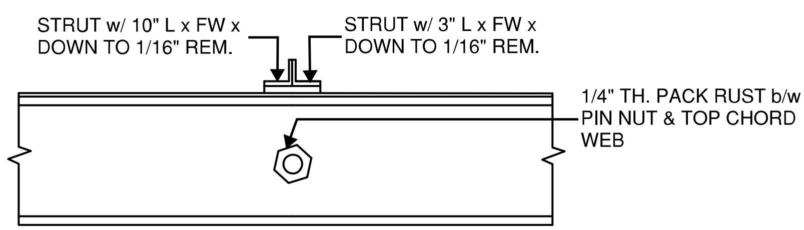


U6 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U6 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

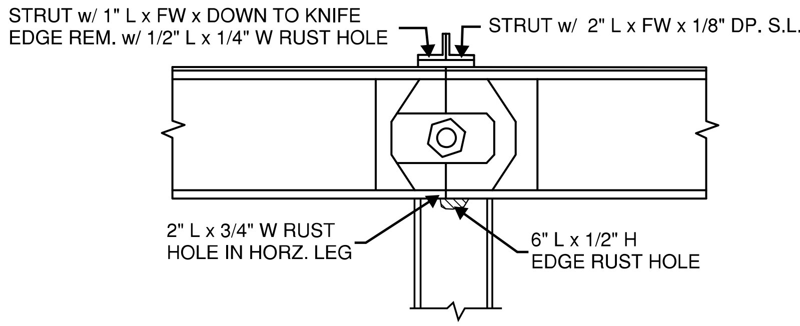
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 45)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

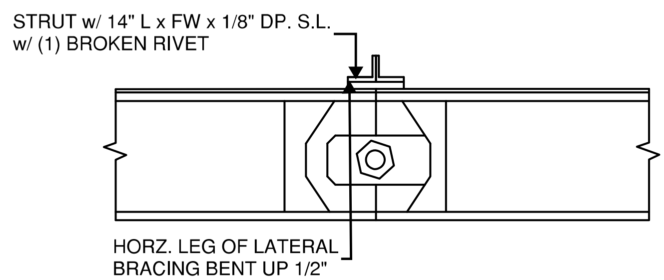
CREW: BJS, SR (GM2) | DATE: 6/29/2017 | BRIDGE NO.: 03984



U7 SOUTH TRUSS NORTH ELEVATION
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U7 SOUTH TRUSS SOUTH ELEVATION
(N.T.S.)

DETERIORATION LEGEND:

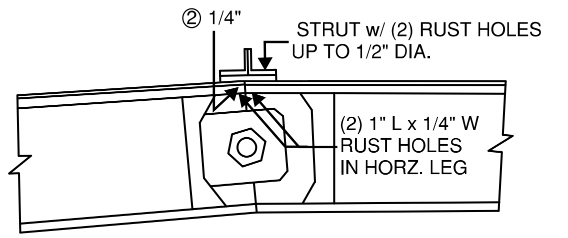
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 46)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

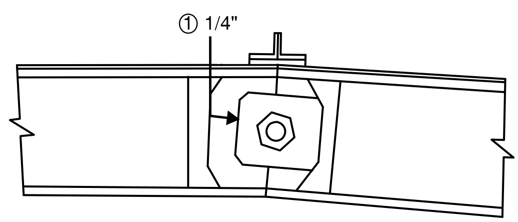


U8 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U8 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

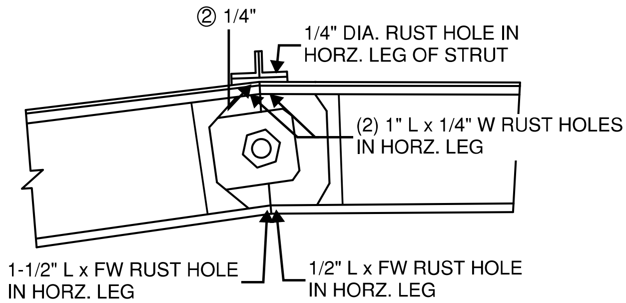
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 47)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

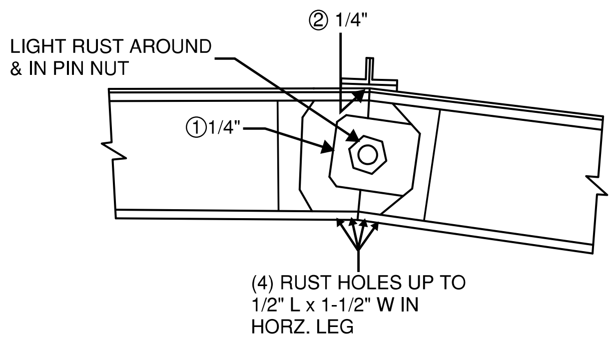


U9 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U9 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

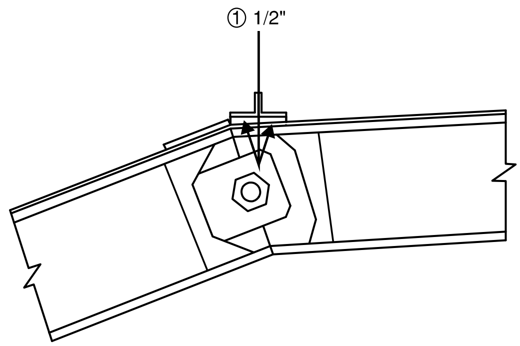
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 48)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

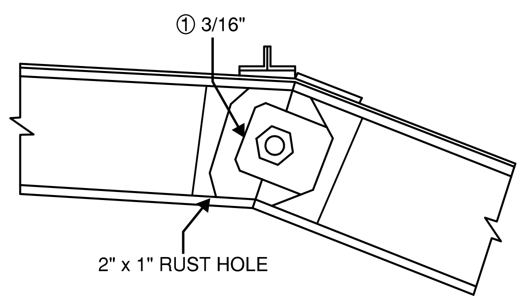


U10 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U10 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

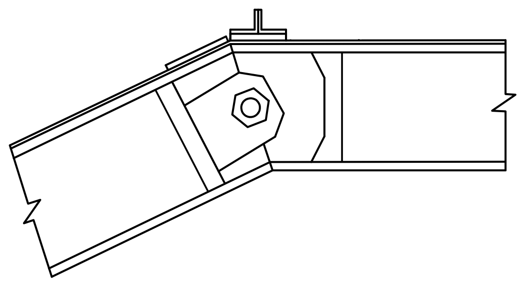
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 49)

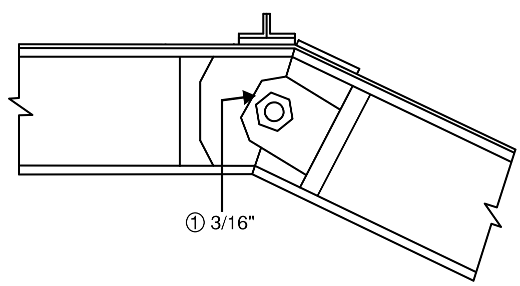
REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



U11 SOUTH TRUSS NORTH ELEVATION
(N.T.S.)

GENERAL NOTES:
- See "GENERAL NOTES - TOP CHORD" sheet.



U11 SOUTH TRUSS SOUTH ELEVATION
(N.T.S.)

DETERIORATION LEGEND:
-① GAP B/W PLATES DUE TO PACK RUST
-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 50)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

GENERAL NOTES - TOP CHORD:

- Random areas of peeling paint with light to moderate rust.

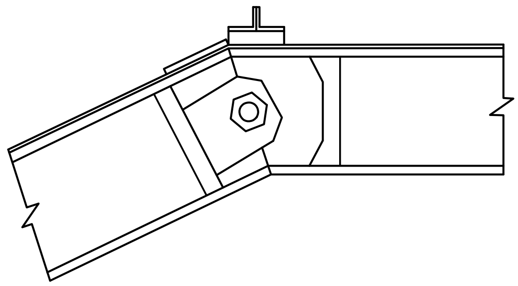
- Random rivets with peeling paint and rust; isolated locations with missing welds and rivet heads with minor head section loss.

- Random locations with bird nests at the truss upper nodes.

(SKETCH 51)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

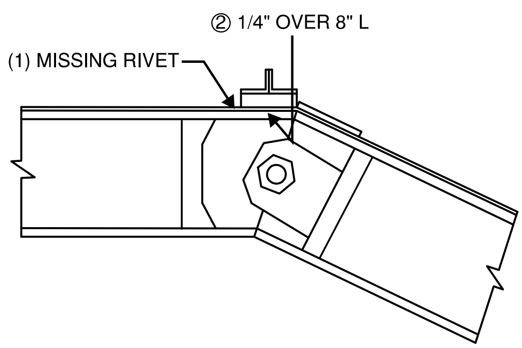


U1 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U1 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

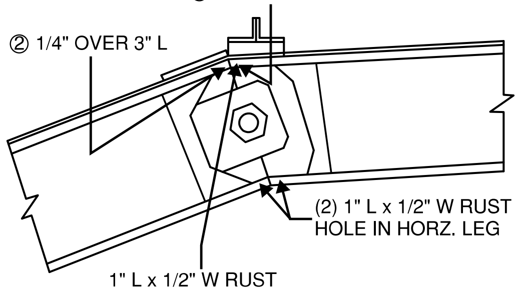
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 52)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

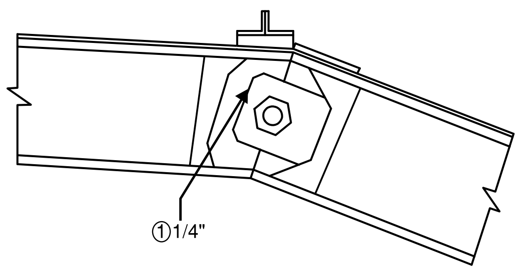


U2 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U2 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

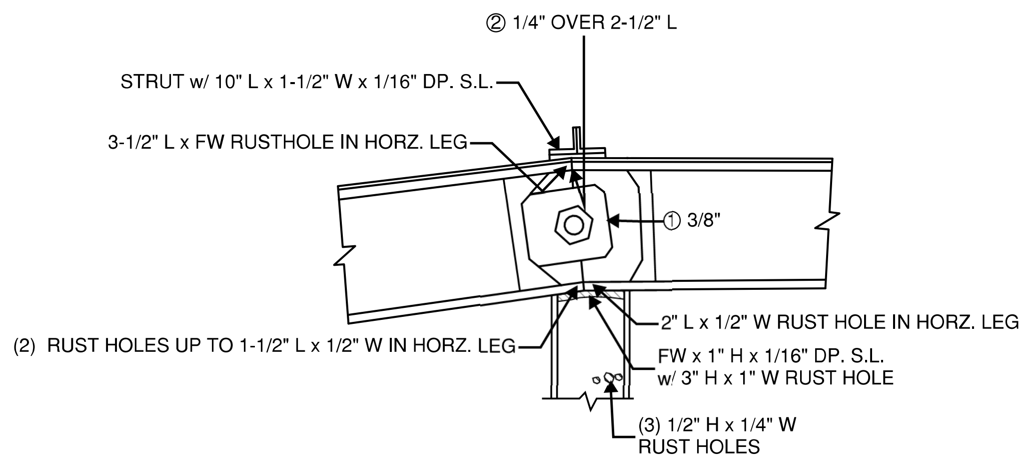
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 53)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

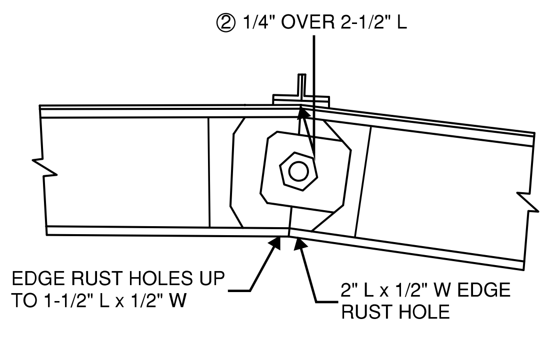


U3 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U3 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

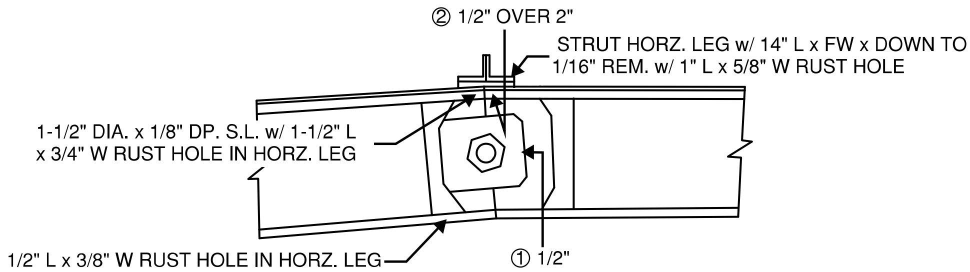
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 54)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

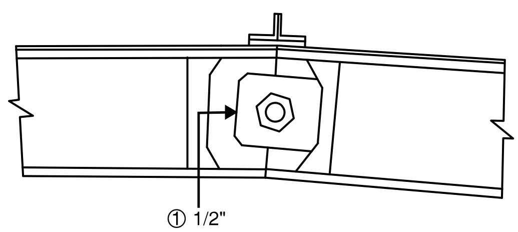


U4 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U4 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

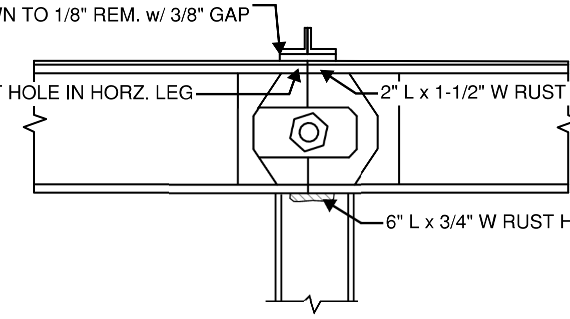
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 55)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2) | DATE: 6/29/2017 | BRIDGE NO.: 03984

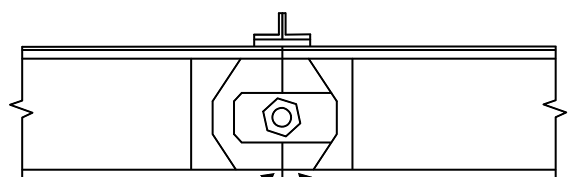


U5 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U5 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

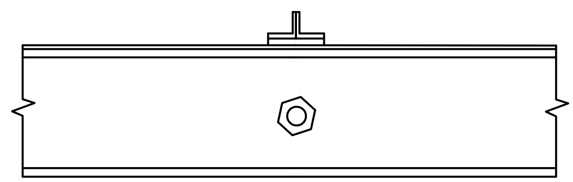
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 56)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

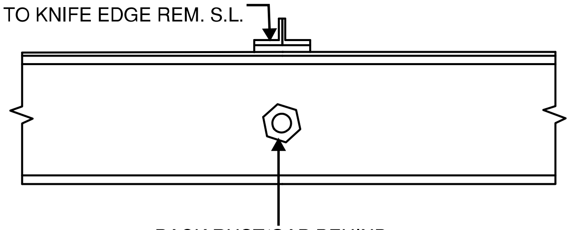
CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



U6 NORTH TRUSS SOUTH ELEVATION
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U6 NORTH TRUSS NORTH ELEVATION
(N.T.S.)

DETERIORATION LEGEND:

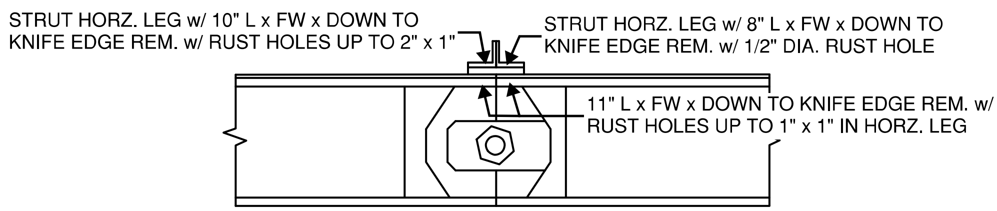
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 57)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

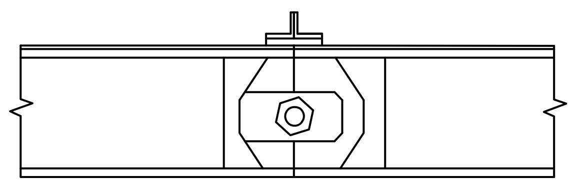


U7 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U7 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

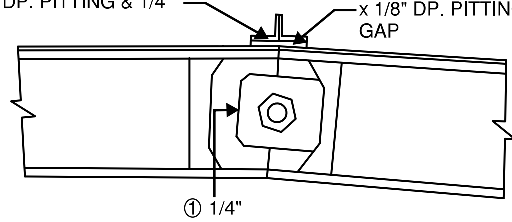
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 58)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

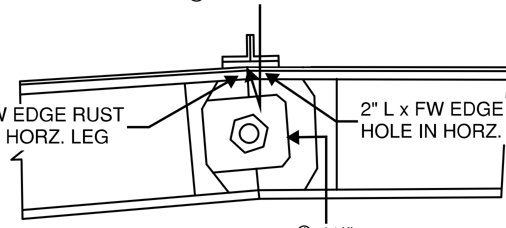


U8 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U8 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

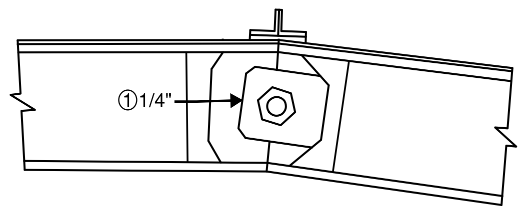
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 59)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

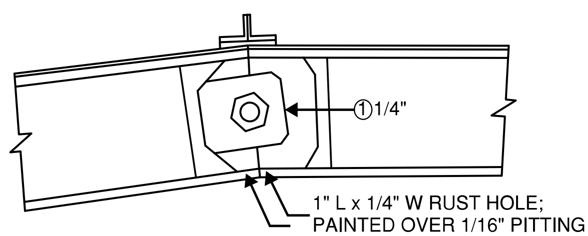


U9 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U9 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

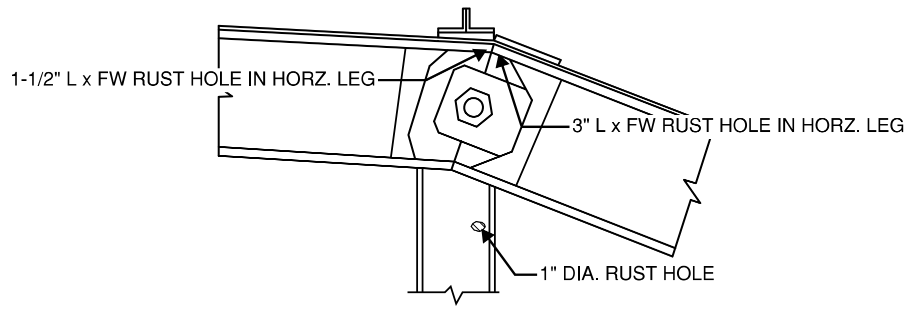
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 60)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

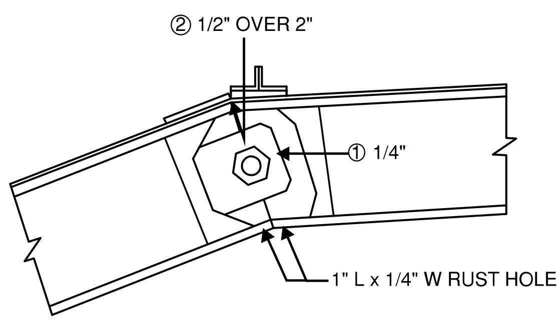


U10 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U10 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

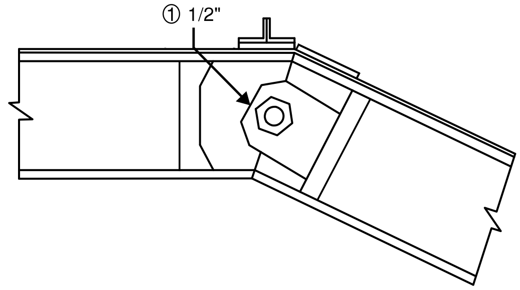
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 61)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

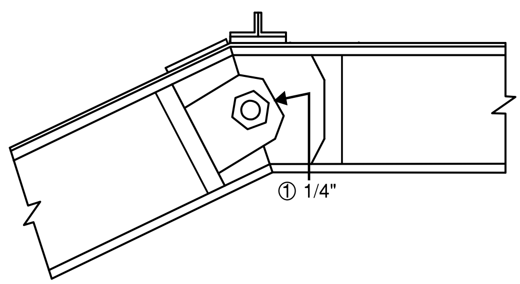


U11 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U11 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

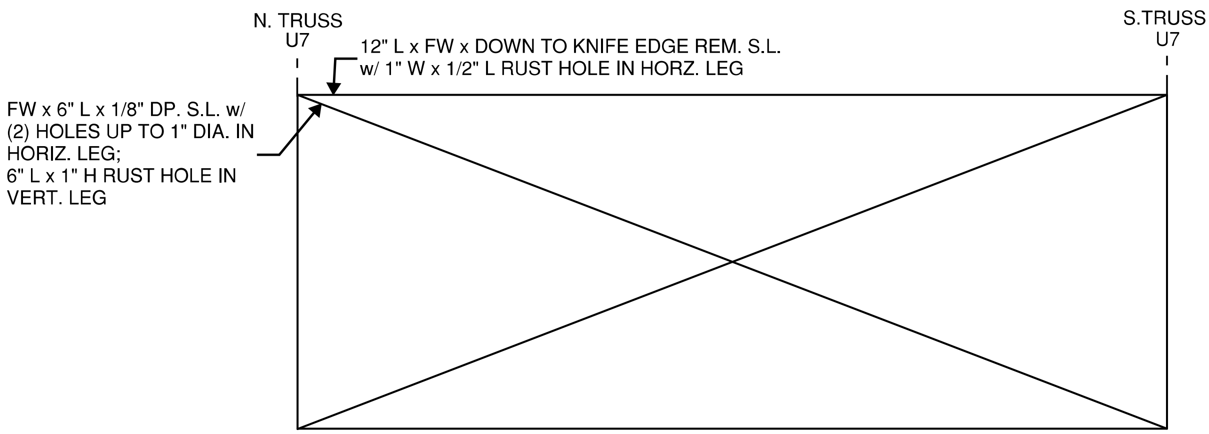
DETERIORATION LEGEND:

-① GAP B/W PLATES DUE TO PACK RUST

(SKETCH 62)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

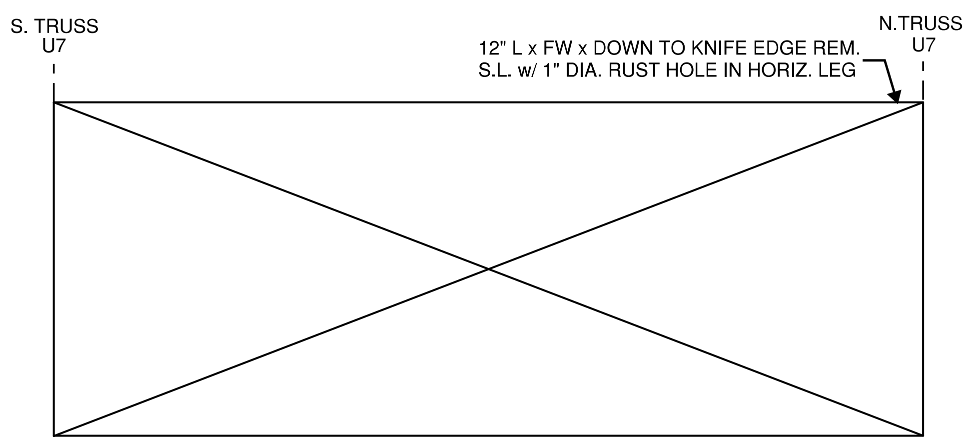


SWAY BRACING AT TOP FOR TRUSS AT U7 LOOKING EAST

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - SWAY BRACING" sheet.



SWAY BRACING AT TOP FOR TRUSS AT U7 LOOKING WEST

(N.T.S.)

(SKETCH 63)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

GENERAL NOTES - SWAY BRACING:

- Sway bracing present at L3, L5, L7, L9 chords.

- Bracings with peeling paint and light to moderate rust.

- Gaps up to 3/8" between top strut and diagonal sway bracing members.

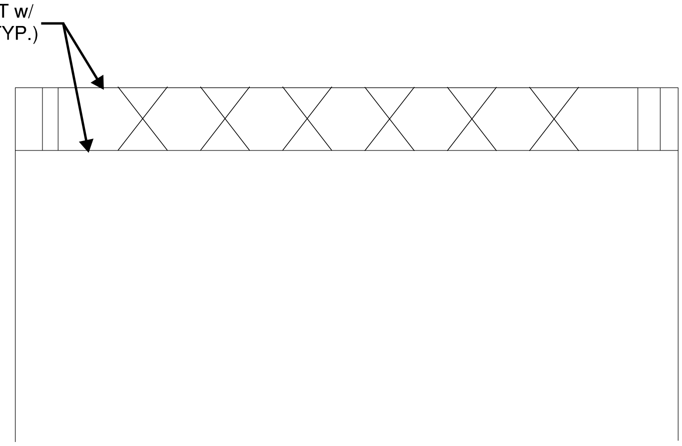
- Bracings atop the top chords with section loss up to full length x full width x down to knife edge remaining (maximum noted in sketches).

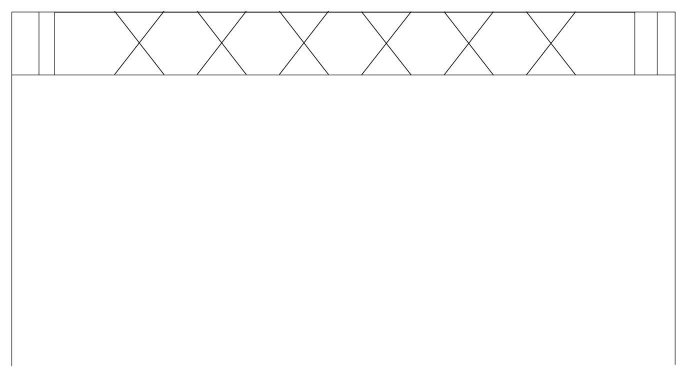
- Horizontal legs of the top struts bent up up to 1/2" due to pack rust between the bracing and top chord of truss.

(SKETCH 64)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984





GENERAL NOTES:

- Portal framing present at L1 and L11 chords.
- Portals with peeling paint and light to moderate rust.

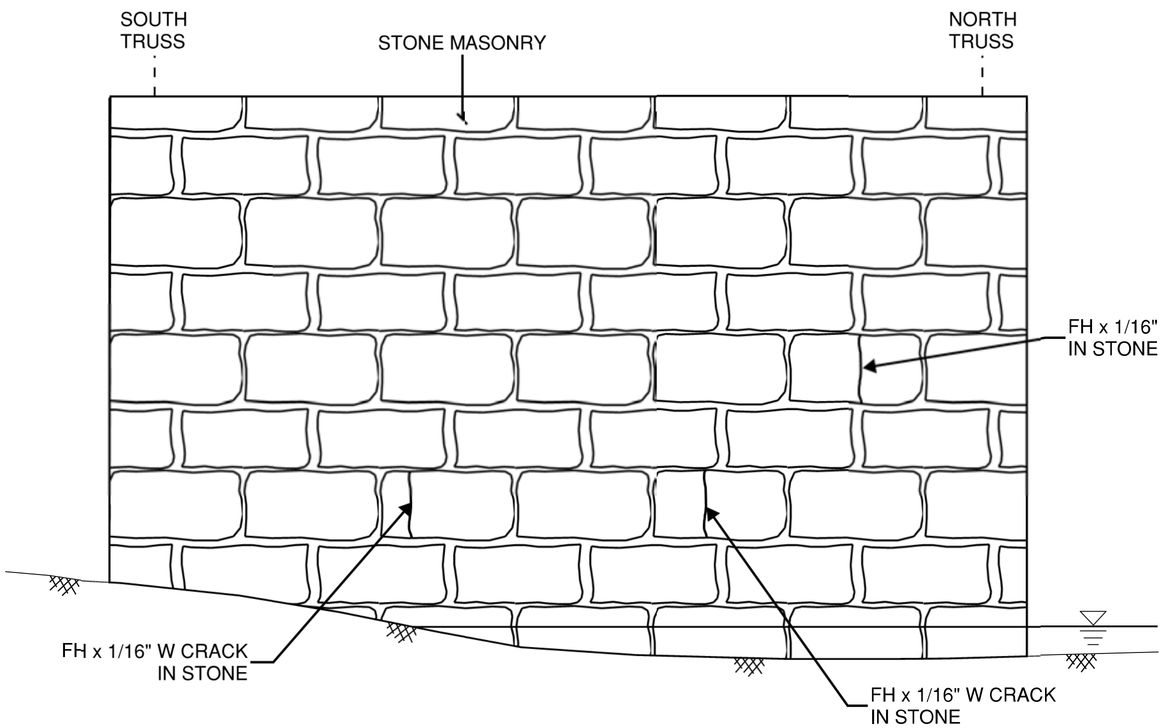
PORTAL FRAMING (TYP.)

(N.T.S.)

(SKETCH 65)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- Random voids between stones due to loss of joint mortar along the base of the stem.
- Random hairline cracks in the joint mortar between stones with efflorescence.

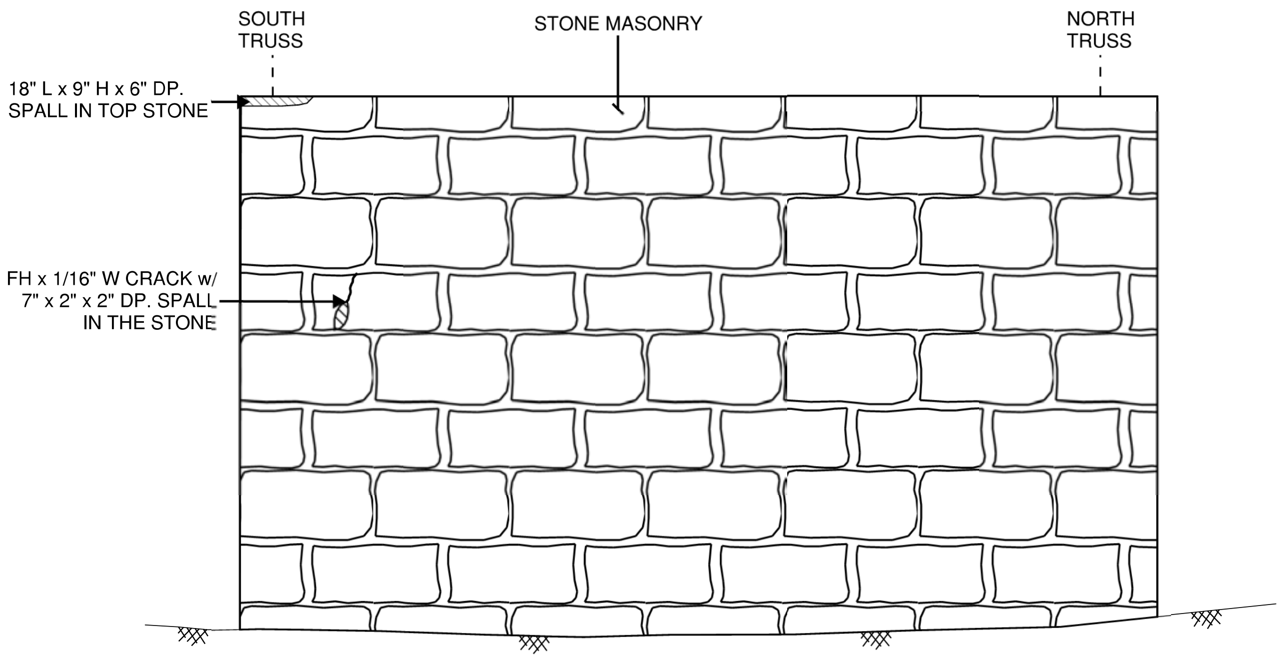
WEST ABUTMENT (1)

(N.T.S.)

(SKETCH 66)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- See "WEST ABUTMENT (1)" sheet.

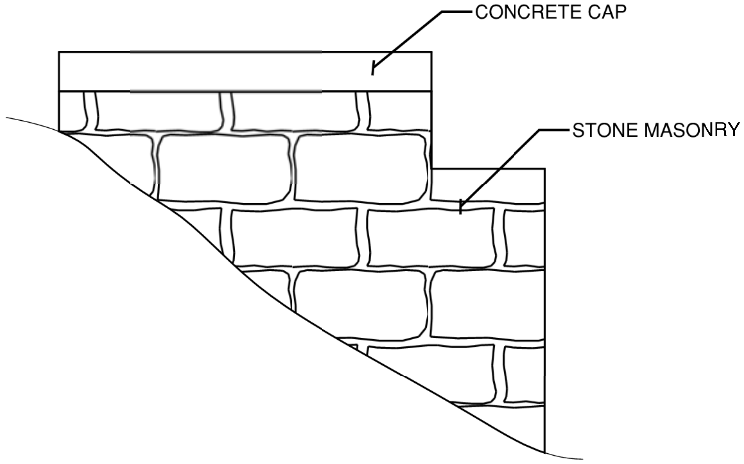
EAST ABUTMENT (2)

(N.T.S.)

(SKETCH 67)

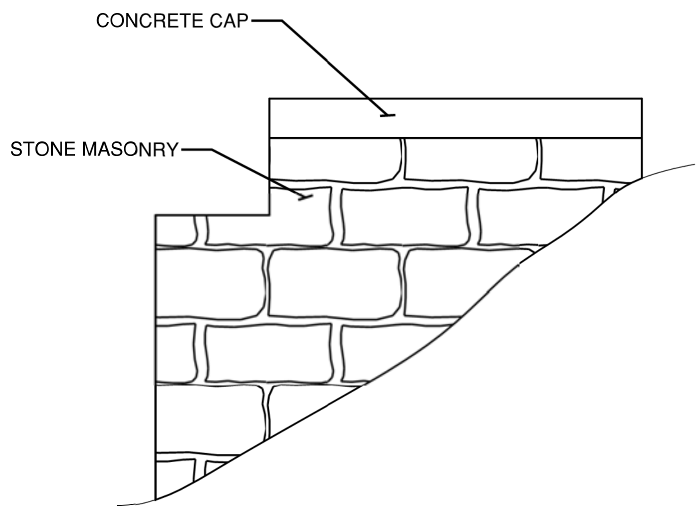
REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



SOUTHWEST WINGWALL (1B)

(N.T.S.)



NORTHWEST WINGWALL (1A)

(N.T.S.)

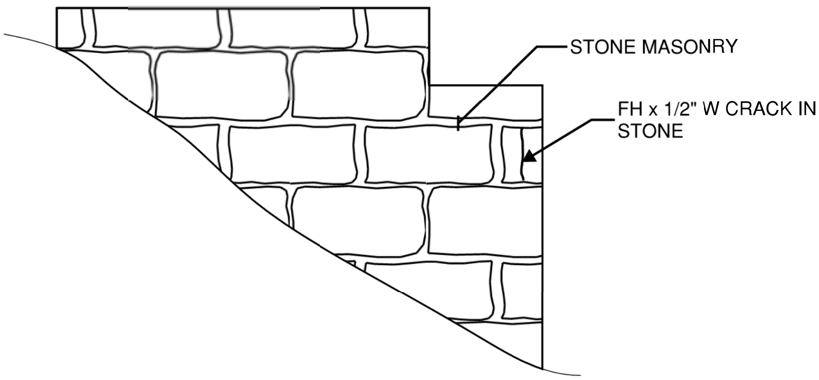
GENERAL NOTES:

- Horizontal hairline cracks in the stones at isolated locations.
- Random hairline cracks in the joint mortar between stones.
- Heavy growth of vegetation along the wingwalls.

(SKETCH 68)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

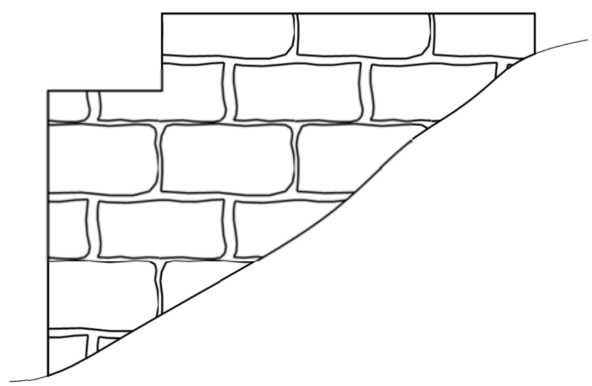


NORTHEAST WINGWALL (2A)

(N.T.S.)

GENERAL NOTES:

- See previous sheet.



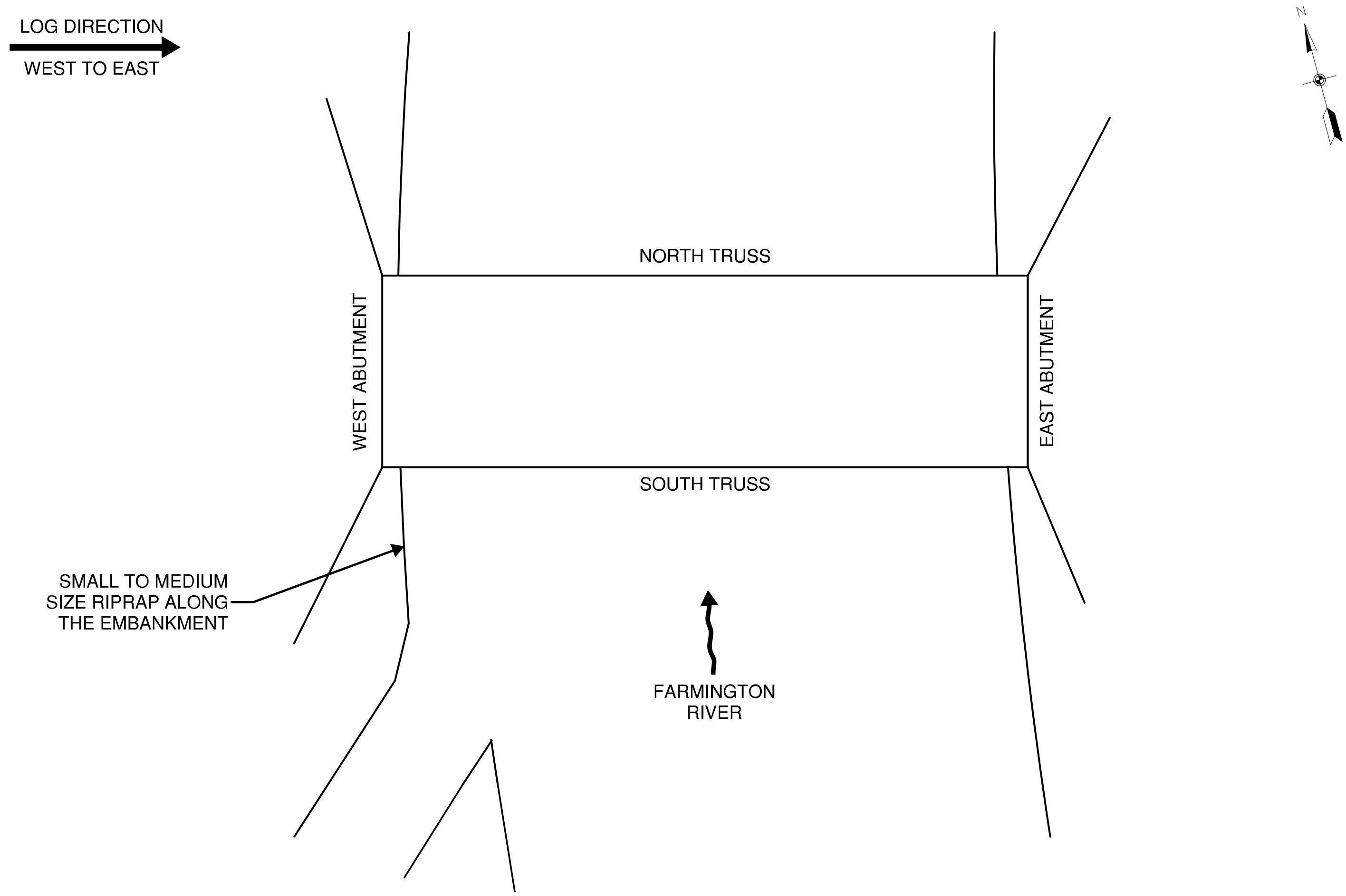
SOUTHEAST WINGWALL (2B)

(N.T.S.)

(SKETCH 69)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- Channel bottom consists of sand with small to medium sized stones.
- Erosion along the channel embankments for up to 3' high x 3' deep with exposed tree roots.
- Heavy growth of vegetation along the channel embankments which is overhanging the channel.

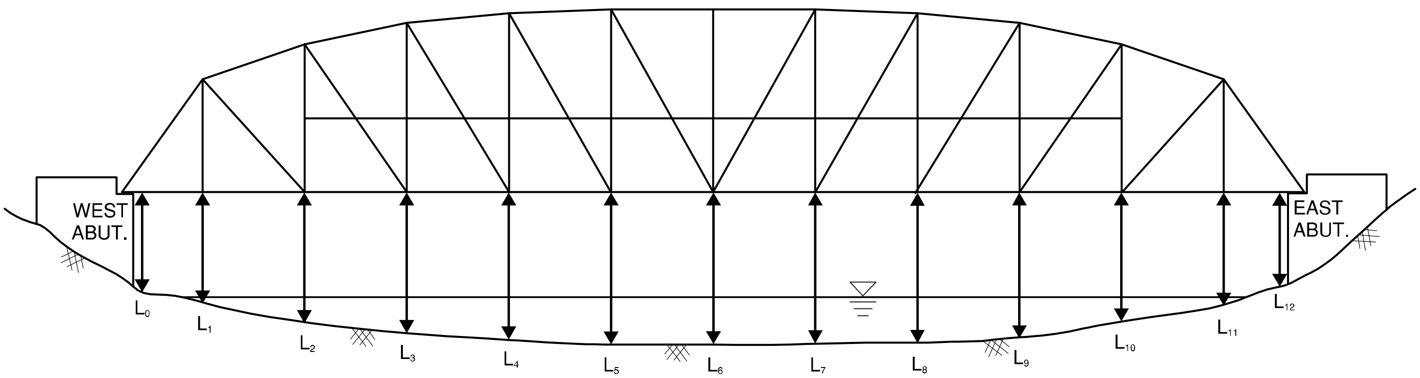
CHANNEL DIAGRAM

(N.T.S.)

(SKETCH 70)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2) | DATE: 6/29/2017 | BRIDGE NO.: 03984



NODE	L_0	L_1	L_2	L_3	L_4	L_5	L_6	L_7	L_8	L_9	L_{10}	L_{11}	L_{12}	Water Depth (L_6)
DROPLINE MEASUREMENT	13'-8"	15'-11"	17'-6"	17'-8"	17'-4"	19'-7"	18'-10"	18'-6"	17'-0"	18'-5"	18'-6"	15'-11"	13'-8"	4'-6"

NOTE:

- Dropline measurements were taken at each node from top of south truss bottom chord.

DROPLINE MEASUREMENTS (SOUTH TRUSS INLET)

(N.T.S.)

(SKETCH 71)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 1: South elevation.

Photo # 2: North elevation.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 3: Bridge from the west approach.

Photo # 4: West approach from the bridge.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 5: Bridge from the east approach.

Photo # 6: East approach from the bridge.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 7: View of the top side of timber deck.

Photo # 8: Isolated timber rot in the deck near midspan with exposed nails.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 9: View of the underside of deck and framing. Note the gaps between the deck planks.

Photo # 10: View of the underside of timber deck between L1 & L2.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 11: Bridge railing and flower beds along the south fascia.

Photo # 12: Bridge railing and flower beds along the north fascia.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 13: Deck end joint at West Abutment. Note the deteriorating joint sealant material.

Photo # 14: Southwest approach railing.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 15: East approach pavement. Note the depressed area.

Photo # 16: Expansion bearing for South Truss at West Abutment.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 17: Fixed bearing for North Truss at East Abutment. Note the section loss with rust holes.

Photo # 18: Expansion bearing for North Truss at West Abutment. Note the missing gusset plate. Also, note the rust hole in horizontal leg of top chord bottom angle.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 19: Floorbeam FB3 between L1 & L2 in South Truss. Note the section loss with rust hole in top flange of the floorbeam.

Photo # 20: Clip angle at Floorbeam FB2 between L1 & L2. Note peeling paint with rust. Also, note the painted over pitting along the bottom of floorbeam web.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 21: Interior view of the bottom chord at L2 in North Truss. Note the deterioration along the pin connection and the repair plates.

Photo # 22: Interior view of the bottom chord at L2 in North Truss. Note the pitting along the interior bottom angle.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 23: Exterior view of bottom chord at L3 in North Truss. Note the deterioration along the pin connection.

Photo # 24: Bottom chord splice connection between L2 & L3 in South Truss.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 25: Pack rust and section loss at the bottom chord splice connection between L2 & L3 in South Truss.

Photo # 26: South elevation of vertical member in South Truss at L3. Note the section loss in the diagonal eye bar and web of truss vertical member. Also, note the repair plates and channel atop the bottom chord.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 27: Vertical member in U5 South Truss. Note the rust hole.

Photo # 28: North elevation of North Truss diagonal member L6-U5 at L6. Note the section loss and rust hole at the connection plates (typical). Also, note the repair plates and retrofit gusset plate.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 29: Section loss in diagonal member L7-U8 at L7 on south side of North Truss. Note the repair plate.

Photo # 30: North elevation of North Truss diagonal member L6-U5 at L6. Note the section loss with rust holes in the channel web.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 31: South elevation of typical repair for eye bar at L10 in South Truss.

Photo # 32: North elevation of top chord member in South Truss at U2. Note the bent down top angles at the connection due to pack rust (typical).

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 33: View of the bottom angles of top chord connection at U3 in North Truss. Note the edge rust holes in the horizontal leg (typical).

Photo # 34: U6 in North Truss. Note the gap behind the pin nut.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 35: Bottom lateral bracing connection near L7 in North Truss. Note the short bolts.

Photo # 36: View of the top strut, sway bracing and lateral bracing between north and south trusses.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 37: Bottom of top strut at North Truss above U4.

Photo # 38: Sway bracing at U7 above North Truss. Note the section loss with rust holes.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 39: North side of U1 in South Truss. Note the crack in the weld.

Photo # 40: West Abutment elevation.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 41: East Abutment elevation.

Photo # 42: Crack in the stone masonry at East Abutment.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 43: Southeast Wingwall elevation.

Photo # 44: Crack in the stone masonry at Northeast Wingwall.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 45: Upstream view of the channel.

Photo # 46: Downstream view of the channel.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 47: Northeast embankment.

Photo # 48: Southeast embankment.

ADDITIONAL BACK-UP MATERIAL

Quick fix keeps bridge in service

Welded reinforcements took load off rusting supports to extend the life of an 85-year-old simple-span through-truss bridge.

The "bridge crisis" is only one of a seemingly unending series of crises that assail us each evening when we open the newspaper or turn on the television set. One small bridge in Connecticut must be included somewhere in the bridge statistics, but it will never be the subject of more than local attention. No one was killed in a sudden collapse, nor were school children forced to dismount and walk across it while their bus followed.

The Drake Hill Road bridge over the Farmington River in Simsbury, Connecticut, is a 185-foot-long, one-lane simple span through-truss structure built in 1892. Town officials were concerned about its evident deterioration, and requested that the Connecticut Department of Transportation inspect the bridge at the town's expense.

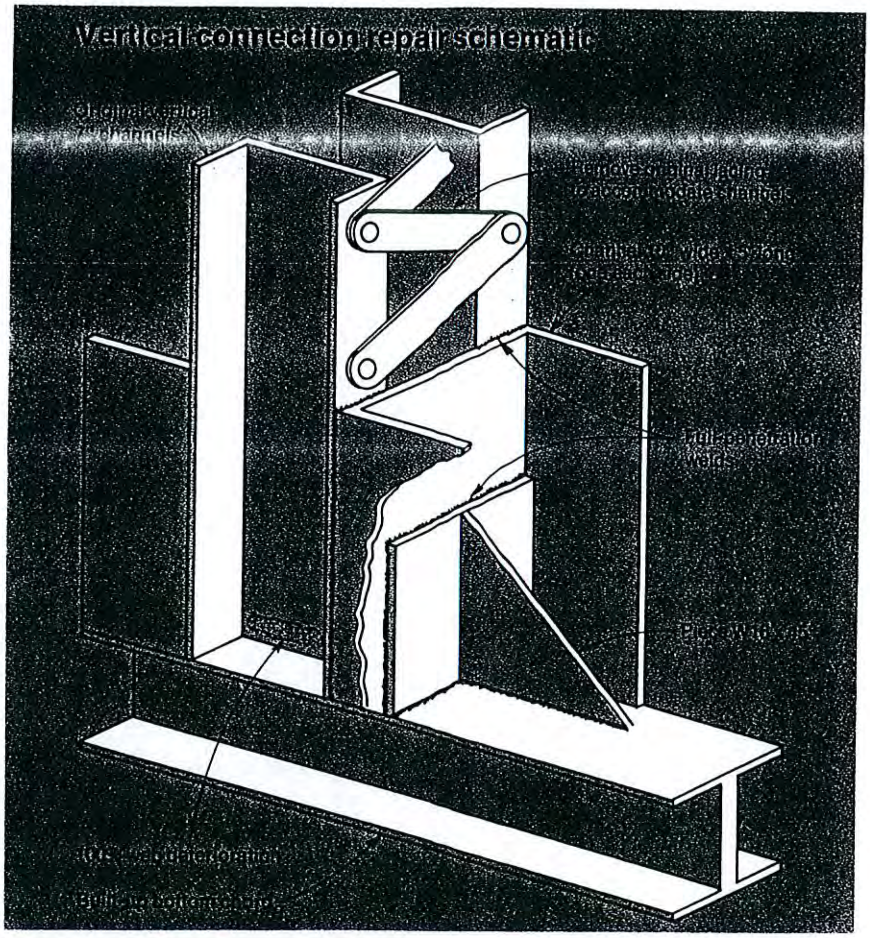
Conn DOT inspectors found severe deterioration of web truss members above the bottom chord. The deterioration was in the splash zone of deicing salts. They recommended that the bridge be declared unsafe, and the town responded by immediately closing it to traffic. This happened in June 1977.

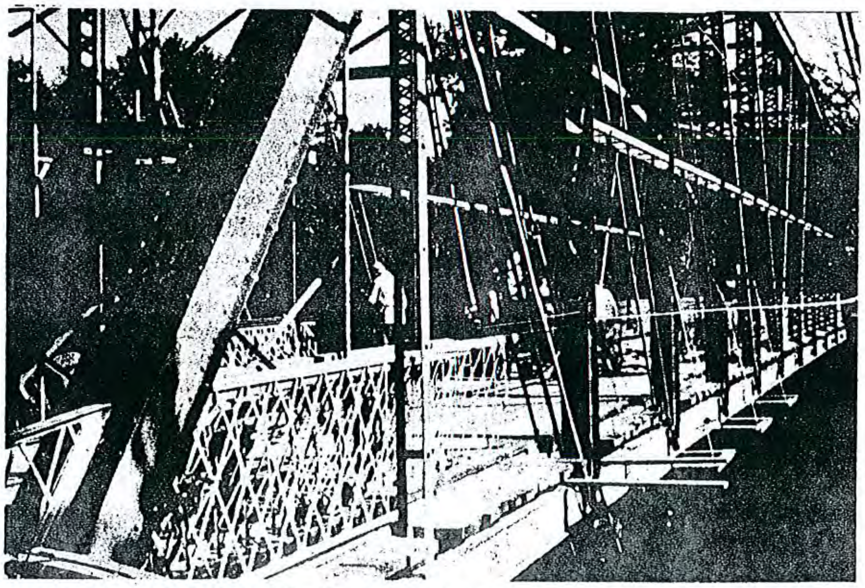
Simsbury is a town of about 22,000, and several thousand of these residents were cut off from the rest of the town by the bridge closing. To reach them, police, fire, and emergency medical vehicles were forced to detour five or six miles.

The bridge also provided an alternate route to and from Hartford, the state capital. When this alternative was available, traffic at the main intersection of the other route was controlled quite adequately by a single traffic light. After the bridge was closed, traffic at this intersection backed up heavily, morning and night, and the town was forced to assign a police patrolman to traffic control duty there.

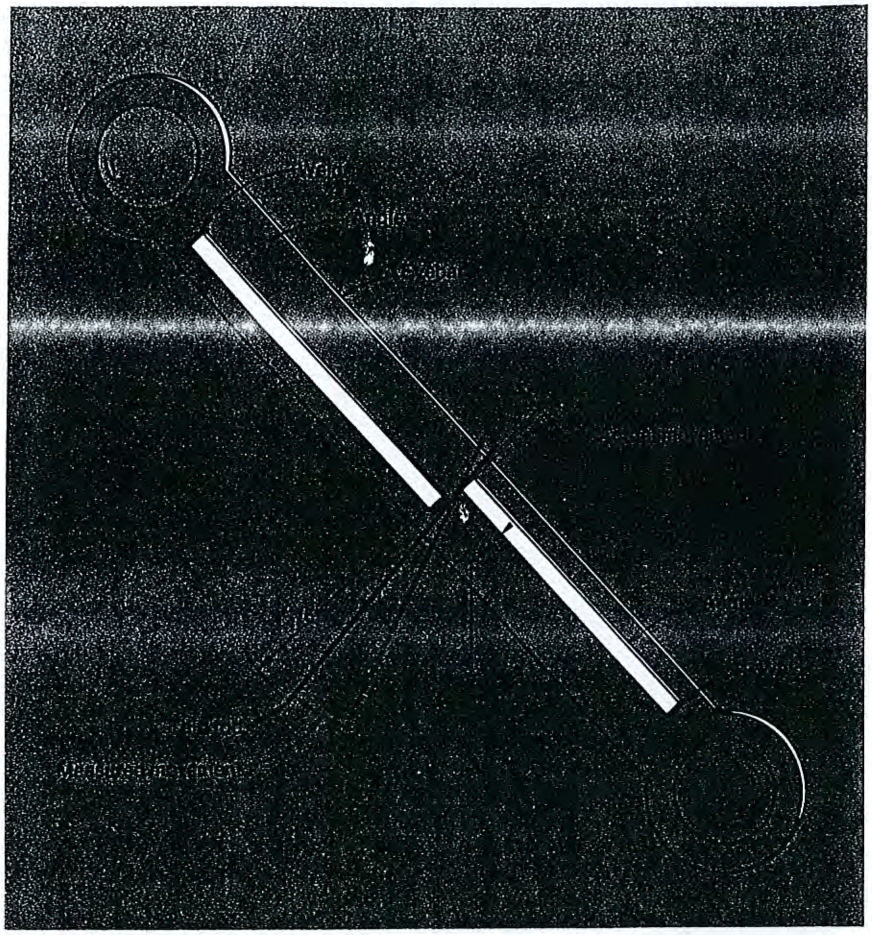
Because of the town's critical need for the bridge, it elected to investigate the possibility that temporary repairs might keep it in service until a permanent replacement could be designed and constructed. It engaged Macchi Engineers to evaluate the bridge's

By A. J. Macchi
Macchi Engineers
Hartford, Connecticut





Prestressing bars relieved the load while the diagonals were repaired.



Measuring stress relief

Tension diagonals were paired flat eyebars. At one connection, an eyebar had completely corroded, leaving the remaining one to carry the entire load. Before the ineffective eyebar could be repaired and made to carry its share of the load, the remaining eyebar had to be relieved of some of its stress. This was done by temporarily attaching two 1¼ inch diameter bars with turnbuckles, then tightening them to take over a portion of the load.

A simple but effective method was used to determine the extent of stress reduction in the remaining eyebar. Two angles whose total length was slightly less than the length of the eyebar were laid along it. One was tackwelded at each end of the eyebar. As the load was removed from the eyebar, it shortened, moving the unattached ends of the angles closer together. Rather than attempt to measure this small movement, the consultant placed a thin rod between the unattached angle legs. Measuring the magnified movement of the rod made it easy for the engineer to calculate the shortening, and therefore the load reduction, in the eyebar.

Using this method, the stress was reduced from 24 ksi to 6 ksi. ■■

structural condition and to prepare plans for rehabilitation. Russel S. Shaw, Simsbury first selectman, and Frank Rossi, director of public works, stipulated that the bridge be reopened to traffic before the onset of winter.

The consultant found major hazards. The original vertical members were seven-inch channel sections, paired and tied with lacing. The webs of these channels were completely rusted through. The tension diagonals in the first three panels from each end were paired flat eyebars. In one location, an eyebar was completely corroded, and the entire load was carried by the remaining bar. Dead load stresses in this bar were estimated to be 24,000 pounds per square inch (24 ksi).

The vertical struts were rehabilitated by removing the steel lacing and replacing it with five-foot lengths of 10-inch channel. The channel backs were welded to the flanges of the existing seven-inch channels. This formed a box section at the bottom of each vertical strut for added strength.

Load was transferred from the rehabilitated struts to the bottom chord through pieces of W16x45 beam. The web of the W16x45 was tied with a full-penetration weld to the web of the built-up bottom chord. Its flanges were welded to the new 10-inch channels. Ultrasonic testing verified the integrity of the full-penetration welds.

Load from the diagonals was transferred to the W16x45 gusset by four plates. One was welded to the top and bottom of each diagonal on each side of the W16x45 flange. This conversion of the diagonal connections from pinned connections to rigid connections induced some moment into the connected members, but an investigation concluded that these moments were negligible.

Old steel stronger than new

Before the contractor could weld the A36 steel reinforcing to the original structural steel which had been fabricated in 1892, testing was necessary. Test specimens were composed of lacing bars that had been removed from the vertical struts and welded to A36 straps using E7018 electrodes.

The specimens failed at tensile stresses of 61,000 and 64,000 psi. To everyone's surprise, it was the A36 steel which ruptured! When additional tensile tests were made to determine the allowable tensile stresses for the repaired bridge, specimens taken from the vertical strut channels averaged an astonishing yield strength of 53 ksi. One specimen failed at 73,000 pounds per square inch!

The entire rehabilitation project was completed in less than three weeks by Baier Construction Company, Inc., of Hartford. The total contract cost was $22,000. At the completion of the work, the bridge was load tested with an 11-ton truck.

In October 1977, just 111 days after the unexpected closing of the bridge, it was reopened to traffic, posted for a weight limit of five tons and a maximum speed of 20 mph.

The bridge repair has eased Simsbury's problems considerably, but the town will not be adequately served until a new bridge, now under design, is actually in place. Although police cars and ambulances can now respond across the bridge, and private automobiles can use it to relieve commuter congestion, fire protection for the residents on the "wrong side of the bridge" is still inadequate. The volunteer fire fighters who protect Simsbury can take their cars directly to that area, but their pumper still must make the detour. ■■

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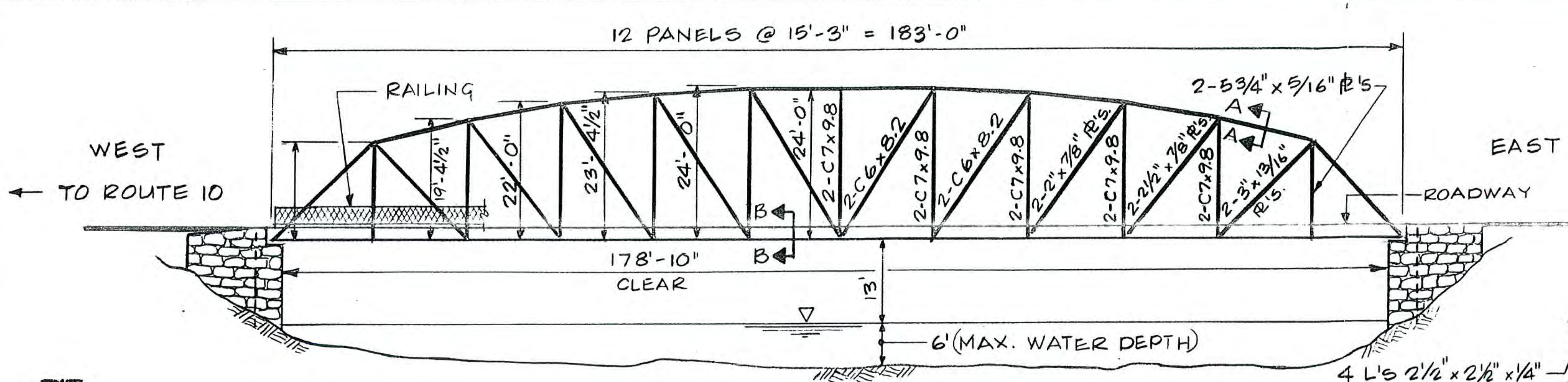
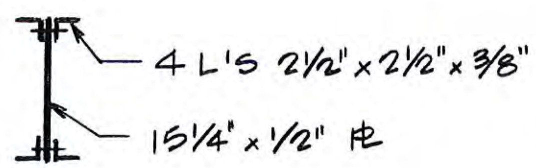
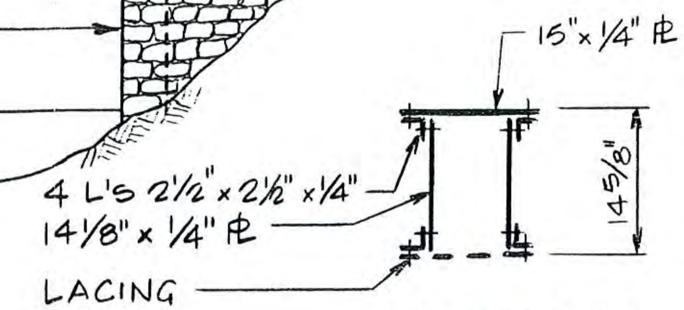


BRIDGE ELEVATION

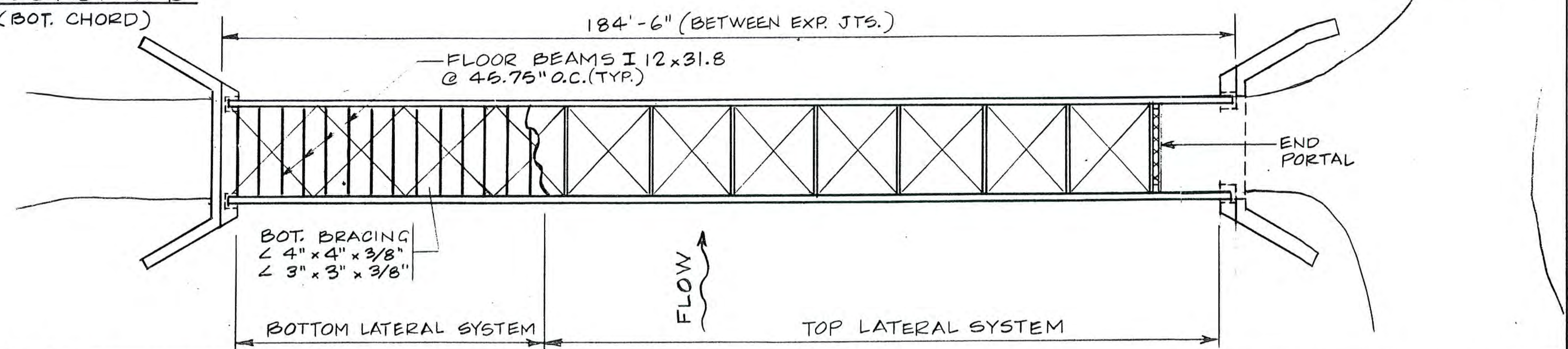
SCALE: 1" = 20'



SECTION B-B
(BOT. CHORD)

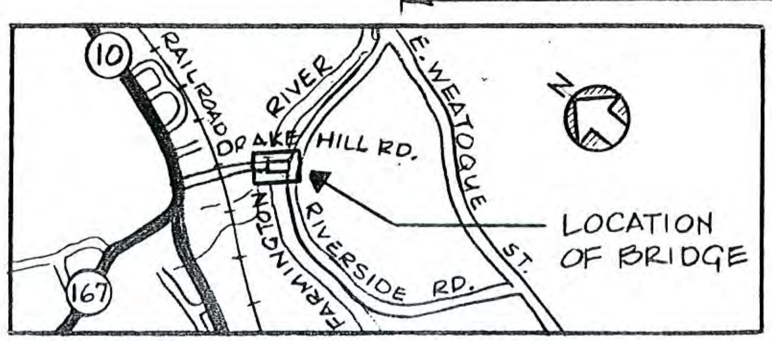


SECTION A-A
(TOP CHORD)



BRIDGE PLAN

SCALE: 1" = 20'

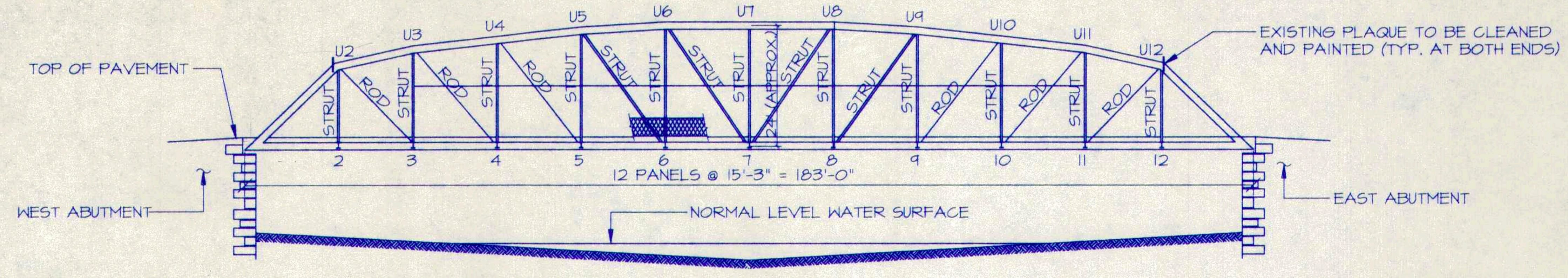


LOCATION PLAN

DRAKE HILL ROAD
OVER
FARMINGTON RIVER
SIMSBURY, CONN.

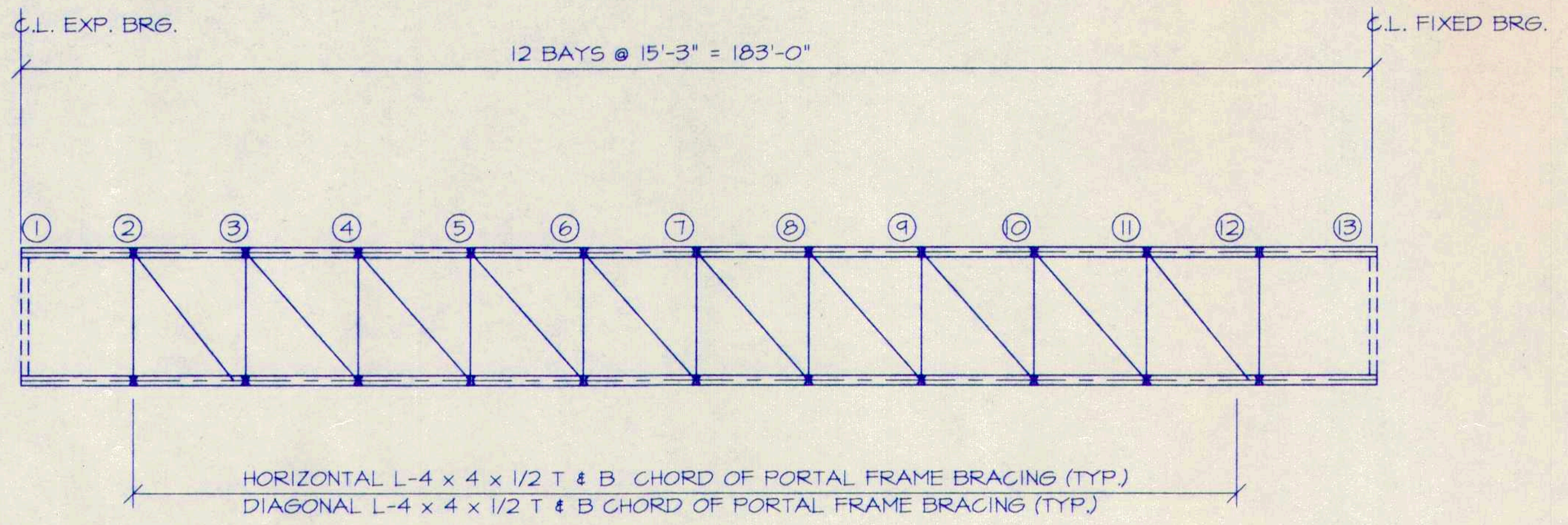
PLAN & ELEVATION	DWG.# 1

PLANS PREPARED BY
MACCHI ENGINEERS - HFTD, CT.
JUNE 1983



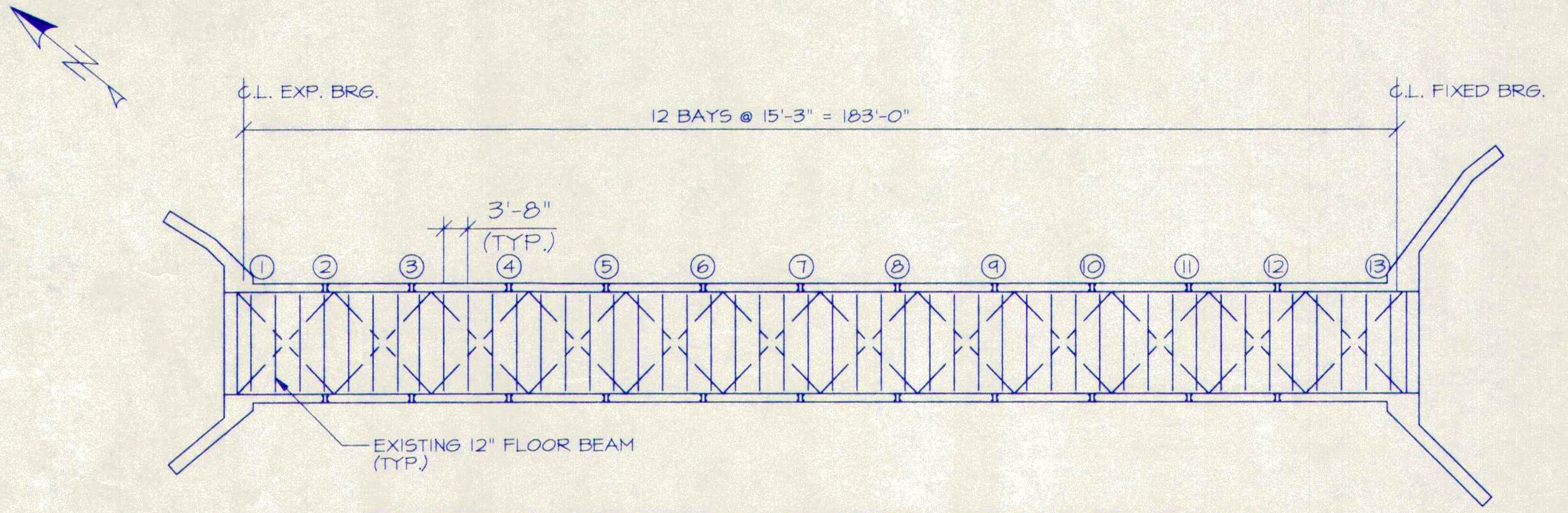
EXISTING BRIDGE ELEVATION (LOOKING UPSTREAM)

SCALE: 1" = 20'



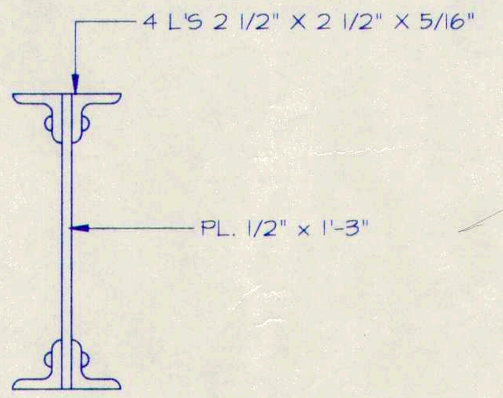
EXISTING TOP CHORD BRACING PLAN

SCALE: 1" = 20'



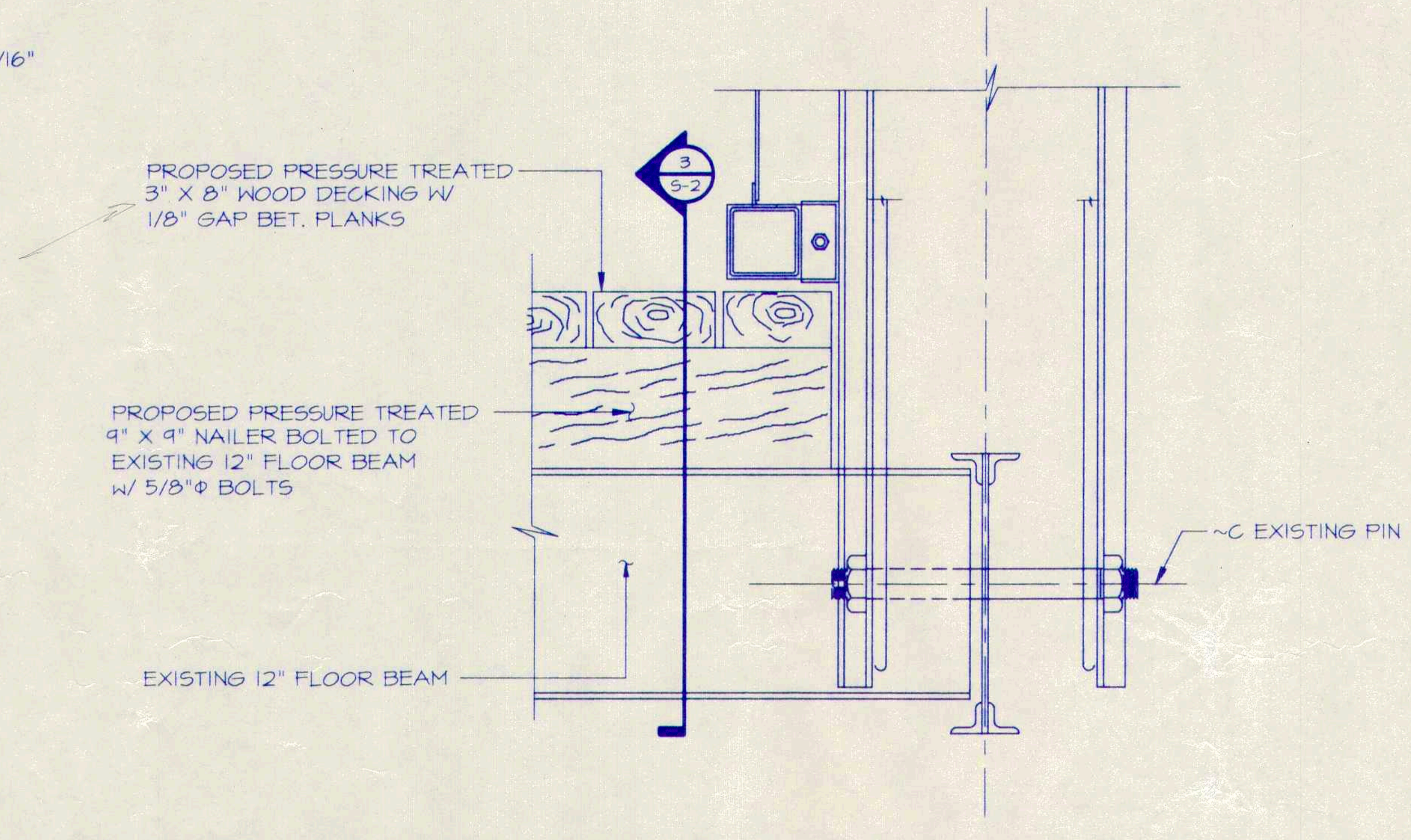
EXISTING BOTTOM CHORD FRAMING PLAN

SCALE: 1" = 20'



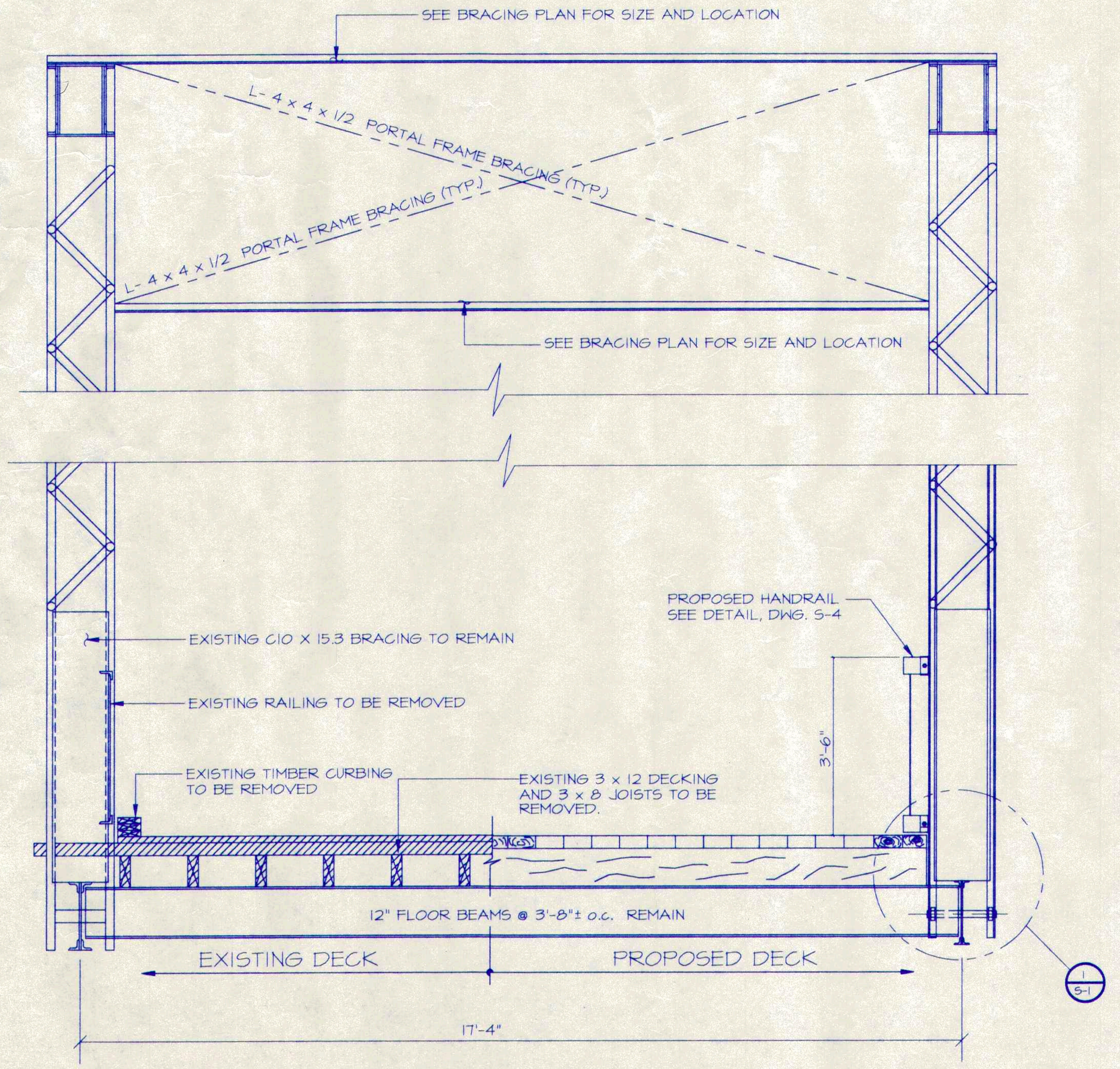
TYPICAL BOTTOM CHORD

SCALE: 1 1/2" = 1'-0"



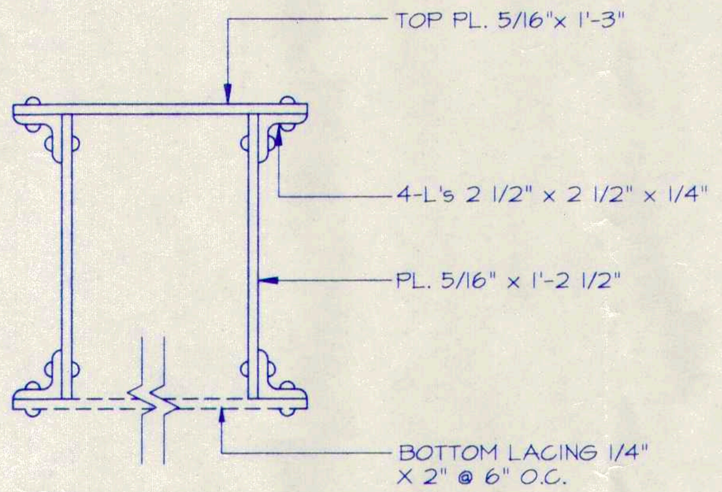
TYPICAL PROPOSED BRIDGE DECK DETAIL

SCALE: 1 1/2" = 1'-0"



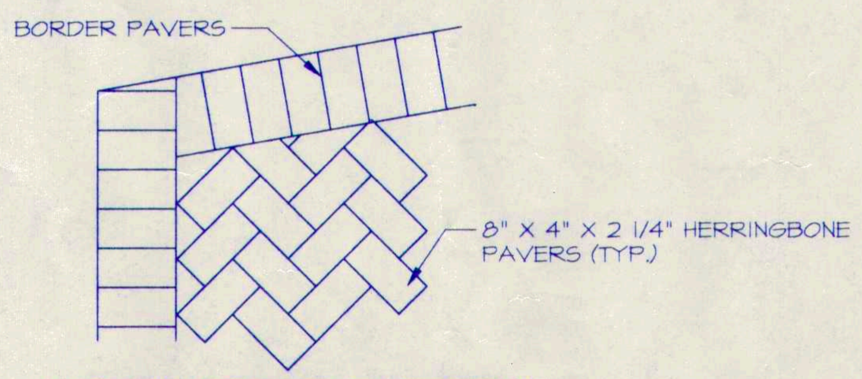
BRIDGE SECTION

SCALE: 1/2" = 1'-0"



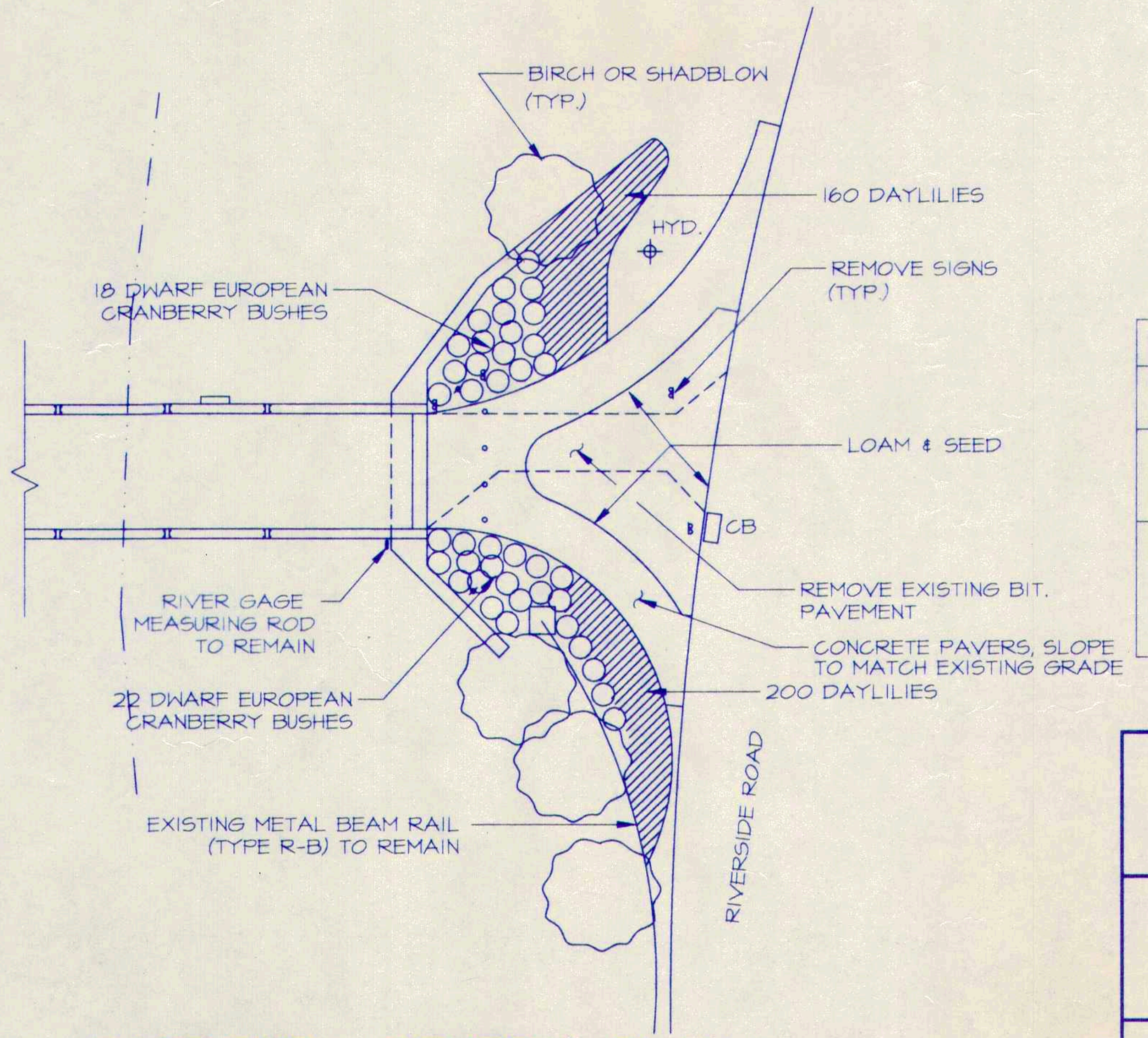
TYPICAL TOP CHORD

SCALE: 1 1/2" = 1'-0"



HERRINGBONE PATTERN BRICK PAVERS

SCALE: 3/4" = 1'-0"



EAST END - GENERAL PLAN
ADD ALTERNATE BID ITEMS

SCALE: 1"=20'

EXISTING TO BE REMOVED

NOTE:
FOR SEDIMENTATION CONTROL SILT FENCE DETAIL, SEE SHEET NO. S-4.

GENERAL NOTES:

DESIGN SPECIFICATIONS:

AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES AND INTERIM SPECIFICATIONS.
BOCA 1987 W/1990 SUPPLEMENT AND CONNECTICUT AMMENDMENTS
BRIDGE WELDING CODE - ANSI/AASHTO/AWS D1.5 - 88.
CONNECTICUT DEPARTMENT OF TRANSPORTATION FORM 814 (1988), INCLUDING SUPPLEMENTS DATED JULY 1993 AND SPECIAL PROVISIONS.

LIVE LOAD: (100 P.S.F.) AFTER REHABILITATION.

EXISTING FLOOR BEAMS AND STRINGERS ARE RATED FOR TEMPORARY H-5 CONSTRUCTION LOADS.

STEEL TYPES AND ALLOWABLE DESIGN STRESSES:

ORIGINAL BRIDGE STEEL: fs = 14,000 P.S.I.

NEW STEEL:
ASTM A-36 STEEL, Fy = 36,000 P.S.I.

REINFORCING STEEL ASTM A-615, GRADE 60. (EPOXY COATED)

SPLASH GUARD PLATES, AND MISC. ITEMS: GALVANIZED AS PER ASTM A-123

REMOVAL OF EXISTING BRIDGE DECK ITEMS AND OTHER MATERIAL:

STAGING SHALL BE PROVIDED UNDER THE BRIDGE FOR THE SAFETY OF WORKERS AND TO PREVENT MATERIALS FROM FALLING INTO FARMINGTON RIVER.

CONSTRUCTION METHODS WHICH MAY DISTORT OR DAMAGE FLOOR BEAMS OR TRUSS MEMBERS WILL NOT BE ALLOWED. SEE SPECIFICATIONS

ALL MATERIAL TO BE REMOVED AND NOT TO BE REUSED SHALL BECOME THE PROPERTY OF THE CONTRACTOR AND WILL BE REMOVED FROM THE SITE AND PROPERLY DISPOSED OF IN ACCORDANCE WITH STATE AND LOCAL ORDINANCES. AS SPECIFIED BY THE ENGINEER, MATERIAL MAY BE DISPOSED OF AT THE LANDFILL ON WOLCOT HILL ROAD. ALL OTHER MATERIAL WILL BE PROPERLY DISPOSED OF BY THE CONTRACTOR.

REPAIRING AND POINTING OF MASONRY WALLS:

SEE SPECIFICATIONS.

EROSION CONTROL

SEDIMENTATION CONTROL SILT FENCE SHALL BE PLACED AT THE TOE OF SLOPES AND AT OTHER LOCATIONS AS REQUIRED TO PREVENT EROSION INTO THE FARMINGTON RIVER.

BRIDGE PAINTING:

ALL STEEL SURFACES, EXCEPT NEW GALVANIZED ITEMS, SHALL BE ABRASIVE BLAST CLEANED IN ACCORDANCE WITH SSPC-SP-10 PRIOR TO THE APPLICATION OF A THREE COAT PAINT SYSTEM AS FOLLOWS:

PRIMER COAT: . . . . ORGANIC ZINC RICH PRIMER
INTERMEDIATE COAT: . . . . EPOXY MASTIC
TOPCOAT: . . . . HIGH BUILD ALIPHATIC URETHANE
COLOR OF TOP COAT: . . . . GREEN
(FED. STD. COLOR NO. 34058)

LOAMING AND SEEDING:

AREAS DISTURBED DURING THIS CONSTRUCTION SHALL BE LOAMED AND SEEDED AS PER THE SPECIFICATIONS.

STRUCTURAL DIMENSIONS:

ALL DIMENSIONS AND ANGLES SHOWN ON THE PLANS ARE BASED ON LIMITED FIELD INVESTIGATION. THE CONTRACTOR SHALL BE RESPONSIBLE FOR FIELD VERIFICATION OF ALL DIMENSIONS AND ANGLES.

CONCRETE:

CLASS 'A' CONCRETE SHALL BE USED FOR ALL WORK ON THE ABUTMENTS AND WINGWALLS. ALLOWABLE DESIGN STRENGTH SHALL BE BASED ON Fc' = 3000 psi.

TIMBER

SEE SPECIFICATIONS.

LANDSCAPE SCHEDULE

(ADD ALTERNATE ITEMS)

BOTANICAL NAME	COMMON NAME	QUANTITY	SIZE AREA
VIBURNAM OPULUS NANUM	DWARF EUROPEAN CRANBERRY BUSH	40	15"-18"SPD.
AMBLANCHIER CANADENSIS OR BETULA PLATYPHYLLA JAPONICA	SHADBLOW WHITESPIRE BIRCH	4	10'-12' HT. MULTI STEM
HEMEROCALLIS FLAVA HEMEROCALLIC HYPERION HEMEROCALLIS STELLA D'ORO HEMEROCALLIS HALLIS PINK	DAYLILIES	90 90 90 90 (QT. CONT.)	12" O.C.

GENERAL PLAN

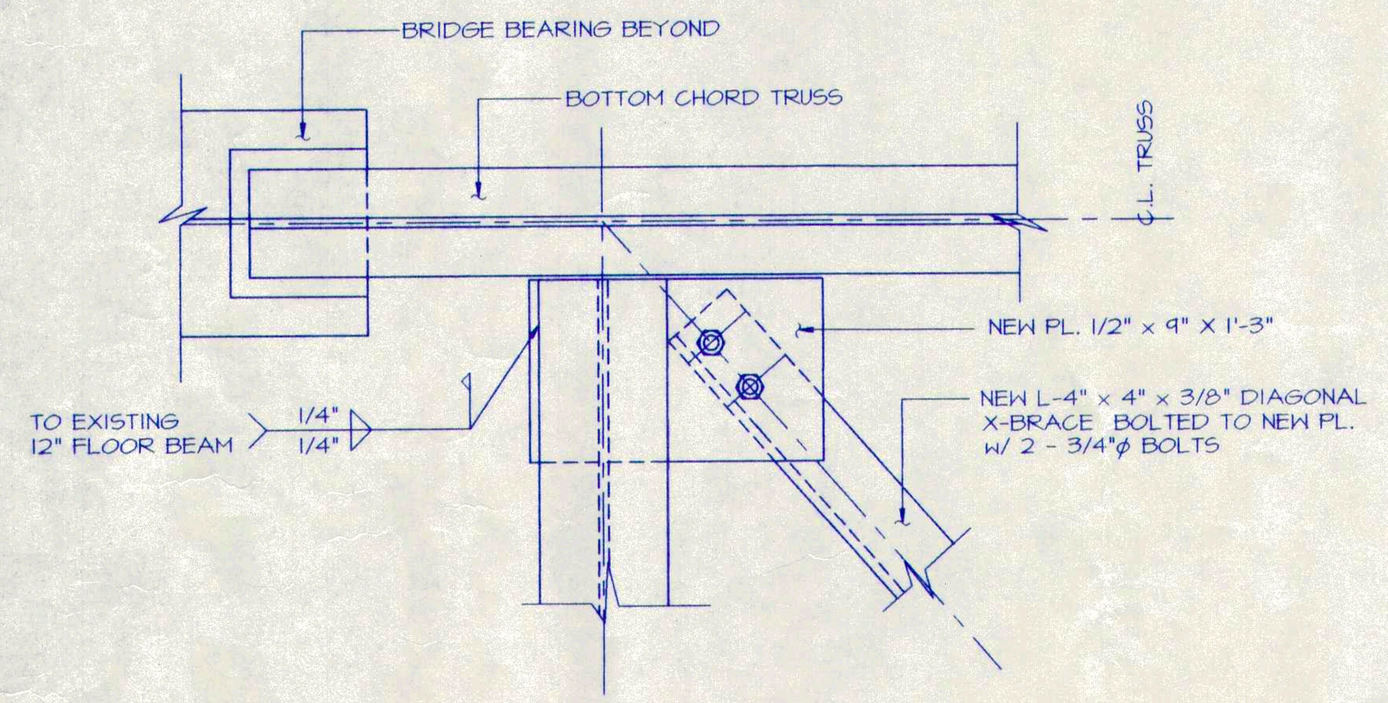
REHABILATATION OF OLD BRIDGE ROAD BRIDGE
SIMSBURY, CONNECTICUT

Macchi Engineers
44 Gillett Street
Hartford, CT 06105
Phone: (203) 549-6190

DRAWN BY:	J.Z.
APPROVED BY:	J.B.
DATE:	8/23/94
SCALE:	AS NOTED
Drawing No.	S-1

BOTTOM CHORD FRAMING PLAN

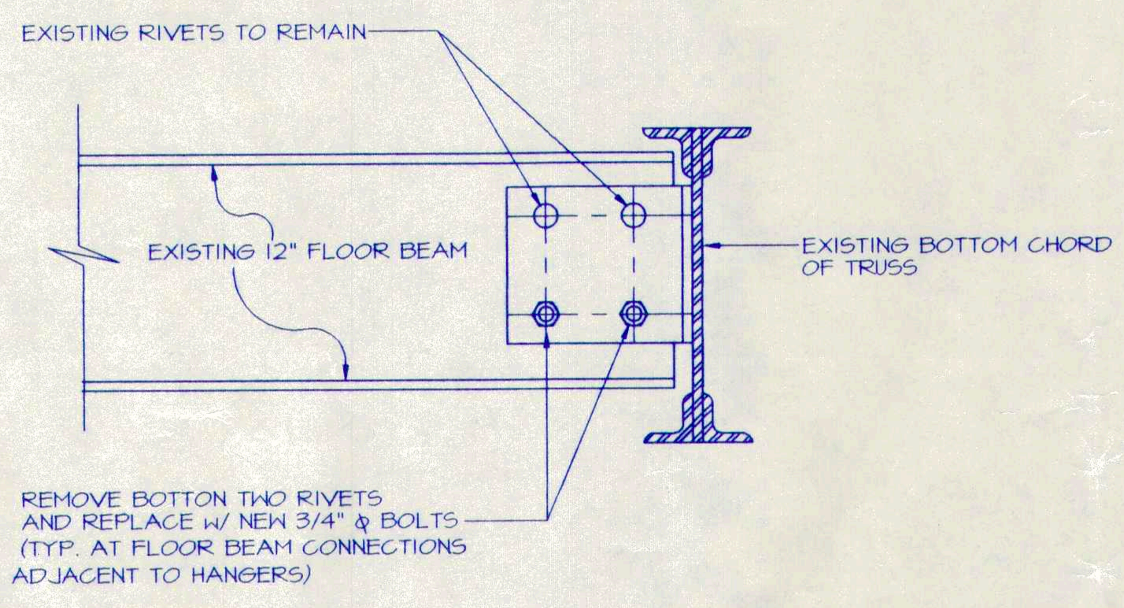
SCALE: 1/8" = 1'-0"



PLAN DETAIL (4/S-2)

SCALE: 1 1/2" = 1'-0"

NOTE: DETAIL 4A (OPPOSITE HAND)

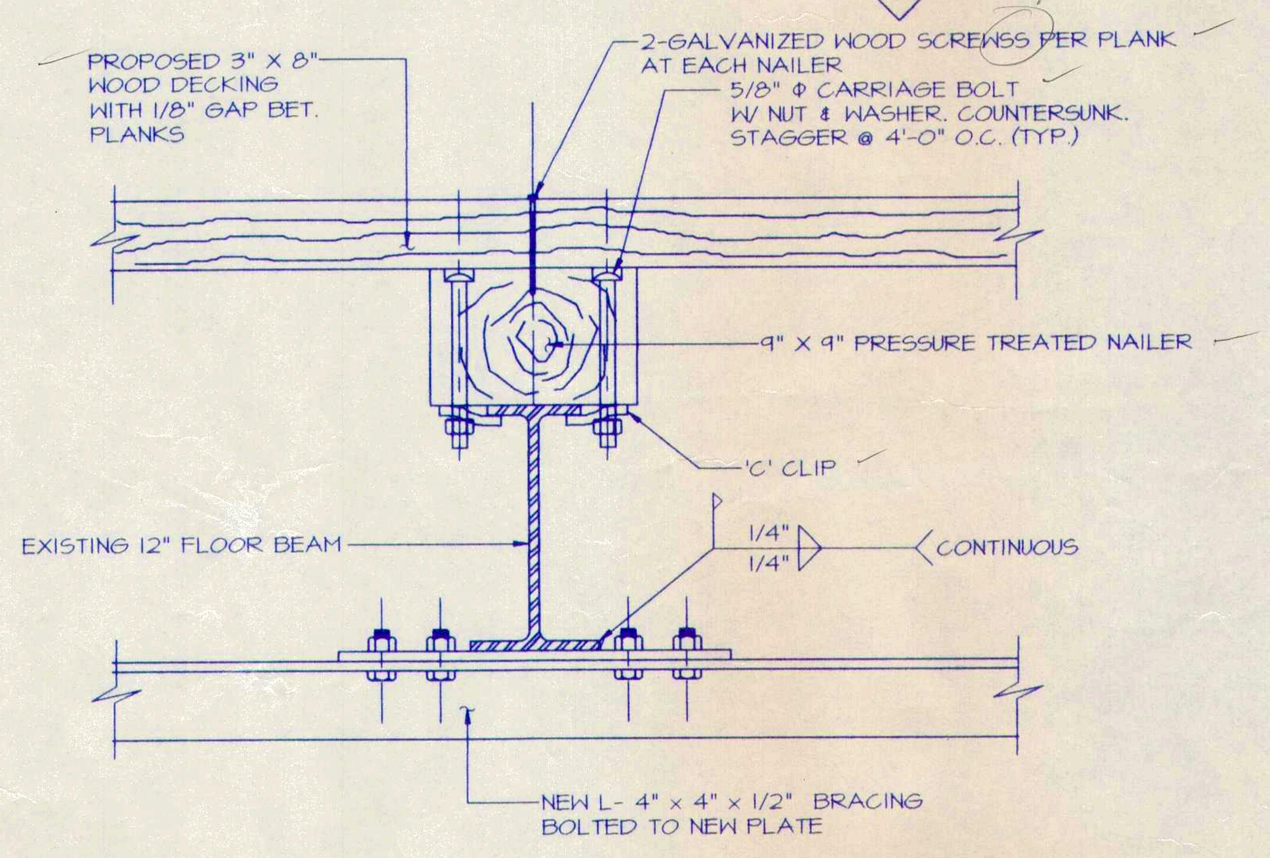


SECTION (5/S-2)

SCALE: 1 1/2" = 1'-0"

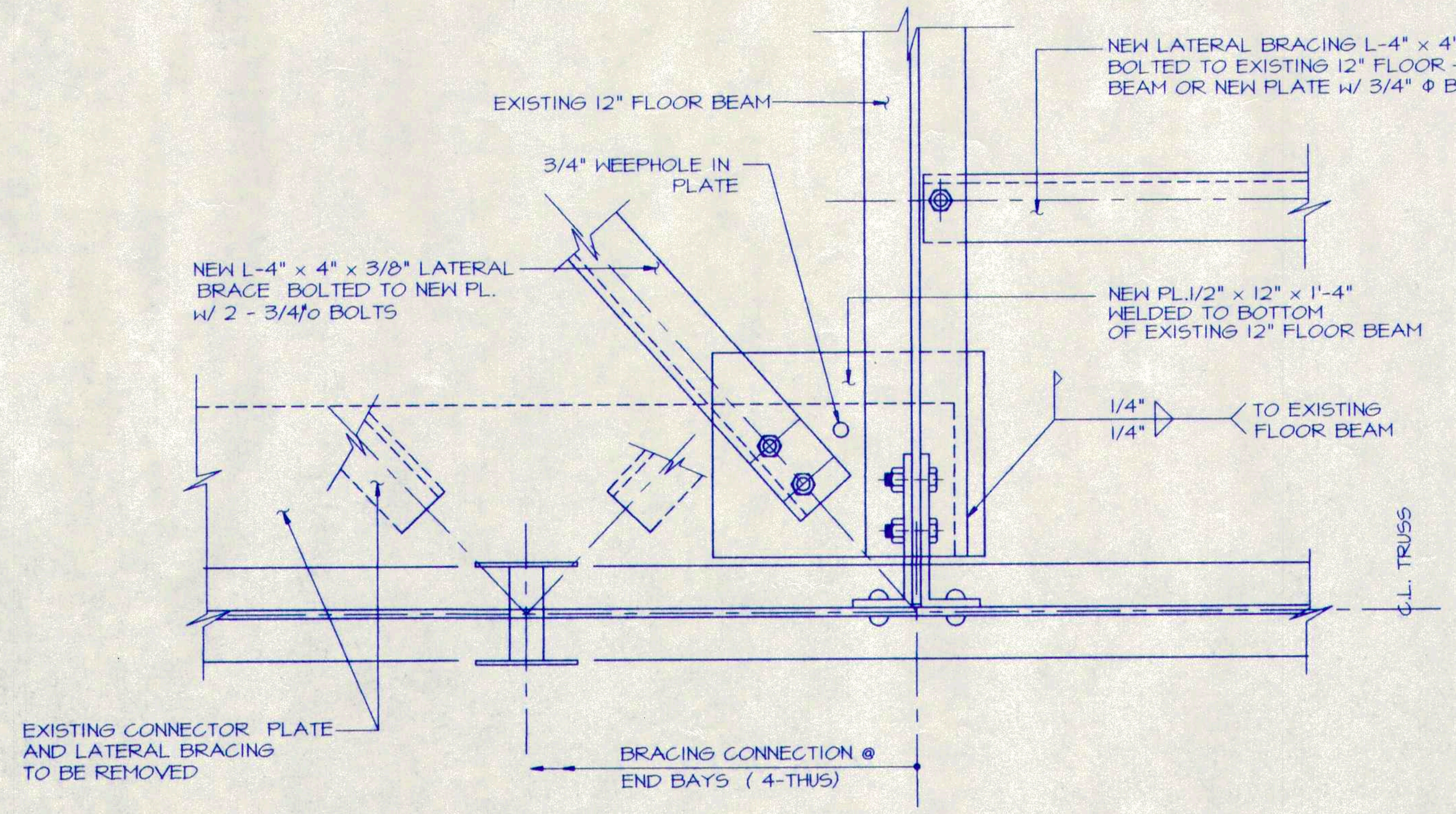
DECKING NOTES:

1. ALL SREWS SHALL BE RING SHANKED AND COUNTERSUNK. WOOD SCREWS SHALL BE A MINIMUM DIAMETER OF .220 INCHES AND SHALL BE A MINIMUM LENGTH OF 5 INCHES.
2. THE DECK ENDS SHALL BE IN A STRAIGHT LINE.
3. ALL TIMBER TO BE TREATED IN ACCORDANCE WITH THE SPECIFICATIONS.
4. FIELD CUTS AND HOLES SHALL BE TREATED IN ACCORDANCE WITH AWPA M4.



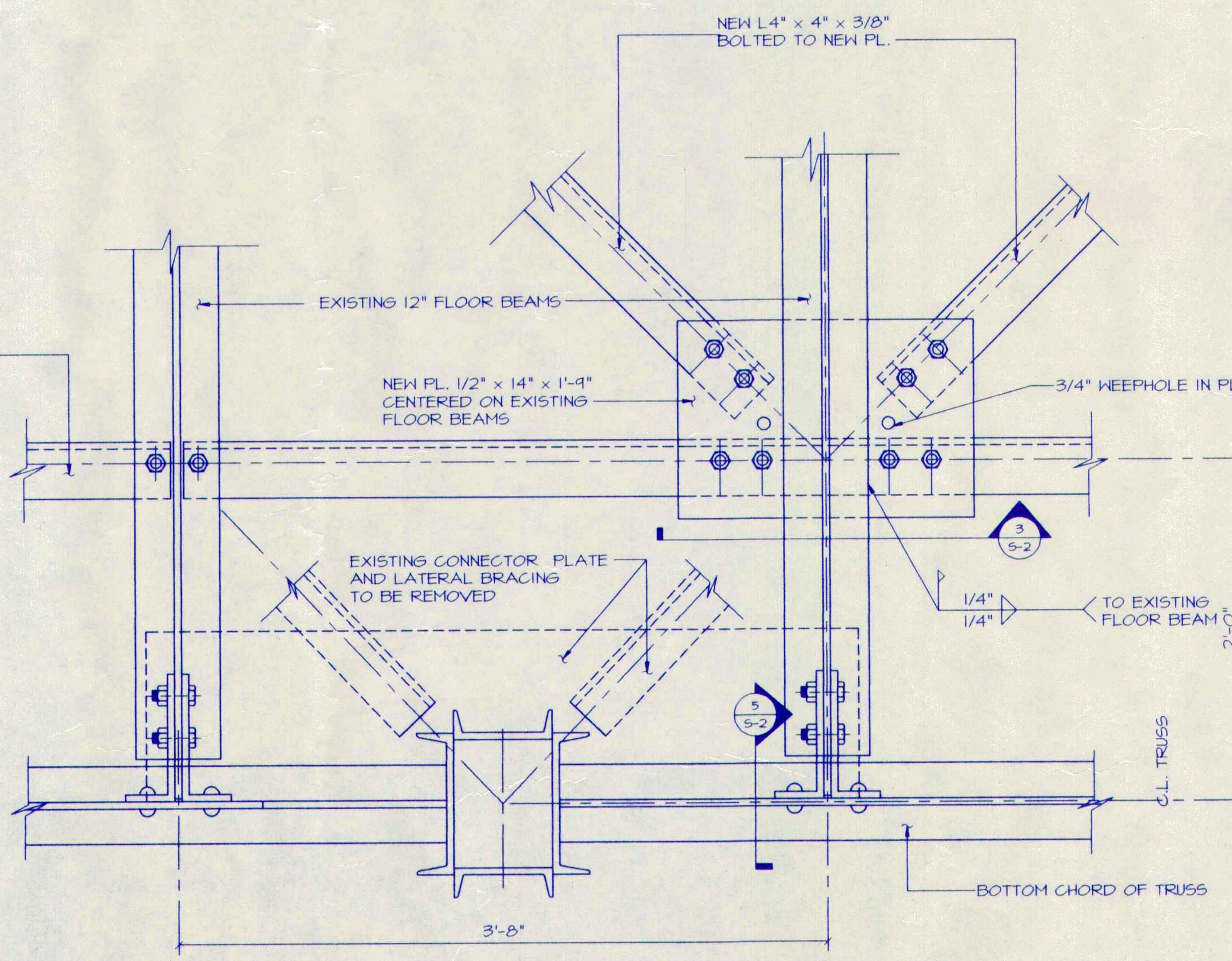
SECTION (3/S-2)

SCALE: 1 1/2" = 1'-0"



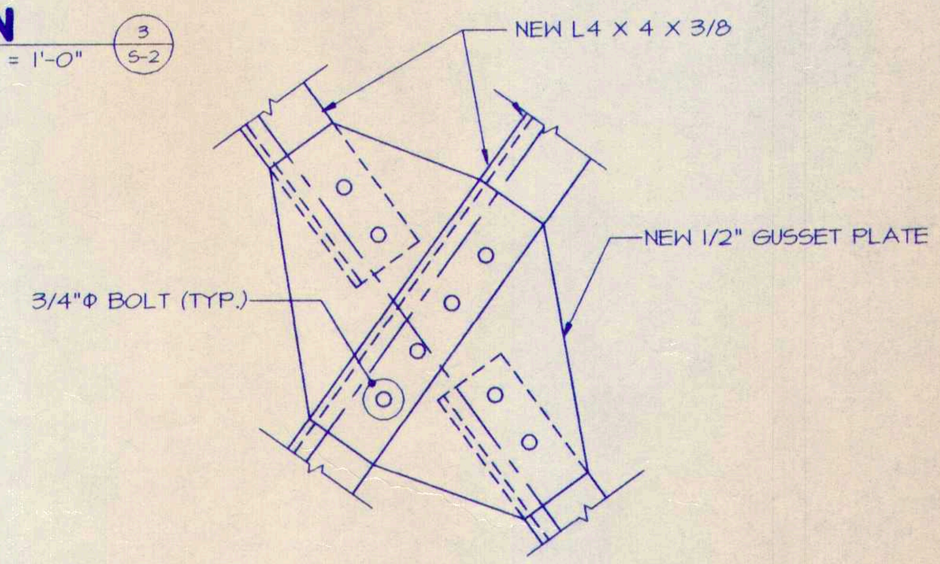
PLAN DETAIL (2/S-2)

SCALE: 1 1/2" = 1'-0"



PLAN DETAIL (1/S-2)

SCALE: 1 1/2" = 1'-0"



SECTION (6/S-2)

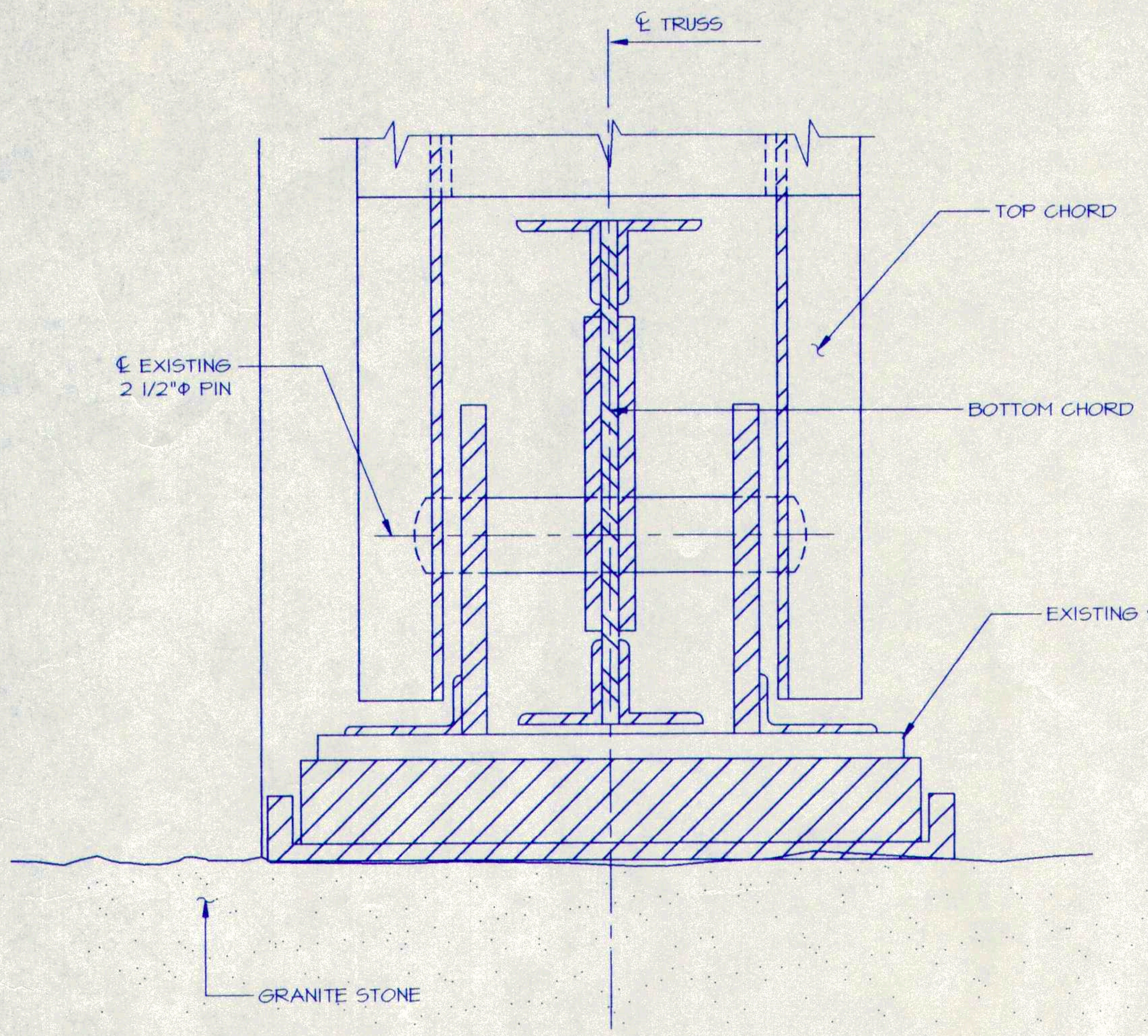
SCALE: 1 1/2" = 1'-0"

BRIDGE DECK AND TYPICAL DETAILS

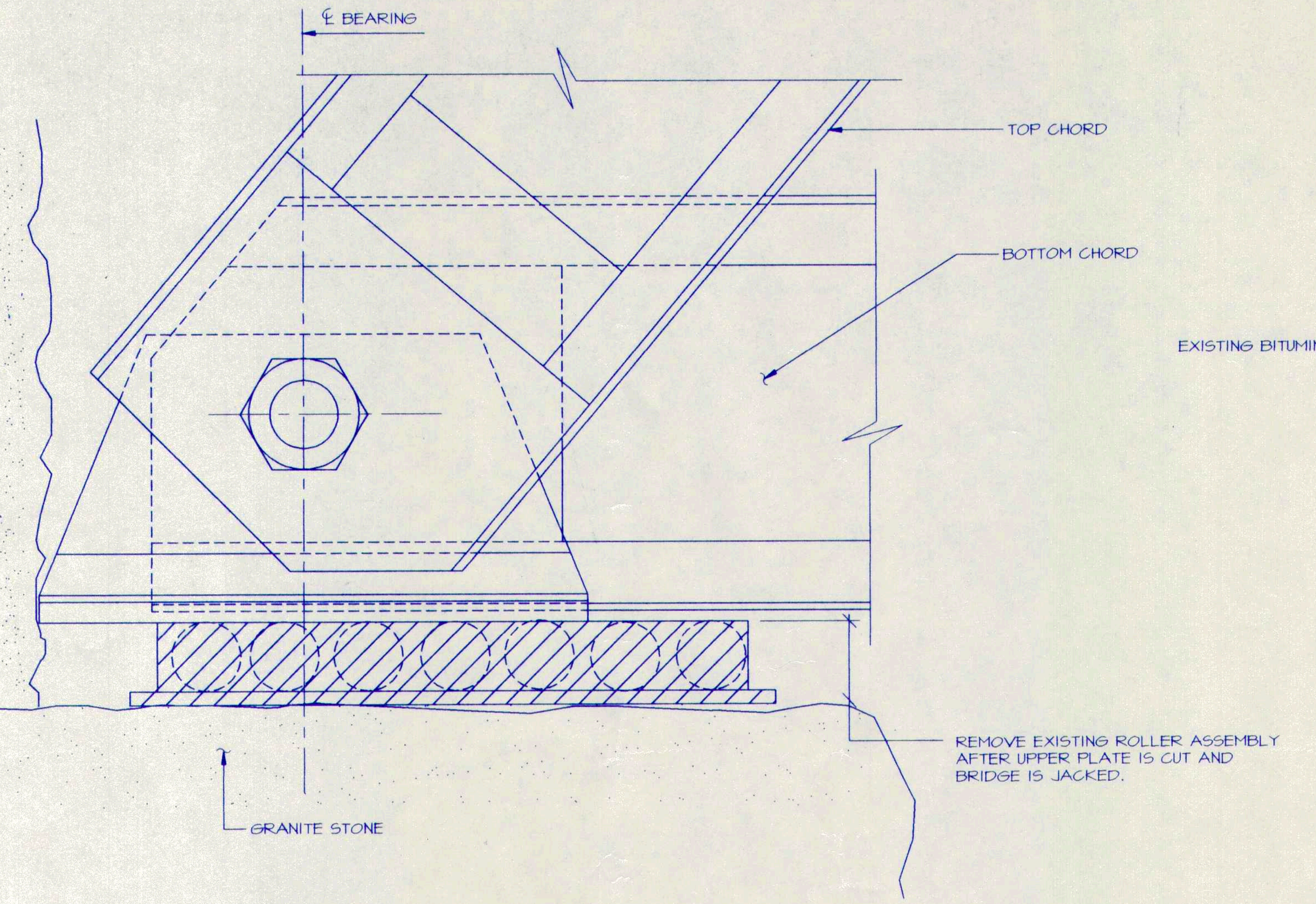
REHABILATATION OF OLD BRIDGE ROAD BRIDGE
SIMSBURY, CONNECTICUT

Macchi Engineers
44 Gillett Street
Hartford, CT 06105
Phone: (203) 549-6190

DRAWN BY: J.Z.	Drawing No. S-2
APPROVED BY: J.B.	
DATE: 8/01/94	
SCALE: AS NOTED	



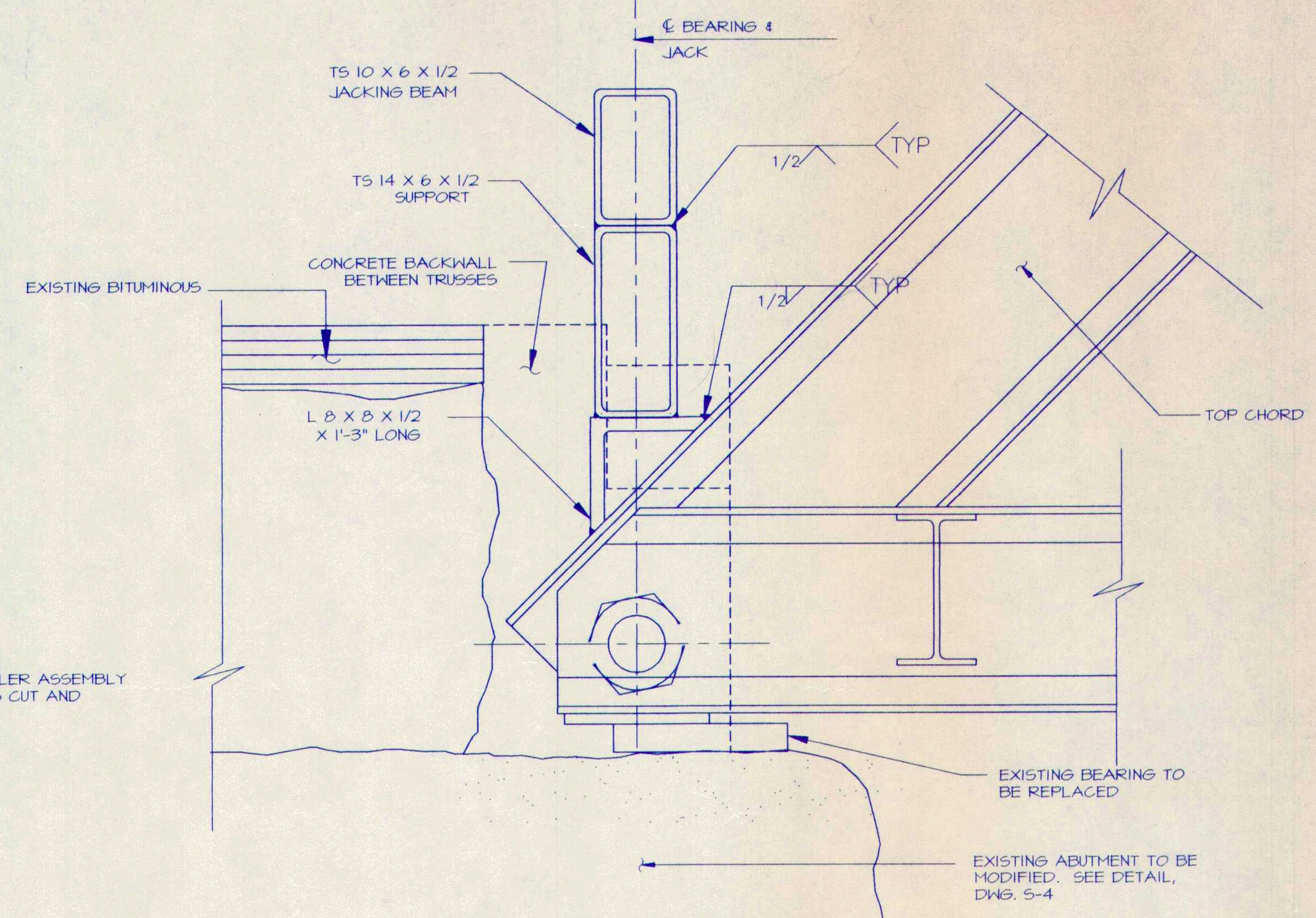
SECTION



ELEVATION

EXISTING EXPANSION BEARING

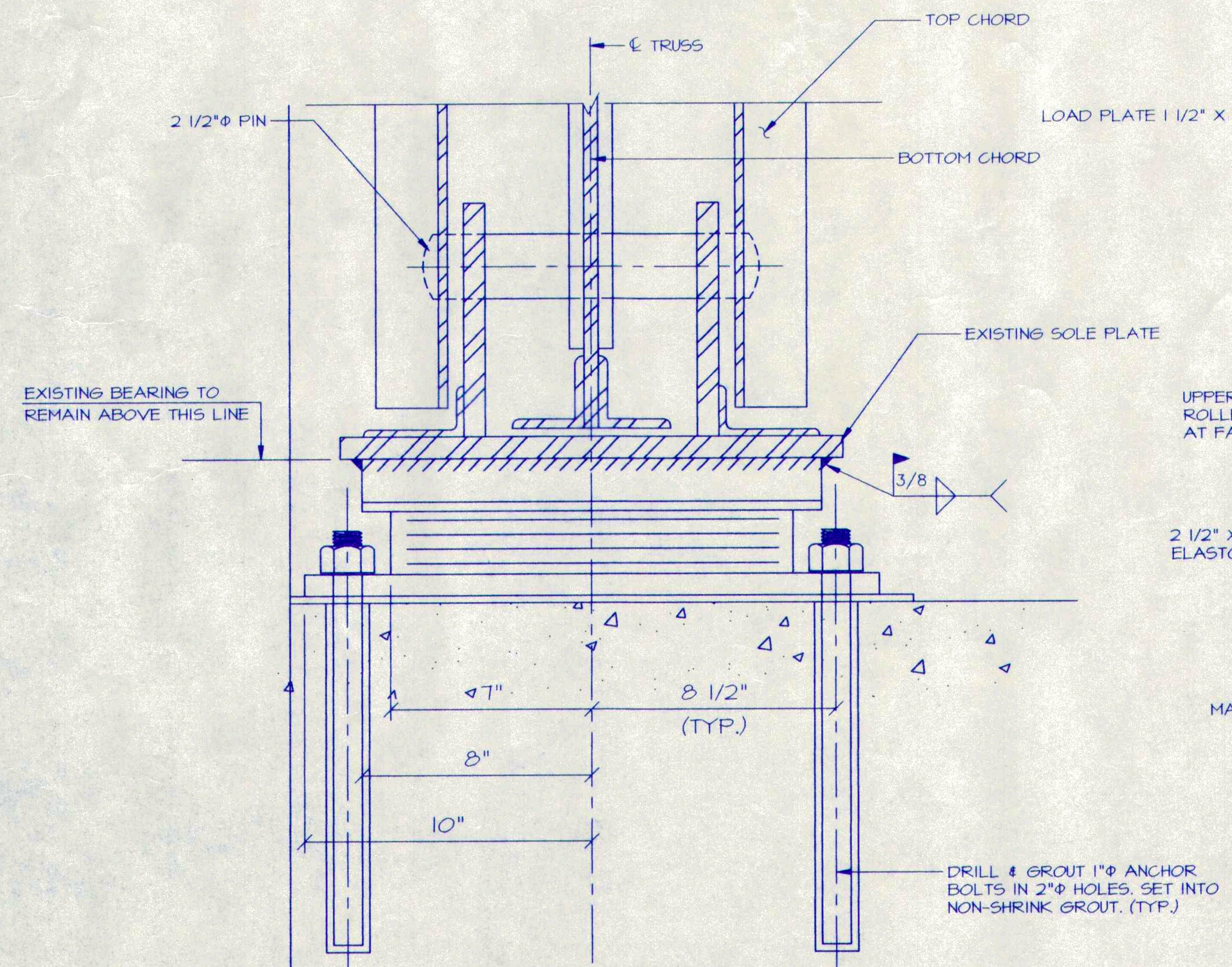
SCALE: 3" = 1'-0"



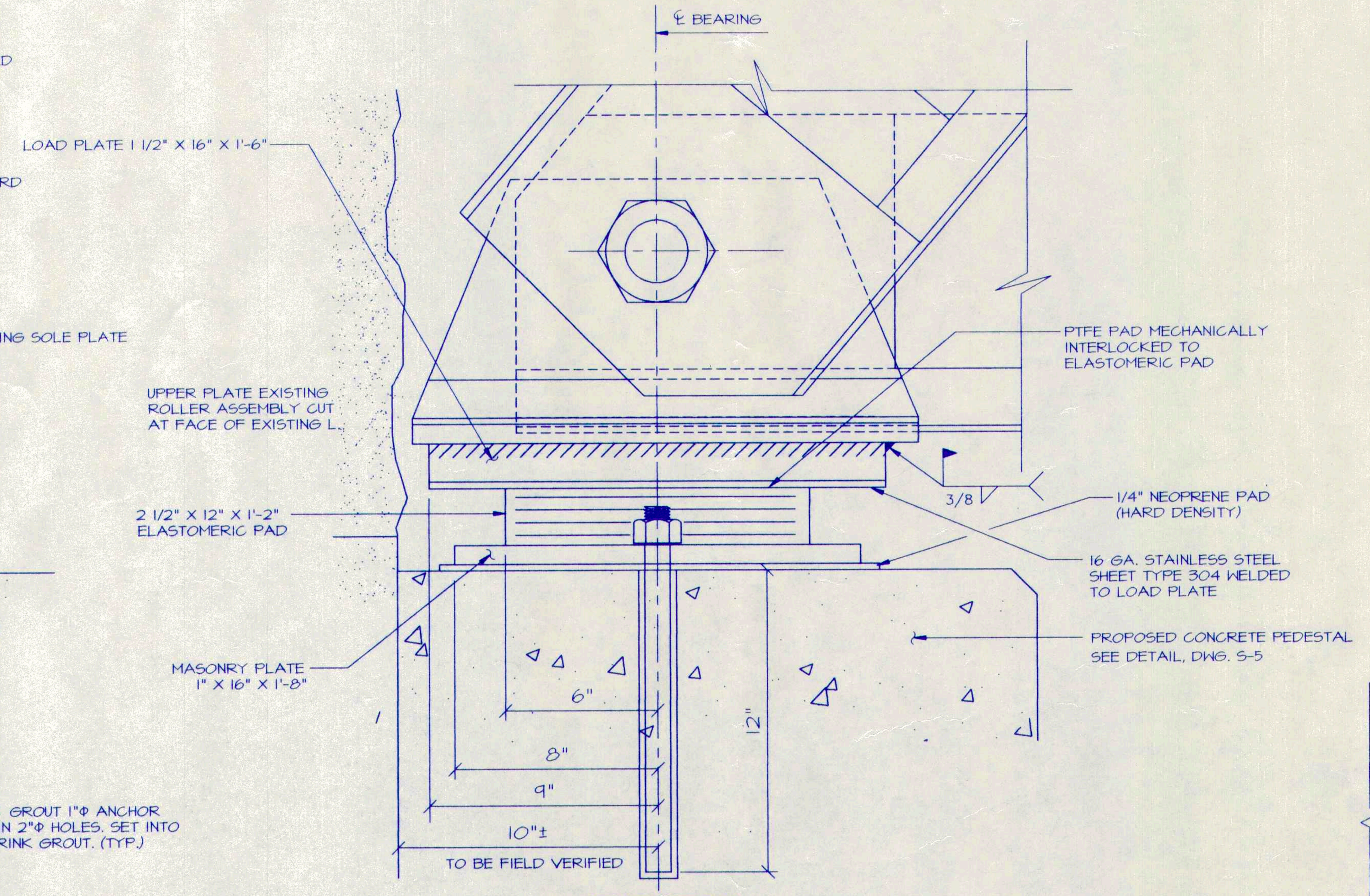
JACKING BEAM • WEST BEARING

SCALE: 1 1/2" = 1'-0"

NOTE:
JACKING FORCE EQUALS
APPROXIMATELY 20 TONS



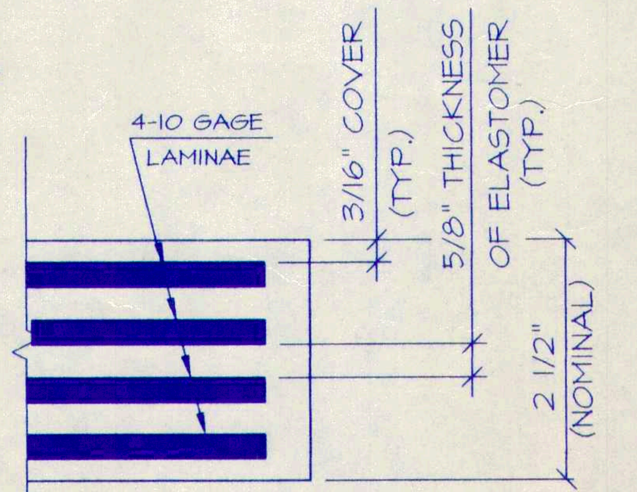
SECTION



ELEVATION

PROPOSED EXPANSION BEARING

SCALE: 3" = 1'-0"



ELASTOMERIC PAD DETAIL

N.T.S.

SEQUENCE OF CONSTRUCTION

1. REMOVE TIMBER DECKING AND JOISTS. REMOVE WINGWALL RAIL AT WEST END OF BRIDGE
2. WELD JACKING TUBE TO TRUSS, PLACE PLATE ON WEST ABUTMENT BACKWALL AND JACK WEST END OF BRIDGE, JACKING BOTH TRUSSES SIMULTANEOUSLY. SEE "JACKING EXISTING TRUSSES" IN SPECIAL PROVISIONS.
3. REMOVE EXISTING ROLLER ASSEMBLY
4. REMOVE TOP COURSE OF STONE MASONRY UNDER BEARING AND REPLACE WITH CONCRETE
5. LOWER TRUSS ONTO NEW BEARINGS AND REMOVE JACKING BEAM
6. PLACE TIMBER DECKING

BEARING NOTES:

1. THE STEEL FOR NEW BEARING ASSEMBLIES SHALL CONFORM TO ASTM A-709, GR. 50W. THE STEEL LAMINAE FOR ELASTOMERIC PADS SHALL CONFORM TO ASTM I-36, OR AN APPROVED EQUAL.
2. THE ELASTOMER BEARING PADS SHALL CONFORM TO ASTM D-4014, TYPE CR, GR.3. IT SHALL HAVE A SHORE 'A' DUROMETER HARDNESS OF 60, AND A SHEAR MODULUS WITHIN THE RANGE OF 170 psi TO 200 psi.
3. THE EXPANSION BEARINGS SHALL BE INSTALLED WHEN THE AMBIENT TEMPERATURE IS IN THE RANGE OF 40°F TO 80°F, AND HAS BEEN IN THIS RANGE FOR AT LEAST TWO HOURS.

EXPANSION BEARING DETAILS

REHABILATATION OF OLD BRIDGE ROAD BRIDGE
SIMSBURY, CONNECTICUT

Macchi Engineers
44 Gillett Street
Hartford, CT 06105
Phone: (203) 549-6190

DRAWN BY: J.Z.	Drawing No. S-3
APPROVED BY: J.B.	
DATE: 8/01/94	
SCALE: AS NOTED	

APPENDIX A - PIN ULTRASONIC TESTING REPORT

TEAM Industrial Services, Inc.

Team Industrial Services, INC.
196 Woodlawn Road
Berlin CT 06037

(860) 828-6333
(860) 828-7488 FAX

REPORT OF NON-DESTRUCTIVE EXAMINATION

Ultrasonic Inspection - Report # 13771299

Client: GM2
Address: 115 GLASTONBURY BLVD
GLASTONBURY, CT
06033

Jobsite: OLD DRAKE HILL FLOWER BRIDGE
Address: FLOWER · ST.
SIMSBURY, CT.

Contact Name: FAISAL AZIZ
Site Contact(s): BRIAN SWANSON

Components Inspected: (48) BRIDGE PIN UT INSPECTION
LOCATIONS AS FOLLOWS:
22 UPPER PINS: 11 ON SOUTH TRUSS L1 THRU L11
AND 11 ON NORTH TRUSS L1 THRU L11
26 LOWER PINS: 13 ON SOUTH TRUSS U0 THRU U12 AND 13 ON
NORTH TRUSS U0 THRU U12.

Results:
NO REJECTABLE INDICATIONS NOTED.
ALL PINS ACCEPTABLE TO UT INSPECTION.
WEAR GROOVES NOTED AND FOUND ACCEPTABLE.

Inspector's Name (Print): GREG BENWAY
Level: II
Inspector's Signature: [signature]
Date: ~~6/28/17~~ 6/29/17

Specification: ASME V
Purchase Order # PROJECT # 40212.00
Procedure: 22-H-800 REV.0
Acceptance: REPORT FINDINGS TO CLIENT / NO CRACKS ALLOWED
Ultrasonic: [X] A-Scan [ ] B-Scan [ ] C-Scan [X] Contact [ ] Immersion
Equipment: Mfg: KB Model: USN-60 S/N: 00R286
Transducer: Mfg.: UTX Model: CX-352 S/N 0708222 Angle: 0°
Size: .500" Ø Frequency: 2.25 MHz
Mfg.: KBA Model: GAMMA S/N 42746 Angle: 15°L
Size: .500" Ø Frequency: 2.25 MHz
Calibration Block: Type: ACTUAL PIN W/NOTCHES Material: STEEL S/N: N/A
Scanning: [X] Manual [ ] Automatic Couplant: ULTRASONIX Batch # 506 02
Pattern: PARALLEL PATH Scanning Speed < 4 IPS % Overlap 50

TEAM	Quality System Supplement Corporate	FORM 103.10 Rev: 8 Page 1 of 1

VISION ACUITY RECORD

Name: Gregory Benway **Employee #:** 655451

Vision Acuity Results

Near Vision Requirements
Required for All Personnel

	Left Eye		Right Eye	
Uncorrected	J -	@ "	J -	@ "
Corrected	J - 1	@ 16"	J - 1	@ 16"

Check one of the following:

☐ **Satisfactory Near Vision Without Corrective Lenses** (J-1 minimum required in at least one eye).

☒ **Satisfactory Near Vision With Corrective Lenses** (J-1 minimum required in at least one eye).

☐ **Unsatisfactory Near Vision**

Check if applies:

☒ Reading card has been verified IAW 8.1.2.1 of 33.G.103-S8 for personnel certifying to 33.G.103-S4 (CP-189/ASME XI)

Distance Vision Requirements
Branch is Required to Determine Applicability

	Left Eye		Right Eye	
Uncorrected	20/20	Snellen	20/20	Snellen
Corrected	20/	Snellen	20/	Snellen

Check one of the following:

☒ **Satisfactory Distance Vision Without Corrective Lenses** (20/30 Snellen minimum required in at least one eye).

☐ **Satisfactory Distance Vision With Corrective Lenses** (20/30 Snellen requirement in at least one eye).

☐ **Unsatisfactory Distance Vision**

☐ **N/A** (Branch determined non-applicable by Code or contractual agreements)

Color Vision Requirements
Required for All Personnel (Use Form 103.10a "Color Vision Examination Charts")

☒ **Satisfactory** – Can differentiate and distinguish between colors or shades of gray used in method(s)

☐ **Unsatisfactory** – Cannot differentiate and distinguish between colors or shades of gray used in method(s)

Deficiencies/Limitations: ☒ N/A

☐ Limitations reviewed and approved by Responsible Level 3 for NAS410 personnel.

Responsible Level 3 Signature ____________________

Brightness Discrimination Requirements
Branch is Required to Determine Applicability

Check all that apply:

☒ N/A ☐ Satisfactory ☐ Unsatisfactory ☐ Corrective Lenses Required

Remarks/Restrictions:

Administered By:		Reviewed & Approved By:	
Signature:	Jeff Watkins	NDT Level III Signature:	Jeff Watkins
Name:	Jeff Watkins	NDT Level III Name:	Jeff Watkins
Location:	1237/Hartford	Date:	11/18/2016
Date:	11/18/2016	Next Examination Date:	11/18/2017

TEAM® Industrial Services

Personnel Qualification and Certification

Employee Name: Gregory S. Benway **Employee ID#:** 655451

Vision Acuity Expiration Date: 11/18/2017

METHOD	LEVEL	DATE CERTIFIED	EXPIRATION DATE	GENERAL-I/II METHOD-III SCORE	SPECIFIC SCORE	PRACTICAL SCORE	COMPOSITE SCORE %	EXPERIENCE HOURS	TRAINING HOURS	LIMITED TO	COMMENTS
UT	II-L	1/4/2016	1/4/2019	95	90	96	93.7	72276	80	Contact: All Angles; Immersion: 0 Degree	
UT	II-L	1/4/2016	1/4/2019	95	90	96	93.7	72276	80	Contact: All Angles; Immersion: 0 Degree	MIL-STD-2132 Inspector, Contact Only

The above named individuals qualification history has been reviewed and found to be acceptable IAW TEAM's requirements for certification; 33.G.103-S1, SNT-TC-1A-2011 and earlier editions (1992, 2001 and 2006), as published by the American Society for Nondestructive Testing and/or any additional certification standards listed in the comments section above.

Certifying Authority: **Date:** 11/21/2016

Charles M. Lee
Corporate Level III
ASNT Cert # 58053

APPENDIX B - BRIDGE PAINT EVALUATION REPORT

Coating Condition Assessment of the Drake Hill Road Bridge

GM2 Associates, Inc

Prepared for:

Mr. Manish K. Gupta
Executive Vice President
GM2 Associates, Inc.
115 Glastonbury, Blvd.
Glastonbury, CT 06033

Prepared by:

KTA-TATOR, INC.
115 Technology Drive
Pittsburgh, PA 15275
(412) 788-1300
(412) 788-1306 – fax
www.kta.com

Robert Lanterman

Robert Lanterman
Coatings Consultant

July 21, 2017

RSL/MPR/AGB:lmb
JN370441
Drake Hill Road Bridge Report.doc

TABLE OF CONTENTS

INTRODUCTION

As authorized by an agreement (Proposal No. 17792) between GM2 Associates, Inc. (GM2) and KTA-Tator, Inc. (KTA), KTA has completed a coating condition assessment of the Drake Hill Road Bridge over the Farmington River located in Simsbury, Connecticut.

The purpose of this assessment was to determine the condition of the existing coatings on the structure in order to develop a maintenance painting strategy. This report contains the results of the field inspection and testing, laboratory analysis of field samples, a discussion of results and recommendations. Photographs depicting typical conditions found during the field visit are included as part of this report.

Photo 1 –General view of bridge.

SUMMARY

The existing coating system on the Drake Hill Road Bridge is in fair to good condition overall. The degree of coating failure typically ranged from 0. 3% to 3% of the surface area. Randomly scattered areas of spot corrosion were observed throughout the structure. Based on the percentage of visible corrosion, the coating is at a point where maintenance painting is economically advantageous. Spot repair of the corroded areas is recommended. Spot repairs will result in a patchwork appearance (of new vs. old paint color) and may not be acceptable based on aesthetics. If aesthetics are critical, then an overcoat can be applied to the entire structure. Application of a test patch is always strongly recommended prior to overcoating the entire area.

BACKGROUND

The Drake Hill Road Bridge is owned and maintained by the town of Simsbury. The bridge is over the Farmington River located in Simsbury, Connecticut. The bridge design is a Parker through truss. The bridge was erected in 1892 and has a length of 183 feet. The bridge no longer carries vehicular traffic and is used as a pedestrian/bicycle bridge. It is also referred to as the "Flower Bridge" as it is decorated with flower boxes and hanging baskets by a group of volunteers. Specifications from 1995 indicate the bridge was to have been blasted and painted with a zinc rich primer, epoxy intermediate coat, and urethane top coat. KTA was contacted to conduct a coating condition assessment and provide recommendations for future coatings maintenance work.

FIELD VISIT

The field visit to the Drake Hill Road Bridge was conducted by Mr. Jeff Towill of KTA on June 28, 2017. The bridge steel members were accessed from the road deck, a safety boat in the river, and using an extension ladder. The tests and inspections performed, including the observations made and measurement findings from the investigation, are discussed herein.

The following methods, standards, and practices were used to evaluate the existing coating and underlying substrate conditions.

- **Visual** – A visual assessment of the coated surfaces was conducted to determine the type, extent, and location of coating breakdown and corrosion on the structure. Visual Standard SSPC VIS 2, "Standard Method for Evaluating Rusting on Painted Steel Surfaces," was used.

- **Coating Thickness** – The dry film thickness was determined using a Positector 6000. The Positector 6000 is a portable, battery operated, digital coating thickness gage that non-destructively measures non-magnetic coating thickness over ferrous substrates using a magnetic principle. Gage calibration was verified prior to and after use with the National Institute of Standards and Technology (NIST) thickness standards.

- **Adhesion** – Adhesion testing was conducted in accordance with ASTM D 3359, "Measuring Adhesion by Tape Test," Method A. This method involves cutting an "X" through the coating down to the substrate using a razor knife, followed by the application of pressure sensitive tape. The tape is then rapidly removed from the X-cut and the adhesion is then rated according to the amount of coating removed using an ASTM rating scale. Typical ratings of 4A to 5A are considered by KTA to represent good adhesion, 2A to 3A represent fair adhesion, while 0A to 1A represent poor adhesion. Coating adhesion was also assessed in general accordance with ASTM D 6677, "Standard Test Method for Evaluating Adhesion by Knife." These methods involve scribing the coating with a knife and evaluating the adhesion in accordance with an ASTM rating scale. The location of the forced separation within the system is also reported.

- **Paint Samples** – Samples were removed for further laboratory examination to determine the generic coating type, to measure the number and thickness of coats, and to check the presence and amount of heavy metals (lead, cadmium, and chromium) in the lab.

- **Photographs** – Photographs of typical coating conditions were taken and are included as part of the report.

Visual Inspection

General

For purposes of the visual inspection, the bridge was broken down into simple component members (i.e. truss members, floor stringers, guard rails, cables and towers). Overall, the visual coating condition was rated fair to good. The overall rate of coating deterioration (spot rust, pinpoint rust, and cracks in the existing coating) was minimal when compared to all the steel surfaces. Coating blisters or application defects such as excessive runs or sags were minimal. There were isolated spot areas of corrosion. Areas of graffiti were found on the bridge at the abutments. A summary of the typical coating condition on the various structural members of the bridge is presented below.

Truss Members

Spot corrosion on the truss members typically ranged from approximately 0.3% to 1% of the surface area. There were several isolated areas with spot corrosion on the North truss ranging from 1% to 3%. Areas of spot corrosion were scatters across the length and most often occurring at the connections. Conditions were typical for upper and lower truss chords, verticals, diagonals, and bracing members. See Photographs 2 through 11 below.

Photo 2 –Typical view of truss.

Photo 3 –Spot corrosion on truss connection.

Photo 4 –Typical view of interior truss top chord.

Photo 5 –Bird nest in truss top chord.

Photo 6 –Spot corrosion at truss connection.

Photo 7 –Spot corrosion on truss bracing.

Photo 8 – Spot corrosion on truss lower chord.

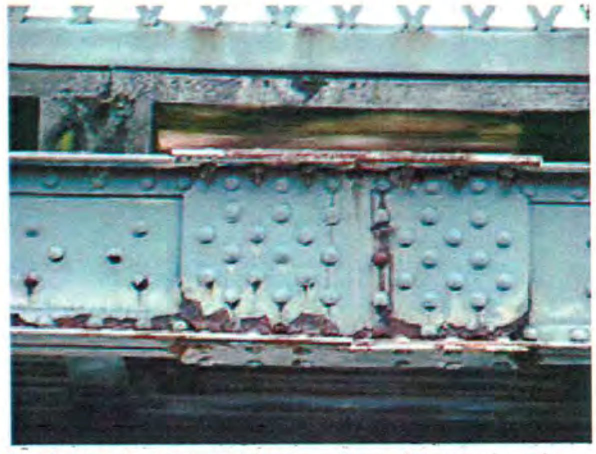


Photo 9 –Spot corrosion on truss lower chord.

A white discoloration was observed on the lower truss and adjacent steel. The discolored areas had the appearance of salt deposits. The discoloration may also be caused by runoff from the flower boxes.

Photo 10 –White stain on lower chord.

Photo 11 – White stain on lower chord.

Floor Beams

Coating deterioration on the floor beams ranged from approximately 0. 3% to 1% of the surface area. Higher levels of corrosion were typically observed at the connections with the lower chords.

Photo 12 –Typical condition of floor beams.

Photo 13 – Typical condition of floor beams.

Photo 14 – Typical condition of floor beams.

Photo 15 – Typical condition of floor beams.

Dry Film Thickness

Total coating system dry film thickness measurements were obtained on the existing coating system. The following table, Table 1 – Dry Film Thickness Measurements, summarizes the range of the thicknesses obtained with a Positector 6000, magnetic-type dry film thickness gage:

Table 1 – Dry Film Thickness Measurements

Member	Minimum (mils)	Maximum (mils)	Average (mils)
North Truss	9.6	17.4	14.9
North Verticals	6.7	15.7	11.5
North Member Diagonals	4.3	26.4	11.2
North Rail and Lattice	7.0	20.1	12.7
South Truss	6.0	17.7	9.6
South Verticals	4.4	15.1	11.2
South Rail and Lattice	6.7	21.7	13.7
Floor Beams	5.4	13.2	8.9
Floor Beam Bracing	4.7	13.0	9.8

Adhesion

Adhesion testing was performed in accordance with ASTM D3359 Method A (X-cut). Adhesion was typically rated 4A (good) in all areas tested. Coating adhesion was also assessed in general accordance with ASTM D 6677. Adhesion was typically rated 8 to 10 (good).

Substrate Examination

The substrate was examined at the sample and adhesion test areas. The substrate appears to have been abrasive blast cleaned.

Chloride Testing

Two chloride tests were performed using a Chlor-Test kit from Chlor Rid International. Test #1 was performed over intact coating on the bottom surface of the lower south truss in an area that had a white discoloration that could be wiped off with a wet cloth. The result was 10 $\mu g/cm^2$. Test #2 was performed nearby on the top of the bottom flange in an area where the coating was removed and the steel was rusting. Loose rust and debris was removed with a scraper. The result was non-detectable.

LABORATORY INVESTIGATION

The laboratory investigation consisted of infrared spectroscopy, visual and microscopic examination, and lead, cadmium, and chromium testing. A description of the test methods employed and results of the investigation are provided below.

Visual and Microscopic Examination

Visual and microscopic examination of the samples was conducted using a Keyence VHX-5000 digital microscope with magnification to 200X. The samples had between two and three coating layers. Table 2 - Coating Thickness Data, below lists the magnification at which each cross-section was examined, the number of layers observed, the color of the individual layers, and the minimum and maximum thickness of the individual layers, measured in mils.

Table 2 - Coating Thickness Data

Sample ID	Sample Description	Magnification	Layer/Coat	Thickness (mils)
KTA-1	West Portal	150X	*Two Coating Layers* Top – Green Bottom – dark gray	 2.6 – 3.2 10.3 – 11.0
KTA-2	Floor beam, first panel point	150X	*Three Coating Layers* Top – green Dark gray Bottom – Metallic dark gray	 3.0 – 3.2 2.9 – 4.3 7.9 – 8.9
KTA-3	South Lower Truss Chord	150X	*Three Coating Layers* Top – Green Dark gray Bottom – metallic dark gray	 5.7 – 7.6 4.3 – 5.5 3.0 – 4.7

Infrared Spectroscopy

Infrared spectroscopic analysis was performed using a Mattson Galaxy Model 3020 Fourier transform infrared spectrometer. This technique involved combining sample scrapings with potassium bromide powder and forming pellets under high pressure. The pellets were then placed in the optical path of the spectrometer and spectra were obtained over the range of 4000 to 400 cm^{-1}. Three spectra were obtained and are appended.

The green topcoat scrapings when combined with potassium bromide of Samples KTA-1 (West Portal), KTA-2 (Floor Beam) and KTA-3 (South Lower Truss) were consistent with a urethane resin. The urethane resin was evidenced by the doublet near 1730/1690 cm^{-1}, and spectral bands near 1520, 1460, 1240, and 1160 cm^{-1}. Talc was evidenced by the bands near 3600, 1020, 670 and 530 cm^{-1}.

Lead, Cadmium and Chromium Testing

Samples KTA-1 thru KTA-3 were tested for lead, cadmium, and chromium in accordance to EPA Method 6010C and EPA Method 3050B. The testing was performed by Schneider Laboratory, Inc., in Richmond, VA. The lead, cadmium and chromium results (ppm by weight) are shown in the table below.

Schneider Laboratories Testing Designation

Sample ID	Sample Description	Total Lead, ppm	Total Cadmium, ppm	Total Chromium, ppm
KTA-1	West Portal	177	ND*	160
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*ND – Test results were below detectible limits of test

DISCUSSION

General Discussion on Maintenance Painting

The purpose of this coating assessment was to assess the condition of the existing coatings on the structures and make recommendations for maintenance painting. Many factors affect the service life of a coating system. These include the type of coating originally applied, the type and quality of surface preparation, service environment, number of coats and film thickness, and the history of maintenance painting activities.

If a coating has provided satisfactory corrosion prevention and remains in relatively good condition, it is cost effective to extend the life of the system through overcoating, retaining as much of that original coating as possible. When the coatings are in poor condition, a "full removal" strategy is used, which removes all existing coatings. This strategy effectively places the bridge at the beginning of a new maintenance painting cycle. Little work will be required for at least 10 years, and then, it should involve only minor touch-up. This strategy, while safe and effective, is also expensive. A discussion of the various types of maintenance painting activities follows.

Maintenance painting options for bridge structures fall into four main categories: (1) deferral of maintenance, (2) spot repairs, (3) spot repairs with full overcoats, and (4) complete coating removal and replacement.

Each of these options is progressively more complex, and requires progressively more work. Correspondingly, each option also offers greater long-term protection to the structure, but at additional costs. When paints containing hazardous metals are present, the issues associated with removing these paints impact the decision-making process.

Deferral of Maintenance

Maintenance painting can be deferred if the existing coating system is in good condition, if the service life of the structure is limited, or there is some other benefit for postponing the work. If extensive corrosion is found and maintenance painting is deferred for a period of time, the level of surface preparation required to properly prepare the surface increases correspondingly, and if left unattended for too long, total removal will ultimately be required. In some cases, when the structure is corroding extensively, but is still structurally sound, painting is deferred because the highest level of surface preparation (abrasive blast cleaning) is already needed, whether performed today or several years from now. The strategy in this case is to allocate the money to repair coatings on other structures that are not so badly deteriorated in order to stop the corrosion from propagating to the point that total removal is the only option for those structures as well.

Spot Repairs

Spot repairs, as the name suggests, involves surface preparation and coating application only to the individual spots of corrosion or coating breakdown. The amount of coating being removed is minimized, reducing the impact of hazardous materials handling, containment, and worker protection when toxic metals are present. Spot repairs also serve to repair the existing coating film only where it is needed, repairing the corroded areas, and stopping the propagation of the breakdown. Coatings in essentially any condition may be spot repaired, but it is only practical when the level of breakdown is minor and somewhat isolated and covers a small percentage of the surface (e.g., 1 or 2%). A disadvantage of this approach involves aesthetics. The repair spots are clearly visible.

A variation of this type of localized repair includes zone or area repairs. This involves surface preparation and coating application over a larger area that exhibits more concentrated levels of breakdown, but the work is limited to those areas. For example, the bearing areas of girders are often zone painted on either side of an expansion joint, without any significant painting on the rest of the structure.

Spot Repairs with Full Overcoat(s)

The application of a full overcoat serves two primary purposes: the additional coat provides additional barrier protection and helps to seal minor defects that are not apparent when conducting spot repairs. It also offers an improved appearance when compared to spot repairs. The addition of the overcoat also adds complexity and cost to the overall project. The complexity increases because a contractor must now gain access to all areas of the structure to apply the full coat. The existing surface must also be thoroughly cleaned (i.e., power washed) to remove chalk and surface debris. The adhesion of the existing coating must also be good and sound; otherwise the stresses imparted by the overcoat can cause disbonding of the existing system, especially under freeze/thaw conditions. In some cases, two full overcoats are applied.

This strategy is typically used when the amount of visible corrosion and coating deterioration covers less than 15% of the surface.

Total Coating Removal and Replacement

Total removal and replacement is the final option for maintenance painting and is the costliest option, especially when removing existing coatings that contain toxic metals. However, it offers the greatest opportunity for long-term protection. All of the mill scale, rust, and paint are completely removed and a new system with a new design life is applied. This method also provides the most pleasing appearance.

When total removal and replacement is performed, a new maintenance cycle begins. As the coatings age and weather, isolated spot repairs will be required. Several spot repairs may be made to the individual structure until a full overcoat is necessary. More spot repairs may then be made and additional overcoats applied until extensive corrosion develops, significant coating breakdown occurs, or the mechanical properties of the coatings (e.g., the adhesion) degrade to the point where additional work (spot touch-up or overcoating) is no longer practical. At this time, complete removal may again be required, but only after the maximum effective life of the original coating system has been extended through the planned maintenance activities.

RECOMMENDATIONS

The existing coating system on the Drake Hill Road Bridge is in fair to good condition overall. The degree of coating failure typically ranged from 0. 3% to 3% of the surface area. Randomly scattered areas of spot corrosion were observed throughout the structure. Based on the percentage of visible corrosion, the coating is at a point where maintenance painting is economically advantageous. When maintenance work is performed, there are two recommended options.

Option 1 – Spot Repairs: Under this option, surface preparation on areas of spot corrosion/coating failure would be performed in accordance with SSPC SP-3, "Power Tool Cleaning." Vacuum shrouded power tools should be used to minimize the containment requirements, but nuisance tarps will be required to capture the paint chips that are dislodged by the tools, but not captured by the vacuum.

The spot repair coating system should involve three coats, consisting of an epoxy mastic prime coat, an epoxy intermediate coat, and a polyurethane finish coat, with stripe coats of the primer and intermediate coats applied to edges, crevices, rivets, and other irregular surfaces. One benefit to this option would be a reduced total project cost for maintenance painting. Spot repairs will leave a patchwork like appearance and may not be acceptable based on aesthetics.

Option 2 – Spot Repairs with Full Overcoat: Under this option, surface preparation on areas of spot corrosion/coating failure would be performed the same as in option 1. Based on the current assessment data and visual observations, in order to apply an

overcoat, all surfaces must also be cleaned by pressure washing to remove chalk, chlorides, dirt, and other debris.

The overcoat system should involve two coats, consisting of a penetrating sealer tie coat and a polyurethane finish coat. Stripe coats of the intermediate should be applied to edges, crevices, rivets, and other irregular surfaces. Application of a test patch is always strongly recommended prior to overcoating the entire area.

Chloride Remediation

It is imperative that residual chloride levels (salt contamination) be maintained at acceptable concentrations prior to coating. The level of salt contamination when applying organic coatings should be kept below 7 $\mu g/cm^2$. The specifications should require testing after surface preparation has been performed and prior to painting. In many instances, chloride contamination can be reduced to acceptable levels by pressure water cleaning and/or abrasive blast cleaning with a combination of finely graded and coarser abrasive media. Chloride removal agents can also be added to the pressure washing water. Other options include abrasive blast cleaning the steel, and allowing it to rust over night followed by re-blast cleaning.

Dealing with Lead

Laboratory testing reported detectable concentrations of lead present in the existing coatings on the bridge. The OSHA Lead in Construction Standard (29 CFR 1926.62) requires that controls be implemented if any detectable concentrations of lead are present. The OSHA Compliance Directive issued for the OSHA Lead in Construction Standard, Instruction CPL 2-2.58, states that if an employer has appropriately tested for lead (e.g., tested all layers of paints or coatings that may be disturbed) utilizing a valid detection method, and found no detectable levels of lead, then the standard does not apply. Paints with detectable concentrations of lead require the contractor performing the work to implement interim controls and assess actual employee exposures during the work in accordance 29 CFR 1926.62. Based on the lead results provided by the laboratory testing, 29 CFR 1926.62 is invoked during any activities that disturb the paint (e.g., abrasive blast cleaning, scraping, burning, and grinding).

It should be noted that other hazardous metals are also present in the existing coating. Any disturbance of paint containing heavy metals in addition to lead must be performed in accordance with the requirements of the applicable OSHA standards.

In addition, containment will be required for the protection of the environment and the public, and the hazardous waste must be properly managed.

Opinion of Probable Coating Replacement Costs

A cost analysis was prepared for the recommended maintenance options. This analysis involved making various assumptions, based upon KTA and industry experience, of how a contractor might staff and proceed with the aforementioned recommendations. Crew sizes,

production rates, material and equipment requirements are evaluated and man-days and project-days are calculated. From the estimated project duration, costs associated with labor, materials, and equipment are factored in and the costs are developed. Overhead and profit are added as a multiplier to the base cost. For the purposes of this opinion of probable coating cost, labor was considered to be prevailing wage and equipment was calculated with rental rates.

Production days were calculated from the square footage of paintable steel surfaces and an allocated production rate. The surface areas for the bridge were calculated from the provided drawings. The Drake Hill Road Bridge surface area is estimated to be 22,000 total square feet. Finally, a variance multiplier is used on the final cost to develop a range of anticipated bid prices. This multiplier allows for the variations in contractor bidding techniques, new technology, and scheduling of the work within the painting season. The opinion of probable cost to perform spot repairs with a full overcoat ranges from $244,300 to $295,700 and the cost to perform spot repairs only ranges from $50,100 to $60,700. Spot repairs with a full overcoat would take approximately one month for total production time while spot repairs only would take approximately one week.

Opinions of probable construction costs are prepared on the basis of KTA's experience and qualifications and represent KTA's judgment as field professionals generally familiar with the industry. However, since KTA has no control over the cost of labor, materials, equipment, or services furnished by others, over contractor's methods of determining prices, or over competitive bidding of market conditions, KTA cannot and does not guarantee that proposals, bids, or actual construction costs will not vary from KTA's opinions of probable cost.

Appendix G - KTA Paint Analysis Report

APPENDIX B - BRIDGE PAINT EVALUATION REPORT

Coating Condition Assessment of the Drake Hill Road Bridge

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Robert Lanterman
Coatings Consultant

July 21, 2017

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Drake Hill Road Bridge Report.doc

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INTRODUCTION

As authorized by an agreement (Proposal No. 17792) between GM2 Associates, Inc. (GM2) and KTA-Tator, Inc. (KTA), KTA has completed a coating condition assessment of the Drake Hill Road Bridge over the Farmington River located in Simsbury, Connecticut.

The purpose of this assessment was to determine the condition of the existing coatings on the structure in order to develop a maintenance painting strategy. This report contains the results of the field inspection and testing, laboratory analysis of field samples, a discussion of results and recommendations. Photographs depicting typical conditions found during the field visit are included as part of this report.

Photo 1 –General view of bridge.

SUMMARY

The existing coating system on the Drake Hill Road Bridge is in fair to good condition overall. The degree of coating failure typically ranged from 0. 3% to 3% of the surface area. Randomly scattered areas of spot corrosion were observed throughout the structure. Based on the percentage of visible corrosion, the coating is at a point where maintenance painting is economically advantageous. Spot repair of the corroded areas is recommended. Spot repairs will result in a patchwork appearance (of new vs. old paint color) and may not be acceptable based on aesthetics. If aesthetics are critical, then an overcoat can be applied to the entire structure. Application of a test patch is always strongly recommended prior to overcoating the entire area.

BACKGROUND

The Drake Hill Road Bridge is owned and maintained by the town of Simsbury. The bridge is over the Farmington River located in Simsbury, Connecticut. The bridge design is a Parker through truss. The bridge was erected in 1892 and has a length of 183 feet. The bridge no longer carries vehicular traffic and is used as a pedestrian/bicycle bridge. It is also referred to as the "Flower Bridge" as it is decorated with flower boxes and hanging baskets by a group of volunteers. Specifications from 1995 indicate the bridge was to have been blasted and painted with a zinc rich primer, epoxy intermediate coat, and urethane top coat. KTA was contacted to conduct a coating condition assessment and provide recommendations for future coatings maintenance work.

FIELD VISIT

The field visit to the Drake Hill Road Bridge was conducted by Mr. Jeff Towill of KTA on June 28, 2017. The bridge steel members were accessed from the road deck, a safety boat in the river, and using an extension ladder. The tests and inspections performed, including the observations made and measurement findings from the investigation, are discussed herein.

The following methods, standards, and practices were used to evaluate the existing coating and underlying substrate conditions.

- **Visual** – A visual assessment of the coated surfaces was conducted to determine the type, extent, and location of coating breakdown and corrosion on the structure. Visual Standard SSPC VIS 2, "Standard Method for Evaluating Rusting on Painted Steel Surfaces," was used.

- **Coating Thickness** – The dry film thickness was determined using a Positector 6000. The Positector 6000 is a portable, battery operated, digital coating thickness gage that non-destructively measures non-magnetic coating thickness over ferrous substrates using a magnetic principle. Gage calibration was verified prior to and after use with the National Institute of Standards and Technology (NIST) thickness standards.

- **Adhesion** – Adhesion testing was conducted in accordance with ASTM D 3359, "Measuring Adhesion by Tape Test," Method A. This method involves cutting an "X" through the coating down to the substrate using a razor knife, followed by the application of pressure sensitive tape. The tape is then rapidly removed from the X-cut and the adhesion is then rated according to the amount of coating removed using an ASTM rating scale. Typical ratings of 4A to 5A are considered by KTA to represent good adhesion, 2A to 3A represent fair adhesion, while 0A to 1A represent poor adhesion. Coating adhesion was also assessed in general accordance with ASTM D 6677, "Standard Test Method for Evaluating Adhesion by Knife." These methods involve scribing the coating with a knife and evaluating the adhesion in accordance with an ASTM rating scale. The location of the forced separation within the system is also reported.

- **Paint Samples** – Samples were removed for further laboratory examination to determine the generic coating type, to measure the number and thickness of coats, and to check the presence and amount of heavy metals (lead, cadmium, and chromium) in the lab.

- **Photographs** – Photographs of typical coating conditions were taken and are included as part of the report.

Visual Inspection

General

For purposes of the visual inspection, the bridge was broken down into simple component members (i.e. truss members, floor stringers, guard rails, cables and towers). Overall, the visual coating condition was rated fair to good. The overall rate of coating deterioration (spot rust, pinpoint rust, and cracks in the existing coating) was minimal when compared to all the steel surfaces. Coating blisters or application defects such as excessive runs or sags were minimal. There were isolated spot areas of corrosion. Areas of graffiti were found on the bridge at the abutments. A summary of the typical coating condition on the various structural members of the bridge is presented below.

Truss Members

Spot corrosion on the truss members typically ranged from approximately 0.3% to 1% of the surface area. There were several isolated areas with spot corrosion on the North truss ranging from 1% to 3%. Areas of spot corrosion were scatters across the length and most often occurring at the connections. Conditions were typical for upper and lower truss chords, verticals, diagonals, and bracing members. See Photographs 2 through 11 below.

Photo 2 –Typical view of truss.

Photo 3 –Spot corrosion on truss connection.

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Photo 7 –Spot corrosion on truss bracing.

Photo 8 – Spot corrosion on truss lower chord.

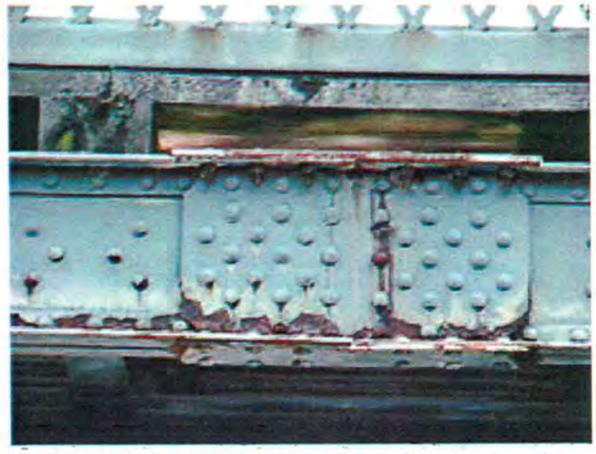

Photo 9 –Spot corrosion on truss lower chord.

A white discoloration was observed on the lower truss and adjacent steel. The discolored areas had the appearance of salt deposits. The discoloration may also be caused by runoff from the flower boxes.

Photo 10 –White stain on lower chord.

Photo 11 – White stain on lower chord.

Floor Beams

Coating deterioration on the floor beams ranged from approximately 0. 3% to 1% of the surface area. Higher levels of corrosion were typically observed at the connections with the lower chords.

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Photo 13 – Typical condition of floor beams.

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Total coating system dry film thickness measurements were obtained on the existing coating system. The following table, Table 1 – Dry Film Thickness Measurements, summarizes the range of the thicknesses obtained with a Positector 6000, magnetic-type dry film thickness gage:

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Spot Repairs with Full Overcoat(s)

The application of a full overcoat serves two primary purposes: the additional coat provides additional barrier protection and helps to seal minor defects that are not apparent when conducting spot repairs. It also offers an improved appearance when compared to spot repairs. The addition of the overcoat also adds complexity and cost to the overall project. The complexity increases because a contractor must now gain access to all areas of the structure to apply the full coat. The existing surface must also be thoroughly cleaned (i.e., power washed) to remove chalk and surface debris. The adhesion of the existing coating must also be good and sound; otherwise the stresses imparted by the overcoat can cause disbonding of the existing system, especially under freeze/thaw conditions. In some cases, two full overcoats are applied.

This strategy is typically used when the amount of visible corrosion and coating deterioration covers less than 15% of the surface.

Total Coating Removal and Replacement

Total removal and replacement is the final option for maintenance painting and is the costliest option, especially when removing existing coatings that contain toxic metals. However, it offers the greatest opportunity for long-term protection. All of the mill scale, rust, and paint are completely removed and a new system with a new design life is applied. This method also provides the most pleasing appearance.

When total removal and replacement is performed, a new maintenance cycle begins. As the coatings age and weather, isolated spot repairs will be required. Several spot repairs may be made to the individual structure until a full overcoat is necessary. More spot repairs may then be made and additional overcoats applied until extensive corrosion develops, significant coating breakdown occurs, or the mechanical properties of the coatings (e.g., the adhesion) degrade to the point where additional work (spot touch-up or overcoating) is no longer practical. At this time, complete removal may again be required, but only after the maximum effective life of the original coating system has been extended through the planned maintenance activities.

RECOMMENDATIONS

The existing coating system on the Drake Hill Road Bridge is in fair to good condition overall. The degree of coating failure typically ranged from 0. 3% to 3% of the surface area. Randomly scattered areas of spot corrosion were observed throughout the structure. Based on the percentage of visible corrosion, the coating is at a point where maintenance painting is economically advantageous. When maintenance work is performed, there are two recommended options.

Option 1 – Spot Repairs: Under this option, surface preparation on areas of spot corrosion/coating failure would be performed in accordance with SSPC SP-3, "Power Tool Cleaning." Vacuum shrouded power tools should be used to minimize the containment requirements, but nuisance tarps will be required to capture the paint chips that are dislodged by the tools, but not captured by the vacuum.

The spot repair coating system should involve three coats, consisting of an epoxy mastic prime coat, an epoxy intermediate coat, and a polyurethane finish coat, with stripe coats of the primer and intermediate coats applied to edges, crevices, rivets, and other irregular surfaces. One benefit to this option would be a reduced total project cost for maintenance painting. Spot repairs will leave a patchwork like appearance and may not be acceptable based on aesthetics.

Option 2 – Spot Repairs with Full Overcoat: Under this option, surface preparation on areas of spot corrosion/coating failure would be performed the same as in option 1. Based on the current assessment data and visual observations, in order to apply an

overcoat, all surfaces must also be cleaned by pressure washing to remove chalk, chlorides, dirt, and other debris.

The overcoat system should involve two coats, consisting of a penetrating sealer tie coat and a polyurethane finish coat. Stripe coats of the intermediate should be applied to edges, crevices, rivets, and other irregular surfaces. Application of a test patch is always strongly recommended prior to overcoating the entire area.

Chloride Remediation

It is imperative that residual chloride levels (salt contamination) be maintained at acceptable concentrations prior to coating. The level of salt contamination when applying organic coatings should be kept below 7 $\mu g/cm^2$. The specifications should require testing after surface preparation has been performed and prior to painting. In many instances, chloride contamination can be reduced to acceptable levels by pressure water cleaning and/or abrasive blast cleaning with a combination of finely graded and coarser abrasive media. Chloride removal agents can also be added to the pressure washing water. Other options include abrasive blast cleaning the steel, and allowing it to rust over night followed by re-blast cleaning.

Dealing with Lead

Laboratory testing reported detectable concentrations of lead present in the existing coatings on the bridge. The OSHA Lead in Construction Standard (29 CFR 1926.62) requires that controls be implemented if any detectable concentrations of lead are present. The OSHA Compliance Directive issued for the OSHA Lead in Construction Standard, Instruction CPL 2-2.58, states that if an employer has appropriately tested for lead (e.g., tested all layers of paints or coatings that may be disturbed) utilizing a valid detection method, and found no detectable levels of lead, then the standard does not apply. Paints with detectable concentrations of lead require the contractor performing the work to implement interim controls and assess actual employee exposures during the work in accordance 29 CFR 1926.62. Based on the lead results provided by the laboratory testing, 29 CFR 1926.62 is invoked during any activities that disturb the paint (e.g., abrasive blast cleaning, scraping, burning, and grinding).

It should be noted that other hazardous metals are also present in the existing coating. Any disturbance of paint containing heavy metals in addition to lead must be performed in accordance with the requirements of the applicable OSHA standards.

In addition, containment will be required for the protection of the environment and the public, and the hazardous waste must be properly managed.

Opinion of Probable Coating Replacement Costs

A cost analysis was prepared for the recommended maintenance options. This analysis involved making various assumptions, based upon KTA and industry experience, of how a contractor might staff and proceed with the aforementioned recommendations. Crew sizes,

production rates, material and equipment requirements are evaluated and man-days and project-days are calculated. From the estimated project duration, costs associated with labor, materials, and equipment are factored in and the costs are developed. Overhead and profit are added as a multiplier to the base cost. For the purposes of this opinion of probable coating cost, labor was considered to be prevailing wage and equipment was calculated with rental rates.

Production days were calculated from the square footage of paintable steel surfaces and an allocated production rate. The surface areas for the bridge were calculated from the provided drawings. The Drake Hill Road Bridge surface area is estimated to be 22,000 total square feet. Finally, a variance multiplier is used on the final cost to develop a range of anticipated bid prices. This multiplier allows for the variations in contractor bidding techniques, new technology, and scheduling of the work within the painting season. The opinion of probable cost to perform spot repairs with a full overcoat ranges from $244,300 to $295,700 and the cost to perform spot repairs only ranges from $50,100 to $60,700. Spot repairs with a full overcoat would take approximately one month for total production time while spot repairs only would take approximately one week.

Opinions of probable construction costs are prepared on the basis of KTA's experience and qualifications and represent KTA's judgment as field professionals generally familiar with the industry. However, since KTA has no control over the cost of labor, materials, equipment, or services furnished by others, over contractor's methods of determining prices, or over competitive bidding of market conditions, KTA cannot and does not guarantee that proposals, bids, or actual construction costs will not vary from KTA's opinions of probable cost.

Exhibit B

Simsbury Flower Bridge
Load Rating Report

Bridge Load Rating

Prepared for:

Town of Simsbury

SIMSBURY, CT

DEPT. OF ENGINEERING

OLD DRAKE HILL ROAD BRIDGE (FLOWER BRIDGE)

OVER FARMINGTON RIVER

Bridge No. 03984

Date of Inspection: 27 June 2017

Date of Rating: 29 August 2019

Prepared by:

GM2 Associates, Inc.

115 Glastonbury Blvd.

Glastonbury, CT 06033

INDEX OF REPORT

EXECUTIVE SUMMARY

Bridge No. 03984 carries Old Drake Hill Road Bridge (Flower Bridge) over Farmington River in Simsbury, Connecticut. The overall length of the structure is 183 feet and the curb to curb width is 15 feet. This steel thru-truss bridge structure is comprised of two Parker trusses and was built in 1892, with structural repairs performed in 1977, and further rehabilitated in 1993 for pedestrian traffic. Currently the bridge is closed to any vehicular traffic, and is open to carry pedestrian and bicycle traffic only.

During this load rating analysis the bridge was evaluated for pedestrian loading and a H10 vehicle based on the as-inspected condition in compliance with AASHTO Guide Specifications for the Design of Pedestrian Bridges. Both the pedestrian and vehicular loads were applied so as to produce the maximum load effects on the bridge members. The load rating analysis includes deterioration of the truss members, floor beams, and pins as noted in the most recent bridge inspection report (06/27/2017, GM2 Associates, Inc.).

Destructive and non-destructive testing was performed on the structural elements of the bridge to determine the yield strength of the steel. Testing results were found to be consistent with previous tensile tests performed in 1977 and are included in Appendix E. The yield strength of the truss members and pins in the load rating analysis was taken as 38 ksi and 47 ksi, respectively, based on the material tests results. Note that this yield strength exceeds the 26 ksi recommended by the AASHTO Manual for Bridge Evaluation (MBE) for unknown steel based on year of construction.

<u>Pedestrian Load Rating</u>

The rating factor for all main truss members (i.e. top chord, bottom chord, diagonals, and vertical struts) was found to be satisfactory (greater than 1.0) and is controlled by the bottom chord with a rating factor of 1.44, closely followed by the top chord with a rating factor of 1.45.

The rating factor for the floor beams and the timber deck was found to be satisfactory, with a controlling rating factor of 3.08 and 6.29, respectively.

The rating factor for the connections to the bottom chord was found to be unsatisfactory (less than 1.0) with a controlling rating factor of 0.93. The load rating of the connections to the bottom chord is controlled by a gusset plate installed during rehabilitation connecting the diagonal members to the bottom chord at panel points L2 and L10 (see *Findings and Recommendations* section for additional discussion).

The rating factor for the pins at the top chord panel points was found to be satisfactory, with a controlling rating factor of 2.40. The pins at the bearing points (support pins), however, have a rating factor less than 1.0 by a significant margin, with a controlling rating factor of 0.11. The low rating factor of the support pins is mainly due to a missing bearing plate at the northwest support, which results in a different load path at this support from the as-designed condition. Additionally, the existing load path at the southwest and east supports is uncertain due to existing deterioration in the bearing plates at the interface with the pin. Assuming an as-designed load path at the southwest and east supports, the minimum rating factor at these supports was found to be 0.64. Additional discussion is included in the *Findings and Recommendations* section of this report.

H10 Vehicle Load Rating

The rating factor for all main truss members (i.e. top chord, bottom chord, diagonals, and vertical struts) was found to be satisfactory (greater than 1.0) and is controlled by the bottom chord with a rating factor of 2.04.

The rating factor for the floor beams and the timber deck was found to be unsatisfactory (less than 1.0), with a controlling rating factor of 0.81 and 0.09, respectively.

The rating factor for the connections to the bottom chord was found to be satisfactory with a controlling rating factor of 2.74, controlled by a gusset plate installed during rehabilitation connecting the diagonal members to the bottom chord at panel points L2 and L10.

The rating factor for the pins at the top chord panel points was found to be greater than 1.0, with a controlling rating factor of 3.64. The rating factor at support pins (pins at bearing points) was found to be unsatisfactory, with a controlling rating factor of 0.43.

Recommendations

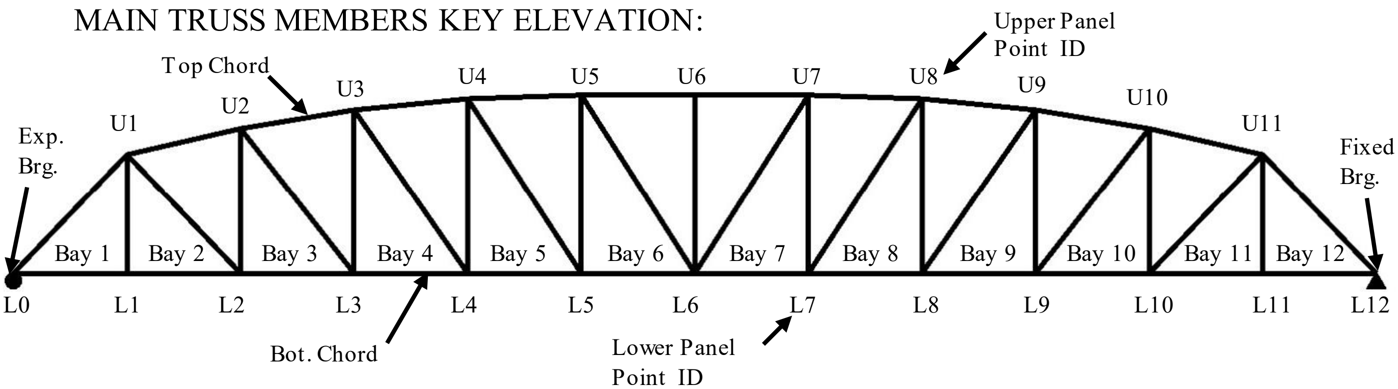
Repairs to the northwest support and further evaluation of the condition at the remaining supports is recommended to improve the live load carrying capacity/rating factor of the bridge. A rehabilitation study report (RSR) outlining the recommended repairs is to follow this load rating analysis.

Based on the load rating analysis, it is recommended to limit the maximum occupancy to 150 persons uniformly distributed on the bridge until necessary repairs to the bearings are performed. Once bearings repairs are performed, the maximum occupancy may be increased to approximately 750 persons uniformly distributed on the bridge. Additionally, it is recommended to maintain the current restriction to vehicular traffic on the bridge.

SUMMARY OF BRIDGE RATING

Town/City:	Simsbury, CT	**Bridge No.:**	03984
Carries:	Pedestrian Walkway	**Crosses:**	Farmington River
Owner:	Town of Simsbury	**Year Built:**	1892
Maintained By:	Town of Simsbury	**Rebuilt/Rehab:**	1977, 1993

MAIN TRUSS MEMBERS KEY ELEVATION:



PEDESTRIAN LOADING RATING FACTORS:

Main Truss Members:

	MAIN TRUSS MEMBERS RF SUMMARY: PEDESTRIAN LOAD						
	Bottom Chord			Struts		Top Chord	
Bay	Flexure	Tension	Shear	Diagonal	Vertical*		Control
1	6.89	3.52	12.59	n/a	5.09	2.23	2.23
2	14.34	3.52	14.60	1.46	3.52	2.24	1.46
3	11.93	2.21	14.83	2.59	6.79	1.83	1.83
4	10.46	1.79	14.68	2.64	10.26	1.62	1.62
5	10.00	1.56	14.83	3.87	146.74	1.50	1.50
6	10.22	1.44	14.89	12.99	n/a	1.45	1.44
7	10.22	1.44	14.90	12.99	146.74	1.45	1.44
8	10.00	1.56	14.82	5.89	10.27	1.50	1.50
9	10.46	1.79	14.70	2.64	6.79	1.62	1.62
10	11.93	2.21	14.84	2.59	3.52	1.83	1.83
11	14.35	3.52	14.61	1.46	5.08	2.24	1.46
12	6.88	3.52	12.57	n/a	n/a	2.23	2.23
							1.44

* Strut located between Bay # reported and following Bay #.

Floor Beams and Deck:

	RATING FACTORS: PEDESTRIAN LOAD				
	Floor Beam		Decking		
Bay	Flexure	Shear	Flexure	Shear	Control
Ped.	3.08	15.55	6.29	8.47	3.08

Connections and Pins:

Bottom chord connections

RATING FACTORS: PEDESTRIAN LOAD				
Diagonal Struts	Diag. Welded Conn.	Conn. Plate	Plate Welded Conn.	Controlling
Panel Point				
L2 & L10	1.99	0.93	1.08	0.93
L3 & L9	3.20	1.56	2.07	1.56
L4 & L8	4.46	2.12	2.85	2.12
L5 & L7	4.88	3.77	5.17	3.77
L6	9.99	8.13	11.68	8.13
				0.93

Floor beam connection

RF, Ped	**9.00**

Top Chord Pins

	RATING FACTORS: PEDESTRIAN LOAD		
	Shear + Moment	Bearing	Controlling
Panel Point			
U1 & U11	3.94	3.04	3.04
U2 & U10	11.22	2.40	2.40
			2.40

Support Pins

	RATING FACTORS: PEDESTRIAN LOAD		
	Shear + Moment	Bearing	Controlling
Pin Location			
Northwest Support	0.11	2.06	0.11
Southwest & East Supports	0.64	1.94	0.64
			0.11

H10 VEHICLE LOADING RATING FACTORS:

Main Truss Members:

	MAIN TRUSS MEMBERS RF SUMMARY: H10						
	Bottom Chord			Struts		Top Chord	
Bay	Flexure	Tension	Shear	Diagonal	Vertical*		Control
1	2.04	13.19	6.03	n/a	4.34	8.36	2.04
2	2.41	13.19	6.08	4.32	7.15	8.48	2.41
3	2.41	8.40	6.07	5.86	12.15	7.01	2.41
4	2.39	6.85	6.12	4.68	12.17	6.22	2.39
5	2.37	5.98	6.09	4.67	11.67	5.77	2.37
6	2.38	5.52	6.10	7.65	n/a	5.58	2.38
7	2.38	5.52	6.10	7.65	11.67	5.58	2.38
8	2.37	5.98	6.09	7.11	12.17	5.77	2.37
9	2.39	6.85	6.12	4.68	12.15	6.22	2.39
10	2.40	8.39	6.07	5.86	7.15	7.01	2.40
11	2.41	13.18	6.08	4.32	4.33	8.48	2.41
12	2.04	13.18	6.02	n/a	n/a	8.36	2.04
							2.04

* Strut located between Bay # reported and following Bay #.

Floor Beams and Deck:

	RATING FACTORS: H10				
	Floor Beam		Decking		
Bay	Flexure	Shear	Flexure	Shear	Control
H10	0.81	3.61	0.09	0.20	0.09

Connections and Pins:

<u>Bottom chord connections</u>

RATING FACTORS: H10				
Diagonal Struts Panel Point	Diag. Welded Conn.	Conn. Plate	Plate Welded Conn.	Controlling
L2 & L10	5.86	2.74	2.93	2.74
L3 & L9	7.23	3.52	3.99	3.52
L4 & L8	7.92	3.76	4.20	3.76
L5 & L7	5.89	4.55	4.53	4.53
L6	5.88	4.79	6.87	4.79
				2.74

<u>Floor beam connection</u>

RF, H10	**2.09**

Top Chord Pins

	RATING FACTORS: H10		
Panel Point	Shear + Moment	Bearing	Controlling
U1 & U11	10.24	3.64	3.64
U2 & U10	27.03	4.89	4.89
			3.64

Support Pins

	RATING FACTORS: H10		
Pin Location	Shear + Moment	Bearing	Controlling
Northwest Support	0.43	7.73	0.43
Southwest & East Supports	2.44	7.25	2.44
			0.43

FINDINGS AND RECOMMENDATIONS

Main Truss Members:

Due to the connection of the floor beams to the bottom chord of the truss at multiple points between panel points, simple analysis investigating only axial load effects in the bottom chord members was deemed inadequate. Therefore, rating factors for axial load, flexure and shear were calculated for the bottom chord. The rating factor for all main truss members (i.e. top chord, bottom chord, diagonals, and vertical struts) was found to be satisfactory for both pedestrian and vehicular loads. The load rating of the main truss members is controlled by the bottom chord, with a rating factor of 1.44 and 2.04 for pedestrian and vehicular loading, respectively. The controlling rating factor for pedestrian loading is closely followed by the top chord, with a rating factor of 1.45.

Floor Beams:

The floor beams are satisfactory under pedestrian load with a rating factor of 3.08. However, an unsatisfactory rating factor of 0.81 was found for vehicular loading. This is due to a floor beam with section loss on the top flange.

Timber Deck:

The timber decking is satisfactory under pedestrian loading (RF = 6.29), but fails to exceed a rating factor of 1.0 for both flexure and shear resistance under vehicular loading with a rating factor of 0.09 and 0.20, respectively.

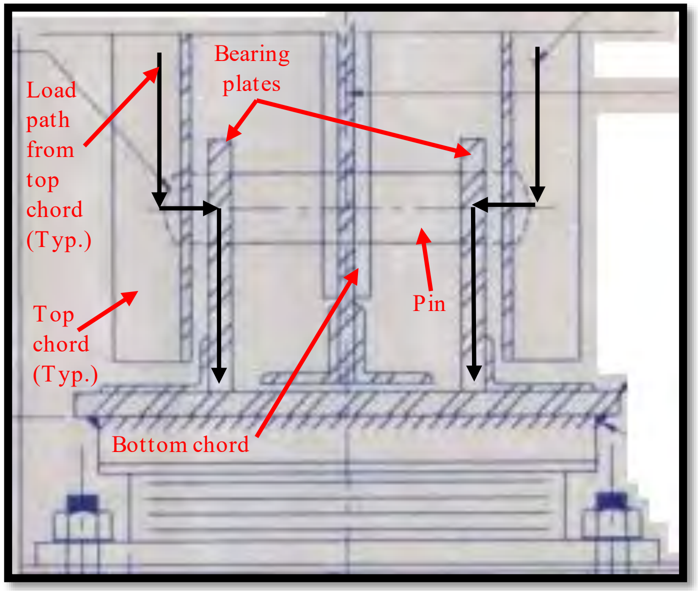
Connections to Bottom Chord:

The repair gusset plates for the diagonal member in panel points L2 and L10 were found to have a rating factor of 0.93 and 2.74 for pedestrian and vehicular loading, respectively. It is noted that the capacity of the gusset plate is directly dependent upon the length of the weld connecting the diagonal member to the gusset plate. Although the rating factor for the connections to the bottom chord under pedestrian load is considered unsatisfactory, it is noted that an additional 0.75" length of weld would bring the rating factor above 1.0.

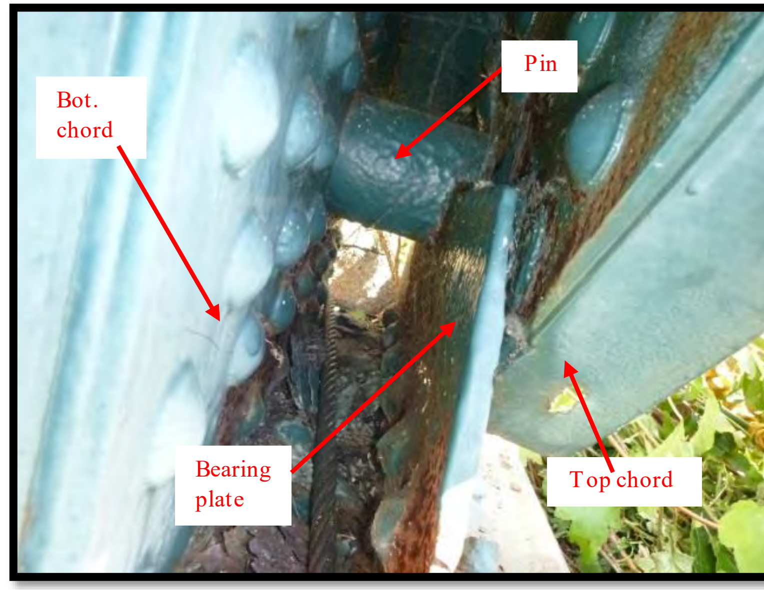
Truss Pins:

The pinned connections at the lower panel points were retrofitted in 1977, bypassing the original pins by adding gusset plates connecting the diagonal members and vertical struts to the bottom chord. Therefore, the pins at the lower panel points are considered to not carry any load and were not included in the load rating analysis

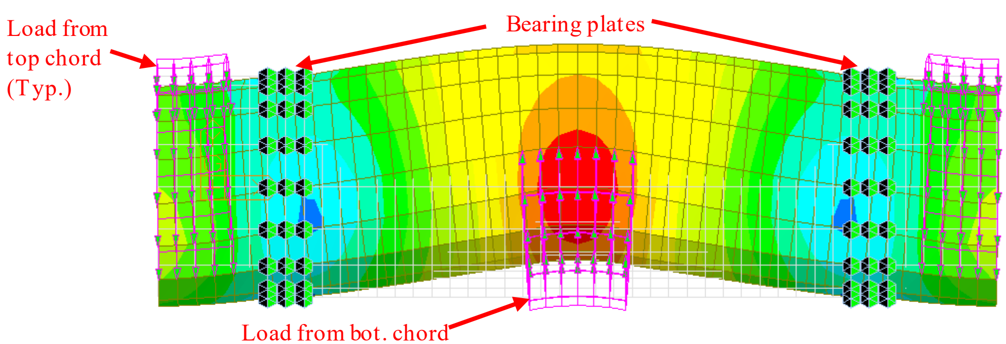
Rating factors for the pins at the top chord panel points were found to be satisfactory for both vehicular and pedestrian loading. However, the pins at the supports (bearing points) were found to have a rating factor less than one for both pedestrian (RF = 0.11) and vehicular loading (RF = 0.43). The low rating factor is primarily due to a missing bearing plate at the northwest support. Figures 1 and 2 below show the typical existing support pin configuration at the southwest and east supports (similar to the as-designed configuration) and the existing support pin configuration at the northwest support, respectively.



(a) As-designed support pin configuration

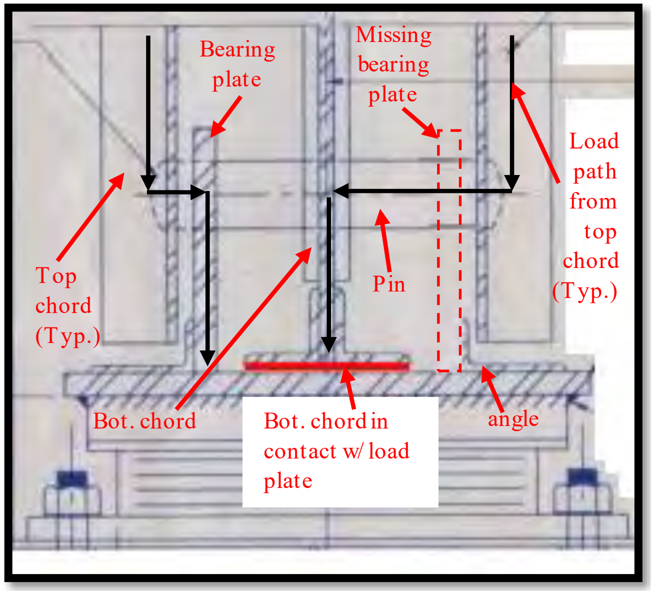


(b) Typical support pin assembly at southwest and east supports

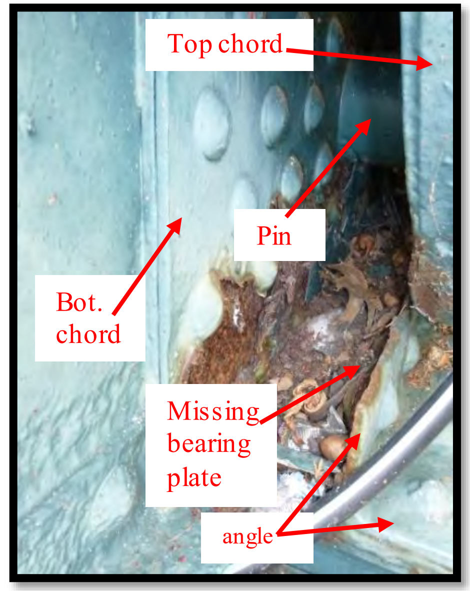


(c) Deflected shape of support pin in the as-designed condition

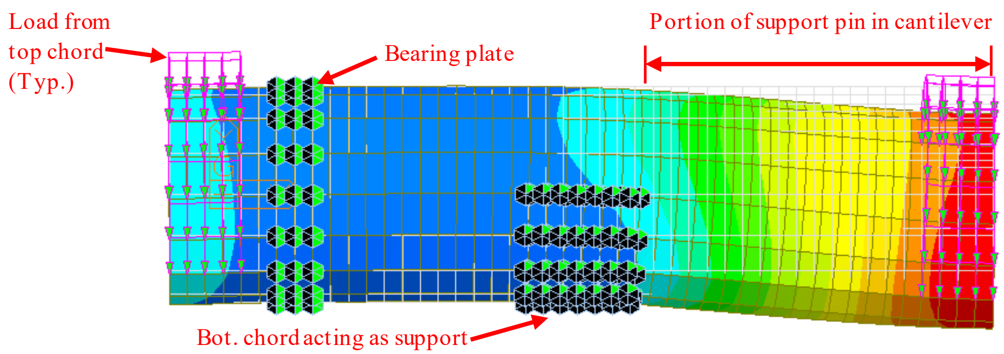
Figure 1: As-designed support pin configuration. Typical condition at southwest and east supports.



(a) As-existing support pin configuration at northwest support, missing bearing plate



(b) Support pin configuration at northwest support



(c) Deflected shape of northwest support pin in the as-inspected condition

Figure 2: As-existing support pin configuration at northwest support

In the as-designed configuration (Fig. 1), the load from the top chord is transferred through the pin, to the bearing plate and consequently to the bearing. Given the close proximity of the top chord to the bearing plates, the load transfer through the pin occurs in pure shear (i.e. the top chord does not exert a bending moment on the pin). The load from the bottom chord is transferred through the pin, in bending and shear, to the bearing plate and consequently to the bearing.

This as-designed load path is no longer valid at the northwest support, where a bearing plate is missing (see Fig. 2). At this location, the load from the top chord is transferred through the pin (in bending and shear) directly to the bottom chord, which acts as the support. This results in larger load effects on the pin when compared to the as-designed condition. It is recommended to perform repairs at the northwest support to restore the as-designed load path.

Based on the remaining live load capacity of the pin at the northwest support, assuming an average weight per person of 200 lbs, it is recommended to limit the maximum occupancy to 150 persons until repairs at the supports are performed.

Additionally, the bearing plates accessible during inspection show heavy corrosion at the pin-bearing plate interface and/or a small gap between the pin and the bearing plate. Therefore, it is uncertain if the remaining bearing plates are currently carrying the load as intended. It is recommended to blast clean the bearing plates to determine the extent of deterioration and need for repairs. It is noted that if the deterioration is such that the bearing plates are not carrying any load, the rating factor at these supports would be equal to that of the northwest support and the bearings must be repaired.

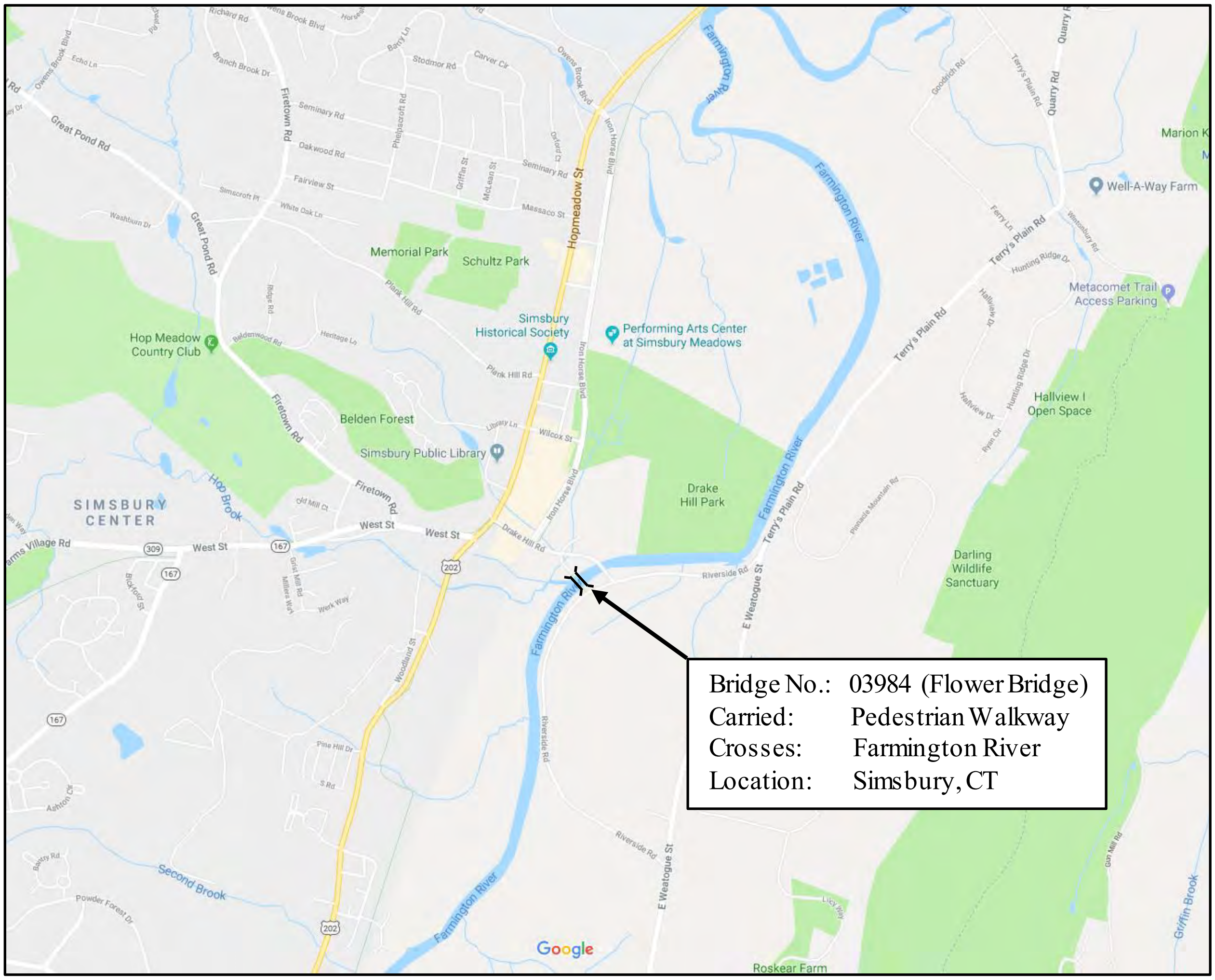
Assuming that the bearing plates at the southwest and east supports provide the intended load path, the minimum rating factor at these supports is equal to 0.64. Although this rating factor is considered unsatisfactory it is noted that it provides sufficient live load capacity to allow a pedestrian load of up to 57 psf, which translates to approximately 750 persons on the bridge.

Recommendations:

Based on the load rating analysis and calculated rating factors, it is recommended to limit the maximum occupancy to 150 persons distributed uniformly on the bridge until repairs at the northwest support are performed and the extent of deterioration and need for repairs at the remaining supports is evaluated.

After the bearings are repaired the maximum occupancy may be increased to approximately 750 persons distributed uniformly on the bridge. Additionally, it is recommended to maintain the current restriction to vehicular traffic on the bridge.

LOCATION MAP



DESCRIPTION OF BRIDGE

General:

Bridge Number:	03984
Owner:	Town of Simsbury
Maintained By:	Town of Simsbury
Location:	Simsbury, CT
Carries:	Pedestrian Walkway
Crosses:	Farmington River
Latest NBI Inspection:	27 June 2017
Date of Construction:	1892
Bridge Type:	Steel Through-Truss
Dates of Rebuild/Rehab:	1977, 1993
Description of Rehab:	Deck Replacement, Lateral Bracing Replacement, Painting
Posting:	n/a

Design:

Superstructure:	The steel thru-truss bridge structure is comprised of two Parker trusses and was built in 1892, with structural repairs performed in 1977, and further rehabilitated in 1993 for pedestrian traffic. The repairs and rehabilitation encompassed weldment of the gusset plates atop the bottom chord member, addition of channel sections to the truss vertical members, steel plates weldment to the truss diagonal members and gusset plates along with new timber deck planks installation.
Bridge Span:	183.0’
Bridge Skew:	0°
Bridge Width:	17.3’ truss-to-truss
Walkway Width:	16.0’ deck width
Walkway Surface:	Timber decking
Bridge Railing:	4”x4” square tubing

Condition:

Truss Condition:	The steel truss members and connections are in fair condition (rated 5) per the Inspection Report date 6/27/17.

RATING ANALYSIS ASSUMPTIONS AND CRITERIA

The objective of this rating report is to present the results of a pedestrian and H10 vehicle load carrying capacity analysis for Bridge Number 03984, Flower Bridge over the Farmington River in Simsbury, CT. The load rating was performed based on the existing conditions found during the latest bridge inspection conducted by GM2 Associates, Inc. on 27 June, 2017.

The bridge rating calculations and bridge rating report were prepared in accordance with the following standards:

a) AASHTO LRFD Bridge Design Specifications, 7th Ed. 2014 (with Interims through 2016)
b) LRFD Guide Specifications for the Design of Pedestrian Bridges, 2nd Ed., 2009 (with Interims through 2015).
c) AASHTO Manual for Bridge Evaluation, 2nd Ed., 2011 (with Interims through 2016)
d) AASHTO Standard Specifications for Highway Bridges, 17th Ed., 2002

The scope of the work for this report consists of the following:

- Review all available plans and bridge inspection reports.
- Utilizing the Load and Resistance Factor Rating (LRFR) Method,
    - Provide Rating Factors for each individual member of the steel truss structure, decking, and floor beams.

The bridge rating calculations and bridge rating report were prepared using the following assumptions:

- Rating factors were calculated for pedestrian loading and H10 truck. Due to the bridge being simply supported the pedestrian load was applied on the entire deck area so as to produce the maximum load effects on the bridge members. A pedestrian load of 90 psf was used in the load rating analysis per AASHTO Guide Specifications for the Design of Pedestrian Bridges. This load is based on the maximum credible pedestrian loading, which in combination with the load factor of 1.75 results in a total loading of 158 psf. A visual representation of the pedestrian load used in the analysis and additional discussion on pedestrian loads can be found in Appendix D.
- Superimposed dead loads from the timber decking, steel floor beams, and steel cross bracing under the deck were calculated and applied as point loads at each floor beam location on the bottom chord along the length of the span.
- Pedestrians and a vehicle will never be on the bridge at the same time. Therefore, pedestrian and vehicular loads are not considered concurrently in the analysis.
- The yield strength of the truss members and pins was taken as 38 ksi and 47 ksi, respectively, based on the results from the material testing (see Appendix E).

1994 REHABILITATION PLANS

TOWN OF SIMSBURY, CONNECTICUT

REHABILATATION OF OLD BRIDGE ROAD BRIDGE OVER THE FARMINGTON RIVER

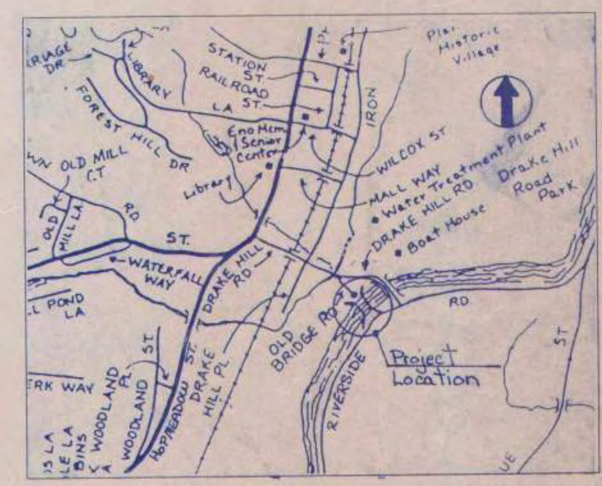
FIRST SELECTMAN
MARY GLASSMAN

DIRECTOR OF PUBLIC WORKS
FRANK ROSSI

AUGUST 29, 1994

STATE PROJECT NO. 128-126
BRIDGE NO 03984

LOCATION MAP

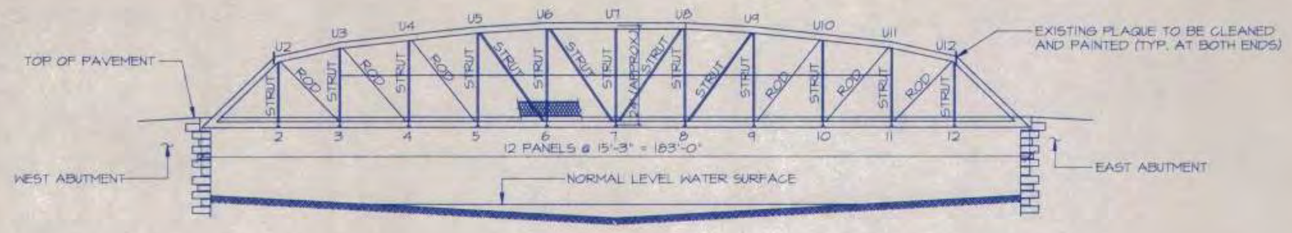


LIST OF DRAWINGS:

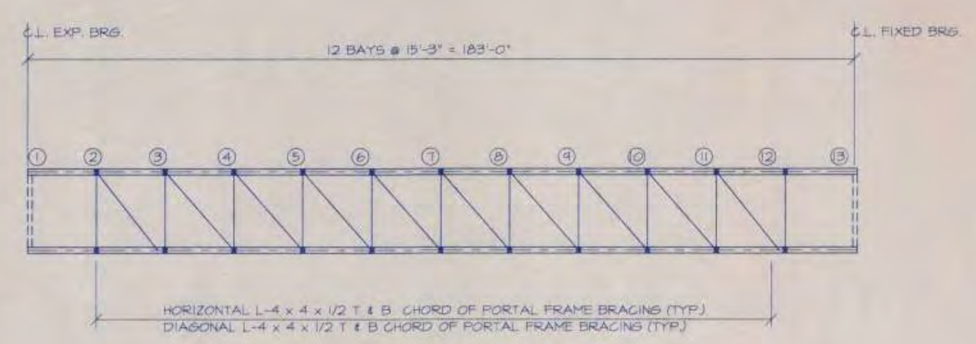
S-1 – GENERAL PLAN
S-2 – BRIDGE DECK AND TYPICAL DETAILS
S-3 – EXPANSION BEARING DETAILS
S-4 – PROTECTIVE FENCE DETAILS
S-5 – ABUTMENT DETAILS

CONSULTANTS:

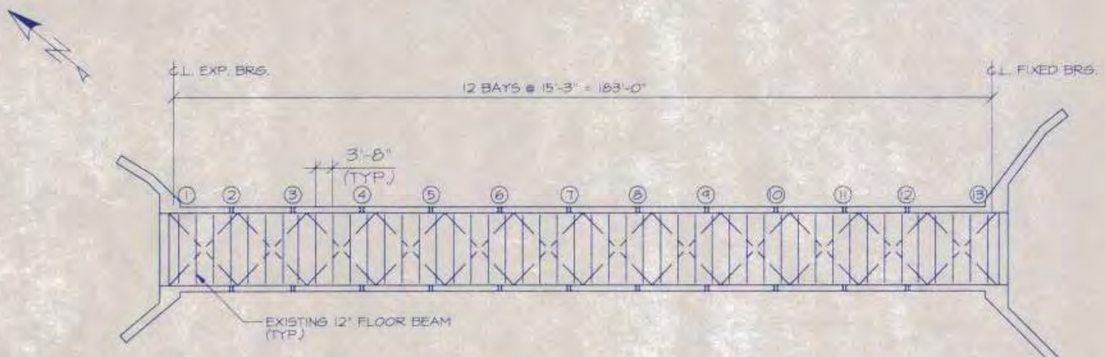
MACCHI ENGINEERS
44 GILLETT STREET, HARTFORD CT. (203) 549-6190



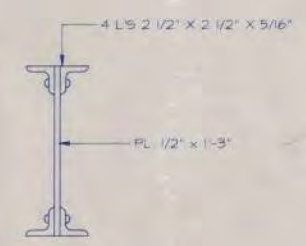
EXISTING BRIDGE ELEVATION (LOOKING UPSTREAM)
SCALE: 1" = 20'



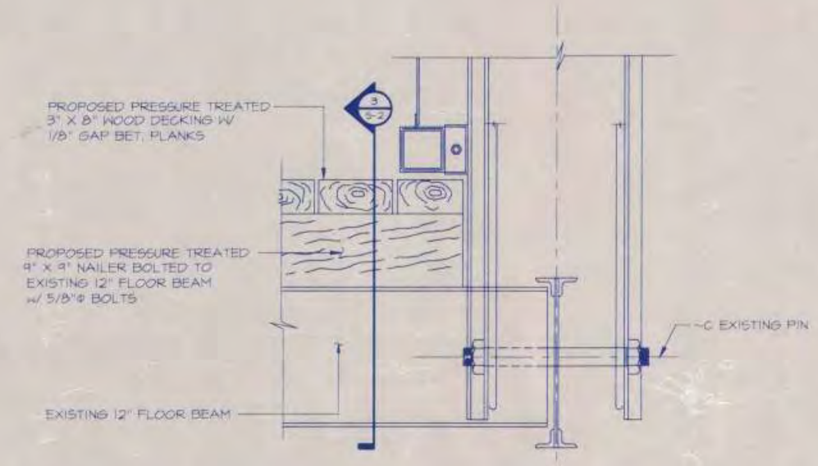
EXISTING TOP CHORD BRACING PLAN
SCALE: 1" = 20'



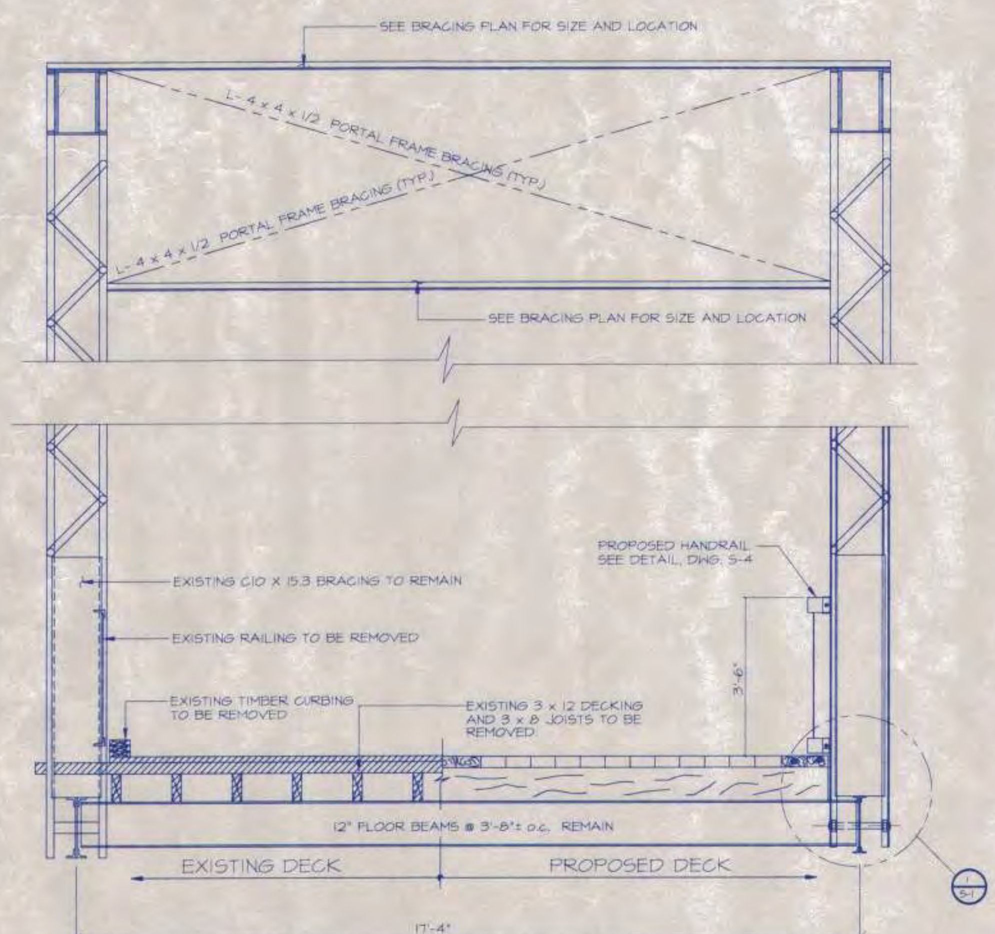
EXISTING BOTTOM CHORD FRAMING PLAN
SCALE: 1" = 20'



TYPICAL BOTTOM CHORD
SCALE: 1 1/2" = 1'-0"

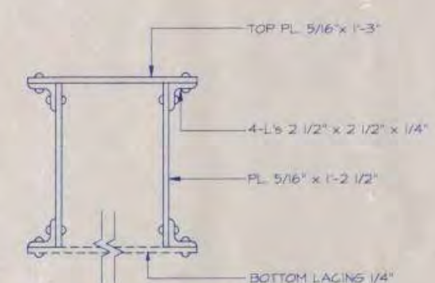


TYPICAL PROPOSED BRIDGE DECK DETAIL
SCALE: 1 1/2" = 1'-0"

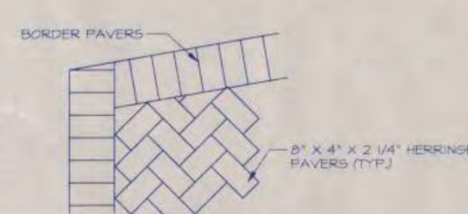


BRIDGE SECTION
SCALE: 1/2" = 1'-0"

EXISTING TO BE REMOVED

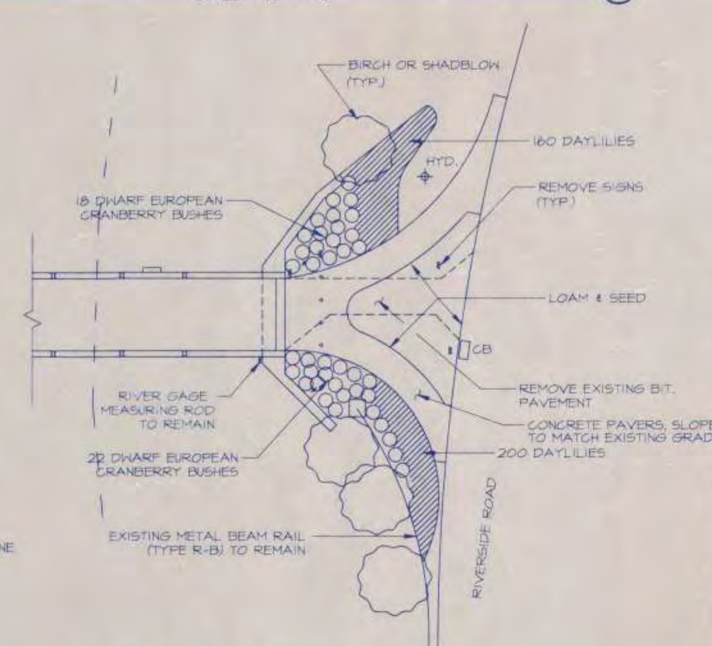


TYPICAL TOP CHORD
SCALE: 1 1/2" = 1'-0"



HERRINGBONE PATTERN BRICK PAVERS
SCALE: 3/4" = 1'-0"

NOTE:
FOR SEDIMENTATION CONTROL SILT FENCE DETAIL, SEE SHEET NO. S-4.



EAST END - GENERAL PLAN ADD ALTERNATE BID ITEMS
SCALE: 1"=20'

GENERAL NOTES:

DESIGN SPECIFICATIONS:

AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES AND INTERIM SPECIFICATIONS.
BOCA 1987 W/1990 SUPPLEMENT AND CONNECTICUT AMMENDMENTS
BRIDGE WELDING CODE - ANSI/AASHTO/AWS D1.5 - 88.
CONNECTICUT DEPARTMENT OF TRANSPORTATION FORM 814 (1988), INCLUDING SUPPLEMENTS DATED JULY 1993 AND SPECIAL PROVISIONS.

LIVE LOAD: (100 P.S.F.) AFTER REHABILITATION.

EXISTING FLOOR BEAMS AND STRINGERS ARE RATED FOR TEMPORARY H-5 CONSTRUCTION LOADS.

STEEL TYPES AND ALLOWABLE DESIGN STRESSES:

ORIGINAL BRIDGE STEEL: f_s = 14,000 P.S.I.

NEW STEEL:
ASTM A-36 STEEL, F_y = 36,000 P.S.I.

REINFORCING STEEL ASTM A-615, GRADE 60. (EPOXY COATED)

SPLASH GUARD PLATES, AND MISC. ITEMS:
GALVANIZED AS PER ASTM A-123

REMOVAL OF EXISTING BRIDGE DECK ITEMS AND OTHER MATERIAL:

STAGING SHALL BE PROVIDED UNDER THE BRIDGE FOR THE SAFETY OF WORKERS AND TO PREVENT MATERIALS FROM FALLING INTO FARMINGTON RIVER.

CONSTRUCTION METHODS WHICH MAY DISTORT OR DAMAGE FLOOR BEAMS OR TRUSS MEMBERS WILL NOT BE ALLOWED. SEE SPECIFICATIONS

ALL MATERIAL TO BE REMOVED AND NOT TO BE REUSED SHALL BECOME THE PROPERTY OF THE CONTRACTOR AND WILL BE REMOVED FROM THE SITE AND PROPERLY DISPOSED OF IN ACCORDANCE WITH STATE AND LOCAL ORDINANCES. AS SPECIFIED BY THE ENGINEER, MATERIAL MAY BE DISPOSED OF AT THE LANDFILL ON WOLCOT HILL ROAD. ALL OTHER MATERIAL WILL BE PROPERLY DISPOSED OF BY THE CONTRACTOR.

REPAIRING AND POINTING OF MASONRY WALLS:

SEE SPECIFICATIONS.

EROSION CONTROL

SEDIMENTATION CONTROL SILT FENCE SHALL BE PLACED AT THE TOE OF SLOPES AND AT OTHER LOCATIONS AS REQUIRED TO PREVENT EROSION INTO THE FARMINGTON RIVER.

BRIDGE PAINTING:

ALL STEEL SURFACES, EXCEPT NEW GALVANIZED ITEMS, SHALL BE ABRASIVE BLAST CLEANED IN ACCORDANCE WITH SSPC-SP-10 PRIOR TO THE APPLICATION OF A THREE COAT PAINT SYSTEM AS FOLLOWS:

PRIMER COAT: .... ORGANIC ZINC RICH PRIMER
INTERMEDIATE COAT: .... EPOXY MASTIC
TOPCOAT: .... HIGH BUILD ALIPHATIC URETHANE
COLOR OF TOP COAT: .... GREEN
(FED. STD. COLOR NO. 34058)

LOAMING AND SEEDING:

AREAS DISTURBED DURING THIS CONSTRUCTION SHALL BE LOAMED AND SEEDED AS PER THE SPECIFICATIONS.

STRUCTURAL DIMENSIONS:

ALL DIMENSIONS AND ANGLES SHOWN ON THE PLANS ARE BASED ON LIMITED FIELD INVESTIGATION. THE CONTRACTOR SHALL BE RESPONSIBLE FOR FIELD VERIFICATION OF ALL DIMENSIONS AND ANGLES.

CONCRETE:

CLASS 'A' CONCRETE SHALL BE USED FOR ALL WORK ON THE ABUTMENTS AND WINGWALLS. ALLOWABLE DESIGN STRENGTH SHALL BE BASED ON F_c' = 3000 psi.

TIMBER

SEE SPECIFICATIONS.

LANDSCAPE SCHEDULE
(ADD ALTERNATE ITEMS)

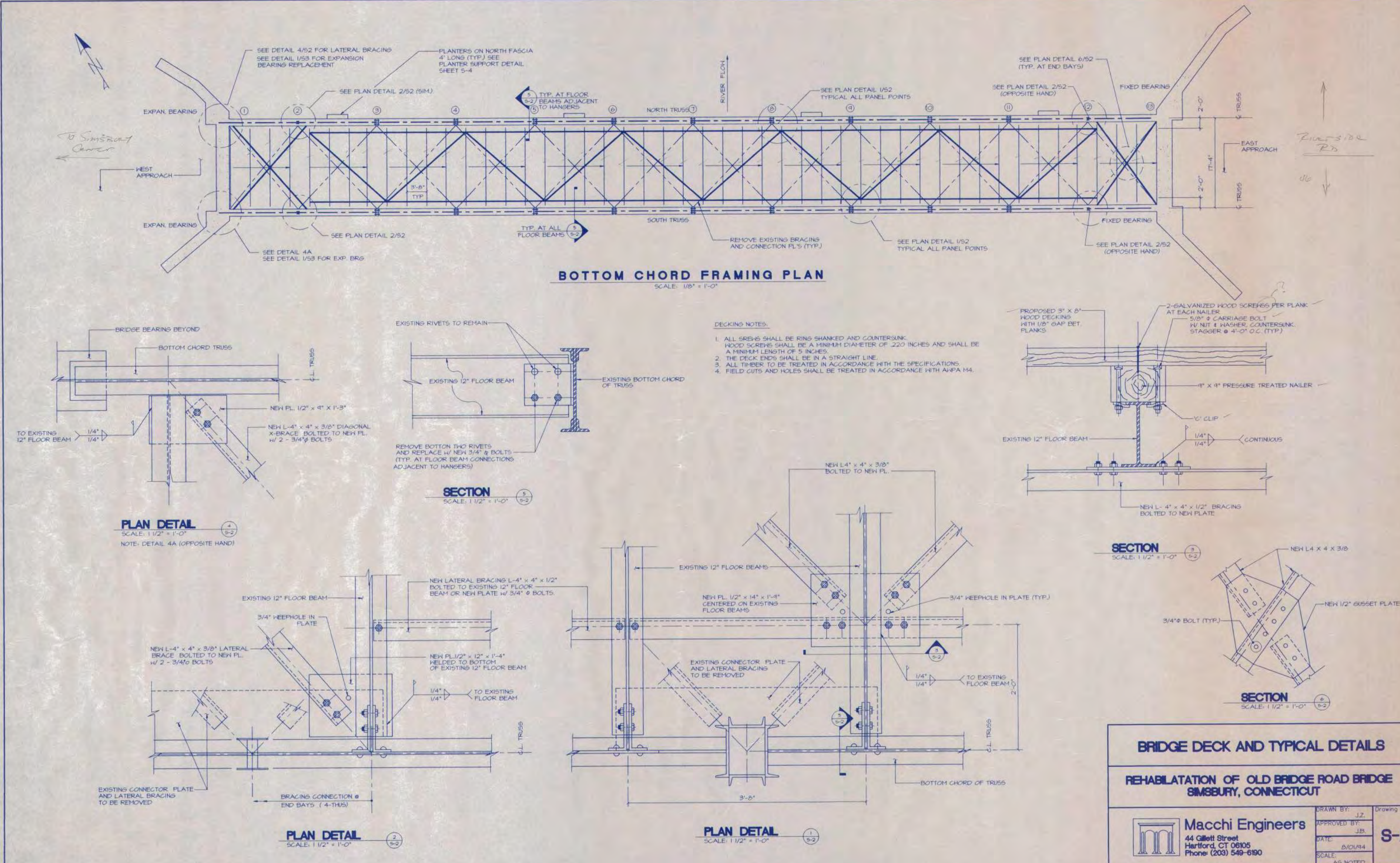
BOTANICAL NAME	COMMON NAME	QUANTITY	SIZE AREA
VIBURNAM OPULUS NANUM	DWARF EUROPEAN CRANBERRY BUSH	40	15"-18"SPD.
AMBLANCHIER CANADENSIS OR BETULA PLATYPHYLLA JAPONICA	SHADBLOW WHITESPIRE BIRCH	4	10'-12' HT. MULTI STEM
HEMEROCALLIS FLAVA HEMEROCALLIC HYPERION HEMEROCALLIS STELLA D'ORO HEMEROCALLIS HALLIS PINK	DAYLILIES	90 90 90 90 (QT. CONT.)	12" O.C.

GENERAL PLAN

REHABILATATION OF OLD BRIDGE ROAD BRIDGE
SIMSBURY, CONNECTICUT

Macchi Engineers
44 Gillett Street
Hartford, CT 06105
Phone: (203) 549-6190

DRAWN BY: J.Z.		Drawing No.
APPROVED BY: J.B.		S-1
DATE: 8/23/94		
SCALE: AS NOTED		



DECKING NOTES:

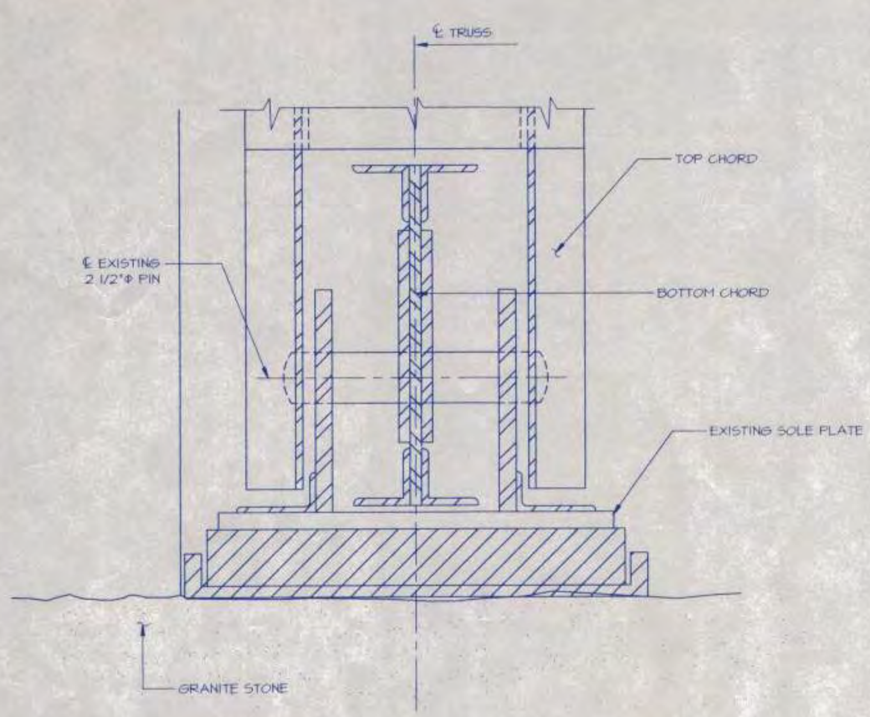
1. ALL SREWS SHALL BE RING SHANKED AND COUNTERSUNK. WOOD SCREWS SHALL BE A MINIMUM DIAMETER OF .220 INCHES AND SHALL BE A MINIMUM LENGTH OF 5 INCHES.
2. THE DECK ENDS SHALL BE IN A STRAIGHT LINE.
3. ALL TIMBER TO BE TREATED IN ACCORDANCE WITH THE SPECIFICATIONS.
4. FIELD CUTS AND HOLES SHALL BE TREATED IN ACCORDANCE WITH AWPA M4.

BRIDGE DECK AND TYPICAL DETAILS

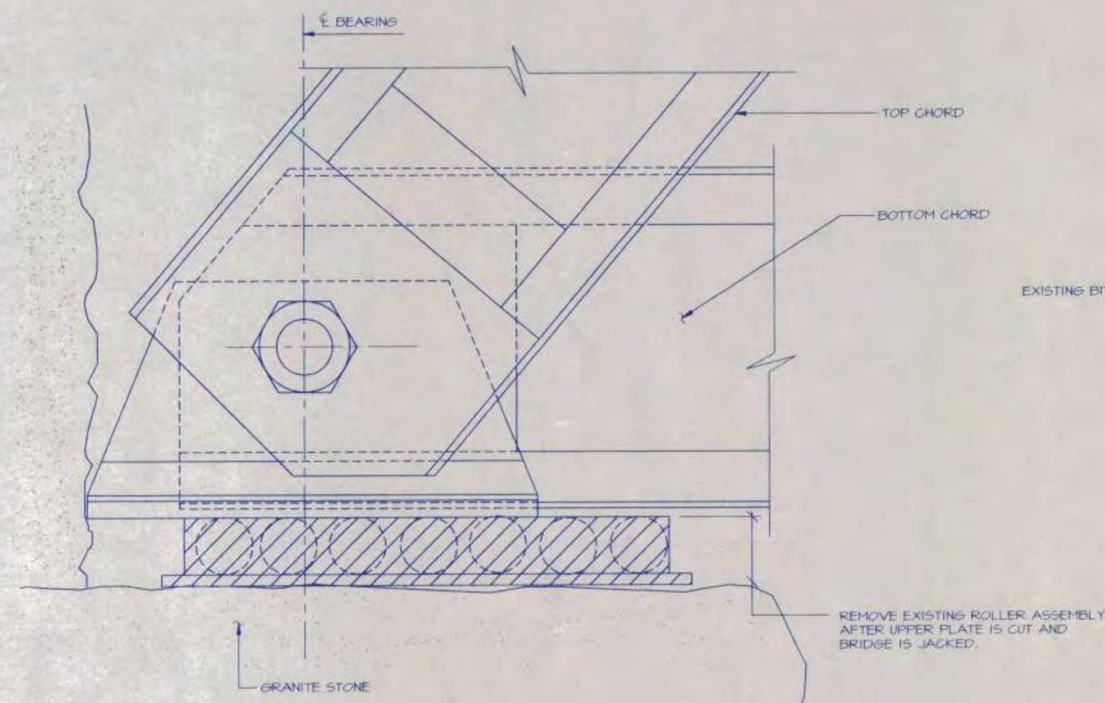
REHABILATATION OF OLD BRIDGE ROAD BRIDGE SIMSBURY, CONNECTICUT

Macchi Engineers
44 Gillett Street
Hartford, CT 06105
Phone: (203) 549-6190

DRAWN BY:	J.Z.
APPROVED BY:	J.B.
DATE:	8/01/94
SCALE:	AS NOTED
Drawing No.	S-2

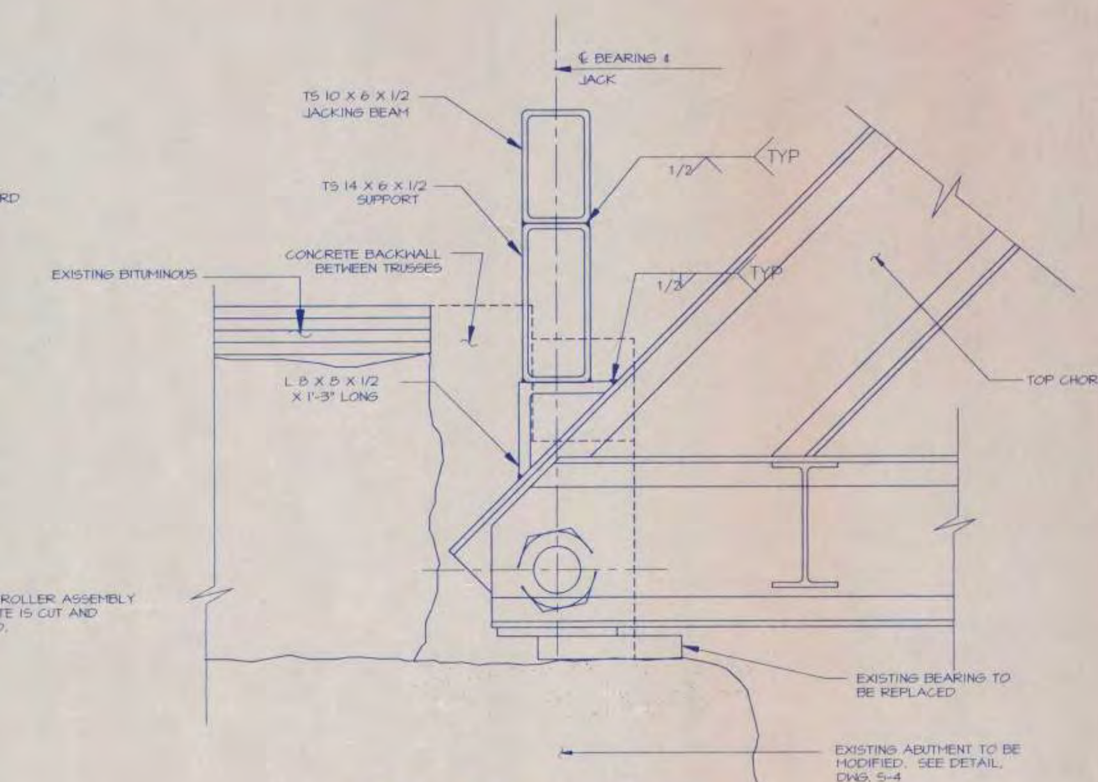


SECTION



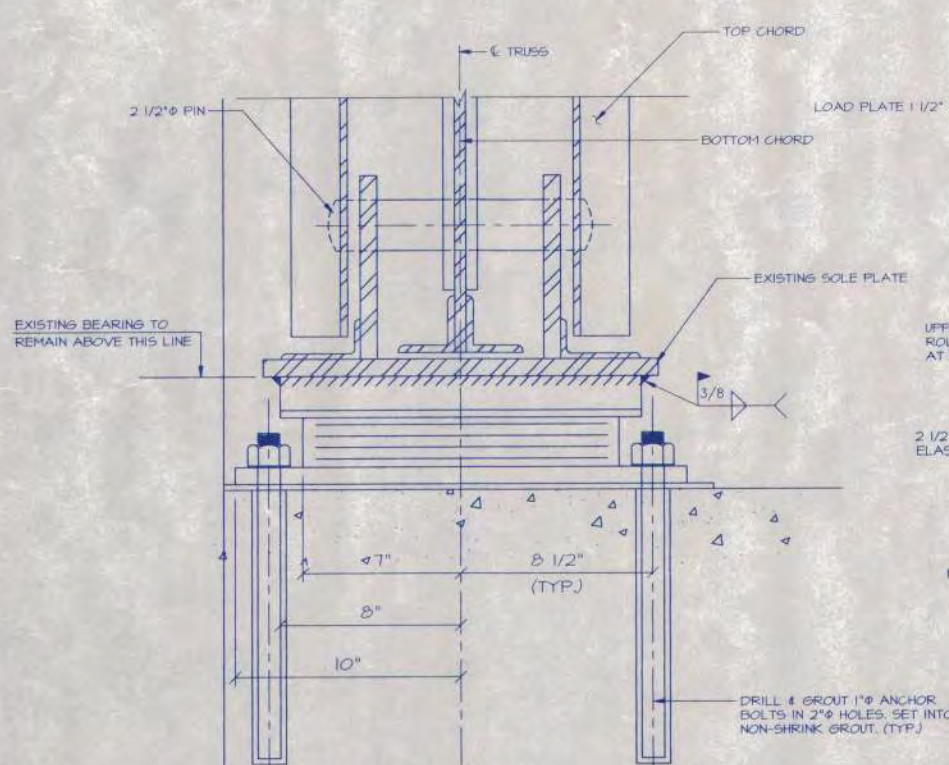
ELEVATION

EXISTING EXPANSION BEARING
SCALE: 3" = 1'-0"

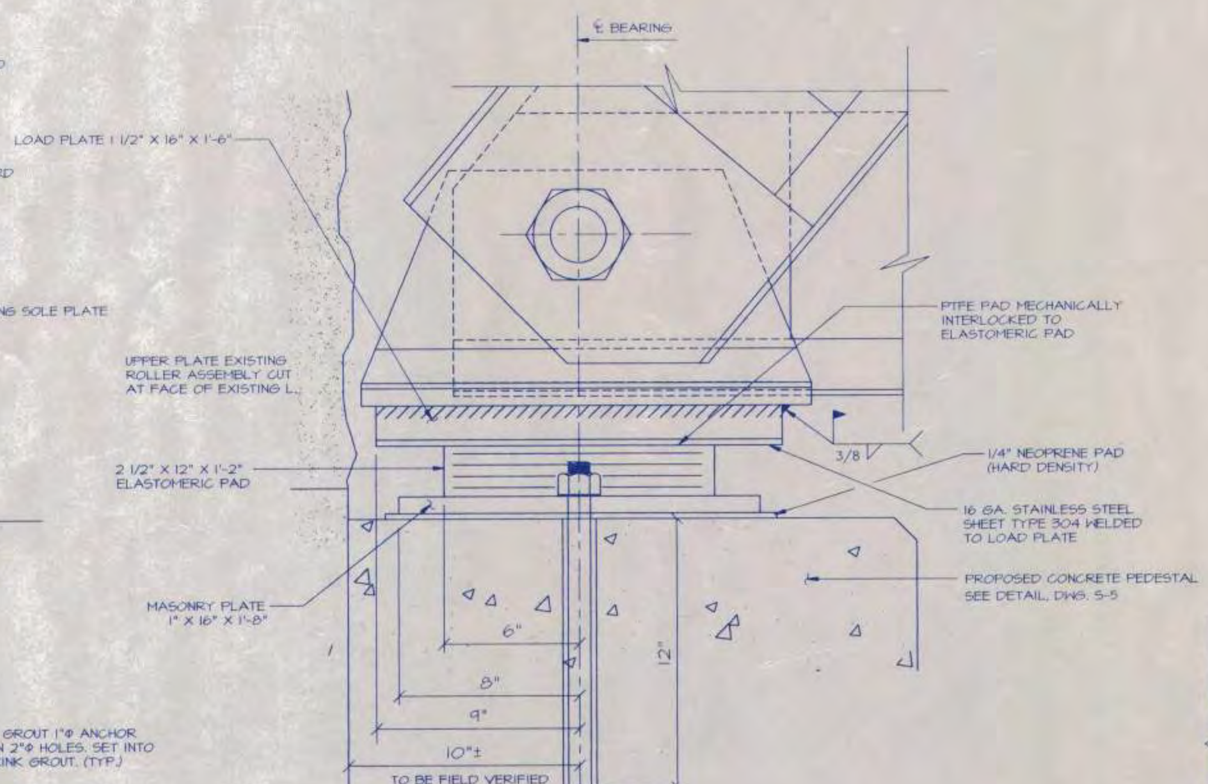


JACKING BEAM • WEST BEARING
SCALE: 1 1/2" = 1'-0"

NOTE:
JACKING FORCE EQUALS APPROXIMATELY 20 TONS

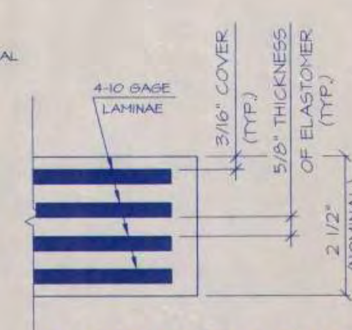


SECTION



ELEVATION

PROPOSED EXPANSION BEARING
SCALE: 3" = 1'-0"



ELASTOMERIC PAD DETAIL
N.T.S.

SEQUENCE OF CONSTRUCTION

1. REMOVE TIMBER DECKING AND JOISTS. REMOVE WINGWALL RAIL AT WEST END OF BRIDGE
2. WELD JACKING TUBE TO TRUSS, PLACE PLATE ON WEST ABUTMENT BACKWALL AND JACK WEST END OF BRIDGE. JACKING BOTH TRUSSES SIMULTANEOUSLY. SEE "JACKING EXISTING TRUSSES" IN SPECIAL PROVISIONS.
3. REMOVE EXISTING ROLLER ASSEMBLY
4. REMOVE TOP COURSE OF STONE MASONRY UNDER BEARING AND REPLACE WITH CONCRETE
5. LOWER TRUSS ONTO NEW BEARINGS AND REMOVE JACKING BEAM
6. PLACE TIMBER DECKING

BEARING NOTES:

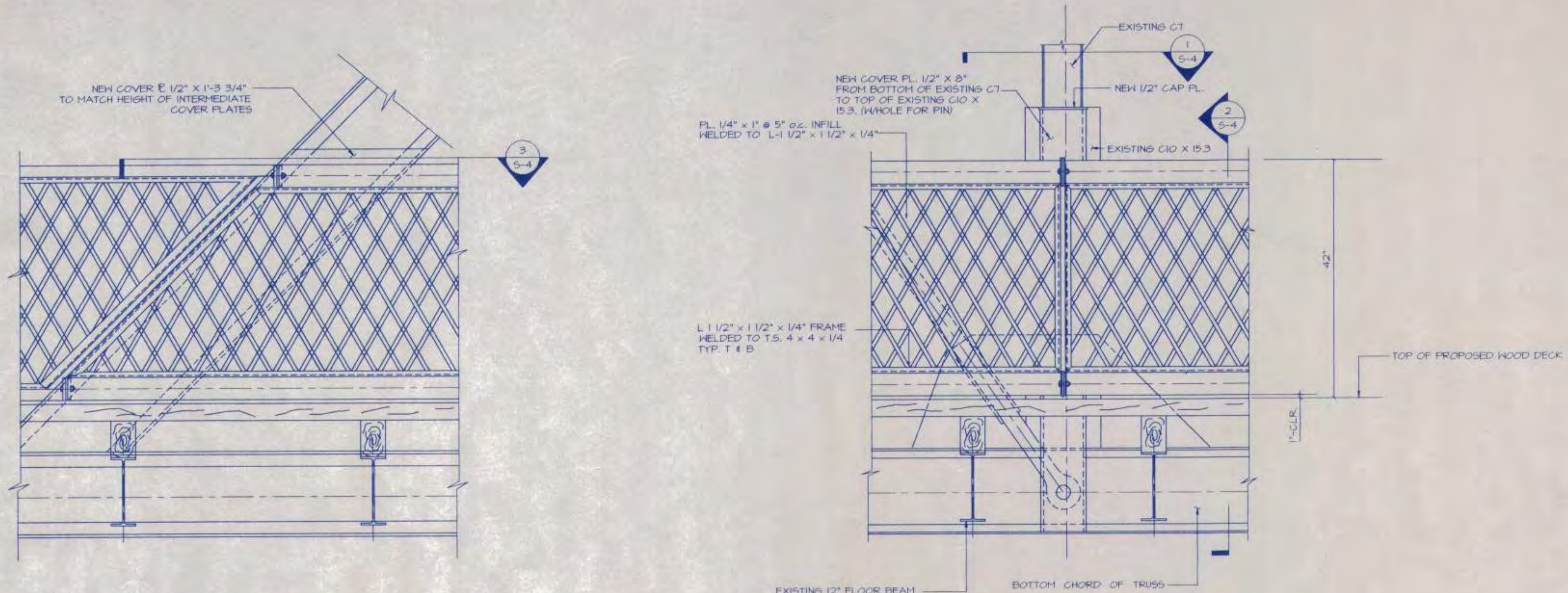
1. THE STEEL FOR NEW BEARING ASSEMBLIES SHALL CONFORM TO ASTM A-709, GR. 50W. THE STEEL LAMINAE FOR ELASTOMERIC PADS SHALL CONFORM TO ASTM I-36, OR AN APPROVED EQUAL.
2. THE ELASTOMER BEARING PADS SHALL CONFORM TO ASTM D-4014, TYPE CR, GR.3. IT SHALL HAVE A SHORE 'A' DUROMETER HARDNESS OF 60, AND A SHEAR MODULUS WITHIN THE RANGE OF 170 psi TO 200 psi.
3. THE EXPANSION BEARINGS SHALL BE INSTALLED WHEN THE AMBIENT TEMPERATURE IS IN THE RANGE OF 40°F TO 80°F, AND HAS BEEN IN THIS RANGE FOR AT LEAST TWO HOURS.

EXPANSION BEARING DETAILS

**REHABILATATION OF OLD BRIDGE ROAD BRIDGE
SIMSBURY, CONNECTICUT**

Macchi Engineers
44 Gillett Street
Hartford, CT 06105
Phone: (203) 549-6190

DRAWN BY: J.Z.	Drawing No.
APPROVED BY: J.B.	S-3
DATE: 8/01/94	
SCALE: AS NOTED	



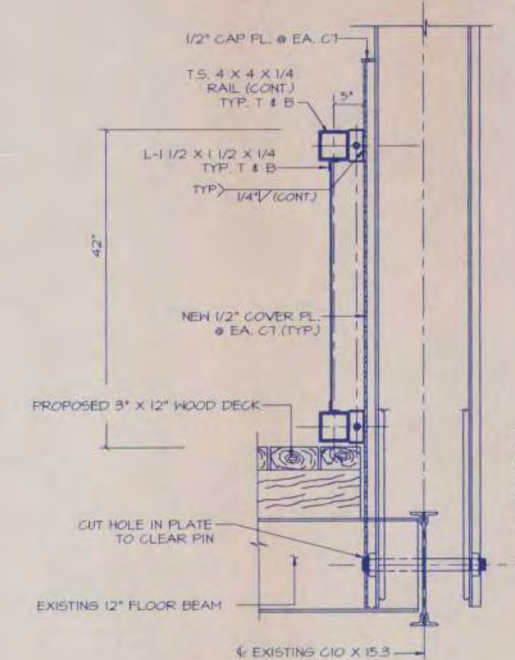
EAST END

PANEL POINTS 3 THRU 11

(PANEL POINTS 2 & 12 SIMILAR - SEE SECTION 1A/S-4)

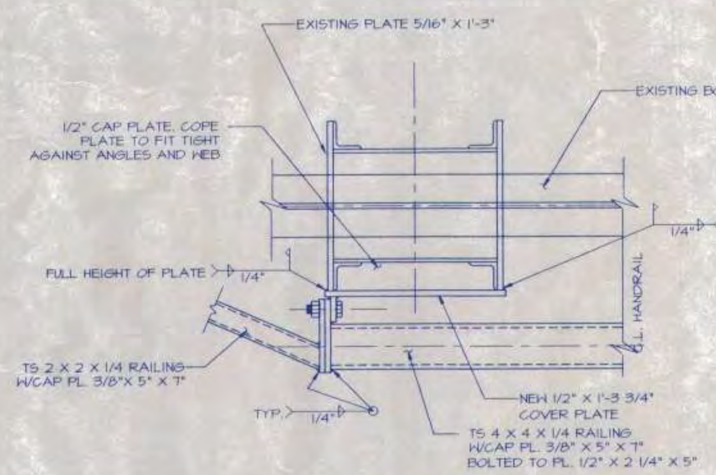
BRIDGE RAIL ELEVATION

SCALE: 1" = 1'-0"



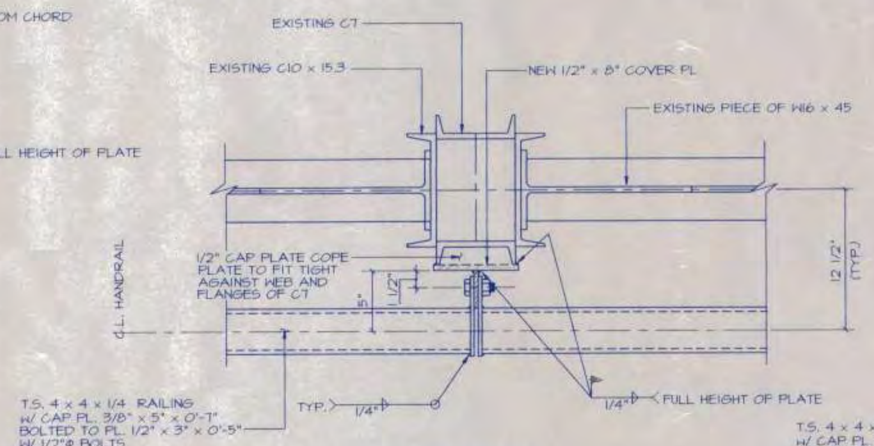
SECTION 2 S-4

SCALE: 1" = 1'-0"



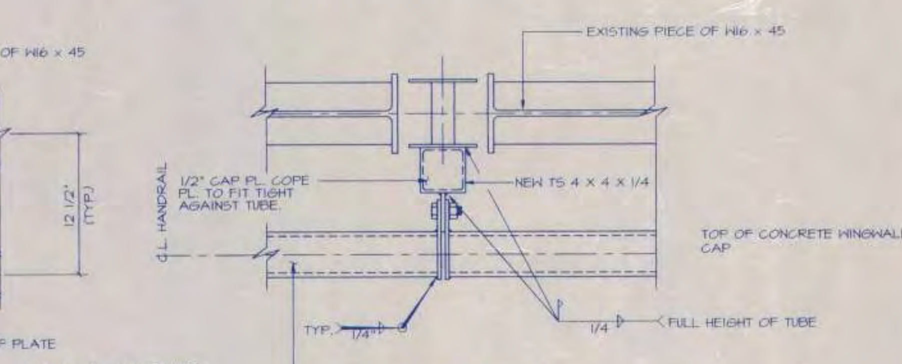
SECTION 3 S-4

SCALE: 1 1/2" = 1'-0"



SECTION 1 S-4

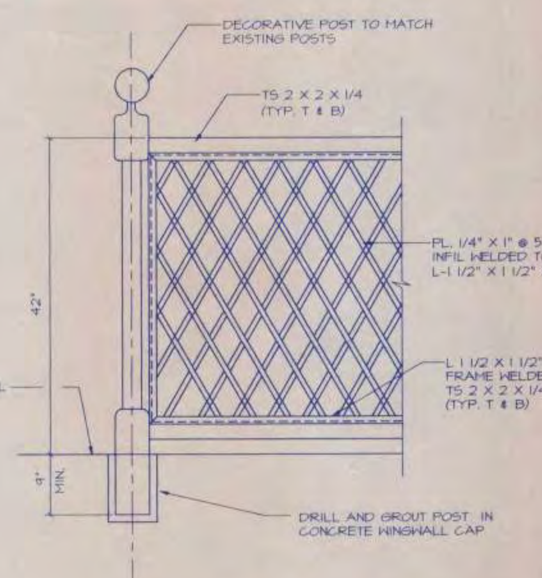
SCALE: 1 1/2" = 1'-0"



SECTION 1A S-4

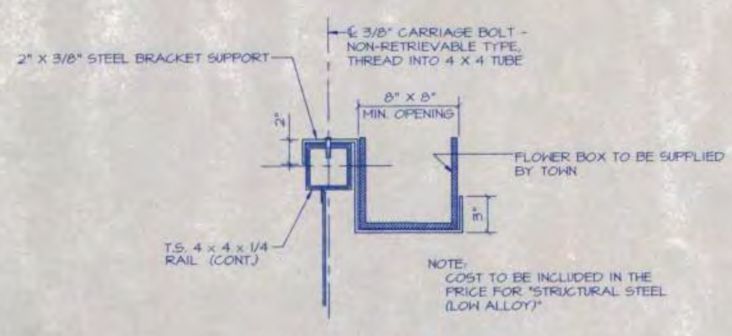
SCALE: 1 1/2" = 1'-0"

(AT PANEL POINTS 2 & 12)



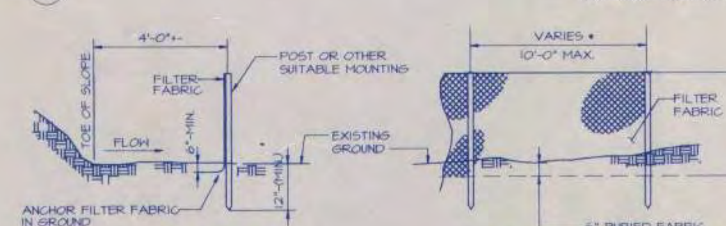
WINGWALL RAIL DETAIL

SCALE: 1" = 1'-0"



PLANTER SUPPORT

(2 PER PLANTER)
N.T.S.



SECTION

ELEVATION

NOTE:
• POST SPACING AND EMBEDMENT VARIES BASED ON THE MANUFACTURERS REQUIREMENTS.

SILT FENCE

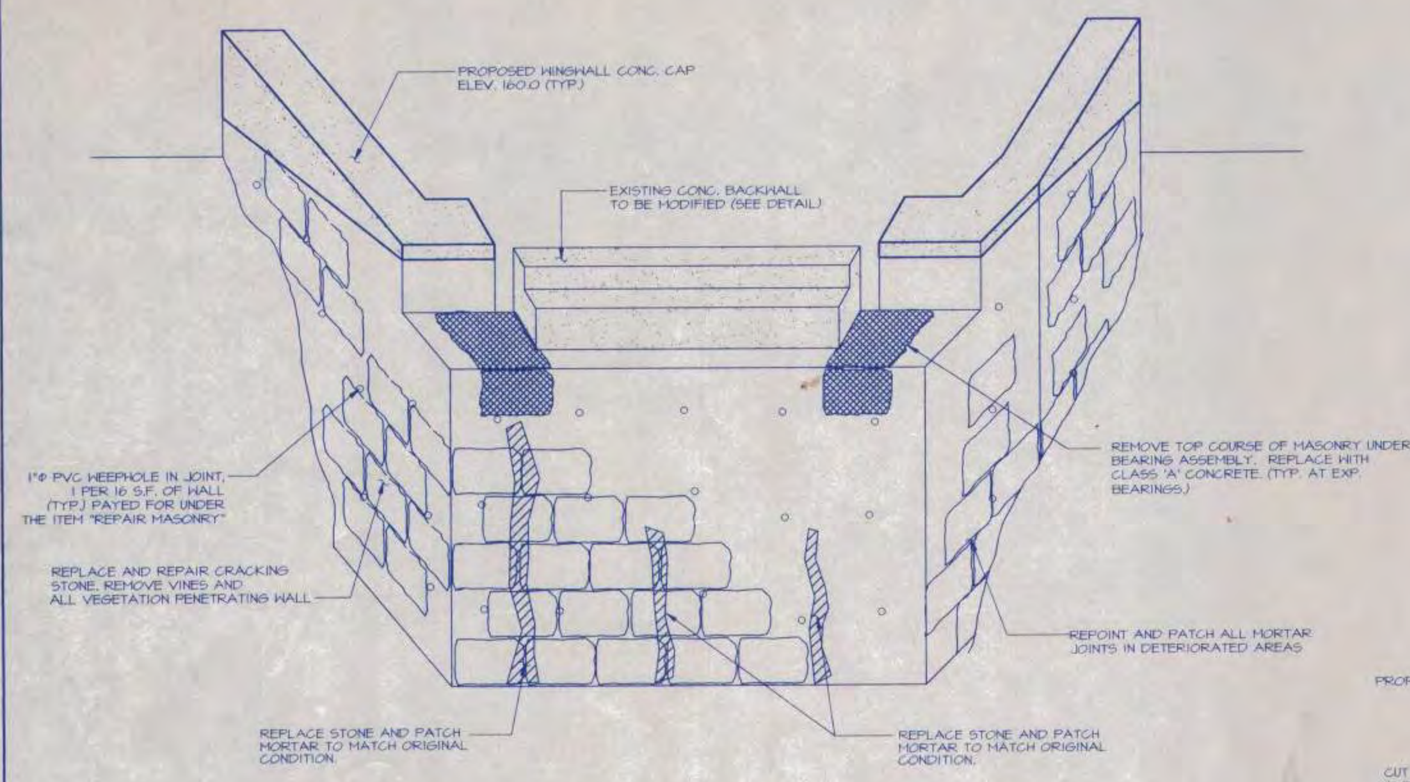
(LIGHT DUTY)

PROTECTIVE FENCE DETAILS

REHABILATATION OF OLD BRIDGE ROAD BRIDGE
SIMSBURY, CONNECTICUT

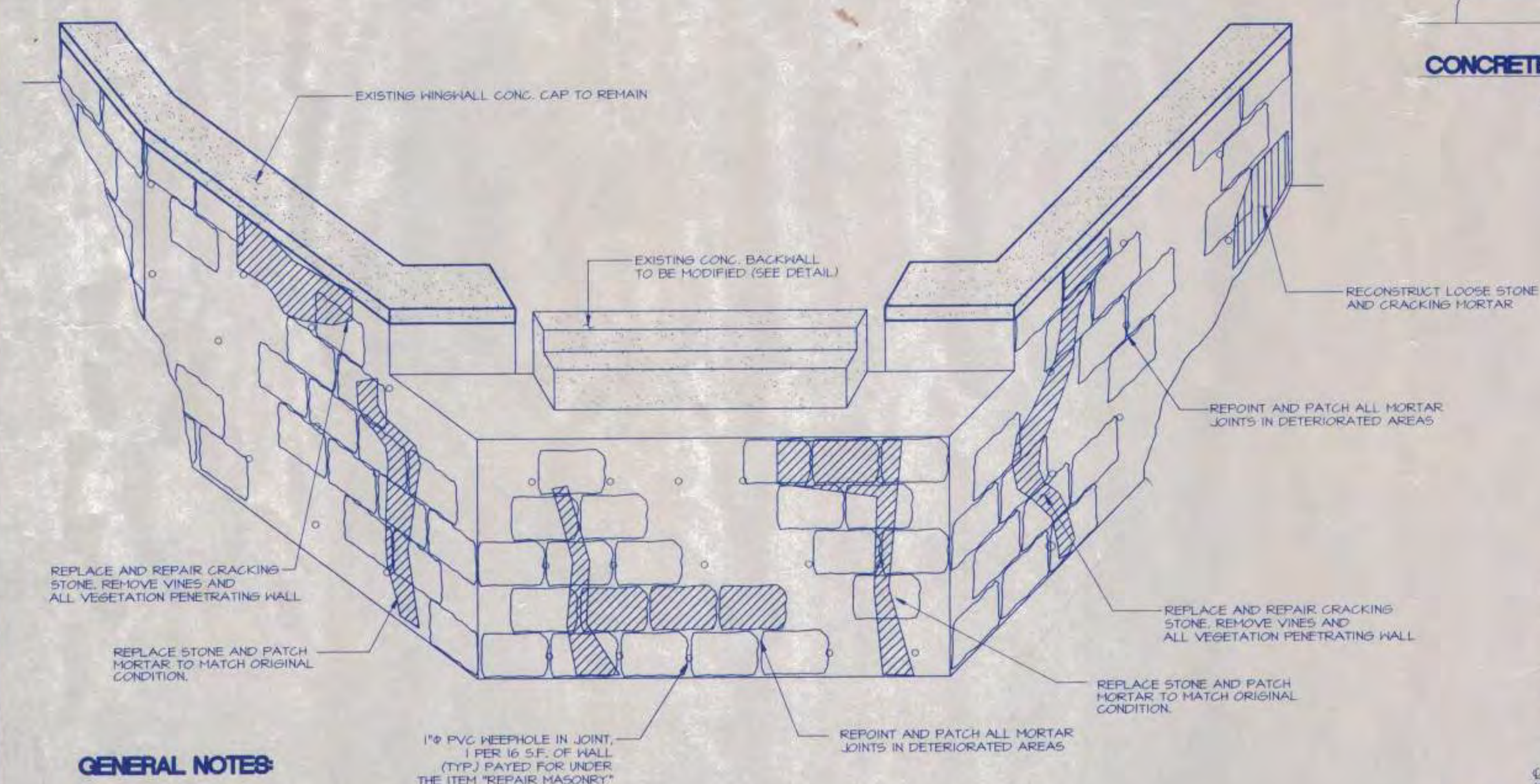
Macchi Engineers
44 Gillett Street
Hartford, CT 06105
Phone: (203) 549-6190

DRAWN BY: J.Z.	Drawing No.	
APPROVED BY: J.B.	S-4	
DATE: 8/01/94		
SCALE: AS NOTED		



WEST ABUTMENT AND WINGWALLS

SCALE: 1/4" = 1'-0"

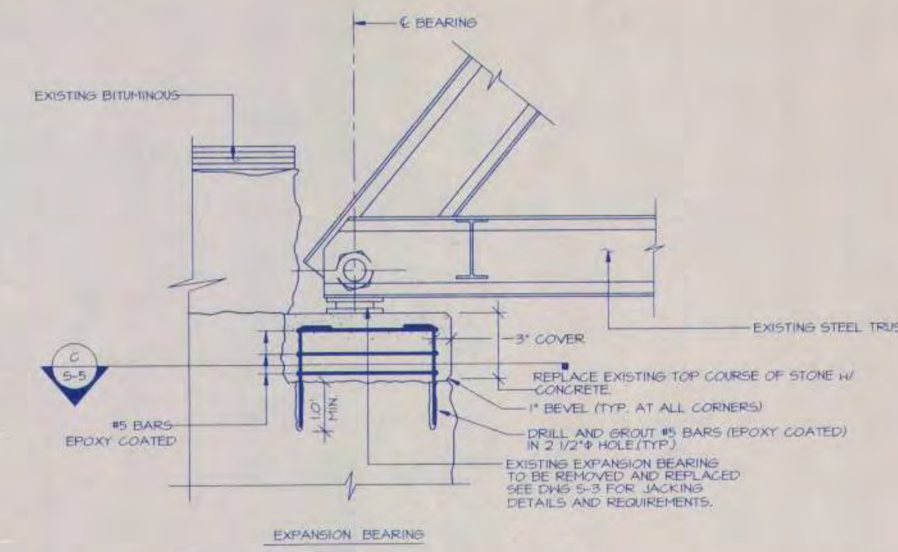


EAST ABUTMENT AND WINGWALLS

SCALE: 1/4" = 1'-0"

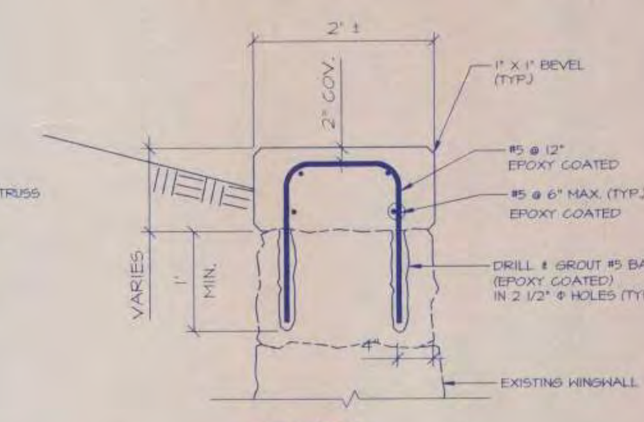
GENERAL NOTES:

- APPROXIMATE AREAS OF REPAIR
- EXISTING MASONRY TO BE REMOVED



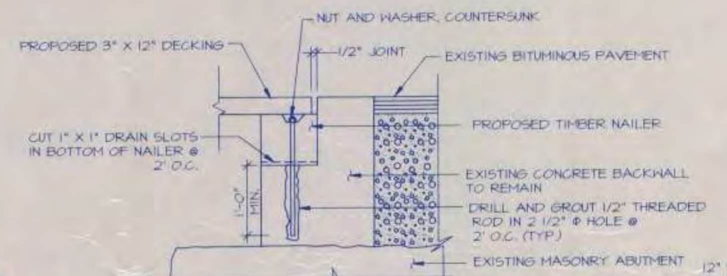
SECTION

N.T.S. (A / S-5)



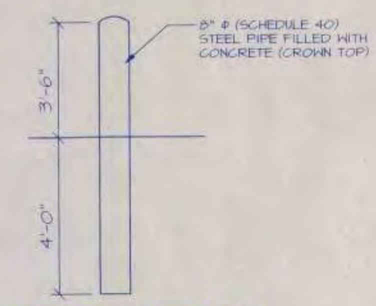
SECTION

SCALE: 1"=1'-0" (B / S-5)



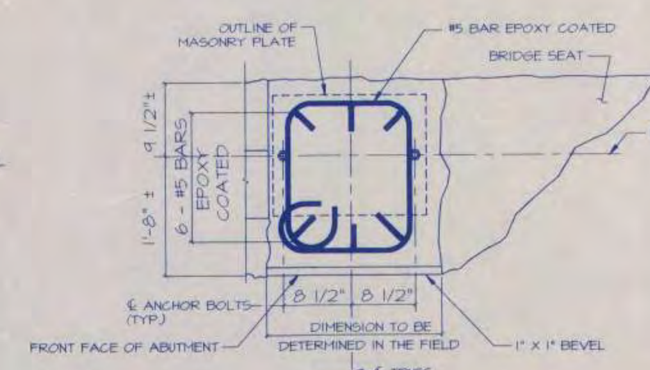
CONCRETE BACKWALL MODIFICATIONS

SCALE: 3/4"=1'-0"
(TYPICAL AT BOTH ABUTMENTS)



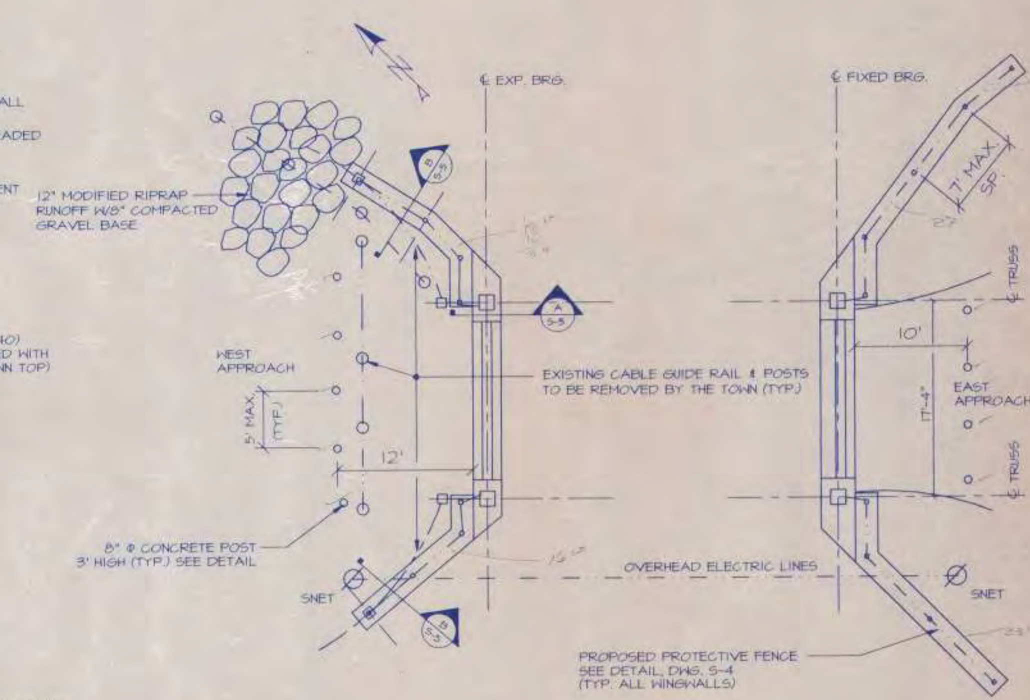
CONCRETE POST DETAIL

SCALE: 1/2"=1'-0"



SECTION

SCALE: 1" = 1'-0" (C / S-5)



ABUTMENT PLAN

SCALE: 1/8" = 1'-0"

ABUTMENT DETAILS

REHABILATATION OF OLD BRIDGE ROAD BRIDGE SIMSBURY, CONNECTICUT

Macchi Engineers 44 Gillett Street Hartford, CT 06105 Phone: (203) 549-6190	DRAWN BY: J.Z. APPROVED BY: J.B. DATE: 8/01/94 SCALE: AS NOTED	Drawing No. S-5

APPENDIX A: 2017 BRIDGE INSPECTION REPORT

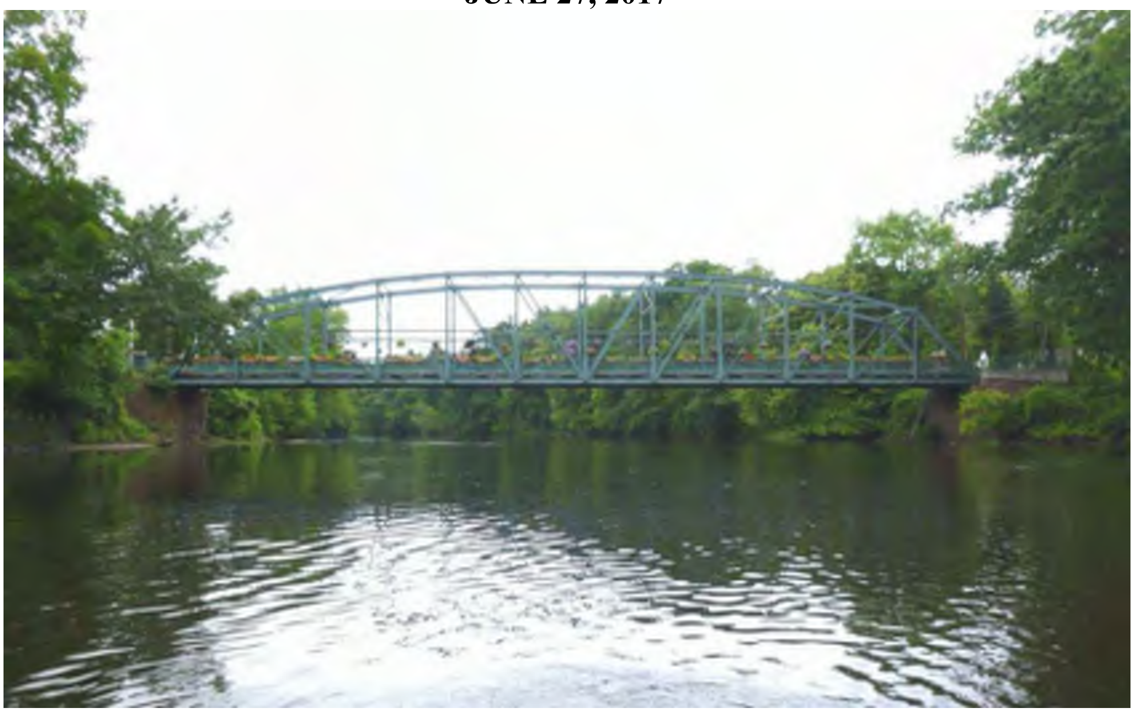
BRIDGE SAFETY INSPECTION

BRIDGE NO. 03984

OLD DRAKE HILL ROAD BRIDGE (FLOWER BRIDGE)
OVER
FARMINGTON RIVER

SIMSBURY, CONNECTICUT

JUNE 27, 2017



Digitally signed by Faisal Aziz
DN: E=faziz@gm2inc.com, CN=Faisal Aziz, O="GM2 Associates, Inc.", L=Glastonbury, S=CT, C=US
Contact Info: 8606591416 x132
Date: 2017.08.09 14:25:16-04'00'

Prepared By:

GM2 ASSOCIATES

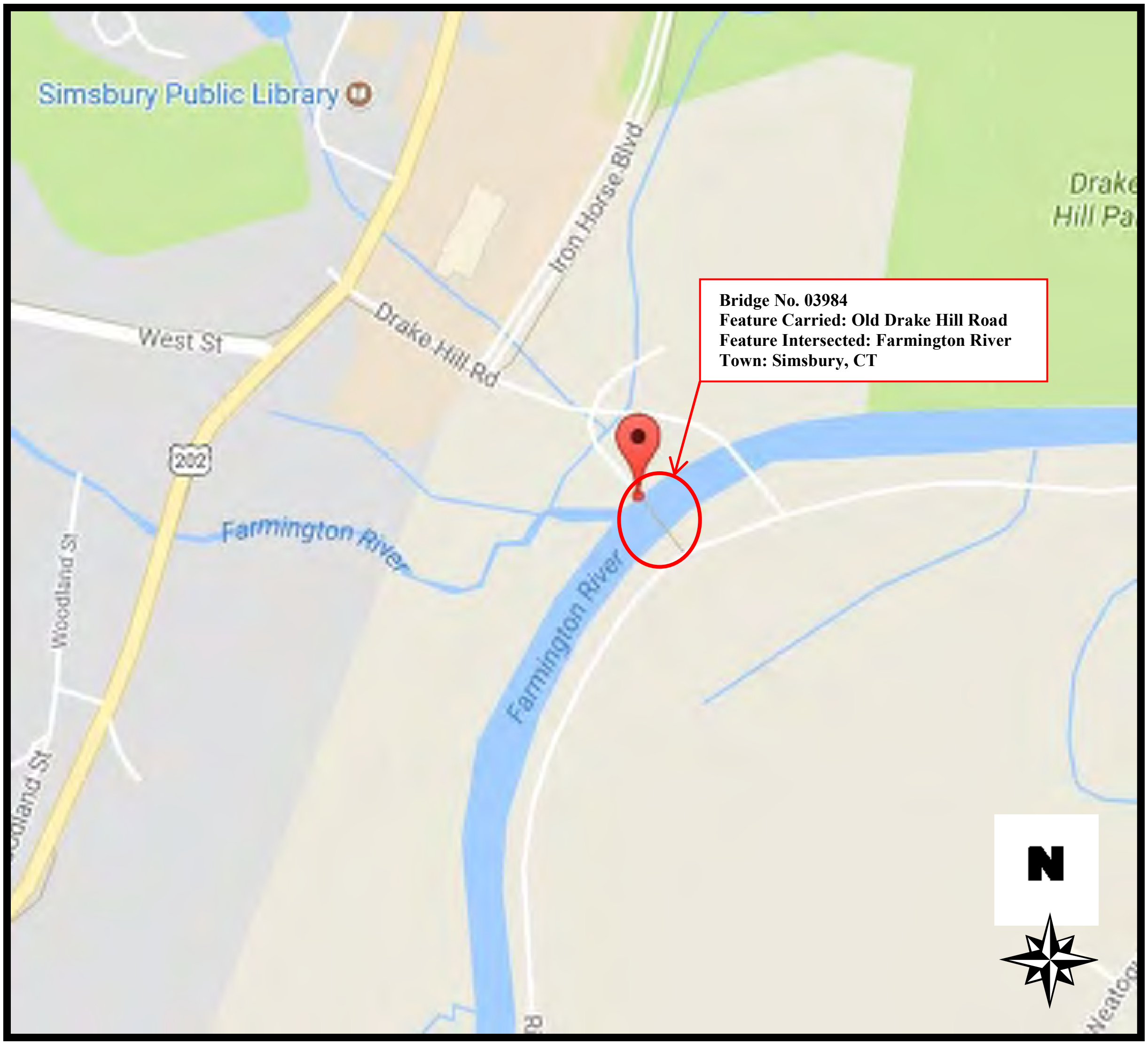
115 GLASTONBURY BLVD.
GLASTONBURY, CT 06033

Prepared For:

TOWN OF SIMSBURY
DEPARTMENT OF ENGINEERING
SIMSBURY, CT

BRIDGE SAFETY INSPECTION
TOWN OF SIMSBURY, CT

LOCATION MAP



TOWN OF SIMSBURY, CT
DEPARTMENT OF ENGINEERING

INSPECTION REPORT TRANSMITTAL FORM

STRUCTURE NO. 03984 TOWN SIMSBURY

Inspectors AKC, BJS, SR Date 06/27/2017

TABLE OF CONTENTS

EXECUTIVE SUMMARY

Bridge No. 03984 carries Old Drake Hill Road Bridge (Flower Bridge) over Farmington River in Simsbury, Connecticut. The overall length of the structure is 183 feet and curb to curb width is 15 feet. This steel thru-truss bridge structure is comprised of two Parker trusses and was built in 1892, with structural repairs performed in 1977, and further rehabilitated in 1993 for pedestrian traffic. The repairs and rehabilitation encompassed weldment of the gusset plates atop the bottom chord member, addition of channel sections to the truss vertical members, steel plates weldment to the truss diagonal members and gusset plates along with new timber deck planks installation. Currently, the bridge is closed to any vehicular traffic; and is open to carry pedestrian and bicycle traffic only.

During this in-depth inspection, completed in June 2017, the footbridge was found to be in “fair” condition. Also, all accessible truss pins were checked for deficiencies, utilizing Ultrasonic Testing (UT), and found to be in “acceptable” condition.

The structure is listed on the National Register of Historic Places in Connecticut; signifying it being a vital asset to the community, and dictating the need to preserve its historic character.

The deficiencies found on the bridge are as follows:

Deck: (Rated – 6 "Satisfactory")

No major deficiencies.

Superstructure: (Rated – 5 "Fair")

1. The vertical gusset plates at the truss bearings exhibit section loss down to 1/8" remaining with rust holes up to 1" x 1/4". In addition, the expansion bearing for the north truss at West Abutment is missing a vertical gusset plate.
2. The truss bottom chords exhibit section loss down to 1/16" remaining with edge rust holes, primarily in the bottom interior angles. The maximum resulting section loss in the bottom chord is approximately 5% (critical zone).
3. There are areas of pack rust up to 1/2" thick between the truss elements at random locations.
4. The channel web of truss vertical members exhibit areas of painted over pitting up to 1/16" deep with up to 1/2" x 3/8" rust holes (less than 5% section loss).
5. Channel webs of truss diagonal members exhibit random rust holes up to 4" diameter, primarily around the bolted tie-rod attachment between the channels (up to 16% section loss in the diagonal member; and 32% section loss in the channels).
6. Isolated locations in the sway bracing exhibit section loss up to full width x 6" long x down to knife edge remaining with up to 1" wide x 1/2" long rust holes.

Note: A condition assessment of the superstructure, in compliance with CTDOT Bridge Inspection Manual and National Bridge Inspection Standards, warrants an overall condition rating of “4 – Poor” or lower. However, a “5 – Fair” condition rating has been assigned due to the structure’s classification as a pedestrian facility only (no vehicular traffic permitted).

<u>Substructure:</u> (Rated – 7 "Good")

No major deficiencies.

<u>Channel and Channel Protection:</u> (Rated – 6 "Satisfactory")

No major deficiencies.

<u>Recommendations:</u>

Based on the extent of deterioration observed on the superstructure steel during this footbridge safety inspection, performed in June 2017, a reanalysis of the structure is recommended to ascertain its safe load capacity and evaluate feasibility of its possible reopening to any vehicular traffic, including the maintenance vehicles.

GM2 also recommends programming this footbridge for rehabilitation, including zone painting, to preserve its historic character and maximize its useful service life.

STATE OF CONNECTICUT
DEPARTMENT OF TRANSPORTATION
BRIDGE SAFETY & EVALUATION

STRUCTURE EVALUATION

SHEET 1 OF 2 FORM BRI-19 REV 10/00

SHEET 1 OF 2 (INSP. REPORT)

Bridge Number: 03984

Inspected By: AMIT KC, BRIAN SWANSON & SAIPAVAN RALLABHANDHI

Sufficiency Rating

Previous Inspection Date: 05/01/1988

BS&E Received ☐ Data Entry By:

Copies Made ☐ Data Entry Date:

90) Inspection Date: 0 6 2 7 1 7 | Inspection Team | 91) Frequency | Class:

Indepth Insp | Deck Survey | Access | Flagman

Access/Frequency: 5 0

RED FLAG

CRITICAL FEATURE INSPECTIONS

	Type	Frequency	Team	Date
Fracture:				
Uwater:				
Special:				

IDENTIFICATION

Bridge Name: OLD DRAKE HILL FLOWER BRIDGE

Town Name: SIMSBURY — Town Code: 128

5) Inventory Route:
- A) Record Type: 1
- B) Signing Prefix: 8
- C) Level of Service: 0
- D) Route Number: 00000
- E) Directional Suffix: 0

6) Feature Intersected: FARMINGTON RIVER

7) Facility Carried: OLD DRAKE HILL ROAD BRIDGE

9) Location: 0.3 M E OF ROUTE 202

11) Milepoint: 0000000 Miles

16) Latitude: 41 deg 52 min 08 sec

17) Longitude: -72 deg 47 min 59.8 sec

98) Border Bridge:
- A) State Code
- B) Percent Responsibility %
- C) Border Town Name

99) Border Bridge Structure No

STRUCTURE TYPE AND MATERIAL

43) Structure Type, Main:
- A) Material: 3
- B) Design Type: 10

44) Structure Type, Approach:
- A) Material: 0
- B) Design Type: 00

45) Number of Spans, Main Unit: 001

46) Number of Approach Spans: 0000

107) Deck Structure Type: 8

108) Wearing Surface/Protective System:
- A) Type of Wearing Surface: 0
- B) Type of Membrane: 0
- C) Type of Deck Protection: 0

AGE AND SERVICE

27) Year Built: 1892 — 106) Year Reconstructed: 1993

42) Type of Service:
- A) On: 3
- B) Under: 5

28) Number of Lanes:
- A) On: 01
- B) Under: 00

29) Average Daily Traffic: N/A — Half ADT?: N/A

109) Percent Truck: N/A %

30) Year of ADT: N/A

19) Bypass, Detour Length: N/A miles

GEOMETRIC DATA

48) Length of Max Span: 183 ft

49) Structure Length: 185 ft

50) Curb or Sidewalk Widths:
- A) Left: 000 ft
- B) Right: 000 ft

51) Brg Rdwy width,curb-curb: 15.2 ft

52) Deck Width, Out-Out: 18.5 ft

32) Approach Roadway Width: N/A ft

33) Bridge Median: 0

Deck Area: 3422 sqft

34) Skew Angle: 0 deg

35) Structure Flared: 0

10)Inv. Rte. Min. Vert Clearance: 20 ft 0 in

47) Log Inv. Rte. Total Horiz Clr.: 15.2 ft

47) RLog Inv. Rte. Total Horiz. Clr.: 0 ft

53) Min Vert Clearance Over Bridge: ft in

54) Min Vert Under Clearance: N Ref 0000 ft in Ref

55) Min Lat Under Clearance on Right: N Ref 0000 ft Ref

56) Min Lat Under Clearance on Left: 000 ft

BRIDGE COMMENTS

ITEM 29 - PEDESTRIAN BRIDGE

STRUCTURE EVALUATION

SHEET 2 OF 2 FORM BRI-19 REV 10/00

SHEET 2 OF 2 (INSP. REPORT)

Bridge Number	03984
NBIS Length	
Town Name	SIMSBURY
Facility Carried	OLD DRAKE HILL ROAD BRIDGE
Feature Crossed	FARMINGTON RIVER

Inspected By: AMIT KC, BRIAN SWANSON & SAIPAVAN RALLABHANDHI

CLASSIFICATION

112) NBIS Bridge Length	
104) Highway System	
26) Functional Class	
100) Defense Highway	
101) Parallel Structure	
102) Direction of Traffic	
103) Temporary Structure	
110) Designated National Network	
20) Toll	
21) Maintain	03
22) Owner	03
Report Class	
37) Historical Significance	1

LOAD RATING AND POSTING

31) Design Load	
63) Operating Rating Type	
64) Operating Rating	.
65) Inventory Rating Type	
66) Inventory Rating	.
Evaluation Code	
Year of Evaluation	
70) Bridge Posting	
41) Structure Status	K

WATERWAY

DrainageBasinCode	
38) Navigation Control	
39) Navigation Vert Clr.	
40) Navigation Horiz Clr.	
116) Vert-Lift Brg Nav Min	
111) Pier Abutment Protection	

CONDITION

	Rating	By
58) Deck	6	AKC
59) Superstructure	5	AKC
60) Substructure	7	AKC
61) Channel & Chan. Protection	6	AKC
62) Culverts	N	AKC

APPRAISALS

	Rating	By
67) Structure Evaluation	5	FA
68) Deck Geometry	N	AKC
69) Under Clear Vert & Horiz	N	AKC
71) Waterway Adequacy	7	AKC
72) Approach Rdwy Alignment	N	AKC
113) Scour Critical		

Items 58 Thru 72 Checked By:

36) Traffic Safety Features:

A) Bridge Railings	N
B) Transitions	N
C) Approach Guardrail	N
D) Approach Guardrail End	N

PROPOSED IMPROVEMENTS

75A) Type of Work Proposed	
75B) Work Done By	
76) Length of Struct. Improvement	ft
94) Bridge Improvement Cost	$
95) Roadway Improvement Cost	$
96) Total Project Cost	$
97) Year of Improvement Cost Est.	
114) Future ADT	
115) Year Future ADT	
List No.	
Project No.	
Advertised	

POSTED SIGNS & UTILITIES

Other Posted Signs 1	
Other Posted Signs 2	
Actual P.L. Single Unit Truck	tons
Rec. P.L. Single Unit Truck	tons
Actual P.L. Semi-TrailerTruck	tons
Rec. P.L. Semi-TrailerTruck	tons
Rec. P.L. All Vehicles	tons
Actual P.L. 4Axle Truck	tons
Rec. P.L. 4Axle Truck	tons
Actual P.L. 3S2 Truck	tons
Rec. P.L. 3S2 Truck	tons
Actual P.L. All Vehicles	tons
Posted Vert Clearance On Bridge	ft in
Posted Vert UnderClearance	ft in
Posted Speed Limit	mph
Utility	

OTHER FEATURES

Fence Required	
Fence Present	
Fence Height	ft
Fence Type	
Fence Material	
Fence Top Type	
Barrel Ladder	
Stand Pipes	
Cat Walks	
Movable Inspection System	
Loose Concrete Checked?	

INSPECTION COMMENTS

Proposed Next Indepth Insp Year

Senior

Supervisor

REVIEWED BY: Faisal Aziz, P.E. Date 08/09/2017

Town of Simsbury
Bridge Inspection Report BRI-18

Bridge No. 03984 **Inspection Date: 06/27/17**

Inspection Type: In-depth	Previous Inspection Date: 1988
Inspection Performed By: AKC, BJS, SR	Feature Carried: Old Drake Hill Road Bridge
Town: Simsbury	Feature Crossed: Farmington River
Year Built: 1892	Main Material: Steel
Year Rehab:1993	Main Design: Parker Through Truss

58. DECK: Overall Rating: 6

	Rating	
Overlay	N	
Deck Str.-Condition	6	The top side of timber deck planks exhibit the following deficiencies: • Random deck planks with splits and checks open up to 1/2". • Random deck plank ends have sections which are broken and lifted up by up to 1/4" high. • Random deck planks with vertical misalignment up to 1/8" high and an isolated location with 1/2" high. • Random deck planks with gaps of up to 1/2" between the segments. • Isolated 7" x 7" x 1" deep area of timber rot with exposed screws near midspan. The underside of timber deck exhibits the following deficiencies: • Random deck planks with longitudinal splits and checks open up to 1/16". • Timber ties atop the floorbeams with longitudinal checks up to 1/16" wide. (See Sketch No. 2 and Photo Nos. 7 - 10)
Curbs	N	
Median	N	
Sidewalks	N	
Parapet	N	
Railing	7	There are metal bridge ornamental railings along both fascia of the bridge, which exhibit isolated areas of peeling paint with light to moderate rust. There are wooden plantation beds for flower pots attached to the outer face of the bridge railings with S-shaped brackets. There are also watering pipes along the railings for irrigating the flower beds. (See Sketch No. 2 and Photo Nos. 11 - 12)
Paint	7	Less than 5% of the painted railing surfaces are peeling with light to moderate rust.
Fence	N	
Drains	N	
Lighting Standard	N	
Utility Type/Size	7	There is an irrigation system in place for the flower beds. A horizontal channel section has been attached to the vertical members of both trusses to accommodate the flower bed irrigation system, which exhibit isolated areas of peeling paint.

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Bridge No. 03984 **Inspection Date: 06/27/17**

		(See Photo Nos. 11 - 12)
Construction Joints	N	
Expansion Joints	6	There is joint sealant material between the timber deck ends and concrete headers at both abutments, which exhibits the following deficiencies: • Deteriorating joint sealant material at random locations. • Minor accumulation of sand along the joints. (See Sketch No. 2 and Photo No. 13)

Approach Condition: Overall Rating: 6

	Rating	
Approach Slab	N	
Relief Joints	N	
Approach Guide Rail	7	There are metal railings at each corner of the bridge which exhibit isolated areas of peeling paint with light to moderate rust. (See Photo No. 14)
Approach Pavement	6	There are stone pavers in both approaches with the following deficiencies: • Minor cracks between the stone pavers. • Isolated depressed area up to 1' long x full width x 1" deep in the east approach. (See Sketch No. 2 and Photo No. 15)
Approach Embankment	N	

Traffic Safety Features:

	Rating	
Bridge Railings	N	Pedestrian bridge.
Transitions	N	Pedestrian bridge.
Approach Guardrails	N	Pedestrian bridge.
Approach Guardrail Ends	N	Pedestrian bridge.

59. Superstructure: Overall Rating: 5

	Rating	
Bearing Devices	4	There are expansion bearings at West Abutment with the following deficiencies: • Vertical gusset plates at the bearings exhibit heavy rust with section loss up to 2" high x 1/16" deep along the bottom. • The bearing for North Truss at West Abutment is missing a vertical gusset plate between the pin and truss members. • Pack rust up to 1/4" thick between the truss members, pin and vertical gusset plate. • Light to moderate accumulation of pack rust and timber debris atop the bearing plates.

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Bridge No. 03984 **Inspection Date: 06/27/17**

		There are fixed bearings at East Abutment with the following deficiencies: • Vertical gusset plate at the bearing exhibit section loss up to 11" long x full height x down to 1/8" remaining with rust holes up to 1" wide x 1/4" high. • Isolated locations with pack rust up to 1/4" thick between the truss members, pin and gusset plate. • Bearing for the North Truss is undermined for 9" long x 1" deep due to spall in the abutment stone, resulting in less than 5% loss of bearing area. • Light to moderate accumulation of pack rust and timber debris atop the bearing plates. (See Sketch Nos. 37 - 39 and Photo Nos. 16 - 18)
Stringers	N	
Girders	N	
Floor Beams	5	There are steel floorbeams (S12 x 31.8), which exhibit the following deficiencies: • Top flanges with up to full length x full width x down to 1/4" remaining section loss and isolated location with 3" long x 3/4" wide rust hole (less critical areas). • Floorbeam webs with up to 6" long x 2" high x 1/16" deep section loss along the bottom at isolated locations (original web thickness = 9/16"). • Bottom flanges with up to full length x full width x 1/16" deep painted over pitting. • Clip angles at the floorbeam bottom chord truss connection exhibit peeling paint with light to moderate rust. (See Sketch Nos. 3 - 10 and Photo Nos. 19 - 20)
Trusses-General	5	The steel superstructure is comprised of two Parker through trusses. The connections at the nodes along the bottom chord has been retrofitted in the past to address severe section losses in the diagonal strut and rod members, and bottom web and flanges of vertical strut members. The bottom chords consist of a built-up rivetted section, which exhibits the following deficiencies: • Areas of peeling paint with moderate to heavy rust, primarily at the interior truss nodes. • Areas of pitting up to 40" long x full width x down to 1/16" remaining, with up to 3" long x 1/4" wide rust holes in the interior bottom angle. The maximum resulting section loss in bottom chord area is approximately 5% (critical zone). • The bottom chord splice connections exhibit pack rust up to 1/2" thick between the bottom/top splice plates and bottom chord angles resulting in the sections bending up/down up to 1/2". The vertical members (2- C7 x 9.8) exhibit the following deficiencies: • Areas of painted over pitting up to 1/16" deep with up to 1/2" x 3/8" rust holes in

Town of Simsbury
Bridge Inspection Report BRI-18

Bridge No. 03984 **Inspection Date: 06/27/17**

		the channel web. • Vertical members at the lower nodes with severe section loss (up to 100%) in the channel webs and flanges (a previously noted condition). Connections have been previously retrofitted. There are diagonal strut members with channel sections (2- C6 x 8.2) between U4-L4 to U8-L8, which exhibit the following deficiencies: • Areas of severe section loss at the lower nodes (up to 100%) in the channel webs (a previously noted condition). Connections have been previously retrofitted. • Channel webs with areas of painted over pitting up to 1/16" deep. Random rust holes in the channel web up to 4" diameter, primarily around the bolted tie-rod attachments between the channels (up to 16% section loss in diagonal member; 32% of the channels). Additional plates have been welded previously at some severely deteriorated locations. There are diagonal eye bar/rod members between U1-L1 to U4-L4 and U8-L8 to U11-L11, which exhibit the following deficiencies: • Areas of severe corrosion at the lower nodes (up to 100%), primarily around the pins (a previously noted condition). Connections have been previously retrofitted. The top chord consists of built-up rivetted section, which exhibits the following deficiencies: • Random areas of peeling paint with light to moderate rust. • Upper truss nodes with pack rust up to 1/2" thick between the top connection plate and top angles of top chord resulting in the sections bending up/down up to 1/4". • Upper truss nodes with pack rust up to 1/4" thick between the connection plate and top chord members. • Upper truss nodes with top angles with up to 11" long x full width x down to knife edge remaining section loss with up to 3-1/2" long x full width rust holes in horizontal legs. • Upper truss nodes with bottom angles of top chords with 9" long x full width x knife edge remaining section loss with 7" long x 1-1/4" wide rust holes in horizontal legs. • Upper chord pins with up to 1/4" thick pack rust/gap between the chord member web and pin. • Random locations in upper chord members with bird nests at the nodes. (See Sketch Nos. 11 - 62 and Photo Nos. 21 - 34)
Trusses-Portal	7	There are steel portals at L1-U1 & L11-U11 chords, with the following deficiency: • Random areas of peeling paint with light rust. (See Sketch No. 65)

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Trusses-Bracing	5	The bottom lateral and diagonal bracing between the floorbeams exhibit the following deficiencies: • Random areas of peeling paint with light to moderate rust. • Isolated bolts are loose/flush with the nuts. • Isolated locations with missing bolts. • Gusset plates with peeling paint and light to moderate rust. The strut and sway bracing exhibits the following deficiencies: • Random areas of peeling paint with light to moderate rust. • Isolated locations in the top strut angle with up to 12" long x full width x down to knife edge remaining section loss with 1" wide x 1/2" long hole in the horizontal leg. • Isolated locations in the diagonal bracing member with up to full width x 6" long x 1/8" deep section loss with up to 1" diameter rust holes. • Isolated locations with gaps up to 3/8" between the diagonal, and top and bottom members of the lateral bracing system. (See Sketch Nos. 3 - 10 & 63 - 64 and Photo Nos. 9 - 10 & 35 - 39)
Paint	7	Less than 10% of the painted surfaces are peeling with light to moderate rust. See items above entitled "Bearing Devices", "Floor Beams", "Trusses-General", "Trusses-Portal" and "Trusses-Bracing".
Rust	4	See items above entitled "Bearing Devices", "Floor Beams", "Trusses-General", "Trusses-Portal" and "Trusses-Bracing".
Machinery Mov. Span	N	
Rivets and Bolts	6	The rivets in the structure exhibit the following deficiencies: • Random rivets with peeling paint and light to moderate rust. • Isolated rivet heads with up to 50% head loss. See item above entitled "Trusses-Bracing". (See Sketch No. 3 - 62 and Photo Nos. 18, 24 - 25, & 37 - 38)
Welds and Cracks	6	There are repair welds in the structure, which exhibit the following deficiencies: • A 2-1/2" long horizontal crack between the top chord and strut at node U1 north side of South Truss (non-critical zone). • Sloppy welds in the repair plates attached to the diagonal truss element. (See Sketch No. 40 and Photo No. 39)
Timber Decay	N	
Concrete Cracking	N	
Collision Damage	N	
Member Alignment	7	Diagonal member, L8-U9 at South Truss is slightly bent.

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Bridge No. 03984 **Inspection Date: 06/27/17**

		(See Sketch Nos. 11 – 12).
Deflect. Under Load	N	(N) Normal; (E) Excessive. Note: Bridge does not carry any vehicular traffic. Open for pedestrian traffic only.
Vibr. Under Load	N	(N) Normal; (E) Excessive. Note: Bridge does not carry any vehicular traffic. Open pedestrian traffic only.
Stand Pipes	N	
Barrel Ladders	N	

60. Substructure: **Overall Rating: 7**

	Rating	
Abutments-Stem	7	There are stone masonry abutment stems, which exhibit the following deficiencies: • Isolated stones with full height cracks open up to 1/16". • East Abutment Stem with isolated 18" long x 9" high x 6" deep spall in stone under the bearing for the North Truss which undermines the bearing up to 9" long x 1" deep. • Isolated stone in East Abutment with full height crack open up to 1/16" and 7" high x 2" wide x 2" deep chipped off. • Random voids in the joint mortar between the stones along the base of stem. • Hairline cracks with and without efflorescence in the mortar between the stones. • Heavy growth of vegetation atop the abutment seats at the bearings. (See Sketch Nos. 66 - 67 and Photo Nos. 40 - 42)
Abutments-Backwall	7	The top of backwalls are exposed along top of the timber deck interface. The west abutment backwall top has cracks up to 1' long x 1/2" wide. (See Sketch No. 2)
Abutments-Footings	N	Not visible.
Abutments-Settlement	8	None observed.
Abutments-Wingwalls	7	There are stone masonry wingwalls with concrete caps, which exhibit the following deficiencies: • Isolated stones with horizontal hairline cracks with efflorescence. • Random hairline cracks in the mortar between the stones. • Moderate to heavy growth of vegetation along the wingwalls. (See Sketch Nos. 68 - 69 and Photo Nos. 43 - 44)
Piers/Bents-Caps	N	
Piers/Bents-Pile Bent	N	
Piers/Bents-Columns	N	
Piers/Bents-Footings	N	
Piers/Bents-	N	

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Bridge No. 03984 **Inspection Date: 06/27/17**

Settlement		
Erosion-Scour	8	Erosion: Rated - '8'. Scour: Rated - '8'.
Concrete Crack-Spall	N	
Steel Corrosion	N	
Paint	N	
Timber Decay	N	
Collision Damage	N	
Debris	7	Light accumulation of timber debris atop the abutment seats.

61. Channel and Channel Protection — Overall Rating: 6

	Rating	
Channel Scour	8	The channel bottom consists of sand with small to medium size stones. (See Sketch No. 70 - 71 and Photo Nos. 45 - 48)
Embankment Erosion	6	Areas of erosion along the embankments up to 3' high x 3' deep with exposed tree roots. (See Sketch No. 70 and Photo Nos. 47 - 48)
Debris	N	
Vegetation	6	Heavy growth of vegetation along the channel embankments, some of which is overhanging the channel. Light to moderate growth of vegetation in the channel. (See Sketch No. 70 and Photo Nos. 45 - 48)
Channel Change	8	The channel flow is perpendicular to the bridge.
Fender System	N	
Spur Dikes & Jetties	N	
Rip Rap	7	Small to medium size riprap is in place along the embankment.

62. Culvert & Retaining Wall: — Overall Rating: N

	Rating	
Barrel	N	
Concrete	N	
Steel	N	
Timber	N	
Headwall	N	
Cutoff Wall	N	
Debris	N	
Retaining Wall System	N	
Footing	N	

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Bridge No. 03984

Inspection Date: 06/27/17

Load Posting: N

Miscellaneous:

Minimum Vertical Under Clearance:	The structure spans over a waterway.
Posted Clearance Under Bridge:	
Posted Clearance on Bridge:	
Advanced Warning:	
Speed Limit:	
Character of Traffic:	Pedestrian bridge.

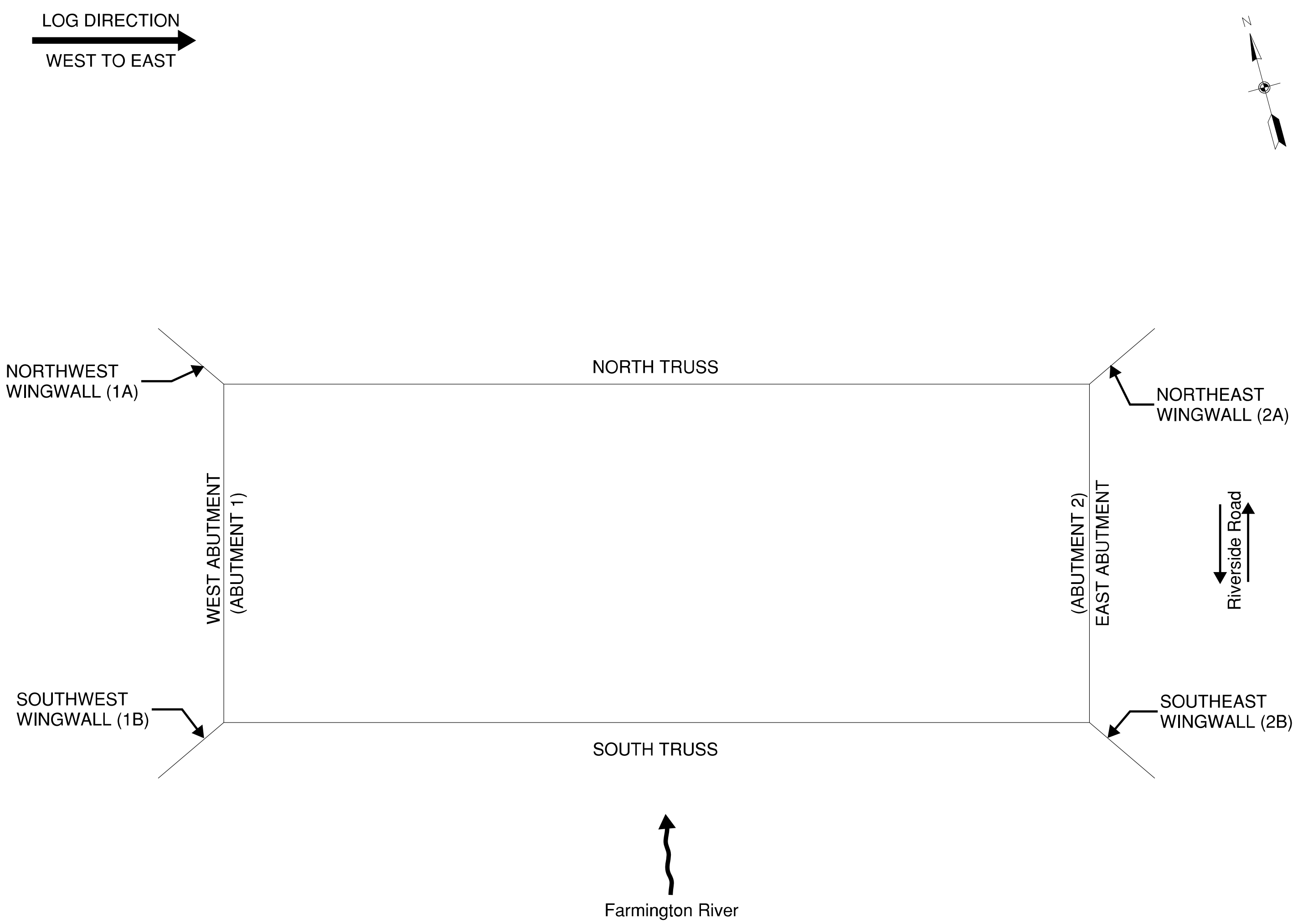
Additional Notes:

Additional Comments:

- Bridge is logged from west to east.
- Farmington River flows from south to north.
- Bridge was inspected using a rigging platform and an extension ladder.
- A safety boat was present during the inspection.

Inspectors' Signatures:	1) (AMIT KC)	Date: 08/09/2017
	2) (SAIPAVAN RALLABHANDHI)	Date: 08/09/2017
	3)	Date:
	4)	Date:
P.E. Signature:	(FAISAL AZIZ)	Date: 8/9/17
P.E. #:	29339	
Reviewed by:	Town of Simsbury	Date:

CREW: AKC, BJS, SR (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984



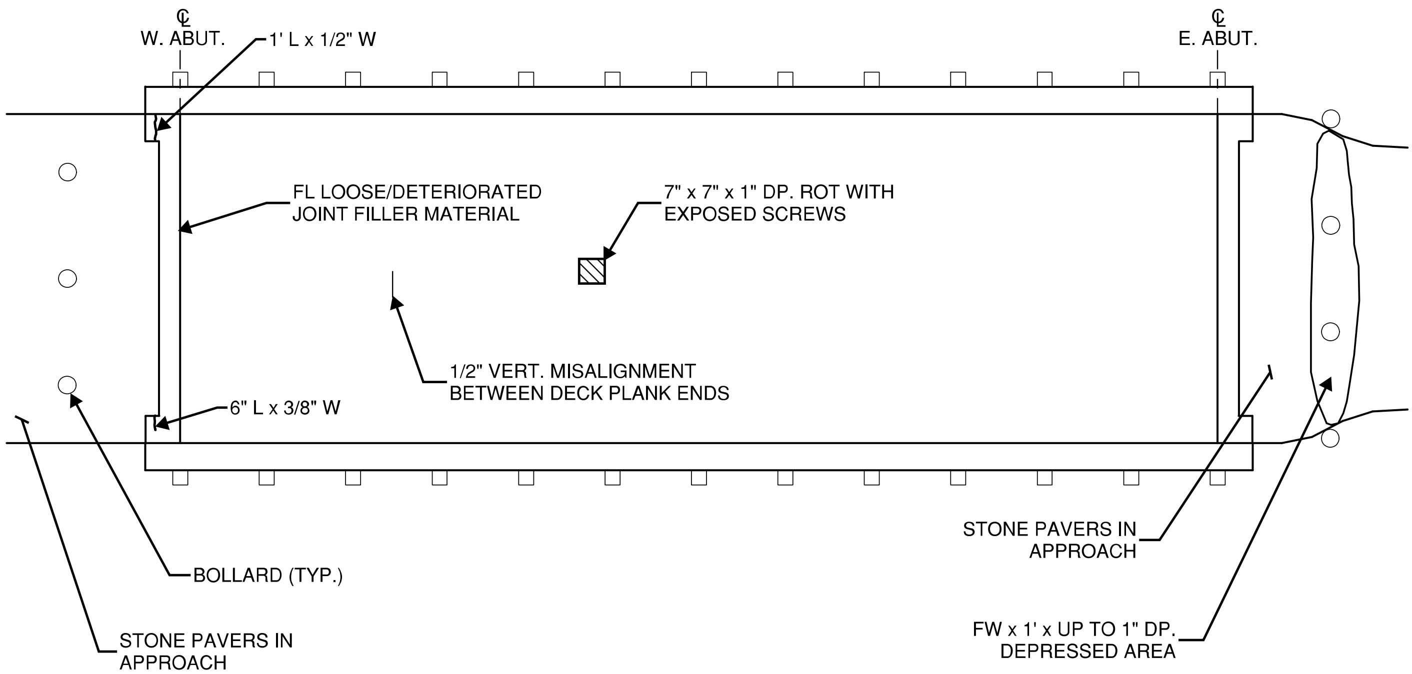
KEY PLAN

(N.T.S.)

(SKETCH 1)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- Top of timber deck planks with random splits and checks open up to 1/2"; random deck plank ends have sections which are broken and lifted up to 1/4" high.
- Timber deck planks are vertically misaligned up to 1/8"; gaps up to 1/2" between the planks.
- Joint sealant material is deteriorating at random locations.
- Metal bridge ornamental railing with random areas of peeling/chipped paint with light rust along the base.

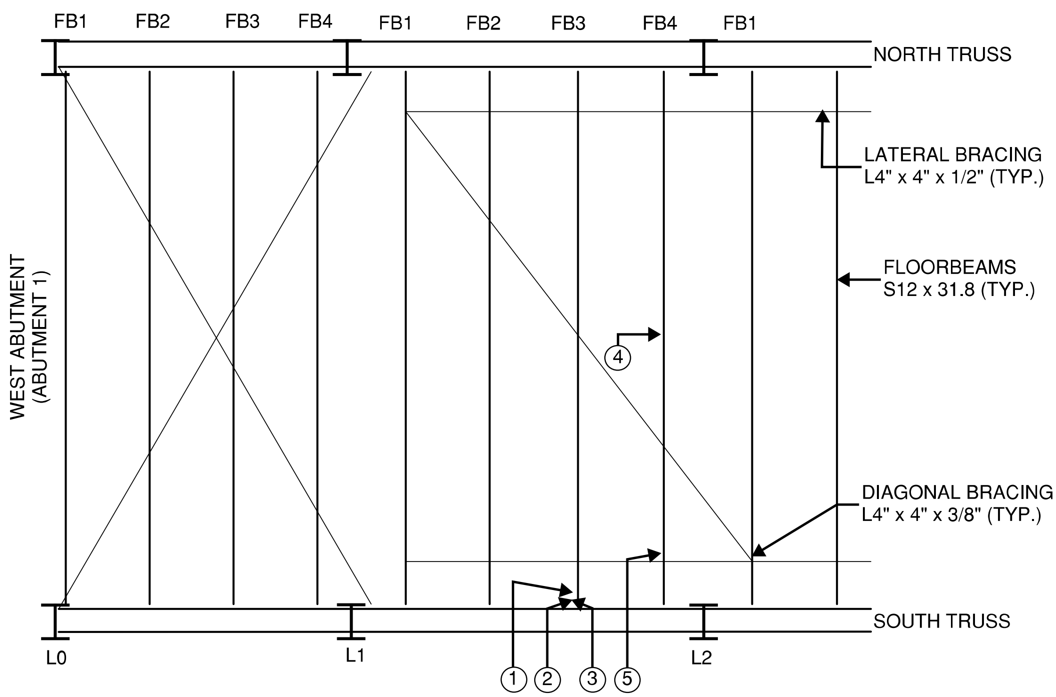
TOP OF DECK

(N.T.S.)

(SKETCH 2)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2) | DATE: 6/27/2017 | BRIDGE NO.: 03984



DETERIORATION NOTES:

- See "DETERIORATION NOTES - UNDERSIDE OF DECK AND FRAMING (L0 TO L2)" sheet.

GENERAL NOTES:

- See "GENERAL NOTES - UNDERSIDE OF DECK AND FRAMING" sheet.

UNDERSIDE OF DECK AND FRAMING (L0 TO L2)

(N.T.S.)

(SKETCH 3)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984

DETERIORATION NOTES - UNDERSIDE OF DECK & FRAMING (L0 TO L2):

① Floorbeam top flange with 6" long x 1" wide x down to 3/16" remaining.

② Floorbeam web bottom with 6" long x 3" high x 1/16" deep pitting on west side at the truss connection.

③ Floorbeam web bottom with 6" long x 2" high x 1/16" deep section loss on east side at the truss connection.

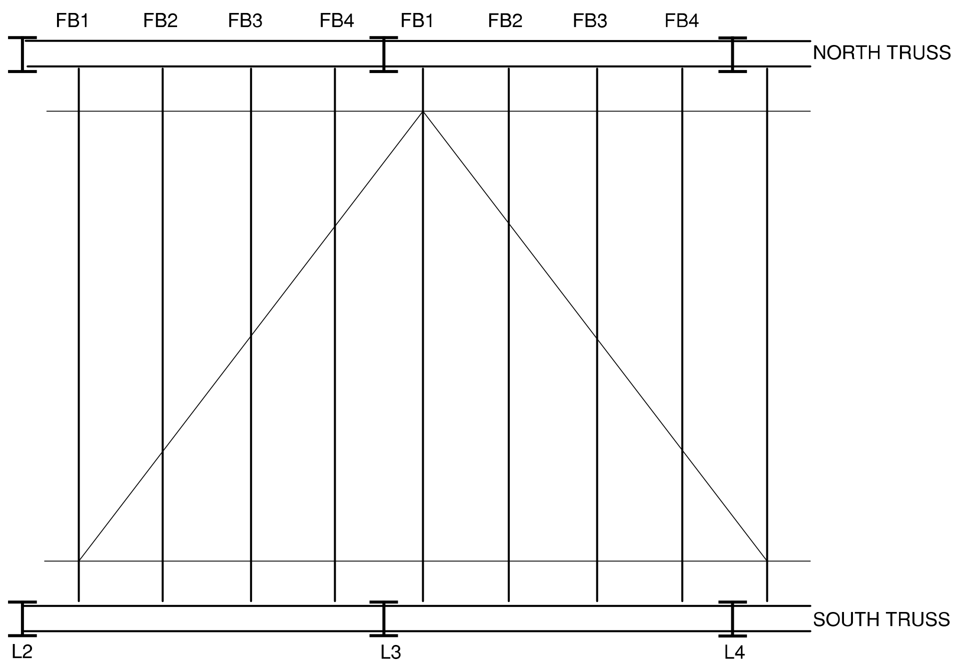
④ Floorbeam top flange with full length x full width x down to 1/4" remaining and bottom flange with full length x full width x 1/16" deep pitting.

⑤ Floorbeam top flange with 4' long x 2.5" x down to knife edge remaining section loss, starting at 10" from South Truss, with a 3" long x 3/4" wide rust hole at 1'± from the longitudinal bracing.

(SKETCH 4)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2) | DATE: 6/27/2017 | BRIDGE NO.: 03984



GENERAL NOTES:

- See "GENERAL NOTES - UNDERSIDE OF DECK AND FRAMING" sheet.

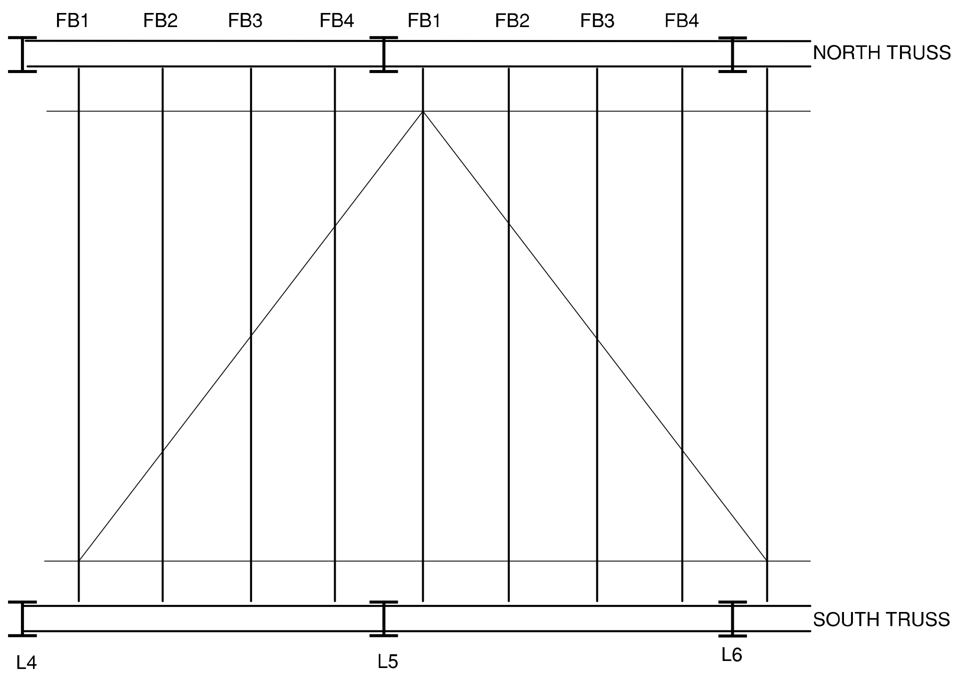
UNDERSIDE OF DECK AND FRAMING (L2 TO L4)

(N.T.S.)

(SKETCH 5)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- See "GENERAL NOTES - UNDERSIDE OF DECK AND FRAMING" sheet.

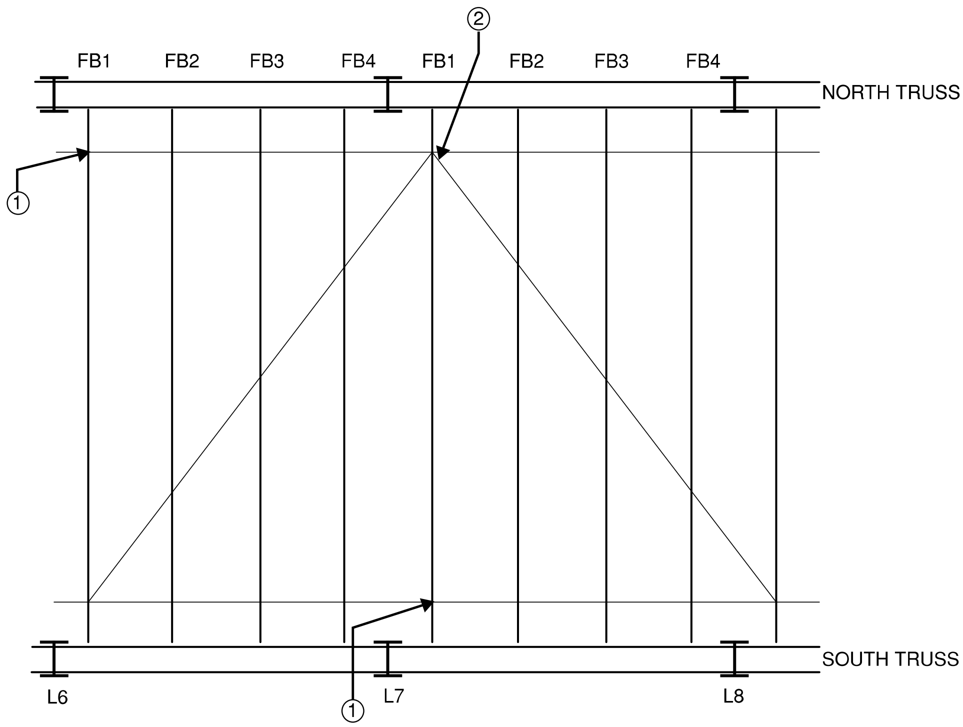
UNDERSIDE OF DECK AND FRAMING (L4 TO L6)

(N.T.S.)

(SKETCH 6)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984



DETERIORATION NOTES - UNDERSIDE OF DECK & FRAMING (L6 TO L8):

① (2) missing bolts between the bottom flange of floorbeam and longitudinal bracing.

② (1) loose bolt at horizontal gusset plate for lateral bracing; random short bolts.

GENERAL NOTES:

- See "GENERAL NOTES - UNDERSIDE OF DECK AND FRAMING" sheet.

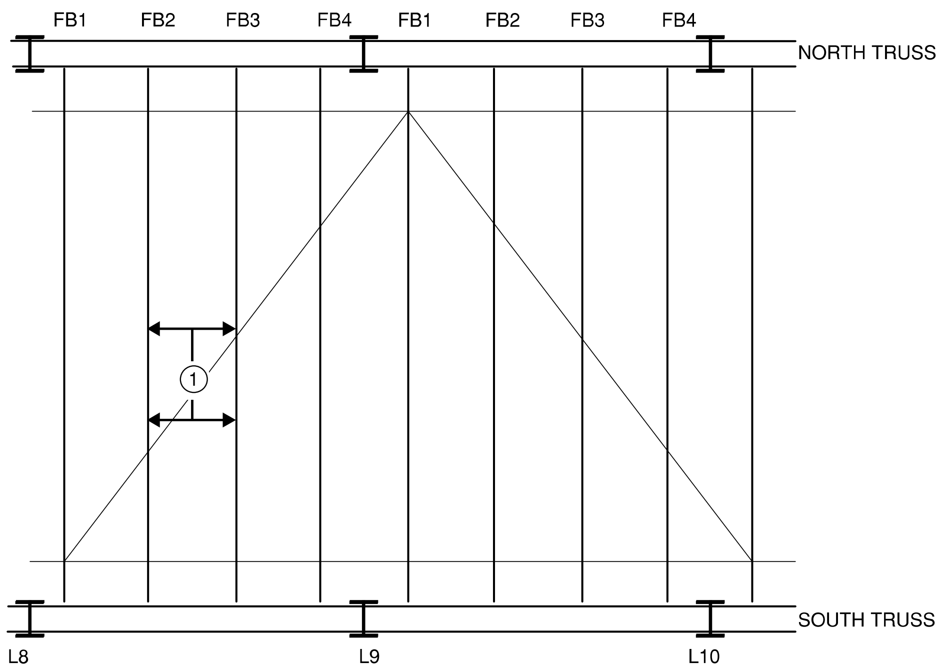
UNDERSIDE OF DECK AND FRAMING (L6 TO L8)

(N.T.S.)

(SKETCH 7)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2) | DATE: 6/27/2017 | BRIDGE NO.: 03984



DETERIORATION NOTES:

① Floorbeam bottom flanges with 1" diameter drilled holes.

GENERAL NOTES:

- See "GENERAL NOTES - UNDERSIDE OF DECK AND FRAMING" sheet.

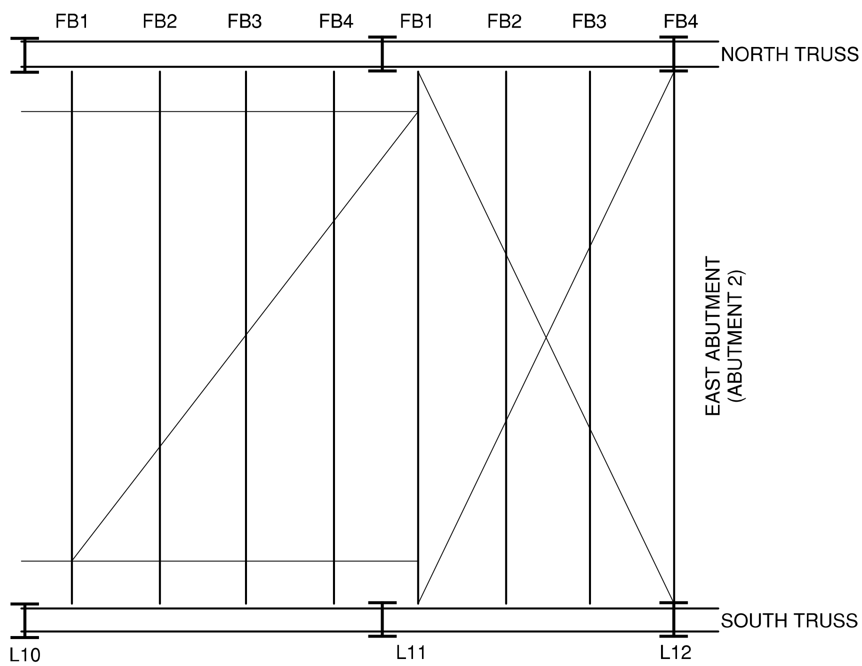
UNDERSIDE OF DECK AND FRAMING (L8 TO L10)

(N.T.S.)

(SKETCH 8)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- See "GENERAL NOTES - UNDERSIDE OF DECK AND FRAMING" sheet.

UNDERSIDE OF DECK AND FRAMING (L10 TO L12)

(N.T.S.)

(SKETCH 9)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: AKC, BJS (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984

GENERAL NOTES - UNDERSIDE OF DECK & FRAMING:

- Timber deck planks with random longitudinal checks open up to 1/16".

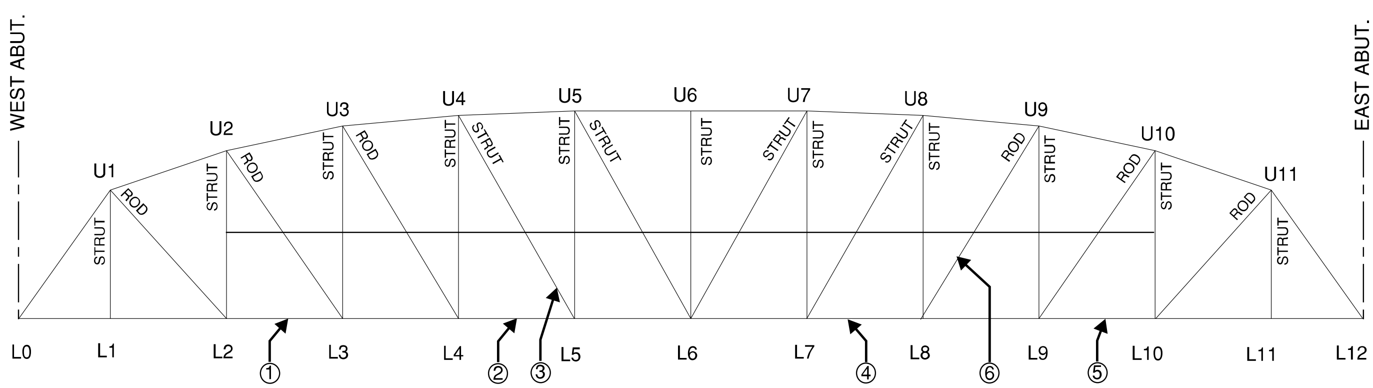
- Timber ties atop the floorbeams with longitudinal checks open up to 1/16".

- Clip angles between the bottom chord web and floorbeams with peeling paint and light rust.

(SKETCH 10)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



DETERIORATION NOTES:

- See "DETERIORATION NOTES - SOUTH TRUSS - SOUTH ELEVATION" sheet.

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.
- See "REPAIR PLATES" sheet for gusset plate dimensions.

SOUTH TRUSS - SOUTH ELEVATON

(N.T.S.)

(SKETCH 11)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

DETERIORATION NOTES - SOUTH TRUSS - SOUTH ELEVATION:

① Bottom chord splice connection between L2 & L3, splice plate bending out due to up to 1/2" thick pack rust.

② Bottom chord splice connection between L4 & L5, splice plate bending out due to up to 1/4" thick pack rust.

③ L5-U4 diagonal member with full width x 3/4" high x down to knife edge remaining with perforations up to 1" long x 1/2" wide. ✳

④ Bottom chord splice connection between L3 & L4, splice plate bending out due to up to 1/8" thick pack rust; missing rivet in the vertical leg of top angle.

⑤ Bottom chord splice connection between L9 & L10, splice plate bending out due to up to 1/4" thick pack rust.

⑥ Outside strut of the diagonal member L8-U9 is slightly bent..

NOTE:

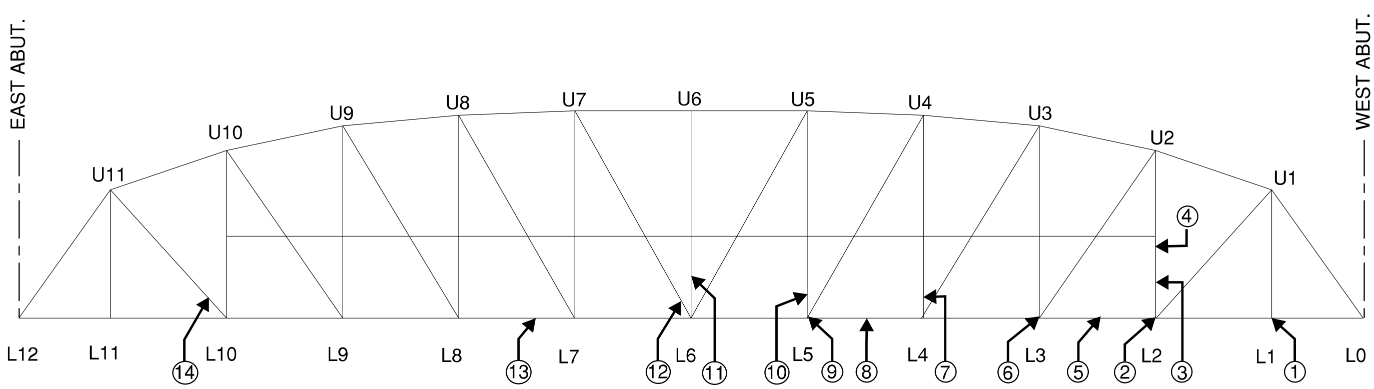
✳ Retrofit assembly in place to address the section losses.

- See "REPAIR PLATES" sheet for retrofit gusset plate dimensions.

(SKETCH 12)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984



DETERIORATION NOTES:

- See "DETERIORATION NOTES - SOUTH TRUSS - NORTH ELEVATION" sheet.

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.
- See "REPAIR PLATES" sheet for gusset plate dimensions.

SOUTH TRUSS - NORTH ELEVATON

(N.T.S.)

(SKETCH 13)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984

DETERIORATION NOTES - SOUTH TRUSS - NORTH ELEVATION:

① Bottom chord, interior bottom angle at L1 with 4' long x full width x down to 1/8" remaining pitting and a 3-1/4" long x1/4" wide rust hole in the horizontal leg (near L1); also at the same location, 4' long x full height x 3/16" deep pitting in vertical leg (approx. 5% section loss in overall chord area).

② Bottom chord, interior angle horizontal leg at L2 with 40" long x full width x down to 1/16" remaining (approx. 5% section loss in overall chord area) with a 3" long x 1/4" wide rust hole at the edge.

③ Vertical member U2-L2 channel web with 1/2" high x 1/4" wide rust hole at the welded repair channel. ✳

④ Vertical member U2-L2 with 1/16" diameter hole in the weld.

⑤ Bottom chord splice connection between L2 & L3, bottom splice plate bent down full width x 3/16" over 9" long due to pack rust; bottom angle horizontal leg with 2" long x 2" wide rust hole; web splice plate with 6" long x 2" high x 1/8" deep section loss at bottom; one rivet head at the bottom with heavy rust and 25% head loss.

⑥ Bottom interior angle, horizontal leg at L3 with 4' long x full width x down to 1/8" remaining (less than 5% loss in overall area).

⑦ Vertical member U4-L4 with a 3" x 1" x 1/8" deep section loss in flange with gap between the vertical member and welded repair channel. ✳

⑧ Bottom chord splice connection between L4 & L5, bottom splice plate is bent down 1/2" over 9" long due to pack rust.

⑨ L5-U4 diagonal member channel web with 1" high x full width x down to knife edge remaining section loss with random perforations.✳

⑩ Vertical member U5-L5 with 2" x 1/2" x 1/8" deep section loss in flange with gap between vertical member and welded channel. ✳

⑪ Vertical member U6-L6 with 2" x 1/2" x 1/8" deep section loss in flange with gap between vertical member and welded channel on both sides. ✳

⑫ L6-U7 diagonal member with 6" x 6" x 1/16" deep pitting at the pin connection. ✳

⑬ Bottom chord splice connection between L7 & L8, web splice plate bent for 6" long x 3/16" due to pack rust;

⑭ L10-U11 diagonal member with 1" diameter x 1/8" deep section loss at the pin connection. ✳

NOTE:

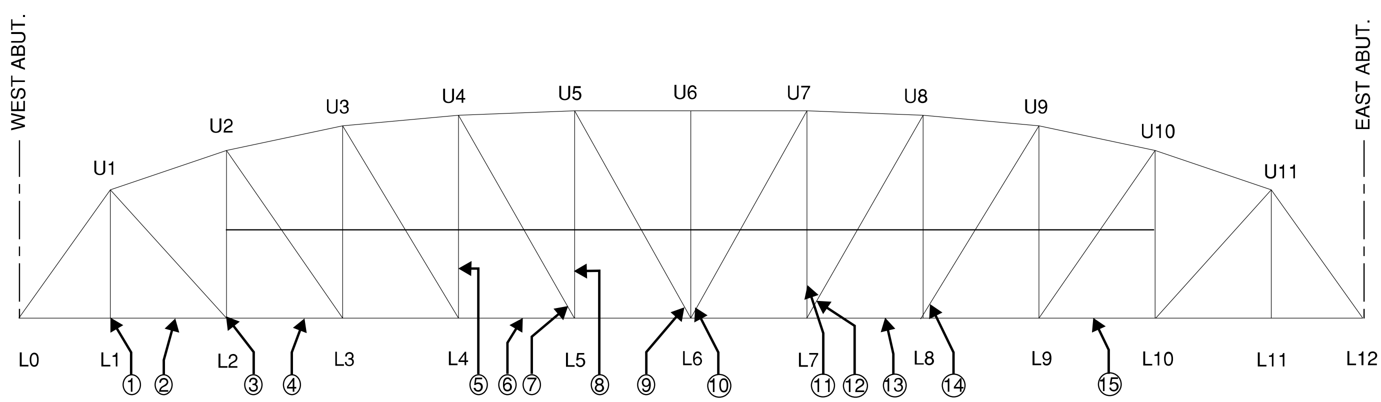
✳ Retrofit assembly in place to address the section losses.

- See "REPAIR PLATES" sheet for retrofit gusset plate dimensions.

(SKETCH 14)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2) | DATE: 6/27/2017 | BRIDGE NO.: 03984



DETERIORATION NOTES:
- See "DETERIORATION NOTES - NORTH TRUSS - SOUTH ELEVATION" sheet.

GENERAL NOTES:
- See "GENERAL NOTES - BOTTOM CHORD" sheet.
- See "REPAIR PLATES" sheet for gusset plate dimensions.

NORTH TRUSS - SOUTH ELEVATON

(N.T.S.)

(SKETCH 15)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/27/2017	BRIDGE NO.: 03984

DETERIORATION NOTES - NORTH TRUSS - SOUTH ELEVATION:

① Bottom chord interior bottom angle at L1 with 3' long x full width x 1/8" remaining pitting (painted over) in horizontal leg (1.5' on each side of L1) and 3' long x full height x 3/16" deep pitting in the vertical leg (approx. 5% section loss in overall chord area).

② Bottom chord interior angle of L1-L2 horizontal leg member with 3' long x full width x 1/8" deep pitting in the vertical and horizontal legs (approx. 5% section loss in overall chord area).

③ Bottom chord interior angle horizontal leg at L2 with 4' long x full width x down to 1/8" remaining (under L2) with a 3" long x 1" wide rust hole (approx. 5% section loss in overall chord area).

④ Bottom chord splice connection between L2 & L3, Splice plate with 1' long x 1" high x up to 1/8" deep section loss (one rivet with 50% head loss); bottom angle horizontal leg with 4" long x full width x up to 1/4" deep section loss and bent out 1/4" due to pack rust; up to 1/2" gap between the top splice plate and top angles of bottom chord due to pack rust.

⑤ Vertical member U4-L4 with full width x 1" high x 1/8" deep section loss in flange with gap between the vertical member and welded repair channel. ✳

⑥ Bottom chord splice connection between L4 & L5, bottom splice plate is bent down 1/4" over 6" long due to pack rust.

⑦ L5-U4 diagonal member channel web with 2" long x 1" high x down to knife edge remaining with a 3/8" diameter rust hole & 4" diameter rust hole and 1/8" thick pack rust between the connection plate and channel web. ✳

⑧ Vertical member U5-L5 with 1-1/2" high x 1/2" wide x 3/16" deep section loss with gap between the vertical member and welded repair channel. ✳

⑨ L6-U5 diagonal member channel web with full width x 3/4" high x 1/4"± deep section loss above the rivet head plate. ✳

⑩ L6-U7 diagonal member with full width x 1" high x down to knife edge remaining with 3" long x 1" high rust hole. ✳

⑪ Vertical member U7-L7 with 1" high x 1/2" wide x 1/8" deep section loss with gap between the vertical member and welded repair channel. ✳

⑫ L7-U8 diagonal member with full width x 2" high x down to knife edge remaining and random perforations. ✳

⑬ Bottom chord splice connection between L7 & L8, web splice plate bent for 6" long x 1/2" due to pack rust;

⑭ L8-U9 diagonal member with 1" high x 1" wide x 1/8" deep section loss with gap between the vertical member and welded repair channel. ✳

⑮ Bottom chord splice connection between L9 & L10 with pack rust up to 1/2" thick between the splice plates and bottom chord angles; 3" long x 2" high x 3/16" deep section loss in the web splice plate along the bottom.

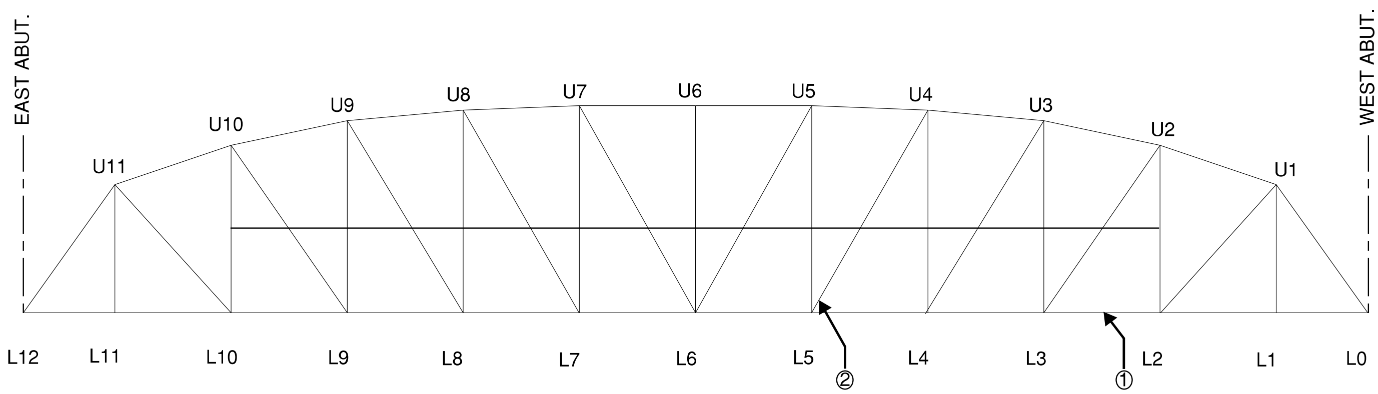
NOTE:

✳ Retrofit assembly in place to address the section losses (See "REPAIR PLATES" sheet for retrofit gusset plate dimensions).

(SKETCH 16)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



DETERIORATION NOTES:

- See "DETERIORATION NOTES - NORTH TRUSS - NORTH ELEVATION" sheet.

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.
- See "REPAIR PLATES" sheet for gusset plate dimensions.

NORTH TRUSS - NORTH ELEVATON

(N.T.S.)

(SKETCH 17)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

DETERIORATION NOTES - NORTH TRUSS - NORTH ELEVATION:

① Bottom chord splice connection between L2 & L3, web splice plate bent up to 1/2" due to pack rust for 6"± long at east edge.

② L5-U4 diagonal member with 1/8" thick pack rust and full width x 1" high x 3/16" deep section loss in channel web.*

NOTE:

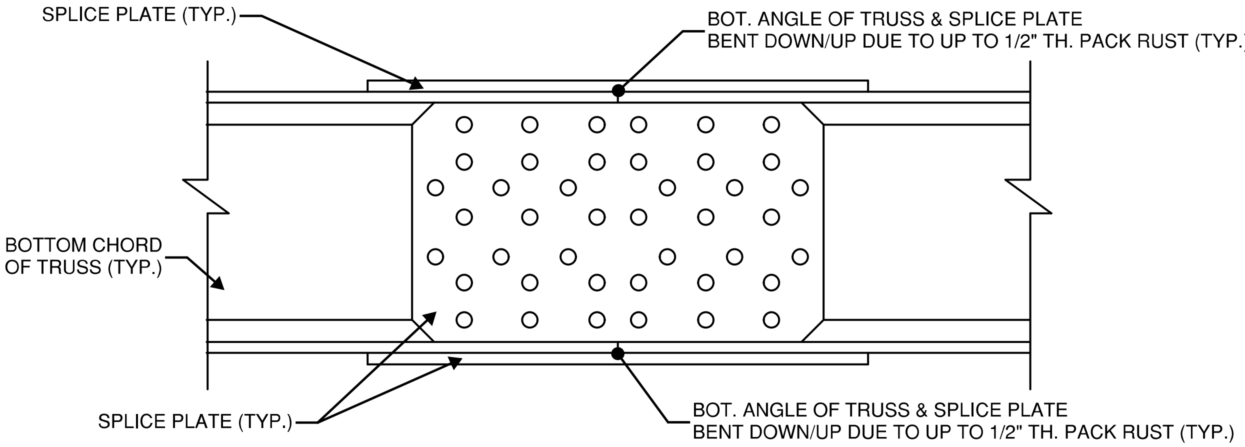
* Retrofit assembly in place to address the section losses.

- See "REPAIR PLATES" sheet for retrofit gusset plate dimensions.

(SKETCH 18)

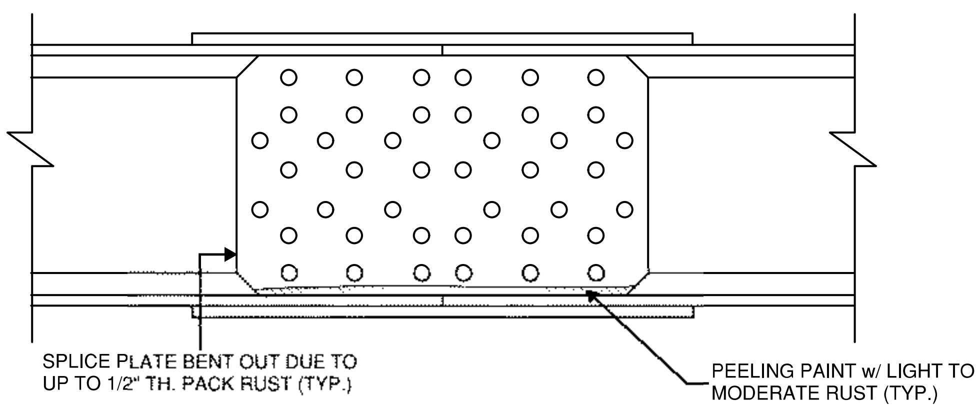
REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



BOTTOM CHORD SPLICE CONNECTION (TYP.) - SOUTH ELEVATION

(N.T.S.)



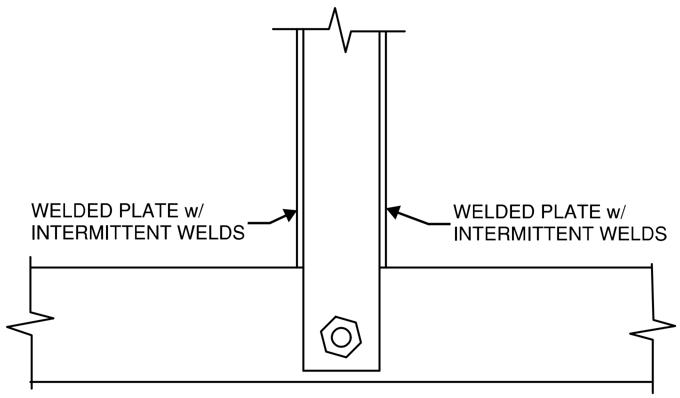
BOTTOM CHORD SPLICE CONNECTION (TYP.) - NORTH ELEVATION

(N.T.S.)

(SKETCH 19)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L1 SOUTH TRUSS SOUTH ELEVATION

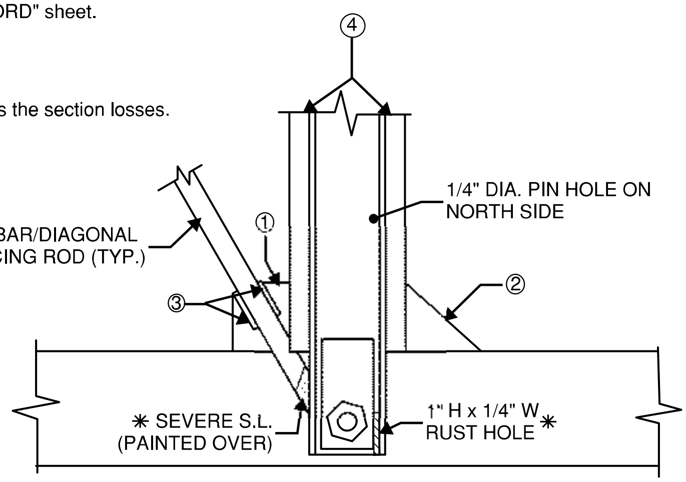
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L2 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 20)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

ORIGINAL TRUSS MEMBER DETAILS:

- Bottom chord is comprised of (1) - 15-1/4" x 1/2" plate & (4) - 2-1/2" x 2-1/2" x 5/16" angles.

- Upper chord is comprised of (1) - 15" x 5/16" plate, (2) - 14-1/2" x 5/16" plate & (4) - 2-1/2" x 2-1/2" x 1/4" angles.

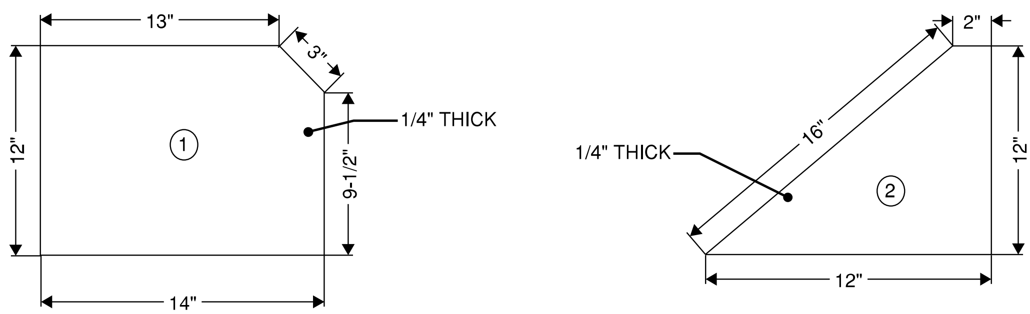
- Vertical members at L2-L10 are comprised of (2) - C7 x 9.8 channels and L1, L11 are comprised of (2) - 2-5 3/4" x 5/16" plates.

- Diagonal members L5-U4, L6-U5, L6-U7, L7-U8 are comprised of (2) - C6 x 8.2 channels.

- Diagonal members L3-U2, L4-U3, L8-U9, L9-U10 are comprised of (2) - 2" x 7/8" plates.

- Diagonal members L2-U1, L10-U11 are comprised of (2) - 3" x 13/16" plates.

- See "ADDITIONAL BACK-UP MATERIAL".



NOTE:
- The repair gusset plates are welded atop the bottom chord and channel bracing of vertical members at chords L2 - L10.

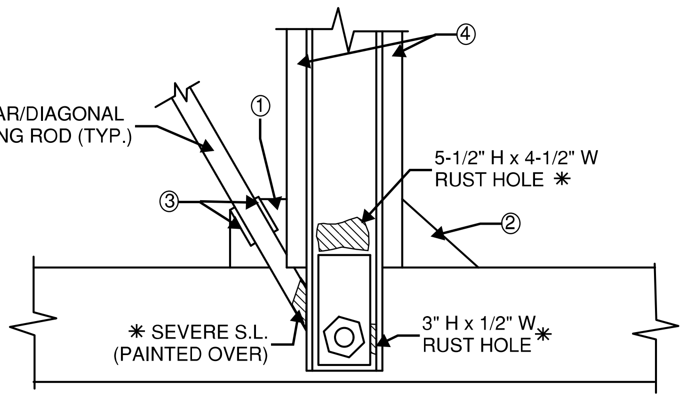
REPAIR PLATES

(N.T.S.)

(SKETCH 21)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L3 SOUTH TRUSS SOUTH ELEVATION

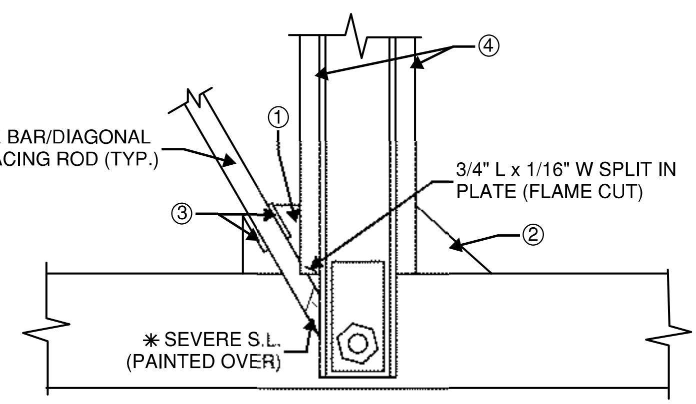
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L4 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

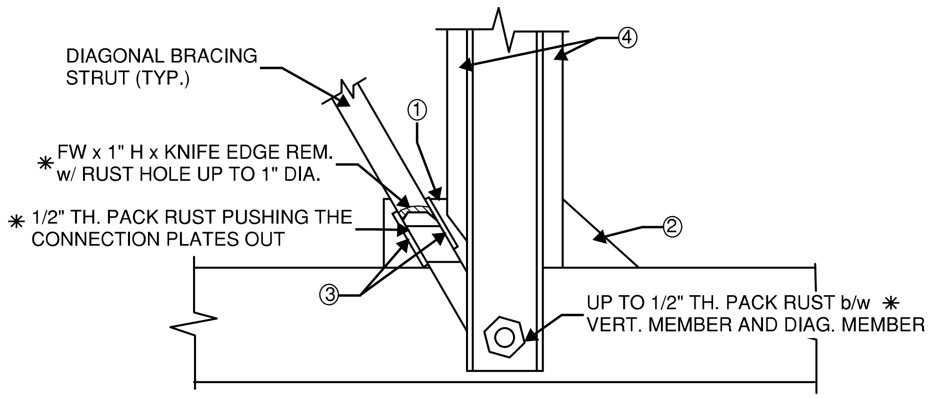
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 22)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L5 SOUTH TRUSS SOUTH ELEVATION

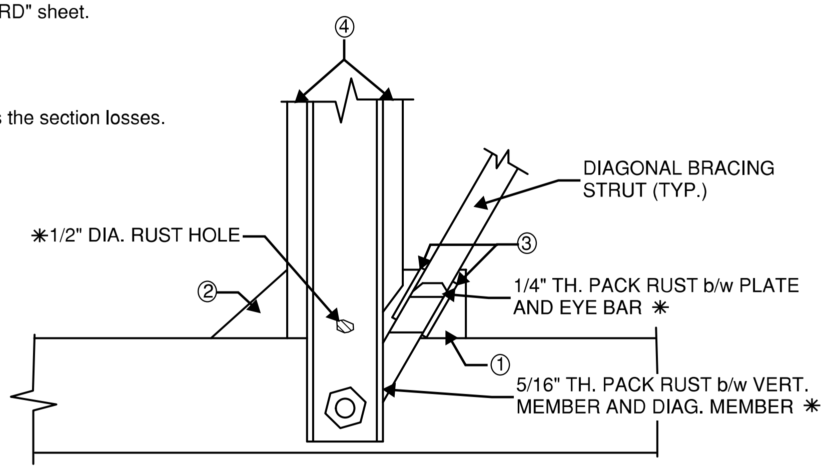
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L7 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

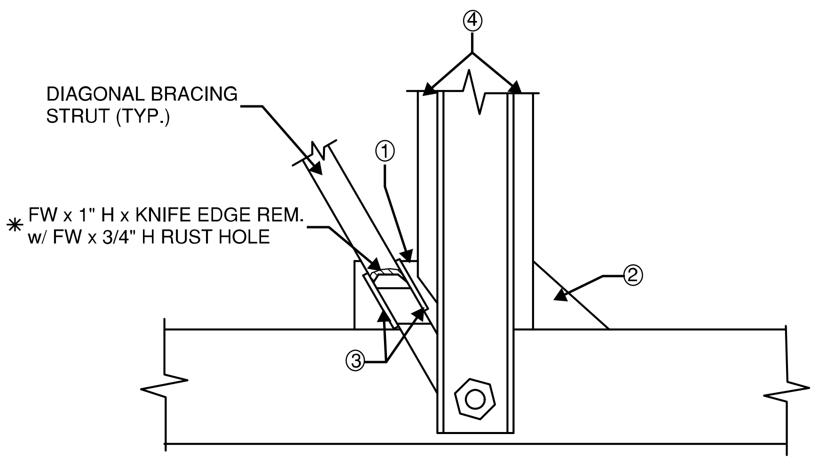
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 23)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L7 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

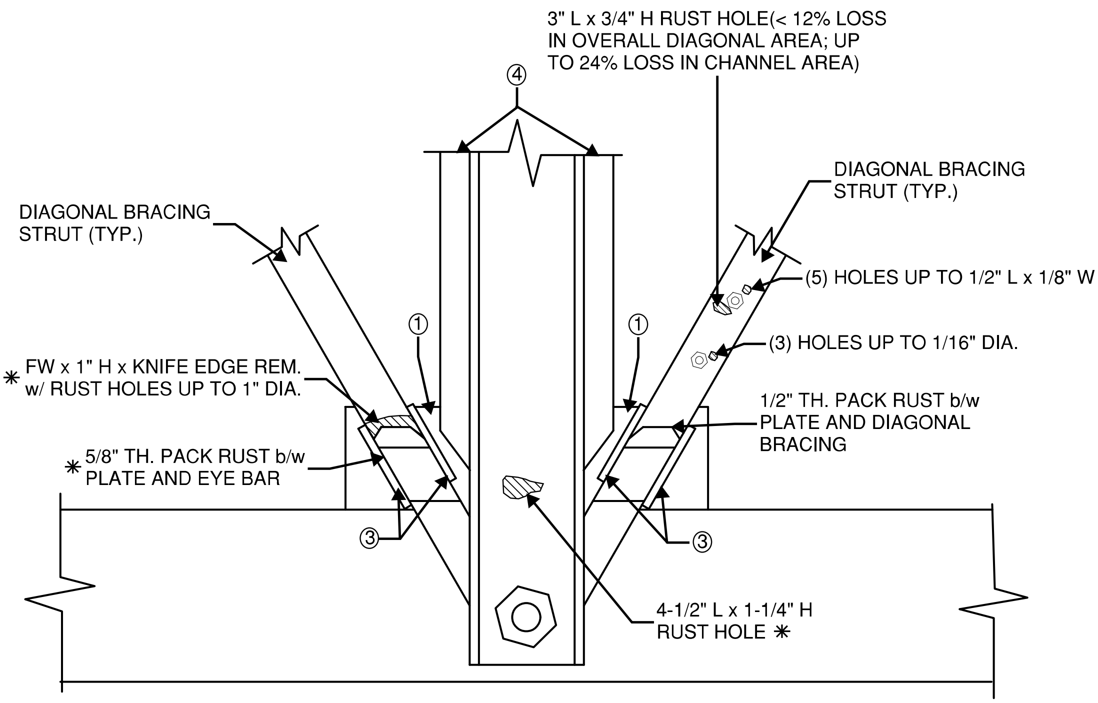
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 24)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L6 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

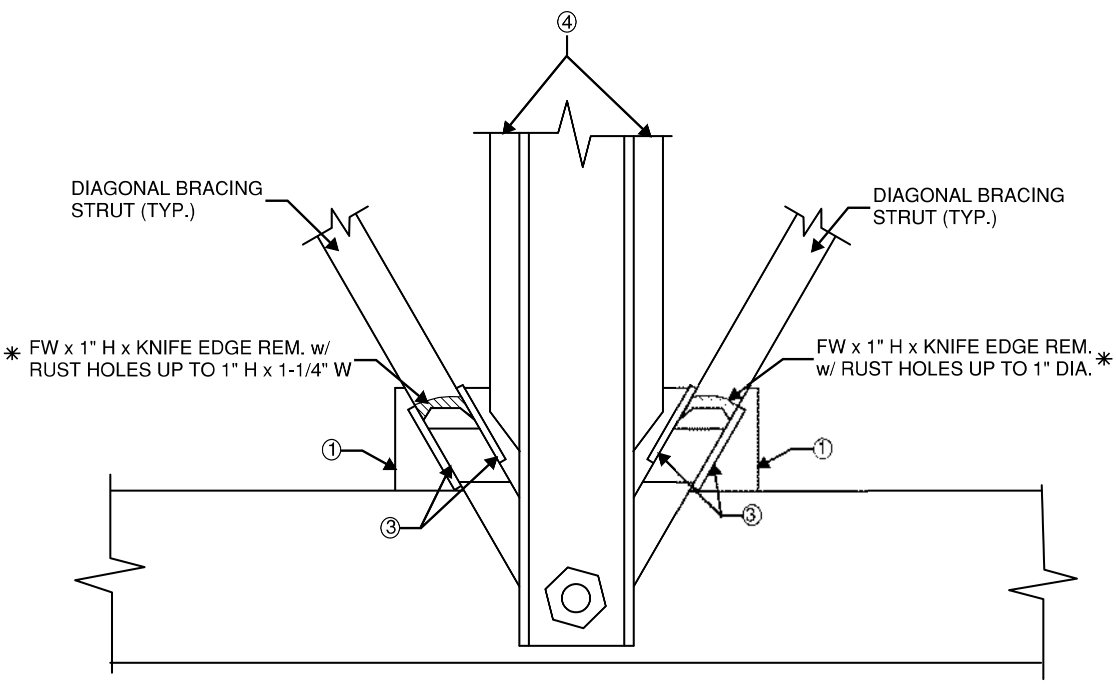
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 25)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L6 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

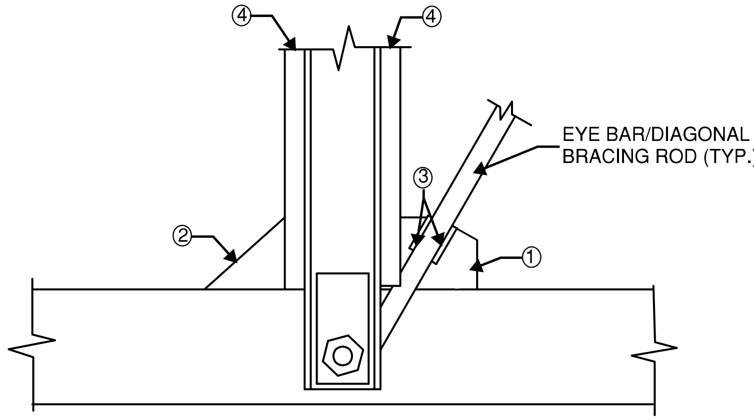
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 26)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L8 SOUTH TRUSS SOUTH ELEVATION

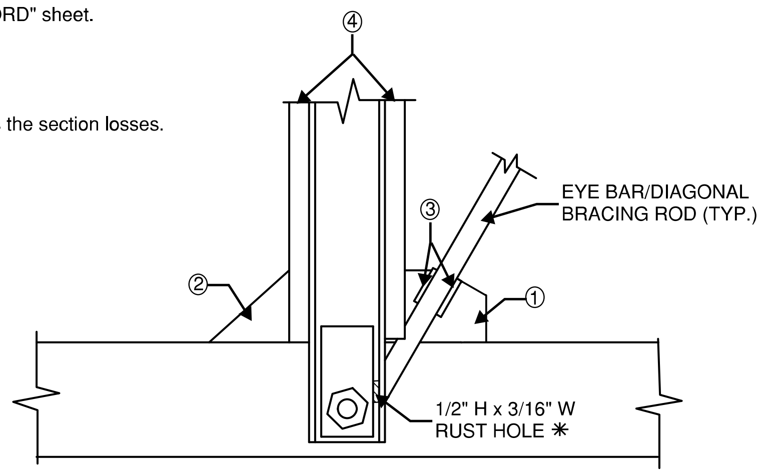
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L9 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

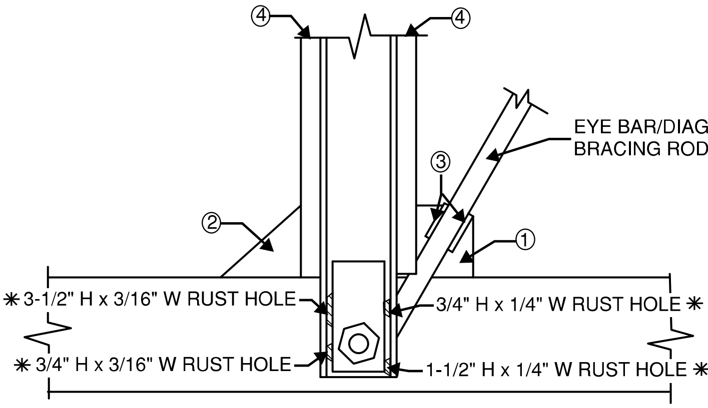
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 27)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L10 SOUTH TRUSS SOUTH ELEVATION

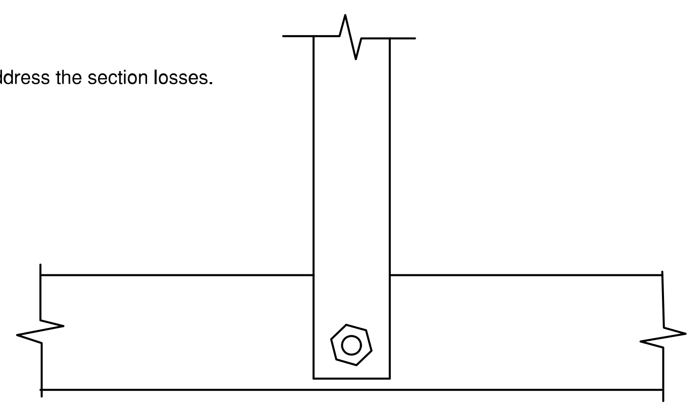
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L11 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 28)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

GENERAL NOTES - BOTTOM CHORD:

- Random areas of peeling paint with moderate to heavy rust.

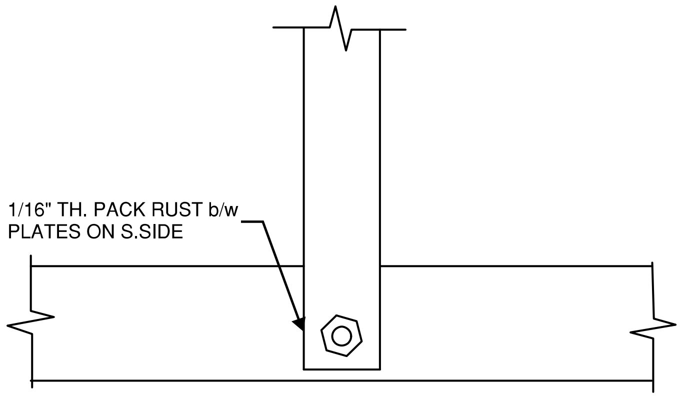
- Severe section loss in the vertical chords and diagonal members were addressed by retrofit gusset plates. The retrofit gusset plates were welded to the bottom chord, diagonal members and vertical chords and painted over during rehabilitation.

- Pack rust up to 1/2" thick between the connection plates and truss members at the pin connections, bottom chord splice connections and diagonal member - truss element connections.

(SKETCH 29)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L1 NORTH TRUSS NORTH ELEVATION

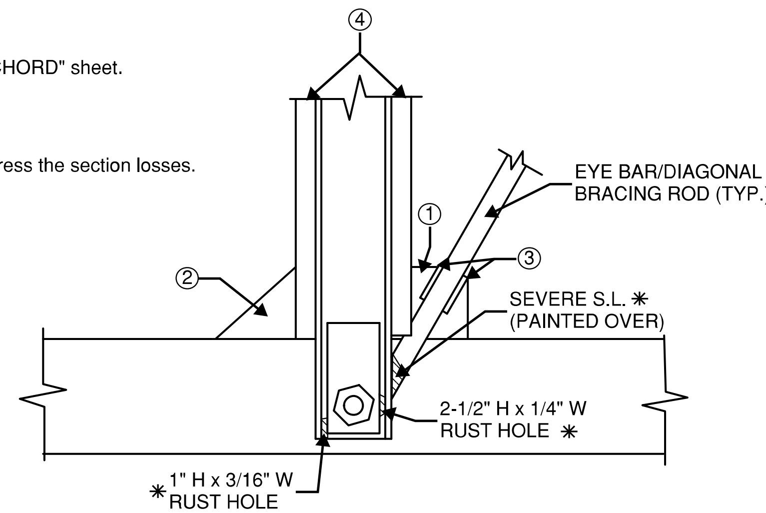
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L2 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

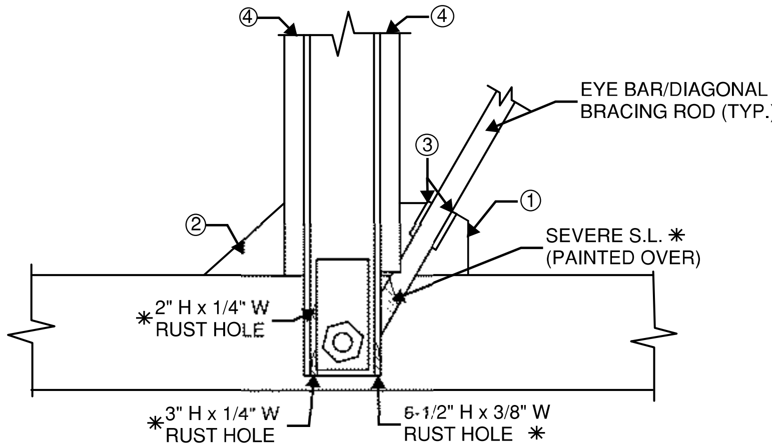
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 30)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L3 NORTH TRUSS NORTH ELEVATION

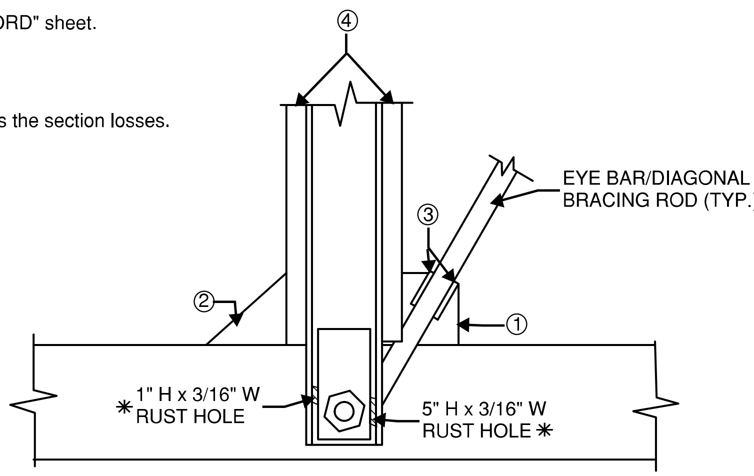
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L4 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

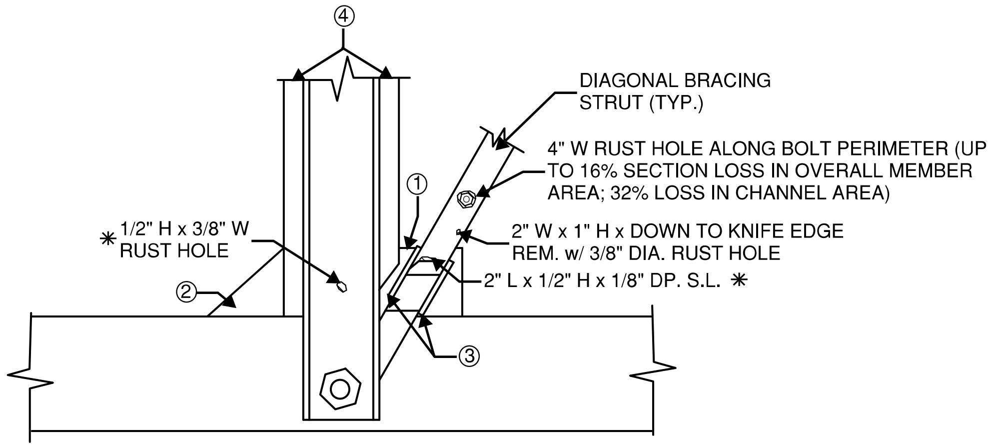
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 31)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L5 NORTH TRUSS NORTH ELEVATION

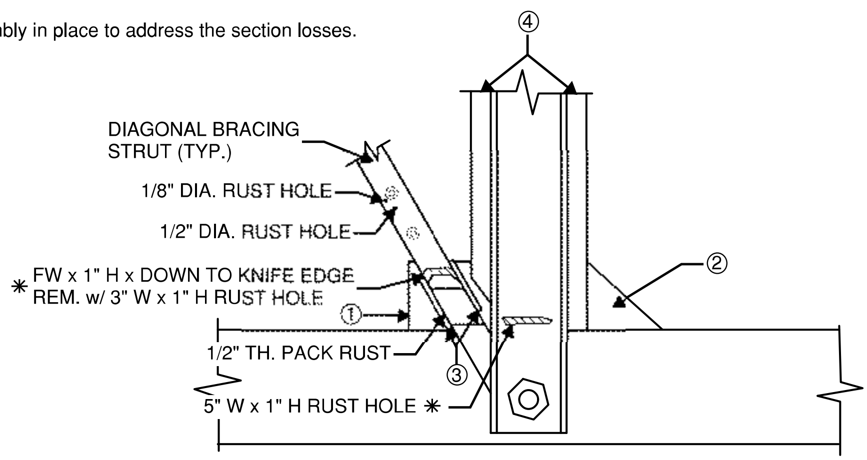
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

* Retrofit assembly in place to address the section losses.



L7 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

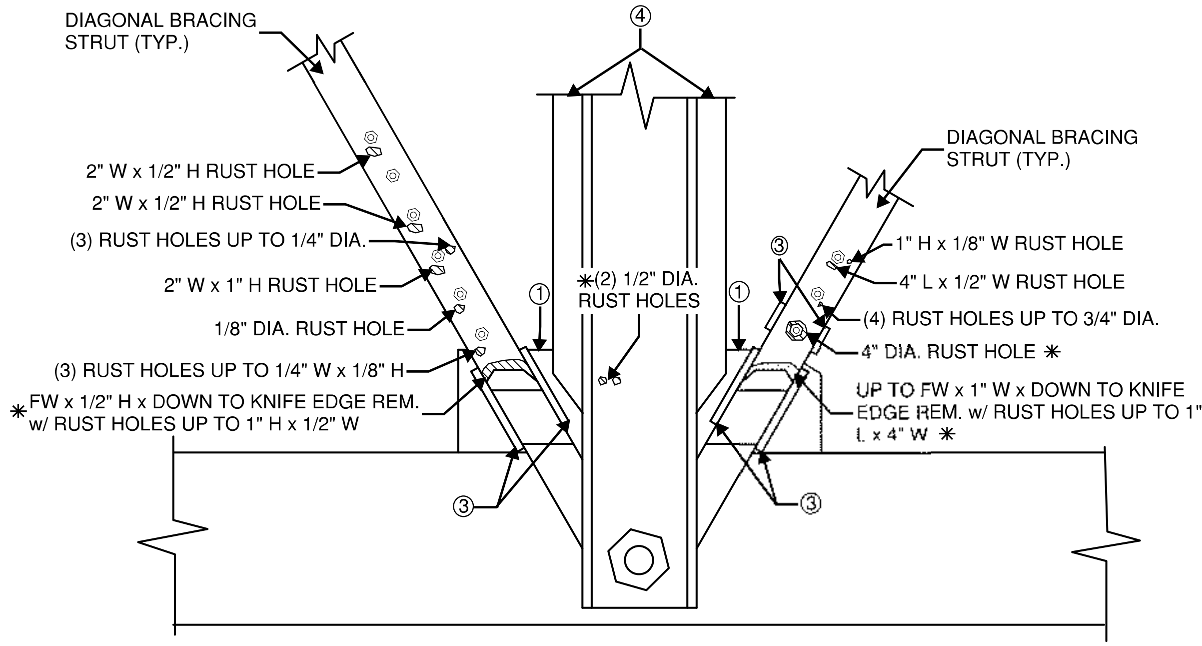
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 32)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L6 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

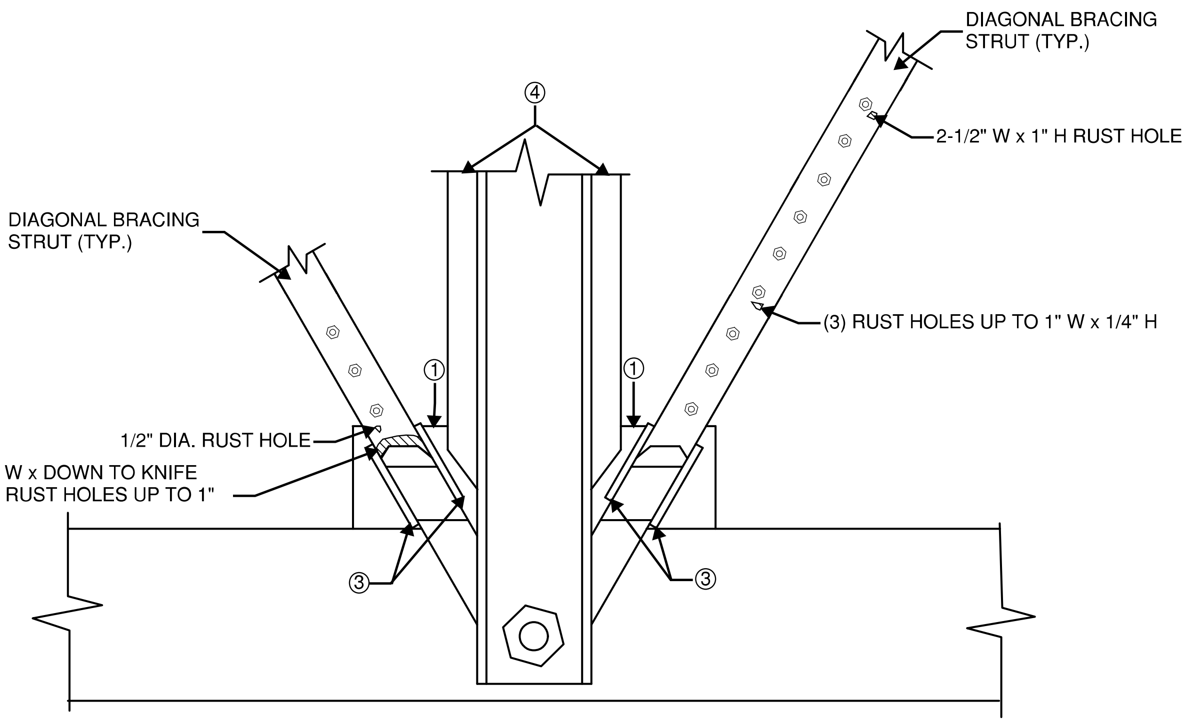
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 33)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L6 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

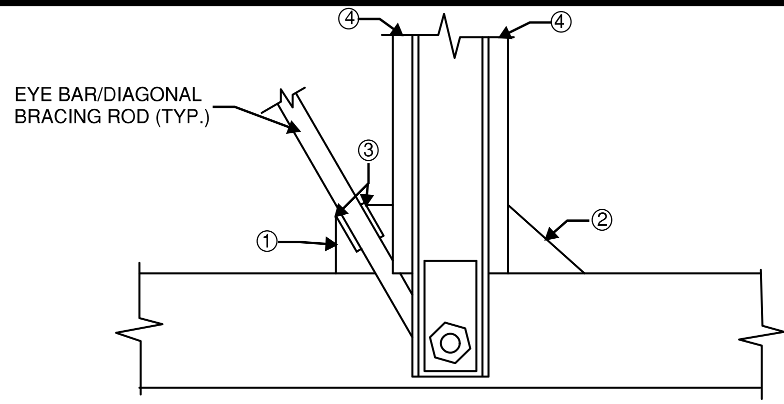
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 34)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L8 NORTH TRUSS NORTH ELEVATION

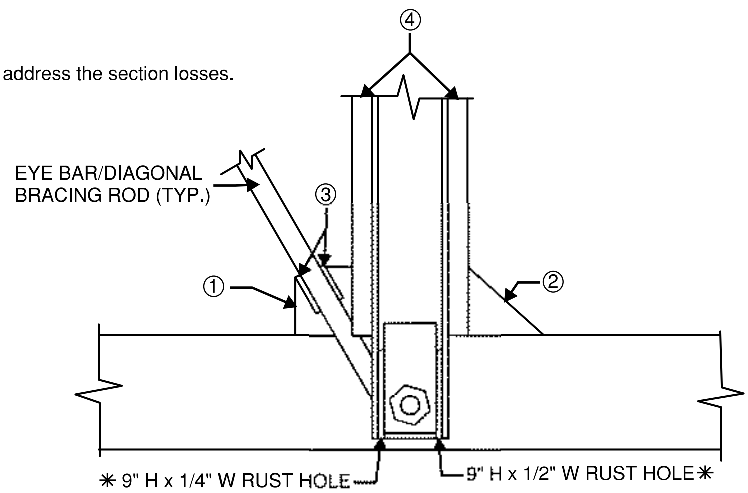
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L9 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

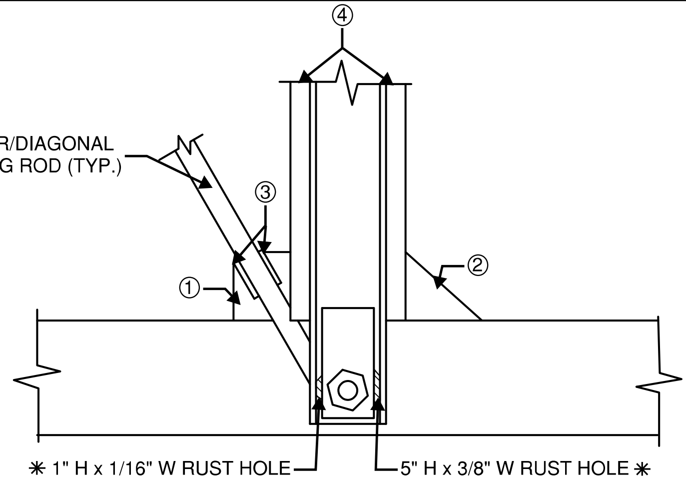
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 35)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984



L10 NORTH TRUSS NORTH ELEVATION

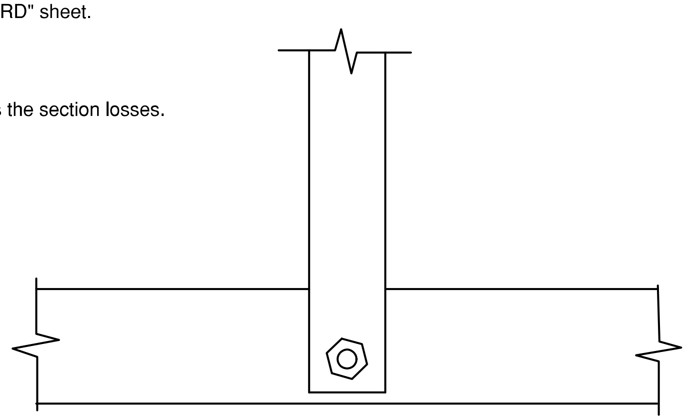
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BOTTOM CHORD" sheet.

NOTE:

✳ Retrofit assembly in place to address the section losses.



L11 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

REPAIR LEGEND:

-①: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

-②: Repair gusset plate (See "REPAIR PLATES" sheet for dimensions)

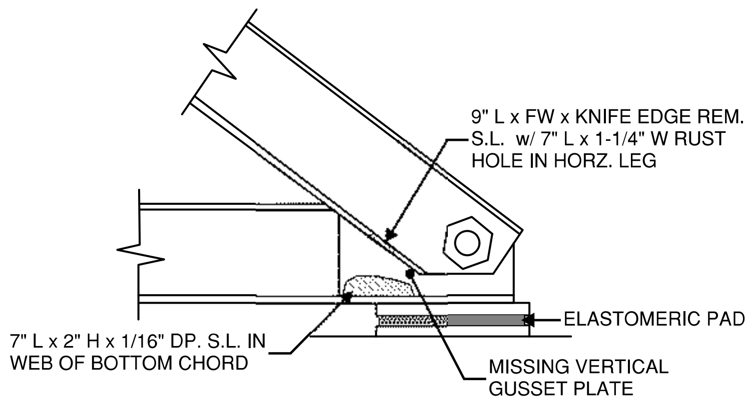
-③: Repair plate 5/8" - 1/2" thick welded to the diagonal member & repair gusset plate

-④: Channel bracing (C10 x 15.3) with intermittent welds

(SKETCH 36)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

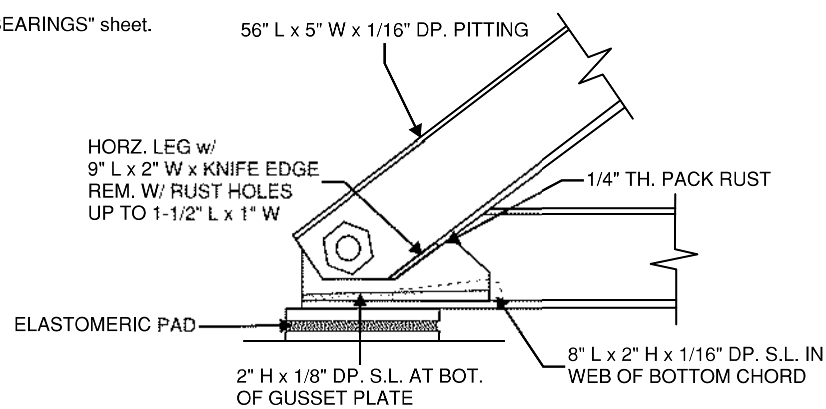


EXPANSION BEARING, L0 NORTH TRUSS AT WEST ABUTMENT

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BEARINGS" sheet.



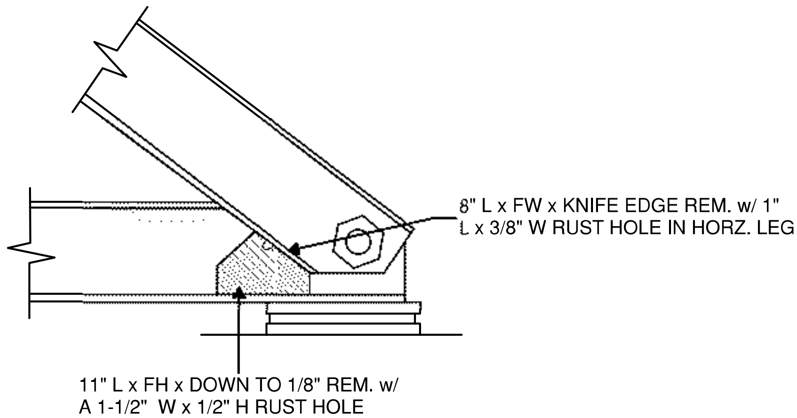
EXPANSION BEARING, L0 SOUTH TRUSS AT WEST ABUTMENT

(N.T.S.)

(SKETCH 37)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

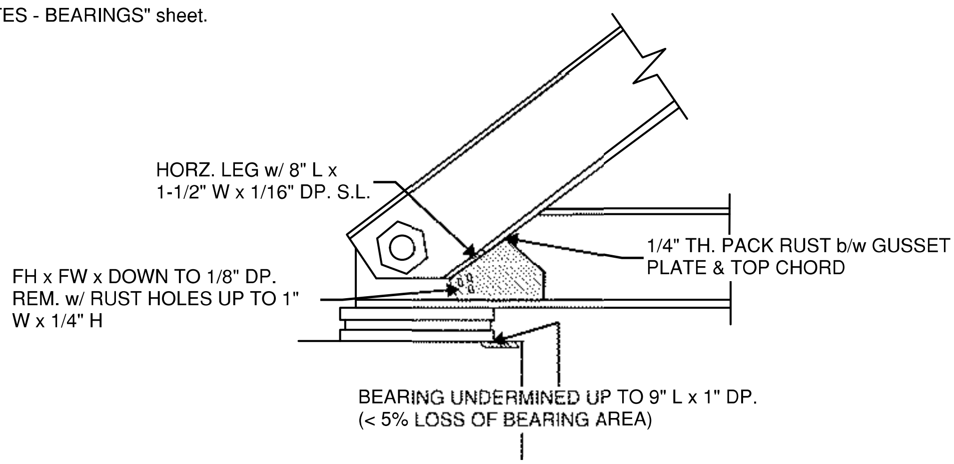


FIXED BEARING, L12 SOUTH TRUSS AT EAST ABUTMENT

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - BEARINGS" sheet.



FIXED BEARING, L12 NORTH TRUSS AT EAST ABUTMENT

(N.T.S.)

(SKETCH 38)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/28/2017	BRIDGE NO.: 03984

GENERAL NOTES - BEARINGS:

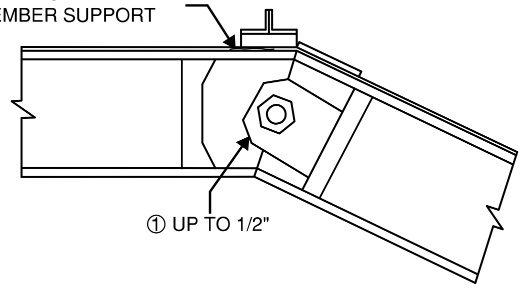
- Moderate to heavy accumulation of pack rust and timber debris atop the bearing plates.

- Areas of peeling paint with light to moderate rust.

(SKETCH 39)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

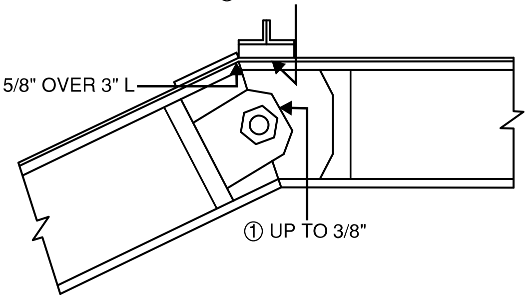


U1 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U1 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

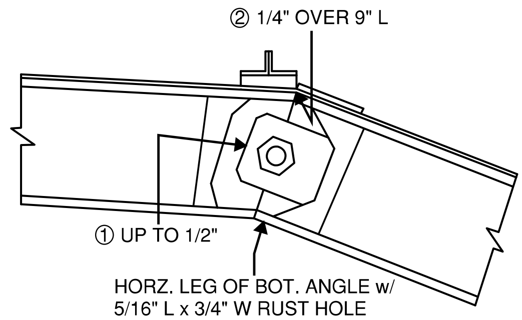
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 40)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

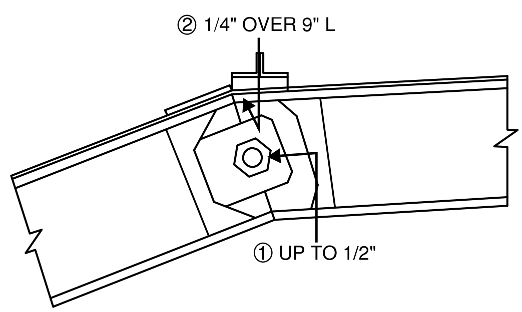


U2 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U2 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

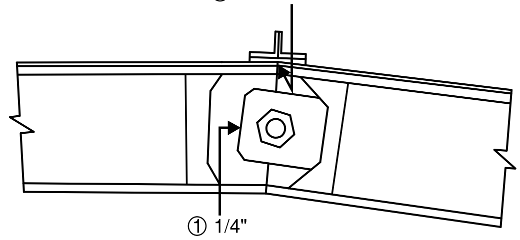
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 41)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

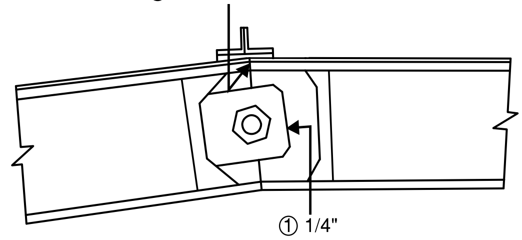


U3 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U3 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

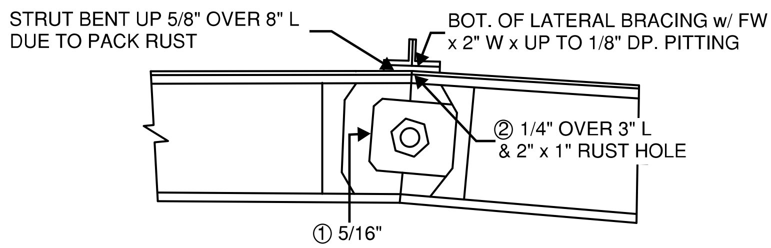
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 42)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

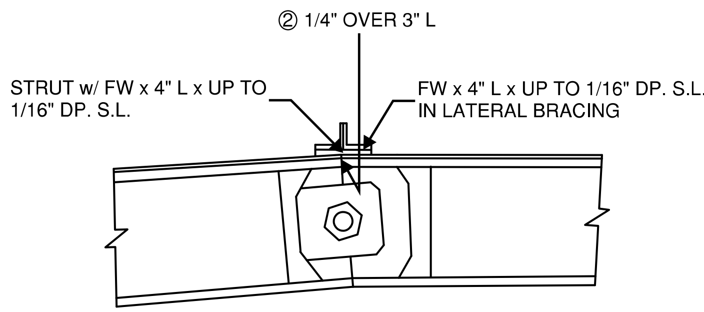


U4 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U4 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

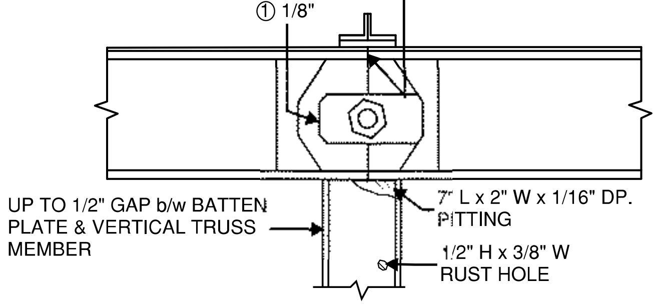
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 43)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

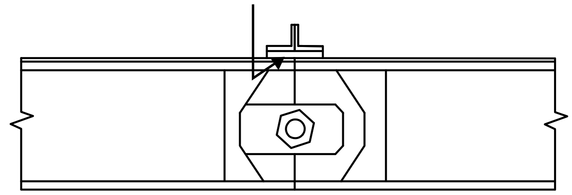


U5 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U5 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

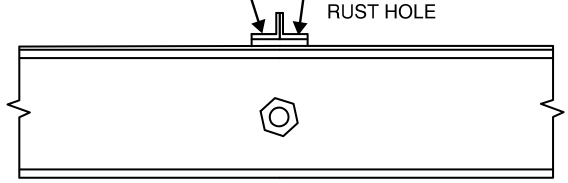
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 44)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

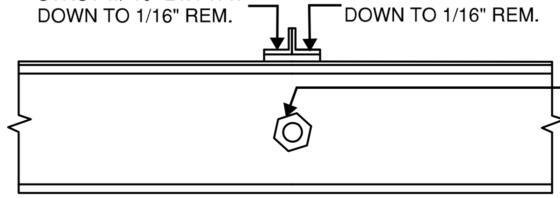
CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



U6 SOUTH TRUSS NORTH ELEVATION
(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U6 SOUTH TRUSS SOUTH ELEVATION
(N.T.S.)

DETERIORATION LEGEND:

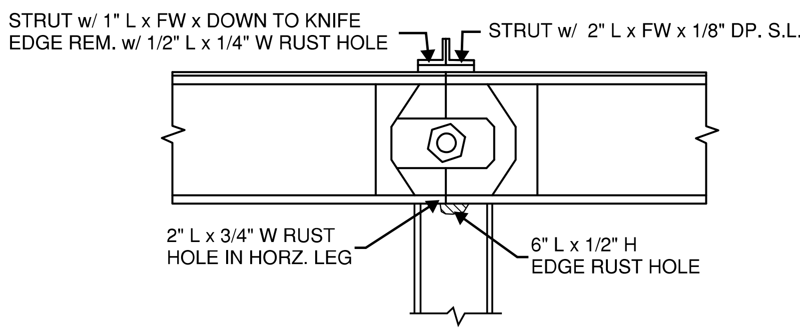
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 45)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

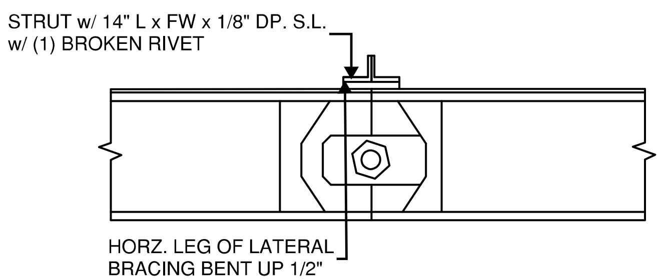


U7 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U7 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

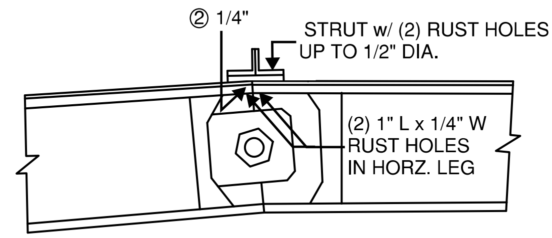
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 46)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

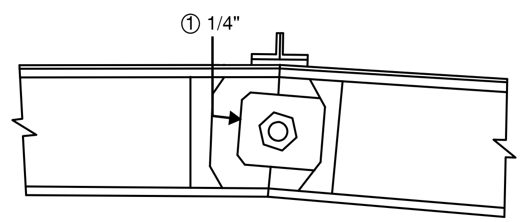


U8 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U8 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

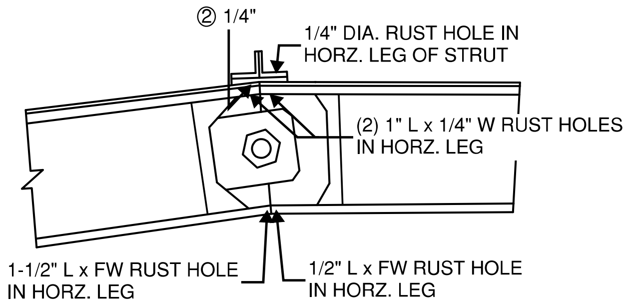
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 47)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

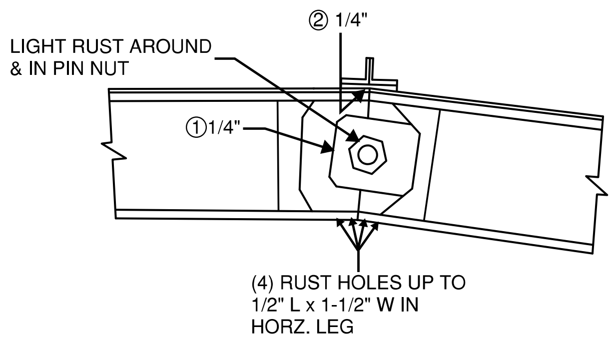


U9 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U9 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

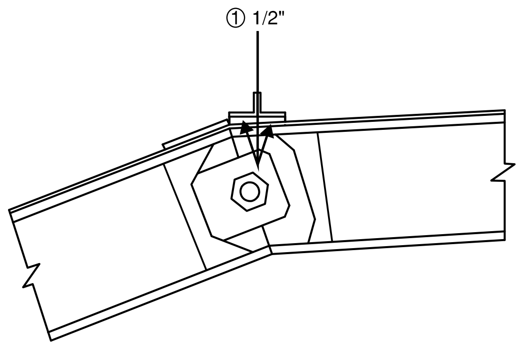
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 48)

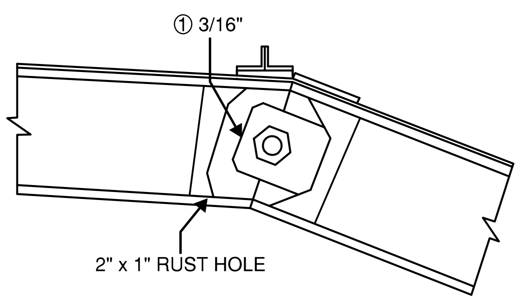
REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



U10 SOUTH TRUSS NORTH ELEVATION
(N.T.S.)

GENERAL NOTES:
- See "GENERAL NOTES - TOP CHORD" sheet.



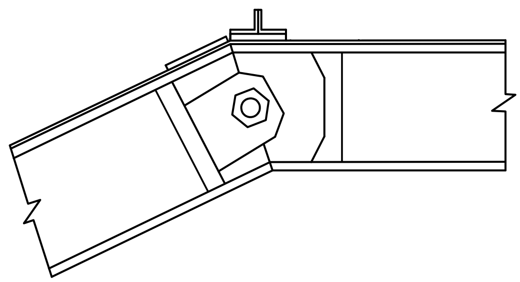
U10 SOUTH TRUSS SOUTH ELEVATION
(N.T.S.)

DETERIORATION LEGEND:
-① GAP B/W PLATES DUE TO PACK RUST
-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 49)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

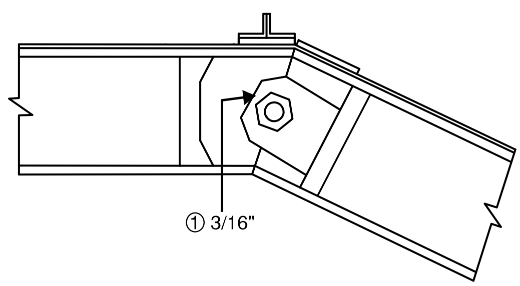


U11 SOUTH TRUSS NORTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U11 SOUTH TRUSS SOUTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 50)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

GENERAL NOTES - TOP CHORD:

- Random areas of peeling paint with light to moderate rust.

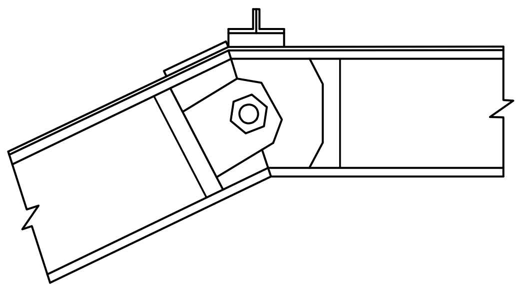
- Random rivets with peeling paint and rust; isolated locations with missing welds and rivet heads with minor head section loss.

- Random locations with bird nests at the truss upper nodes.

(SKETCH 51)

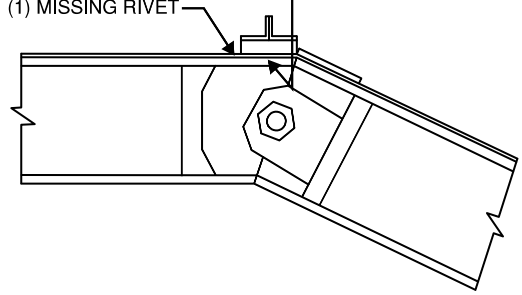
REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



U1 NORTH TRUSS SOUTH ELEVATION
(N.T.S.)

GENERAL NOTES:
- See "GENERAL NOTES - TOP CHORD" sheet.



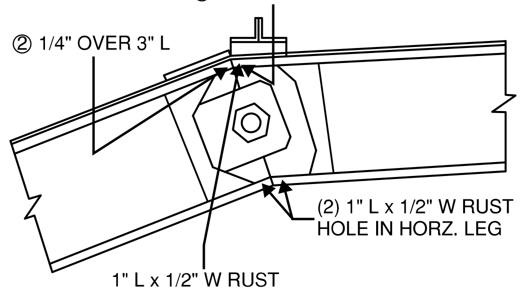
U1 NORTH TRUSS NORTH ELEVATION
(N.T.S.)

DETERIORATION LEGEND:
-① GAP B/W PLATES DUE TO PACK RUST
-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 52)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

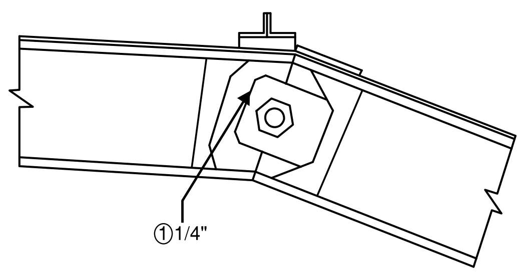


U2 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U2 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

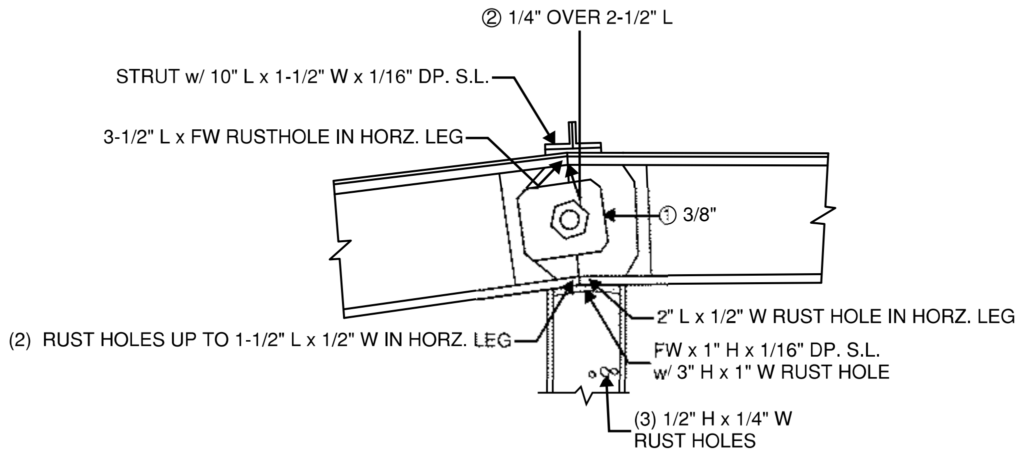
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 53)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

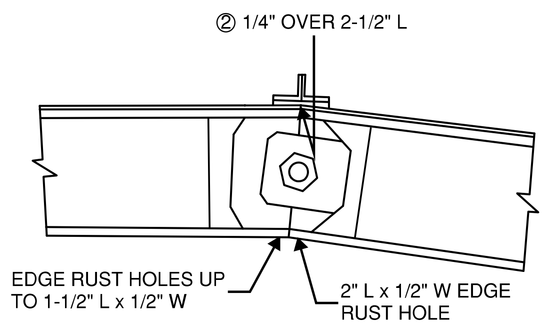


U3 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U3 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

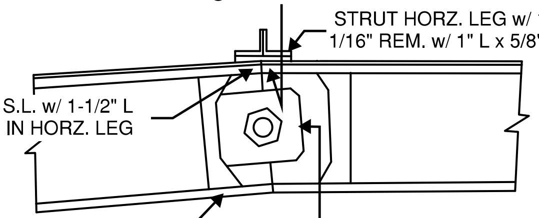
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 54)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

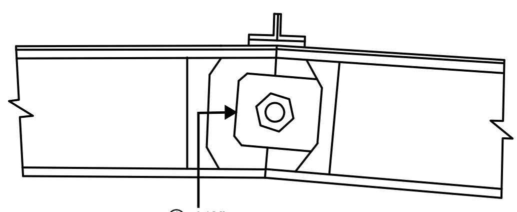


U4 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U4 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

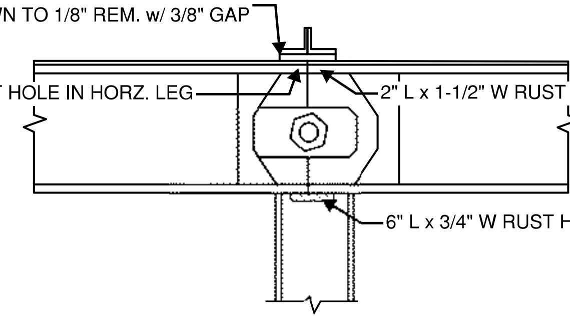
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 55)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

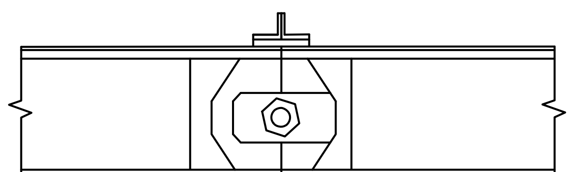


U5 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U5 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

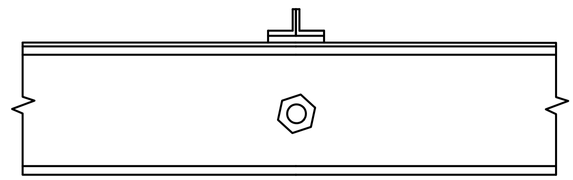
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 56)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

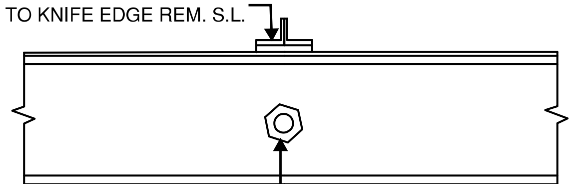


U6 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U6 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

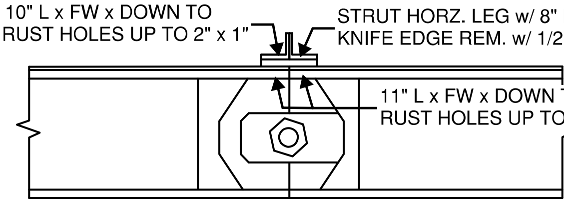
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 57)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

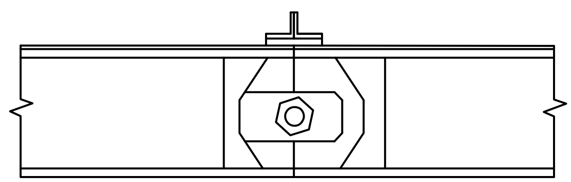


U7 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U7 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

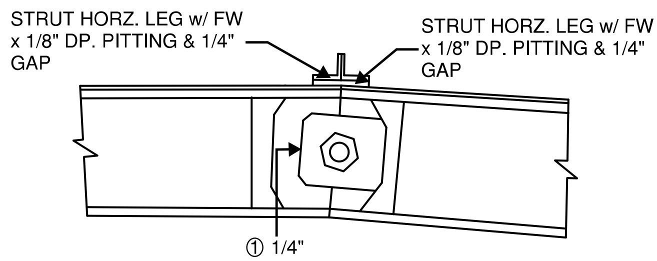
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 58)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2) | DATE: 6/29/2017 | BRIDGE NO.: 03984

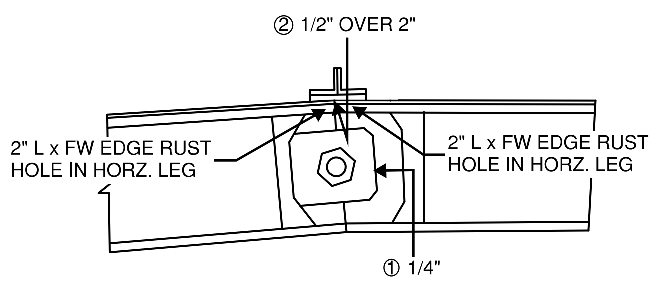


U8 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U8 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

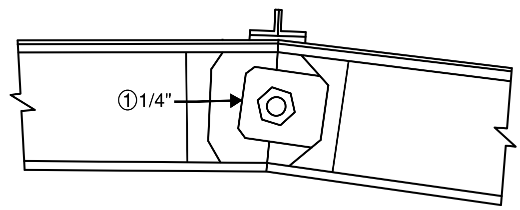
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 59)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

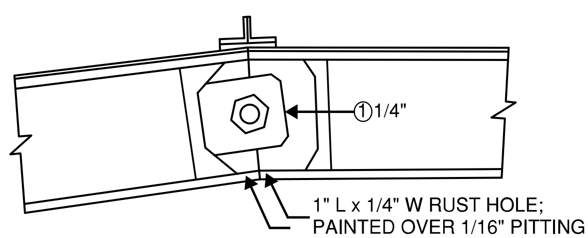


U9 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U9 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

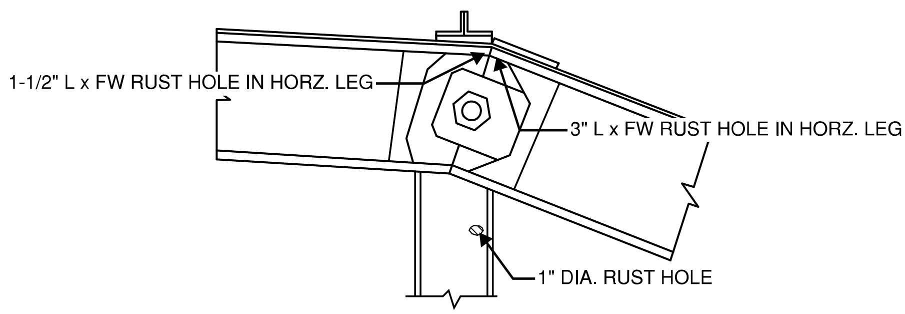
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 60)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

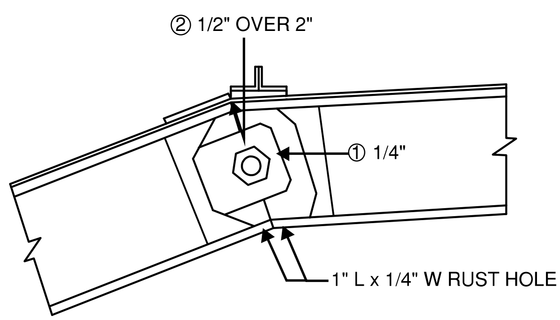


U10 NORTH TRUSS SOUTH ELEVATION

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - TOP CHORD" sheet.



U10 NORTH TRUSS NORTH ELEVATION

(N.T.S.)

DETERIORATION LEGEND:

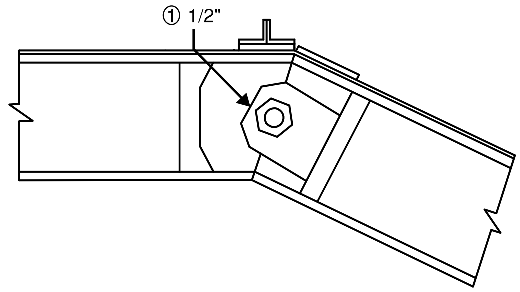
-① GAP B/W PLATES DUE TO PACK RUST

-② HORIZONTAL LEG OF TOP ANGLE BENT DOWN

(SKETCH 61)

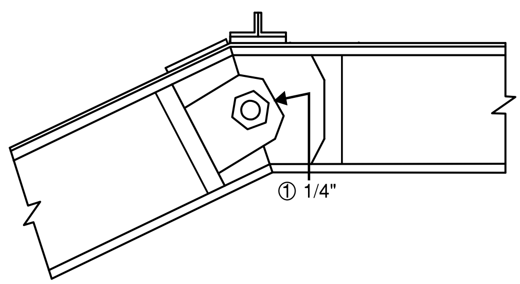
REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



U11 NORTH TRUSS SOUTH ELEVATION
(N.T.S.)

GENERAL NOTES:
- See "GENERAL NOTES - TOP CHORD" sheet.



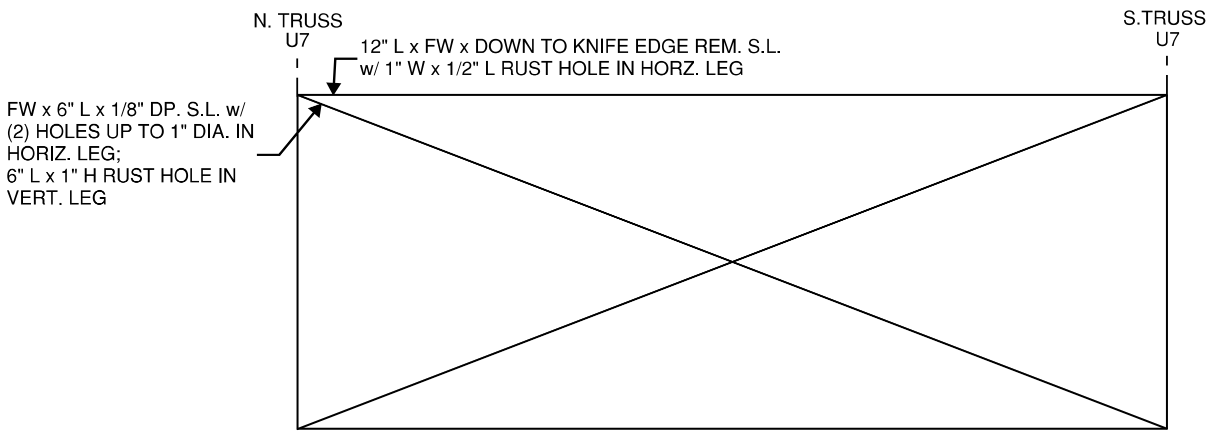
U11 NORTH TRUSS NORTH ELEVATION
(N.T.S.)

DETERIORATION LEGEND:
-① GAP B/W PLATES DUE TO PACK RUST

(SKETCH 62)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

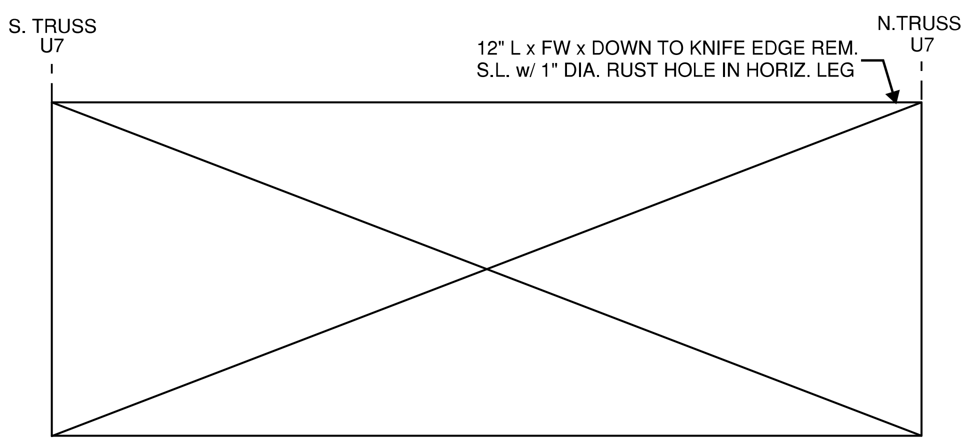


SWAY BRACING AT TOP FOR TRUSS AT U7 LOOKING EAST

(N.T.S.)

GENERAL NOTES:

- See "GENERAL NOTES - SWAY BRACING" sheet.



SWAY BRACING AT TOP FOR TRUSS AT U7 LOOKING WEST

(N.T.S.)

(SKETCH 63)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984

GENERAL NOTES - SWAY BRACING:

- Sway bracing present at L3, L5, L7, L9 chords.

- Bracings with peeling paint and light to moderate rust.

- Gaps up to 3/8" between top strut and diagonal sway bracing members.

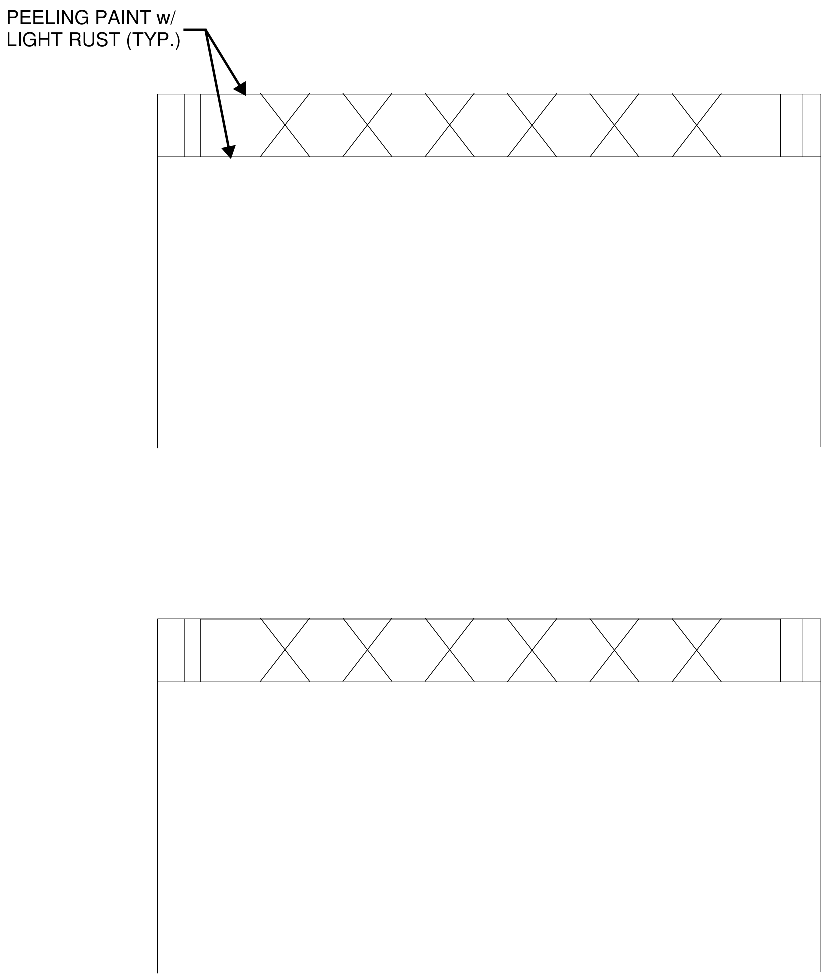
- Bracings atop the top chords with section loss up to full length x full width x down to knife edge remaining (maximum noted in sketches).

- Horizontal legs of the top struts bent up up to 1/2" due to pack rust between the bracing and top chord of truss.

(SKETCH 64)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- Portal framing present at L1 and L11 chords.
- Portals with peeling paint and light to moderate rust.

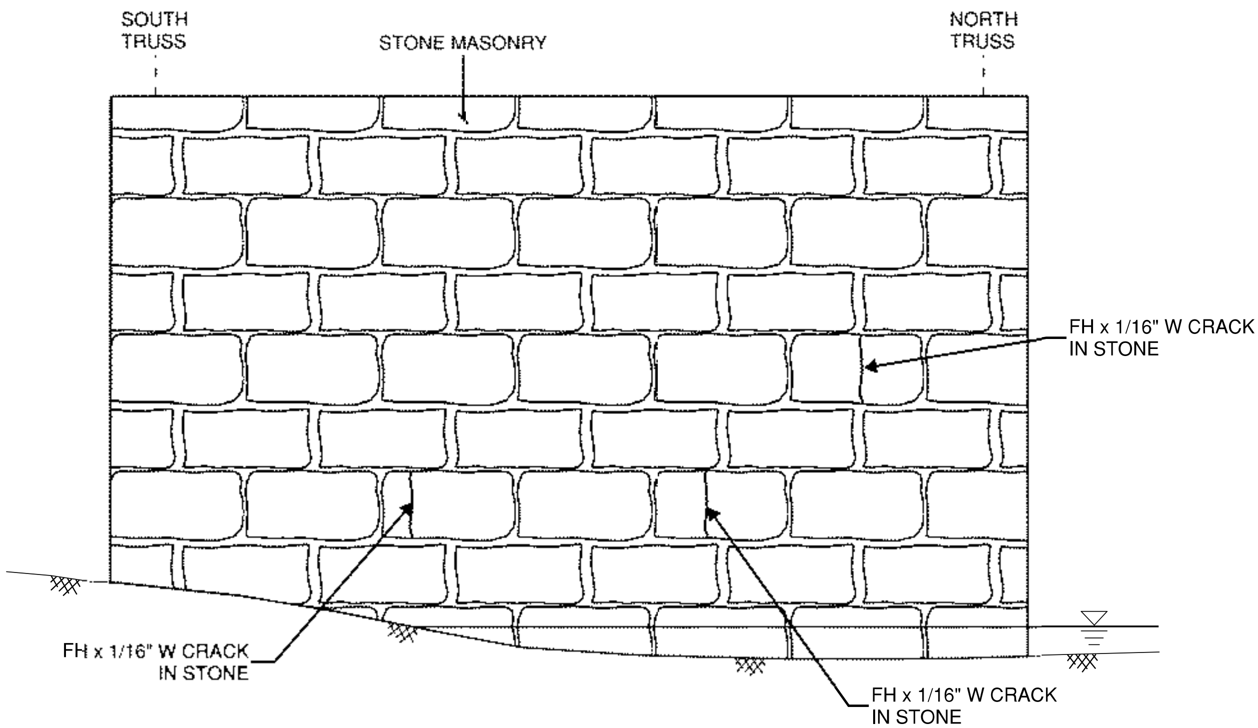
PORTAL FRAMING (TYP.)

(N.T.S.)

(SKETCH 65)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- Random voids between stones due to loss of joint mortar along the base of the stem.
- Random hairline cracks in the joint mortar between stones with efflorescence.

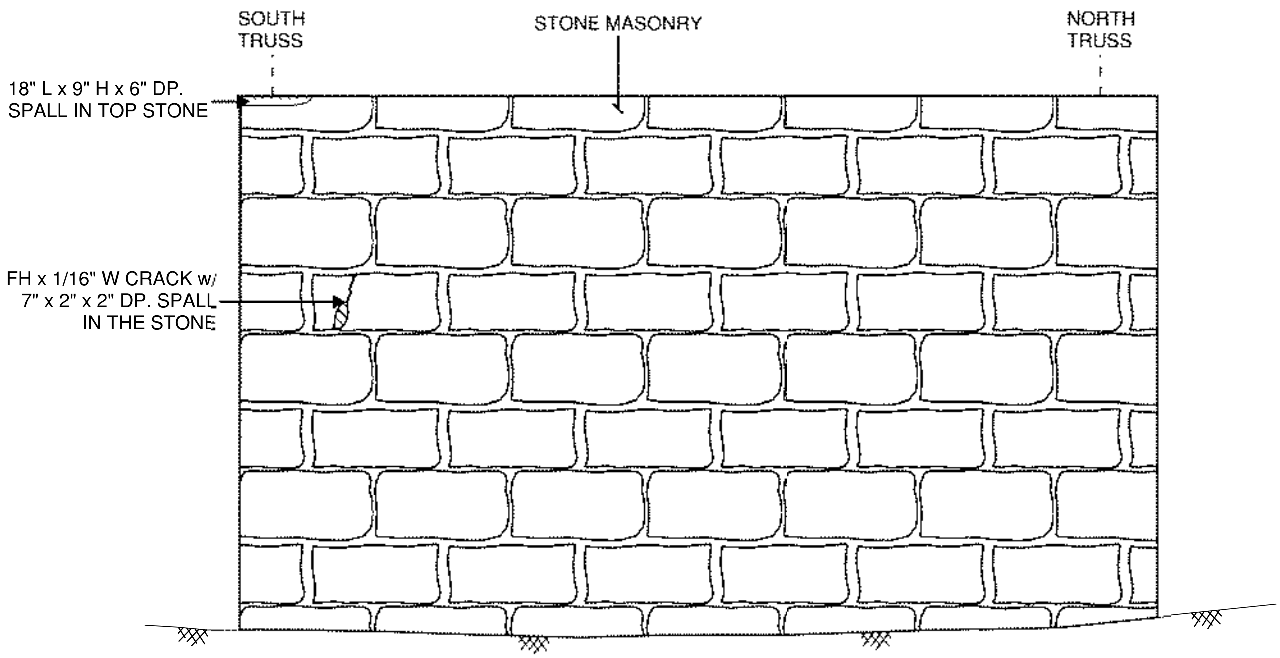
WEST ABUTMENT (1)

(N.T.S.)

(SKETCH 66)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- See "WEST ABUTMENT (1)" sheet.

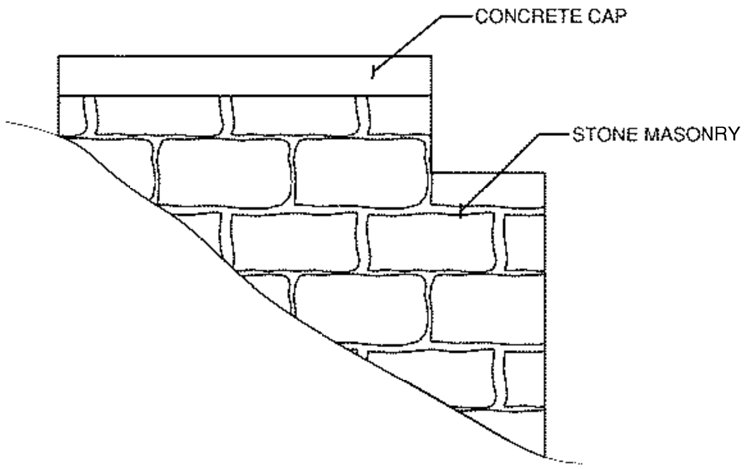
EAST ABUTMENT (2)

(N.T.S.)

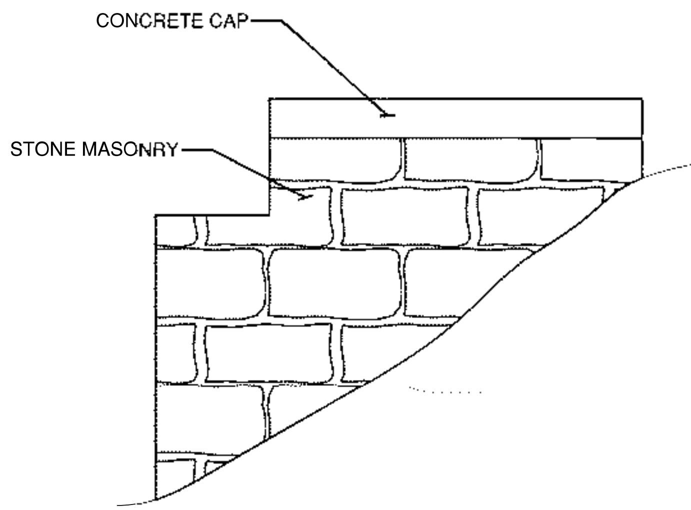
(SKETCH 67)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



SOUTHWEST WINGWALL (1B)
(N.T.S.)



NORTHWEST WINGWALL (1A)
(N.T.S.)

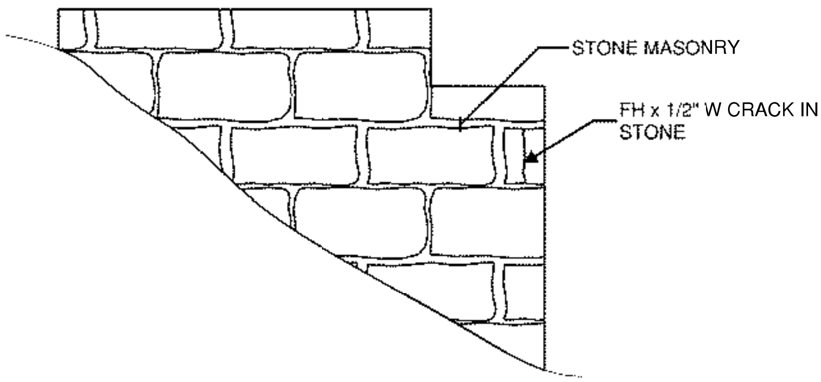
GENERAL NOTES:

- Horizontal hairline cracks in the stones at isolated locations.
- Random hairline cracks in the joint mortar between stones.
- Heavy growth of vegetation along the wingwalls.

(SKETCH 68)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

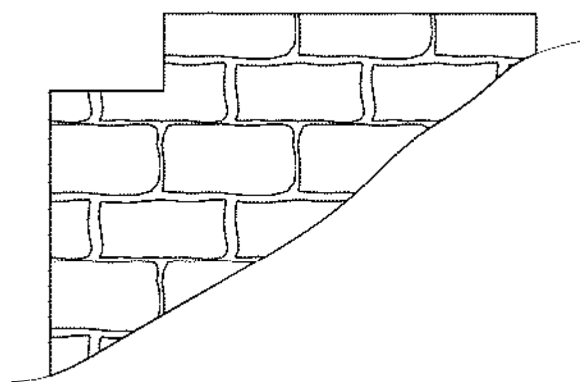
CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



NORTHEAST WINGWALL (2A)
(N.T.S.)

GENERAL NOTES:

- See previous sheet.

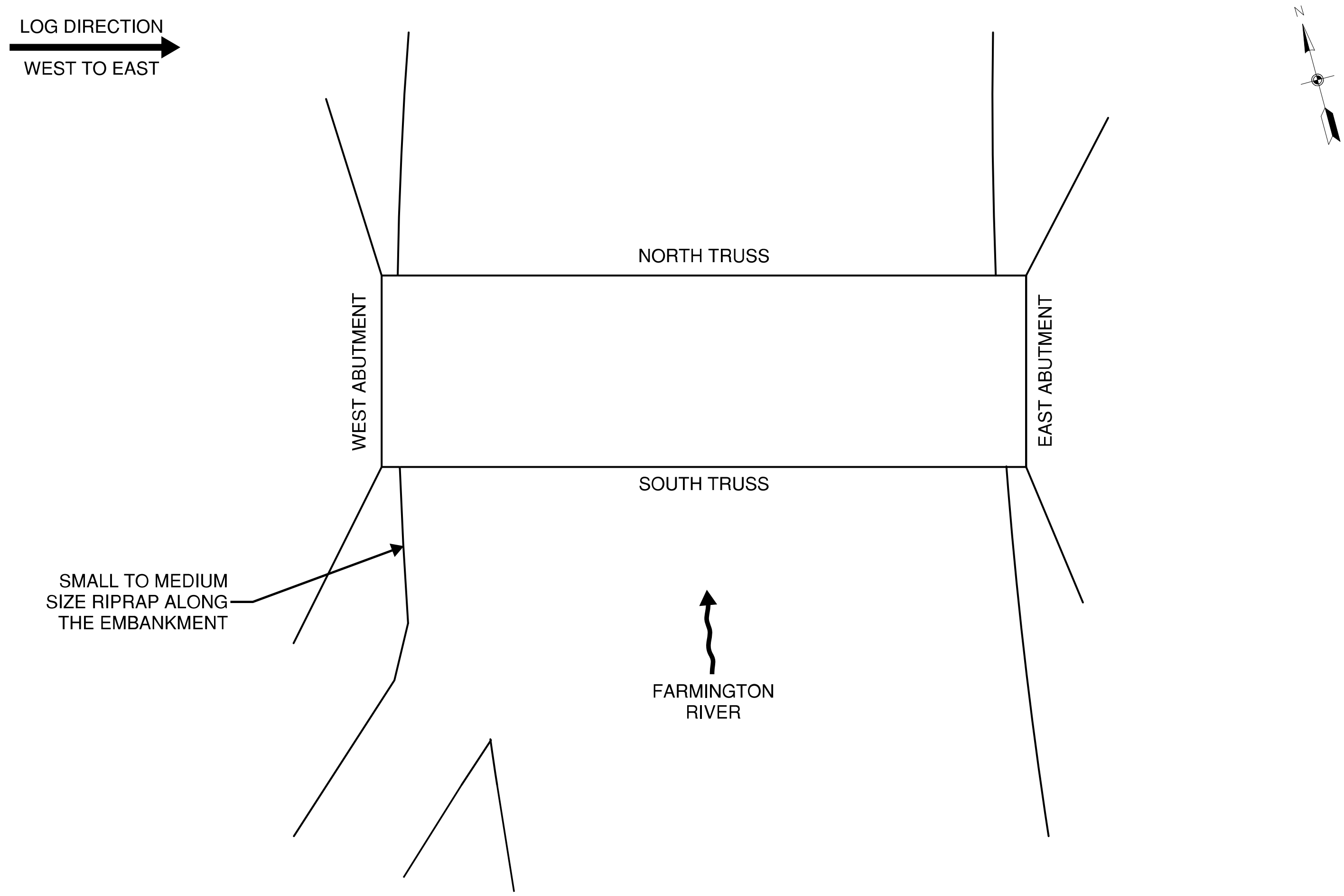


SOUTHEAST WINGWALL (2B)
(N.T.S.)

(SKETCH 69)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2)	DATE: 6/29/2017	BRIDGE NO.: 03984



GENERAL NOTES:

- Channel bottom consists of sand with small to medium sized stones.
- Erosion along the channel embankments for up to 3' high x 3' deep with exposed tree roots.
- Heavy growth of vegetation along the channel embankments which is overhanging the channel.

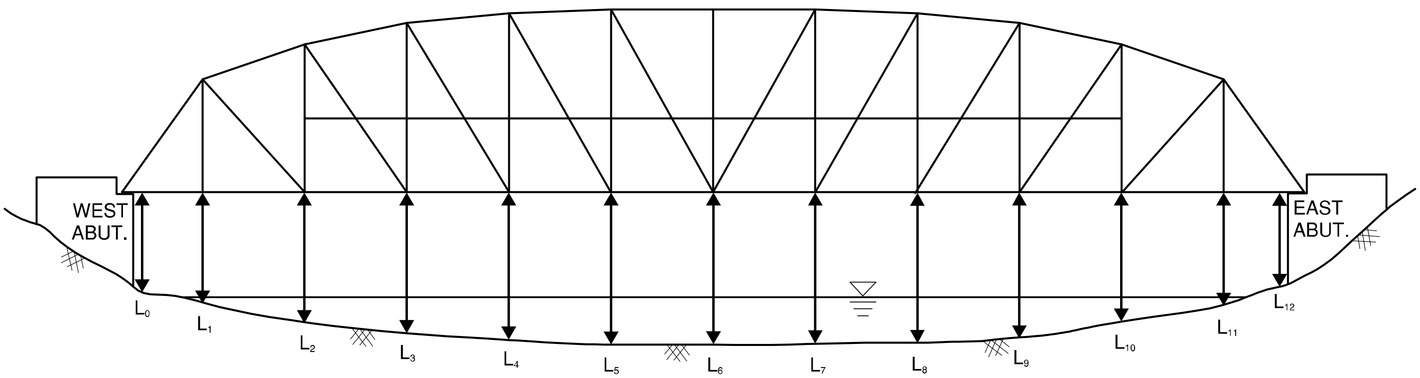
CHANNEL DIAGRAM

(N.T.S.)

(SKETCH 70)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

CREW: BJS, SR (GM2) | DATE: 6/29/2017 | BRIDGE NO.: 03984



NODE	L_0	L_1	L_2	L_3	L_4	L_5	L_6	L_7	L_8	L_9	L_{10}	L_{11}	L_{12}	Water Depth (L_6)
DROPLINE MEASUREMENT	13'-8"	15'-11"	17'-6"	17'-8"	17'-4"	19'-7"	18'-10"	18'-6"	17'-0"	18'-5"	18'-6"	15'-11"	13'-8"	4'-6"

NOTE:

- Dropline measurements were taken at each node from top of south truss bottom chord.

DROPLINE MEASUREMENTS (SOUTH TRUSS INLET)

(N.T.S.)

(SKETCH 71)

REVISION 1	DATE:	CREW:	REVISION 3	DATE:	CREW:
REVISION 2	DATE:	CREW:	REVISION 4	DATE:	CREW:

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 1: South elevation.

Photo # 2: North elevation.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 3: Bridge from the west approach.

Photo # 4: West approach from the bridge.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 5: Bridge from the east approach.

Photo # 6: East approach from the bridge.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 7: View of the top side of timber deck.

Photo # 8: Isolated timber rot in the deck near midspan with exposed nails.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 9: View of the underside of deck and framing. Note the gaps between the deck planks.

Photo # 10: View of the underside of timber deck between L1 & L2.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 11: Bridge railing and flower beds along the south fascia.

Photo # 12: Bridge railing and flower beds along the north fascia.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 13: Deck end joint at West Abutment. Note the deteriorating joint sealant material.

Photo # 14: Southwest approach railing.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 15: East approach pavement. Note the depressed area.

Photo # 16: Expansion bearing for South Truss at West Abutment.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 17: Fixed bearing for North Truss at East Abutment. Note the section loss with rust holes.

Photo # 18: Expansion bearing for North Truss at West Abutment. Note the missing gusset plate. Also, note the rust hole in horizontal leg of top chord bottom angle.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 19: Floorbeam FB3 between L1 & L2 in South Truss. Note the section loss with rust hole in top flange of the floorbeam.

Photo # 20: Clip angle at Floorbeam FB2 between L1 & L2. Note peeling paint with rust. Also, note the painted over pitting along the bottom of floorbeam web.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 21: Interior view of the bottom chord at L2 in North Truss. Note the deterioration along the pin connection and the repair plates.

Photo # 22: Interior view of the bottom chord at L2 in North Truss. Note the pitting along the interior bottom angle.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 23: Exterior view of bottom chord at L3 in North Truss. Note the deterioration along the pin connection.

Photo # 24: Bottom chord splice connection between L2 & L3 in South Truss.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 25: Pack rust and section loss at the bottom chord splice connection between L2 & L3 in South Truss.

Photo # 26: South elevation of vertical member in South Truss at L3. Note the section loss in the diagonal eye bar and web of truss vertical member. Also, note the repair plates and channel atop the bottom chord.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 27: Vertical member in U5 South Truss. Note the rust hole.

Photo # 28: North elevation of North Truss diagonal member L6-U5 at L6. Note the section loss and rust hole at the connection plates (typical). Also, note the repair plates and retrofit gusset plate.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 29: Section loss in diagonal member L7-U8 at L7 on south side of North Truss. Note the repair plate.

Photo # 30: North elevation of North Truss diagonal member L6-U5 at L6. Note the section loss with rust holes in the channel web.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 31: South elevation of typical repair for eye bar at L10 in South Truss.

Photo # 32: North elevation of top chord member in South Truss at U2. Note the bent down top angles at the connection due to pack rust (typical).

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 33: View of the bottom angles of top chord connection at U3 in North Truss. Note the edge rust holes in the horizontal leg (typical).

Photo # 34: U6 in North Truss. Note the gap behind the pin nut.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 35: Bottom lateral bracing connection near L7 in North Truss. Note the short bolts.

Photo # 36: View of the top strut, sway bracing and lateral bracing between north and south trusses.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 37: Bottom of top strut at North Truss above U4.

Photo # 38: Sway bracing at U7 above North Truss. Note the section loss with rust holes.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 39: North side of U1 in South Truss. Note the crack in the weld.

Photo # 40: West Abutment elevation.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 41: East Abutment elevation.

Photo # 42: Crack in the stone masonry at East Abutment.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 43: Southeast Wingwall elevation.

Photo # 44: Crack in the stone masonry at Northeast Wingwall.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 45: Upstream view of the channel.

Photo # 46: Downstream view of the channel.

Bridge No:	03984	**Inspected by:**	Amit KC
Town:	Simsbury	**Inspected by:**	Brian Swanson
Feature Carried:	Old Drake Hill Road Bridge	**Inspected by:**	Saipavan Rallabhandhi
Feature Crossed:	Farmington River	**Date Inspected:**	June 27, 2017

Photo # 47: Northeast embankment.

Photo # 48: Southeast embankment.

ADDITIONAL BACK-UP MATERIAL

Quick fix keeps bridge in service

Welded reinforcements took load off rusting supports to extend the life of an 85-year-old simple-span through-truss bridge.

The "bridge crisis" is only one of a seemingly unending series of crises that assail us each evening when we open the newspaper or turn on the television set. One small bridge in Connecticut must be included somewhere in the bridge statistics, but it will never be the subject of more than local attention. No one was killed in a sudden collapse, nor were school children forced to dismount and walk across it while their bus followed.

The Drake Hill Road bridge over the Farmington River in Simsbury, Connecticut, is a 185-foot-long, one-lane simple span through-truss structure built in 1892. Town officials were concerned about its evident deterioration, and requested that the Connecticut Department of Transportation inspect the bridge at the town's expense.

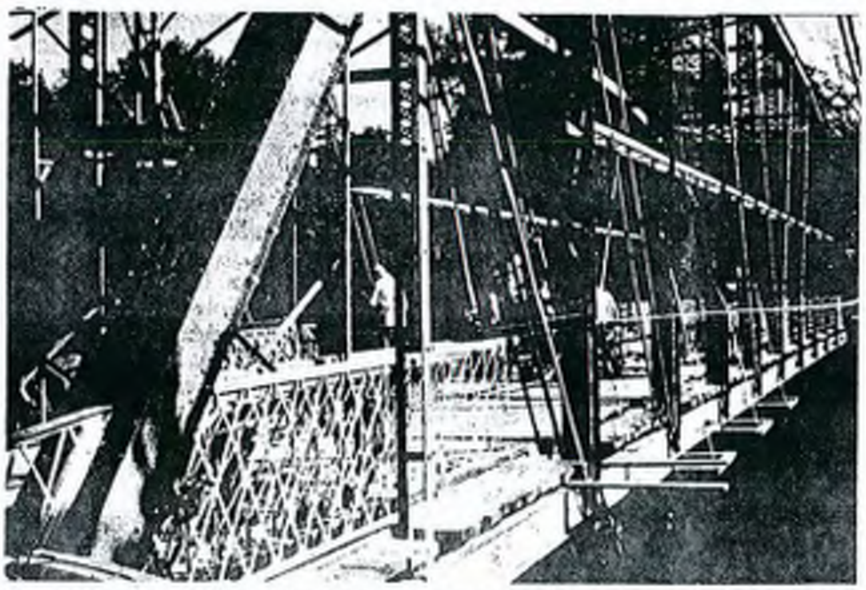
Conn DOT inspectors found severe deterioration of web truss members above the bottom chord. The deterioration was in the splash zone of deicing salts. They recommended that the bridge be declared unsafe, and the town responded by immediately closing it to traffic. This happened in June 1977.

Simsbury is a town of about 22,000, and several thousand of these residents were cut off from the rest of the town by the bridge closing. To reach them, police, fire, and emergency medical vehicles were forced to detour five or six miles.

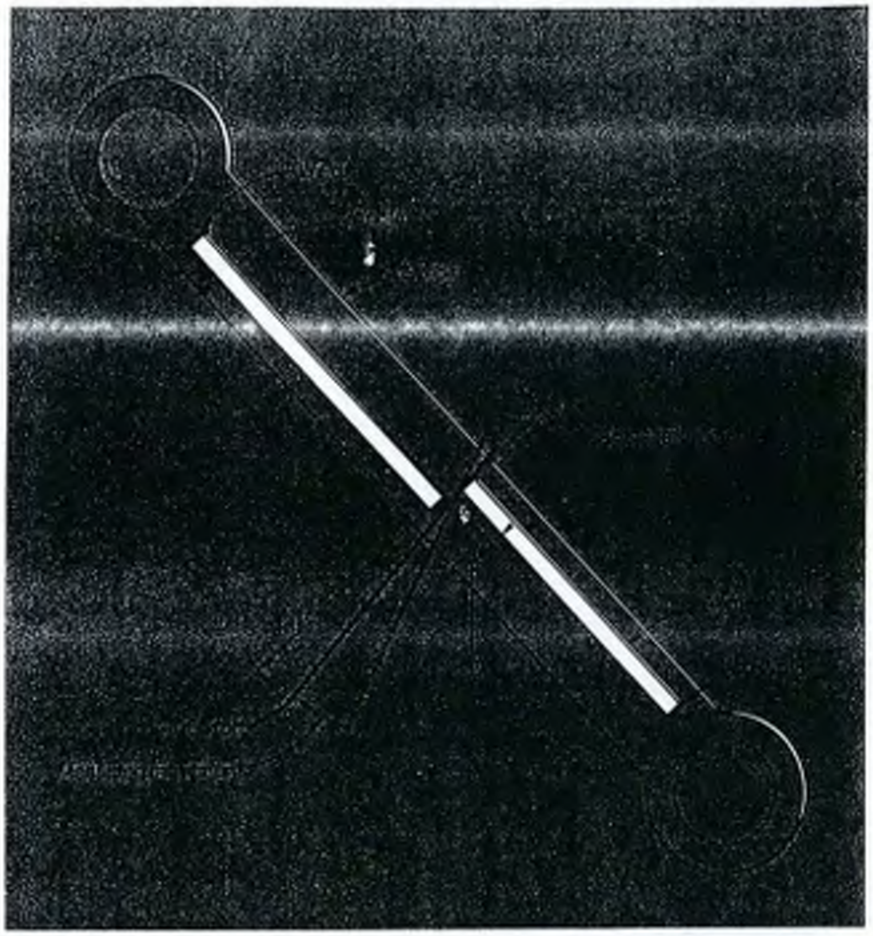
The bridge also provided an alternate route to and from Hartford, the state capital. When this alternative was available, traffic at the main intersection of the other route was controlled quite adequately by a single traffic light. After the bridge was closed, traffic at this intersection backed up heavily, morning and night, and the town was forced to assign a police patrolman to traffic control duty there.

Because of the town's critical need for the bridge, it elected to investigate the possibility that temporary repairs might keep it in service until a permanent replacement could be designed and constructed. It engaged Macchi Engineers to evaluate the bridge's

By A. J. Macchi
Macchi Engineers
Hartford, Connecticut



Prestressing bars relieved the load while the diagonals were repaired.



Measuring stress relief

Tension diagonals were paired flat eyebars. At one connection, an eyebar had completely corroded, leaving the remaining one to carry the entire load. Before the ineffective eyebar could be repaired and made to carry its share of the load, the remaining eyebar had to be relieved of some of its stress. This was done by temporarily attaching two 1¼ inch diameter bars with turnbuckles, then tightening them to take over a portion of the load.

A simple but effective method was used to determine the extent of stress reduction in the remaining eyebar. Two angles whose total length was slightly less than the length of the eyebar were laid along it. One was tackwelded at each end of the eyebar. As the load was removed from the eyebar, it shortened, moving the unattached ends of the angles closer together. Rather than attempt to measure this small movement, the consultant placed a thin rod between the unattached angle legs. Measuring the magnified movement of the rod made it easy for the engineer to calculate the shortening, and therefore the load reduction, in the eyebar.

Using this method, the stress was reduced from 24 ksi to 6 ksi. ■■

structural condition and to prepare plans for rehabilitation. Russel S. Shaw, Simsbury first selectman, and Frank Rossi, director of public works, stipulated that the bridge be reopened to traffic before the onset of winter.

The consultant found major hazards. The original vertical members were seven-inch channel sections, paired and tied with lacing. The webs of these channels were completely rusted through. The tension diagonals in the first three panels from each end were paired flat eyebars. In one location, an eyebar was completely corroded, and the entire load was carried by the remaining bar. Dead load stresses in this bar were estimated to be 24,000 pounds per square inch (24 ksi).

The vertical struts were rehabilitated by removing the steel lacing and replacing it with five-foot lengths of 10-inch channel. The channel backs were welded to the flanges of the existing seven-inch channels. This formed a box section at the bottom of each vertical strut for added strength.

Load was transferred from the rehabilitated struts to the bottom chord through pieces of W16x45 beam. The web of the W16x45 was tied with a full-penetration weld to the web of the built-up bottom chord. Its flanges were welded to the new 10-inch channels. Ultrasonic testing verified the integrity of the full-penetration welds.

Load from the diagonals was transferred to the W16x45 gusset by four plates. One was welded to the top and bottom of each diagonal on each side of the W16x45 flange. This conversion of the diagonal connections from pinned connections to rigid connections induced some moment into the connected members, but an investigation concluded that these moments were negligible.

Old steel stronger than new

Before the contractor could weld the A36 steel reinforcing to the original structural steel which had been fabricated in 1892, testing was necessary. Test specimens were composed of lacing bars that had been removed from the vertical struts and welded to A36 straps using E7018 electrodes.

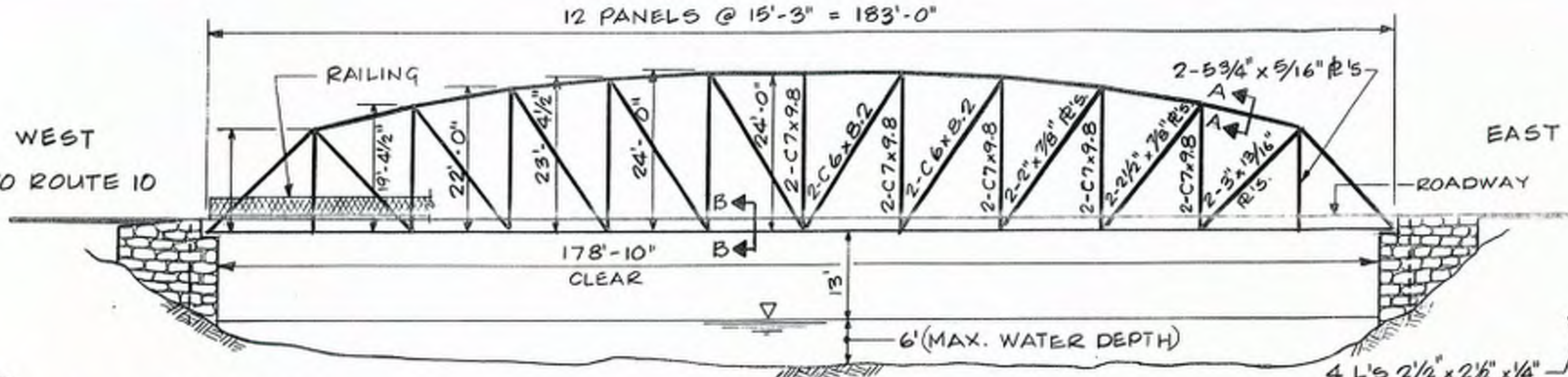
The specimens failed at tensile stresses of 61,000 and 64,000 psi. To everyone's surprise, it was the A36 steel which ruptured! When additional tensile tests were made to determine the allowable tensile stresses for the repaired bridge, specimens taken from the vertical strut channels averaged an astonishing yield strength of 53 ksi. One specimen failed at 73,000 pounds per square inch!

The entire rehabilitation project was completed in less than three weeks by Baier Construction Company, Inc., of Hartford. The total contract cost was $22,000. At the completion of the work, the bridge was load tested with an 11-ton truck.

In October 1977, just 111 days after the unexpected closing of the bridge, it was reopened to traffic, posted for a weight limit of five tons and a maximum speed of 20 mph.

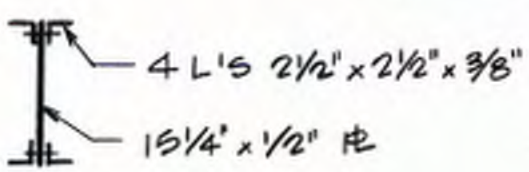
The bridge repair has eased Simsbury's problems considerably, but the town will not be adequately served until a new bridge, now under design, is actually in place. Although police cars and ambulances can now respond across the bridge, and private automobiles can use it to relieve commuter congestion, fire protection for the residents on the "wrong side of the bridge" is still inadequate. The volunteer fire fighters who protect Simsbury can take their cars directly to that area, but their pumper still must make the detour. ■■

For a free copy of this article, circle 178 on the Reader Service Card.

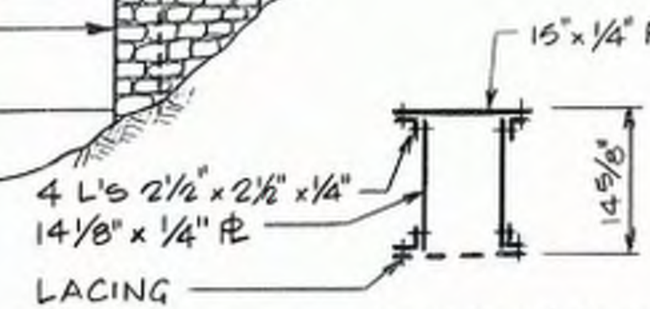


BRIDGE ELEVATION

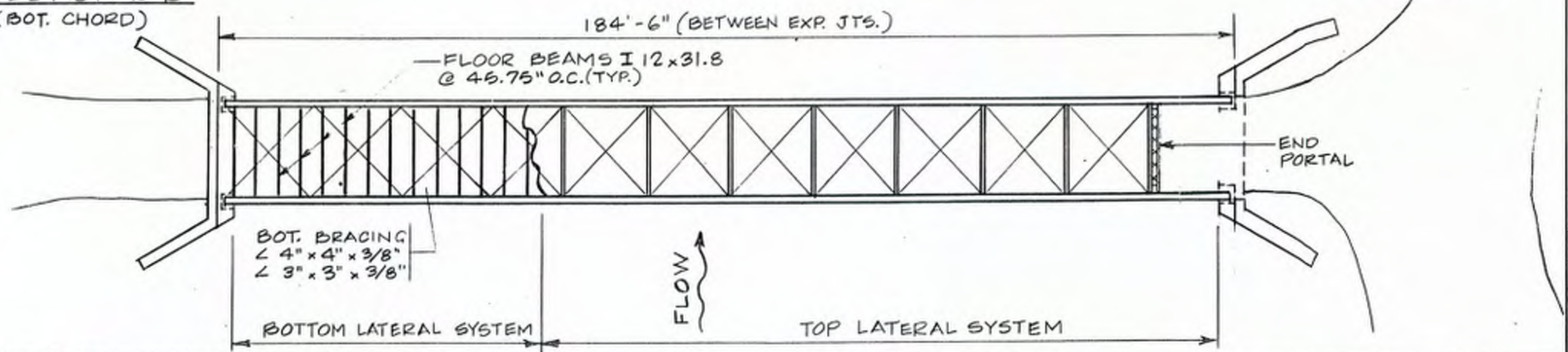
SCALE: 1" = 20'



SECTION B-B
(BOT. CHORD)



SECTION A-A
(TOP CHORD)



BRIDGE PLAN

SCALE: 1" = 20'

LOCATION PLAN

DRAKE HILL ROAD
OVER
FARMINGTON RIVER
SIMSBURY, CONN.

PLAN & ELEVATION | DWG.# 1

PLANS PREPARED BY
MACCHI ENGINEERS - HFTD, CT.
JUNE 1983

APPENDIX A - ULTRASONIC TESTING REPORT

TEAM Industrial Services, Inc.

Team Industrial Services, INC.
196 Woodlawn Road
Berlin CT 06037

(860) 828-6333
(860) 828-7488 FAX

REPORT OF NON-DESTRUCTIVE EXAMINATION

Ultrasonic Inspection - Report # 13771299

Client: GM2
Address: 115 GLASTONBURY BLVD
GLASTONBURY, CT
06033

Jobsite: OLD DRAKE HILL FLOWER BRIDGE
Address: FLOWER ST.
SIMSBURY, CT.

Contact Name: FAISAL AZIZ
Site Contact(s): BRIAN SWANSON

Components Inspected: (48) BRIDGE PIN UT INSPECTION
LOCATIONS AS FOLLOWS:
22 UPPER PINS: 11 ON SOUTH TRUSS L1 THRU L11
AND 11 ON NORTH TRUSS L1 THRU L11
26 LOWER PINS: 13 ON SOUTH TRUSS U0 THRU U12 AND 13 ON
NORTH TRUSS U0 THRU U12.

Results:
NO REJECTABLE INDICATIONS NOTED.
ALL PINS ACCEPTABLE TO UT INSPECTION.
WEAR GROOVES NOTED AND FOUND ACCEPTABLE.

Inspector's Name (Print): GREG BENWAY
Level: II
Inspector's Signature: [signature]
Date: ~~6/28/17~~ 6/29/17

Specification: ASME V
Purchase Order # PROJECT # 40212.00
Procedure: 22-H-800 REV.0
Acceptance: REPORT FINDINGS TO CLIENT / NO CRACKS ALLOWED
Ultrasonic: [X] A-Scan [ ] B-Scan [ ] C-Scan [X] Contact [ ] Immersion
Equipment: Mfg: KB Model: USN-60 S/N: 00R286
Transducer: Mfg.: UTX Model: CX-352 S/N 0708222 Angle: 0°
Size: .500" Ø Frequency: 2.25 MHz
Mfg.: KBA Model: GAMMA S/N 42746 Angle: 15°L
Size: .500" Ø Frequency: 2.25 MHz
Calibration Block: Type: ACTUAL PIN W/NOTCHES Material: STEEL S/N: N/A
Scanning: [X] Manual [ ] Automatic Couplant: ULTRASONIX Batch # 506 02
Pattern: PARALLEL PATH Scanning Speed < 4 IPS % Overlap 50

TEAM	Quality System Supplement Corporate	FORM 103.10 Rev: 8 Page 1 of 1

VISION ACUITY RECORD

Name: Gregory Benway **Employee #:** 655451

Vision Acuity Results

Near Vision Requirements
Required for All Personnel

	Left Eye		Right Eye	
Uncorrected	J -	@ "	J -	@ "
Corrected	J - 1	@ 16"	J - 1	@ 16"

Check one of the following:

☐ **Satisfactory Near Vision Without Corrective Lenses** (J-1 minimum required in at least one eye).

☒ **Satisfactory Near Vision With Corrective Lenses** (J-1 minimum required in at least one eye).

☐ **Unsatisfactory Near Vision**

Check if applies:

☒ Reading card has been verified IAW 8.1.2.1 of 33.G.103-S8 for personnel certifying to 33.G.103-S4 (CP-189/ASME XI)

Distance Vision Requirements
Branch is Required to Determine Applicability

	Left Eye		Right Eye	
Uncorrected	20/20	Snellen	20/20	Snellen
Corrected	20/	Snellen	20/	Snellen

Check one of the following:

☒ **Satisfactory Distance Vision Without Corrective Lenses** (20/30 Snellen minimum required in at least one eye).

☐ **Satisfactory Distance Vision With Corrective Lenses** (20/30 Snellen requirement in at least one eye).

☐ **Unsatisfactory Distance Vision**

☐ **N/A** (Branch determined non-applicable by Code or contractual agreements)

Color Vision Requirements
Required for All Personnel (Use Form 103.10a "Color Vision Examination Charts")

☒ **Satisfactory** – Can differentiate and distinguish between colors or shades of gray used in method(s)

☐ **Unsatisfactory** – Cannot differentiate and distinguish between colors or shades of gray used in method(s)

Deficiencies/Limitations: ☒ N/A

☐ Limitations reviewed and approved by Responsible Level 3 for NAS410 personnel.

Responsible Level 3 Signature ______________________

Brightness Discrimination Requirements
Branch is Required to Determine Applicability

Check all that apply:

☒ N/A ☐ Satisfactory ☐ Unsatisfactory ☐ Corrective Lenses Required

Remarks/Restrictions:

Administered By:		Reviewed & Approved By:	
Signature:	[signature]	NDT Level III Signature:	[signature]
Name:	Jeff Watkins	NDT Level III Name:	Jeff Watkins
Location:	1237/Hartford	Date:	11/18/2016
Date:	11/18/2016	Next Examination Date:	11/18/2017

TEAM® Industrial Services

Personnel Qualification and Certification

Employee Name: Gregory S. Benway **Employee ID#:** 655451

Vision Acuity Expiration Date: 11/18/2017

METHOD	LEVEL	DATE CERTIFIED	EXPIRATION DATE	GENERAL-I/II METHOD-III SCORE	SPECIFIC SCORE	PRACTICAL SCORE	COMPOSITE SCORE %	EXPERIENCE HOURS	TRAINING HOURS	LIMITED TO	COMMENTS
UT	II-L	1/4/2016	1/4/2019	95	90	96	93.7	72276	80	Contact: All Angles; Immersion: 0 Degree	
UT	II-L	1/4/2016	1/4/2019	95	90	96	93.7	72276	80	Contact: All Angles; Immersion: 0 Degree	MIL-STD-2132 Inspector, Contact Only

The above named individuals qualification history has been reviewed and found to be acceptable IAW TEAM's requirements for certification; 33.G.103-S1, SNT-TC-1A-2011 and earlier editions (1992, 2001 and 2006), as published by the American Society for Nondestructive Testing and/or any additional certification standards listed in the comments section above.

Certifying Authority: **Date:** 11/21/2016

Charles M. Lee
Corporate Level III
ASNT Cert # 58053

APPENDIX B - PAINT REPORT

Coating Condition Assessment of the Drake Hill Road Bridge

GM2 Associates, Inc

Prepared for:

Mr. Manish K. Gupta
Executive Vice President
GM2 Associates, Inc.
115 Glastonbury, Blvd.
Glastonbury, CT 06033

Prepared by:

KTA-TATOR, INC.
115 Technology Drive
Pittsburgh, PA 15275
(412) 788-1300
(412) 788-1306 – fax
www.kta.com

Robert Lanterman

Robert Lanterman
Coatings Consultant

July 21, 2017

RSL/MPR/AGB:lmb
JN370441
Drake Hill Road Bridge Report.doc

TABLE OF CONTENTS

INTRODUCTION

As authorized by an agreement (Proposal No. 17792) between GM2 Associates, Inc. (GM2) and KTA-Tator, Inc. (KTA), KTA has completed a coating condition assessment of the Drake Hill Road Bridge over the Farmington River located in Simsbury, Connecticut.

The purpose of this assessment was to determine the condition of the existing coatings on the structure in order to develop a maintenance painting strategy. This report contains the results of the field inspection and testing, laboratory analysis of field samples, a discussion of results and recommendations. Photographs depicting typical conditions found during the field visit are included as part of this report.

Photo 1 –General view of bridge.

SUMMARY

The existing coating system on the Drake Hill Road Bridge is in fair to good condition overall. The degree of coating failure typically ranged from 0. 3% to 3% of the surface area. Randomly scattered areas of spot corrosion were observed throughout the structure. Based on the percentage of visible corrosion, the coating is at a point where maintenance painting is economically advantageous. Spot repair of the corroded areas is recommended. Spot repairs will result in a patchwork appearance (of new vs. old paint color) and may not be acceptable based on aesthetics. If aesthetics are critical, then an overcoat can be applied to the entire structure. Application of a test patch is always strongly recommended prior to overcoating the entire area.

BACKGROUND

The Drake Hill Road Bridge is owned and maintained by the town of Simsbury. The bridge is over the Farmington River located in Simsbury, Connecticut. The bridge design is a Parker through truss. The bridge was erected in 1892 and has a length of 183 feet. The bridge no longer carries vehicular traffic and is used as a pedestrian/bicycle bridge. It is also referred to as the "Flower Bridge" as it is decorated with flower boxes and hanging baskets by a group of volunteers. Specifications from 1995 indicate the bridge was to have been blasted and painted with a zinc rich primer, epoxy intermediate coat, and urethane top coat. KTA was contacted to conduct a coating condition assessment and provide recommendations for future coatings maintenance work.

FIELD VISIT

The field visit to the Drake Hill Road Bridge was conducted by Mr. Jeff Towill of KTA on June 28, 2017. The bridge steel members were accessed from the road deck, a safety boat in the river, and using an extension ladder. The tests and inspections performed, including the observations made and measurement findings from the investigation, are discussed herein.

The following methods, standards, and practices were used to evaluate the existing coating and underlying substrate conditions.

- **Visual** – A visual assessment of the coated surfaces was conducted to determine the type, extent, and location of coating breakdown and corrosion on the structure. Visual Standard SSPC VIS 2, "Standard Method for Evaluating Rusting on Painted Steel Surfaces," was used.

- **Coating Thickness** – The dry film thickness was determined using a Positector 6000. The Positector 6000 is a portable, battery operated, digital coating thickness gage that non-destructively measures non-magnetic coating thickness over ferrous substrates using a magnetic principle. Gage calibration was verified prior to and after use with the National Institute of Standards and Technology (NIST) thickness standards.

- **Adhesion** – Adhesion testing was conducted in accordance with ASTM D 3359, "Measuring Adhesion by Tape Test," Method A. This method involves cutting an "X" through the coating down to the substrate using a razor knife, followed by the application of pressure sensitive tape. The tape is then rapidly removed from the X-cut and the adhesion is then rated according to the amount of coating removed using an ASTM rating scale. Typical ratings of 4A to 5A are considered by KTA to represent good adhesion, 2A to 3A represent fair adhesion, while 0A to 1A represent poor adhesion. Coating adhesion was also assessed in general accordance with ASTM D 6677, "Standard Test Method for Evaluating Adhesion by Knife." These methods involve scribing the coating with a knife and evaluating the adhesion in accordance with an ASTM rating scale. The location of the forced separation within the system is also reported.

- **Paint Samples** – Samples were removed for further laboratory examination to determine the generic coating type, to measure the number and thickness of coats, and to check the presence and amount of heavy metals (lead, cadmium, and chromium) in the lab.

- **Photographs** – Photographs of typical coating conditions were taken and are included as part of the report.

Visual Inspection

General

For purposes of the visual inspection, the bridge was broken down into simple component members (i.e. truss members, floor stringers, guard rails, cables and towers). Overall, the visual coating condition was rated fair to good. The overall rate of coating deterioration (spot rust, pinpoint rust, and cracks in the existing coating) was minimal when compared to all the steel surfaces. Coating blisters or application defects such as excessive runs or sags were minimal. There were isolated spot areas of corrosion. Areas of graffiti were found on the bridge at the abutments. A summary of the typical coating condition on the various structural members of the bridge is presented below.

Truss Members

Spot corrosion on the truss members typically ranged from approximately 0.3% to 1% of the surface area. There were several isolated areas with spot corrosion on the North truss ranging from 1% to 3%. Areas of spot corrosion were scatters across the length and most often occurring at the connections. Conditions were typical for upper and lower truss chords, verticals, diagonals, and bracing members. See Photographs 2 through 11 below.

Photo 2 –Typical view of truss.

Photo 3 –Spot corrosion on truss connection.

Photo 4 –Typical view of interior truss top chord.

Photo 5 –Bird nest in truss top chord.

Photo 6 –Spot corrosion at truss connection.

Photo 7 –Spot corrosion on truss bracing.

Photo 8 – Spot corrosion on truss lower chord.

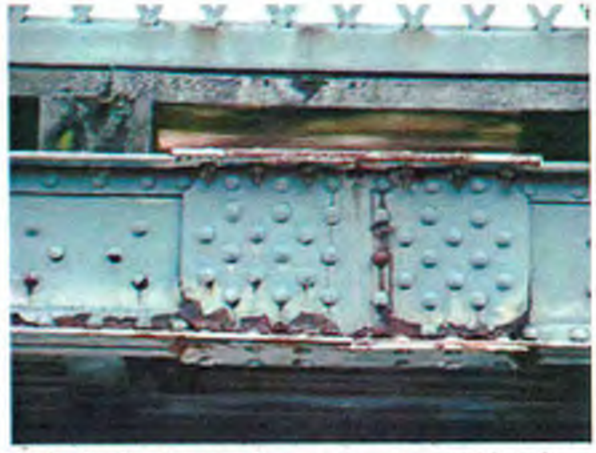


Photo 9 –Spot corrosion on truss lower chord.

A white discoloration was observed on the lower truss and adjacent steel. The discolored areas had the appearance of salt deposits. The discoloration may also be caused by runoff from the flower boxes.

Photo 10 –White stain on lower chord.

Photo 11 – White stain on lower chord.

Floor Beams

Coating deterioration on the floor beams ranged from approximately 0. 3% to 1% of the surface area. Higher levels of corrosion were typically observed at the connections with the lower chords.

Photo 12 –Typical condition of floor beams.

Photo 13 – Typical condition of floor beams.

Photo 14 – Typical condition of floor beams.

Photo 15 – Typical condition of floor beams.

Dry Film Thickness

Total coating system dry film thickness measurements were obtained on the existing coating system. The following table, Table 1 – Dry Film Thickness Measurements, summarizes the range of the thicknesses obtained with a Positector 6000, magnetic-type dry film thickness gage:

Table 1 – Dry Film Thickness Measurements

Member	Minimum (mils)	Maximum (mils)	Average (mils)
North Truss	9.6	17.4	14.9
North Verticals	6.7	15.7	11.5
North Member Diagonals	4.3	26.4	11.2
North Rail and Lattice	7.0	20.1	12.7
South Truss	6.0	17.7	9.6
South Verticals	4.4	15.1	11.2
South Rail and Lattice	6.7	21.7	13.7
Floor Beams	5.4	13.2	8.9
Floor Beam Bracing	4.7	13.0	9.8

Adhesion

Adhesion testing was performed in accordance with ASTM D3359 Method A (X-cut). Adhesion was typically rated 4A (good) in all areas tested. Coating adhesion was also assessed in general accordance with ASTM D 6677. Adhesion was typically rated 8 to 10 (good).

Substrate Examination

The substrate was examined at the sample and adhesion test areas. The substrate appears to have been abrasive blast cleaned.

Chloride Testing

Two chloride tests were performed using a Chlor-Test kit from Chlor Rid International. Test #1 was performed over intact coating on the bottom surface of the lower south truss in an area that had a white discoloration that could be wiped off with a wet cloth. The result was 10 $\mu g/cm^2$. Test #2 was performed nearby on the top of the bottom flange in an area where the coating was removed and the steel was rusting. Loose rust and debris was removed with a scraper. The result was non-detectable.

LABORATORY INVESTIGATION

The laboratory investigation consisted of infrared spectroscopy, visual and microscopic examination, and lead, cadmium, and chromium testing. A description of the test methods employed and results of the investigation are provided below.

Visual and Microscopic Examination

Visual and microscopic examination of the samples was conducted using a Keyence VHX-5000 digital microscope with magnification to 200X. The samples had between two and three coating layers. Table 2 - Coating Thickness Data, below lists the magnification at which each cross-section was examined, the number of layers observed, the color of the individual layers, and the minimum and maximum thickness of the individual layers, measured in mils.

Table 2 - Coating Thickness Data

Sample ID	Sample Description	Magnification	Layer/Coat	Thickness (mils)
KTA-1	West Portal	150X	*Two Coating Layers* Top – Green Bottom – dark gray	 2.6 – 3.2 10.3 – 11.0
KTA-2	Floor beam, first panel point	150X	*Three Coating Layers* Top – green Dark gray Bottom – Metallic dark gray	 3.0 – 3.2 2.9 – 4.3 7.9 – 8.9
KTA-3	South Lower Truss Chord	150X	*Three Coating Layers* Top – Green Dark gray Bottom – metallic dark gray	 5.7 – 7.6 4.3 – 5.5 3.0 – 4.7

Infrared Spectroscopy

Infrared spectroscopic analysis was performed using a Mattson Galaxy Model 3020 Fourier transform infrared spectrometer. This technique involved combining sample scrapings with potassium bromide powder and forming pellets under high pressure. The pellets were then placed in the optical path of the spectrometer and spectra were obtained over the range of 4000 to 400 cm^{-1}. Three spectra were obtained and are appended.

The green topcoat scrapings when combined with potassium bromide of Samples KTA-1 (West Portal), KTA-2 (Floor Beam) and KTA-3 (South Lower Truss) were consistent with a urethane resin. The urethane resin was evidenced by the doublet near 1730/1690 cm^{-1}, and spectral bands near 1520, 1460, 1240, and 1160 cm^{-1}. Talc was evidenced by the bands near 3600, 1020, 670 and 530 cm^{-1}.

Lead, Cadmium and Chromium Testing

Samples KTA-1 thru KTA-3 were tested for lead, cadmium, and chromium in accordance to EPA Method 6010C and EPA Method 3050B. The testing was performed by Schneider Laboratory, Inc., in Richmond, VA. The lead, cadmium and chromium results (ppm by weight) are shown in the table below.

Schneider Laboratories Testing Designation

Sample ID	Sample Description	Total Lead, ppm	Total Cadmium, ppm	Total Chromium, ppm
KTA-1	West Portal	177	ND*	160
KTA-2	Floor Beam – First Panel	17.4	ND*	466
KTA-3	South Lower Truss	49.0	ND*	315

*ND – Test results were below detectible limits of test

DISCUSSION

General Discussion on Maintenance Painting

The purpose of this coating assessment was to assess the condition of the existing coatings on the structures and make recommendations for maintenance painting. Many factors affect the service life of a coating system. These include the type of coating originally applied, the type and quality of surface preparation, service environment, number of coats and film thickness, and the history of maintenance painting activities.

If a coating has provided satisfactory corrosion prevention and remains in relatively good condition, it is cost effective to extend the life of the system through overcoating, retaining as much of that original coating as possible. When the coatings are in poor condition, a "full removal" strategy is used, which removes all existing coatings. This strategy effectively places the bridge at the beginning of a new maintenance painting cycle. Little work will be required for at least 10 years, and then, it should involve only minor touch-up. This strategy, while safe and effective, is also expensive. A discussion of the various types of maintenance painting activities follows.

Maintenance painting options for bridge structures fall into four main categories: (1) deferral of maintenance, (2) spot repairs, (3) spot repairs with full overcoats, and (4) complete coating removal and replacement.

Each of these options is progressively more complex, and requires progressively more work. Correspondingly, each option also offers greater long-term protection to the structure, but at additional costs. When paints containing hazardous metals are present, the issues associated with removing these paints impact the decision-making process.

Deferral of Maintenance

Maintenance painting can be deferred if the existing coating system is in good condition, if the service life of the structure is limited, or there is some other benefit for postponing the work. If extensive corrosion is found and maintenance painting is deferred for a period of time, the level of surface preparation required to properly prepare the surface increases correspondingly, and if left unattended for too long, total removal will ultimately be required. In some cases, when the structure is corroding extensively, but is still structurally sound, painting is deferred because the highest level of surface preparation (abrasive blast cleaning) is already needed, whether performed today or several years from now. The strategy in this case is to allocate the money to repair coatings on other structures that are not so badly deteriorated in order to stop the corrosion from propagating to the point that total removal is the only option for those structures as well.

Spot Repairs

Spot repairs, as the name suggests, involves surface preparation and coating application only to the individual spots of corrosion or coating breakdown. The amount of coating being removed is minimized, reducing the impact of hazardous materials handling, containment, and worker protection when toxic metals are present. Spot repairs also serve to repair the existing coating film only where it is needed, repairing the corroded areas, and stopping the propagation of the breakdown. Coatings in essentially any condition may be spot repaired, but it is only practical when the level of breakdown is minor and somewhat isolated and covers a small percentage of the surface (e.g., 1 or 2%). A disadvantage of this approach involves aesthetics. The repair spots are clearly visible.

A variation of this type of localized repair includes zone or area repairs. This involves surface preparation and coating application over a larger area that exhibits more concentrated levels of breakdown, but the work is limited to those areas. For example, the bearing areas of girders are often zone painted on either side of an expansion joint, without any significant painting on the rest of the structure.

Spot Repairs with Full Overcoat(s)

The application of a full overcoat serves two primary purposes: the additional coat provides additional barrier protection and helps to seal minor defects that are not apparent when conducting spot repairs. It also offers an improved appearance when compared to spot repairs. The addition of the overcoat also adds complexity and cost to the overall project. The complexity increases because a contractor must now gain access to all areas of the structure to apply the full coat. The existing surface must also be thoroughly cleaned (i.e., power washed) to remove chalk and surface debris. The adhesion of the existing coating must also be good and sound; otherwise the stresses imparted by the overcoat can cause disbonding of the existing system, especially under freeze/thaw conditions. In some cases, two full overcoats are applied.

This strategy is typically used when the amount of visible corrosion and coating deterioration covers less than 15% of the surface.

Total Coating Removal and Replacement

Total removal and replacement is the final option for maintenance painting and is the costliest option, especially when removing existing coatings that contain toxic metals. However, it offers the greatest opportunity for long-term protection. All of the mill scale, rust, and paint are completely removed and a new system with a new design life is applied. This method also provides the most pleasing appearance.

When total removal and replacement is performed, a new maintenance cycle begins. As the coatings age and weather, isolated spot repairs will be required. Several spot repairs may be made to the individual structure until a full overcoat is necessary. More spot repairs may then be made and additional overcoats applied until extensive corrosion develops, significant coating breakdown occurs, or the mechanical properties of the coatings (e.g., the adhesion) degrade to the point where additional work (spot touch-up or overcoating) is no longer practical. At this time, complete removal may again be required, but only after the maximum effective life of the original coating system has been extended through the planned maintenance activities.

RECOMMENDATIONS

The existing coating system on the Drake Hill Road Bridge is in fair to good condition overall. The degree of coating failure typically ranged from 0. 3% to 3% of the surface area. Randomly scattered areas of spot corrosion were observed throughout the structure. Based on the percentage of visible corrosion, the coating is at a point where maintenance painting is economically advantageous. When maintenance work is performed, there are two recommended options.

Option 1 – Spot Repairs: Under this option, surface preparation on areas of spot corrosion/coating failure would be performed in accordance with SSPC SP-3, "Power Tool Cleaning." Vacuum shrouded power tools should be used to minimize the containment requirements, but nuisance tarps will be required to capture the paint chips that are dislodged by the tools, but not captured by the vacuum.

The spot repair coating system should involve three coats, consisting of an epoxy mastic prime coat, an epoxy intermediate coat, and a polyurethane finish coat, with stripe coats of the primer and intermediate coats applied to edges, crevices, rivets, and other irregular surfaces. One benefit to this option would be a reduced total project cost for maintenance painting. Spot repairs will leave a patchwork like appearance and may not be acceptable based on aesthetics.

Option 2 – Spot Repairs with Full Overcoat: Under this option, surface preparation on areas of spot corrosion/coating failure would be performed the same as in option 1. Based on the current assessment data and visual observations, in order to apply an

overcoat, all surfaces must also be cleaned by pressure washing to remove chalk, chlorides, dirt, and other debris.

The overcoat system should involve two coats, consisting of a penetrating sealer tie coat and a polyurethane finish coat. Stripe coats of the intermediate should be applied to edges, crevices, rivets, and other irregular surfaces. Application of a test patch is always strongly recommended prior to overcoating the entire area.

Chloride Remediation

It is imperative that residual chloride levels (salt contamination) be maintained at acceptable concentrations prior to coating. The level of salt contamination when applying organic coatings should be kept below 7 $\mu g/cm^2$. The specifications should require testing after surface preparation has been performed and prior to painting. In many instances, chloride contamination can be reduced to acceptable levels by pressure water cleaning and/or abrasive blast cleaning with a combination of finely graded and coarser abrasive media. Chloride removal agents can also be added to the pressure washing water. Other options include abrasive blast cleaning the steel, and allowing it to rust over night followed by re-blast cleaning.

Dealing with Lead

Laboratory testing reported detectable concentrations of lead present in the existing coatings on the bridge. The OSHA Lead in Construction Standard (29 CFR 1926.62) requires that controls be implemented if any detectable concentrations of lead are present. The OSHA Compliance Directive issued for the OSHA Lead in Construction Standard, Instruction CPL 2-2.58, states that if an employer has appropriately tested for lead (e.g., tested all layers of paints or coatings that may be disturbed) utilizing a valid detection method, and found no detectable levels of lead, then the standard does not apply. Paints with detectable concentrations of lead require the contractor performing the work to implement interim controls and assess actual employee exposures during the work in accordance 29 CFR 1926.62. Based on the lead results provided by the laboratory testing, 29 CFR 1926.62 is invoked during any activities that disturb the paint (e.g., abrasive blast cleaning, scraping, burning, and grinding).

It should be noted that other hazardous metals are also present in the existing coating. Any disturbance of paint containing heavy metals in addition to lead must be performed in accordance with the requirements of the applicable OSHA standards.

In addition, containment will be required for the protection of the environment and the public, and the hazardous waste must be properly managed.

Opinion of Probable Coating Replacement Costs

A cost analysis was prepared for the recommended maintenance options. This analysis involved making various assumptions, based upon KTA and industry experience, of how a contractor might staff and proceed with the aforementioned recommendations. Crew sizes,

production rates, material and equipment requirements are evaluated and man-days and project-days are calculated. From the estimated project duration, costs associated with labor, materials, and equipment are factored in and the costs are developed. Overhead and profit are added as a multiplier to the base cost. For the purposes of this opinion of probable coating cost, labor was considered to be prevailing wage and equipment was calculated with rental rates.

Production days were calculated from the square footage of paintable steel surfaces and an allocated production rate. The surface areas for the bridge were calculated from the provided drawings. The Drake Hill Road Bridge surface area is estimated to be 22,000 total square feet. Finally, a variance multiplier is used on the final cost to develop a range of anticipated bid prices. This multiplier allows for the variations in contractor bidding techniques, new technology, and scheduling of the work within the painting season. The opinion of probable cost to perform spot repairs with a full overcoat ranges from $244,300 to $295,700 and the cost to perform spot repairs only ranges from $50,100 to $60,700. Spot repairs with a full overcoat would take approximately one month for total production time while spot repairs only would take approximately one week.

Opinions of probable construction costs are prepared on the basis of KTA's experience and qualifications and represent KTA's judgment as field professionals generally familiar with the industry. However, since KTA has no control over the cost of labor, materials, equipment, or services furnished by others, over contractor's methods of determining prices, or over competitive bidding of market conditions, KTA cannot and does not guarantee that proposals, bids, or actual construction costs will not vary from KTA's opinions of probable cost.

APPENDIX B: COMPUTATIONS

Timber Decking:

Span Length =	L_{timber} =	44.0 in	
Bridge Width =	W_{bridge} =	16.0 ft	
No. of Planks =	N_{plank} =	24	
Plank Width =	W_{plank} =	8.0 in	
Plank Depth =	D_{plank} =	3.0 in	
Timber Weight =	w_{timber} =	60.0 pcf	
Dead Load (Deck, per Floor Beam) =	DL_{plank} =	880.0 lb	

Railing:

Railing Length =	L_{rail} =	183.0 ft	
Railing Weight =	W_{rail} =	37 plf	(2 - TS 4x4x0.25) & (2 - L1.5x1.5x0.25)
No. of Floor Beams =	N_{plank} =	48	
Dead Load (Railing, per Floor Beam) =	DL_{nailer} =	140.6 lb	

Timber Nailer:

Nailer Width =	W_{nailer} =	9.0 in	
Nailer Depth =	D_{nailer} =	9.0 in	
Dead Load (Nailer, per Floor Beam) =	DL_{nailer} =	540.0 lb	

Floor Beam:

Beam Length =	L_{beam} =	17.3 ft	
Floor Beam Weight =	w_{floor_beam} =	31.8 plf	
Dead Load (Floor Beam, per Floor Beam) =	DL_{nailer} =	551.2 lb	

Longitudinal Bracing:

Long. Bracing Length =	$L_{bracing\_L}$ =	293.3 ft	
No. of Beams Braced =	N_{braced_long} =	41	
Long. Bracing Weight =	$w_{bracing\_L}$ =	12.8 plf	L4x4x1/2
Dead Load (Long. Bracing, per Floor Beam) =	DL_{nailer} =	100.7 lb	(Center 41 Floor Beams) (+10% added for plate connections)

Diagonal Bracing:

No. of Beams Braced (West End, per Dia.)=	N_{braced_dia} =	2	
No. of Beams Braced (Center, per Dia.)=	N_{braced_dia} =	2	
No. of Beams Braced (East End, per Dia.)=	N_{braced_dia} =	2	
West End Bracing Length =	L_{w_end} =	22.7 ft	
Center Bracing Length =	L_{center} =	21.7 ft	
East End Bracing Length =	L_{e_end} =	20.5 ft	
Diag. Bracing Weight =	$w_{bracing\_D}$ =	9.8 plf	L4x4x3/8
Dead Load (West End, per Floor Beam) =	DL_w =	244.8 lb	(+10% added for plate connections)
Dead Load (Center, per Floor Beam) =	DL_c =	117.0 lb	(+10% added for plate connections)
Dead Load (East, per Floor Beam) =	DL_e =	221.3 lb	(+10% added for plate connections)

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Pedestrian:

Pedestrian Loading = w_{ped} = ***90.0 psf***

Live Load (per Floor Beam) = LL_{ped} = ***5280.0 lb***

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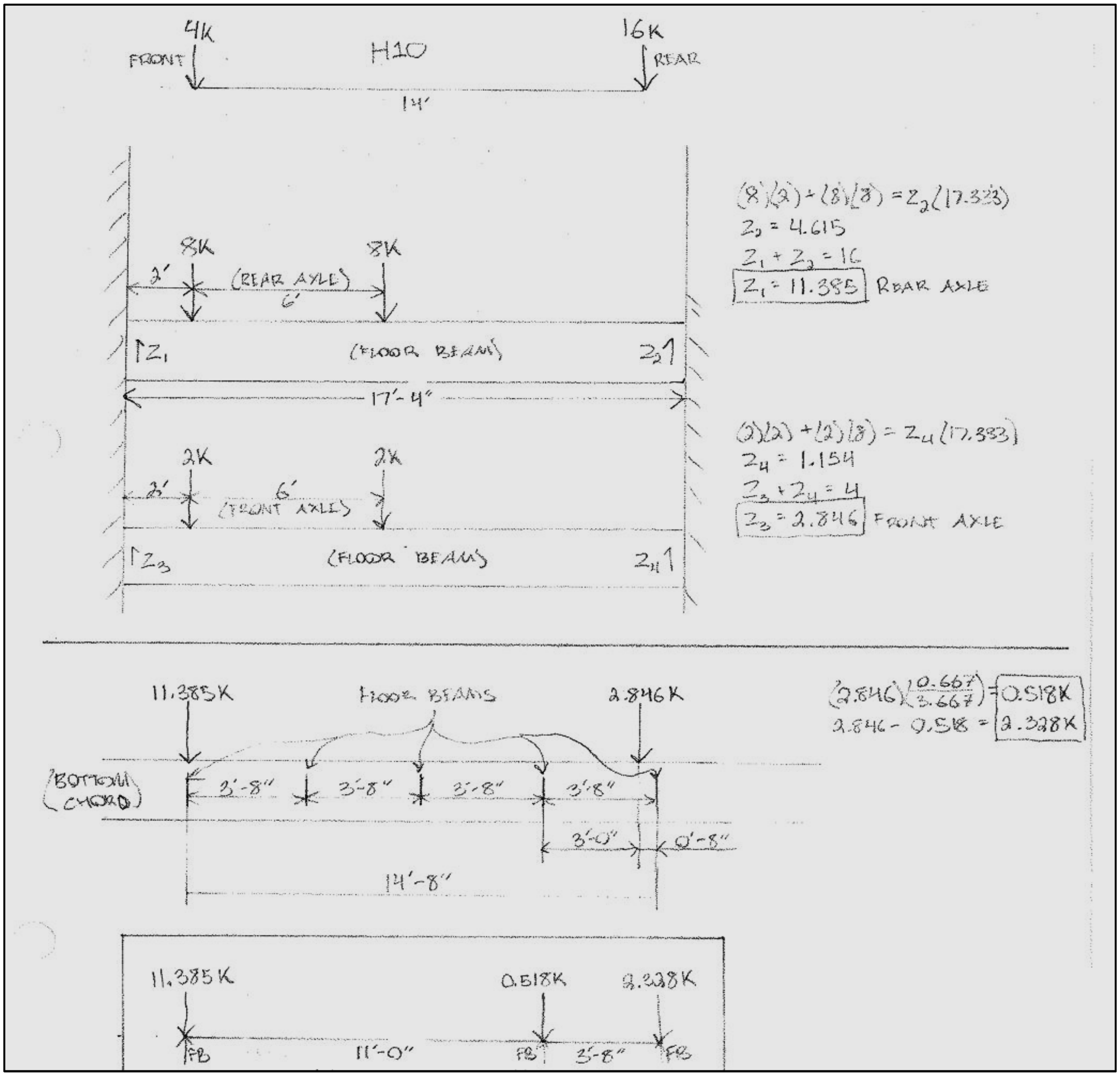
Pedestrian:

Rear Axle (per Floor Beam) = $LL_{H10, RA}$ = **11385 lb**

Front Axle, +3 FBs Away (per Floor Beam) = $LL_{H10, FA1}$ = **518 lb**

Front Axle, +4 FBs Away (per Floor Beam) = $LL_{H10, FA2}$ = **2328 lb**



Bay	MDL (ft-lb)	MLL-PED (ft-lb)	MLL-H10 (ft-lb)	RF (Ped)	RF (H10)
1	5899.2	12723.3	42975.2	6.89	2.04
2	3237.1	6246.9	37242.5	14.34	2.41
3	3747.8	7480.5	37097.2	11.93	2.41
4	4453.8	8480.7	37123.1	10.46	2.39
5	4686.5	8852.8	37294.9	10.00	2.37
6	4651.9	8665.3	37170.5	10.22	2.38
7	4651.7	8665.3	37170.5	10.22	2.38
8	4687.5	8852.8	37294.9	10.00	2.37
9	4449.8	8480.7	37123.1	10.46	2.39
10	3765.4	7480.5	37097.2	11.93	2.40
11	3160.8	6246.9	37242.5	14.35	2.41
12	6058.9	12723.3	42975.2	6.88	2.04

Section Modulus =	S_x =	***59.4 in^3***	AISC Table 1-3, S15x42.9 (Approx.)
Bending Yield Strength =	F_y =	***38000.0 psi***	MBE Table 6A.6.2.1-1
Condition Factor =	ϕ_C =	***0.95***	MBE Table 6A.4.2.3-1
System Factor =	ϕ_S =	***0.9***	MBE Table 6A.4.2.4-1 (riveted member)
Resistance Factor =	ϕ_f =	***1.0***	LRFD 6.5.4.2
	ϕ_{check} =	***0.86***	
	ϕ_{C_S} =	***0.86***	
Slenderness Ratio Check=	λ_f=	***3.33***	(b = 2.5", t = 0.375") (LRFD Eq. 6.10.8.2.2-1)
	λ_{pf}=	***9.20***	(LRFD Table C6.10.8.2.2-1; 50 ksi (conservative))
	Slenderness=	***Non-Slender***	
Nominal Resistance, Flexure Stress=	F_{n_f} =	***38000.0 psi***	(LRFD Eqs. 6.10.8.2.2-1 & 6.10.8.3-1)
Nominal Resistance, Flexure =	R_{n_f} =	***188100 ft-lb***	
DL Load Factor =	γ_{DL} =	***1.25***	
LL Load Factor =	γ_{LL} =	***1.75***	
Flexural Capacity =	C_f =	***160826 ft-lb***	MBE Eq. 6A.4.2.1-1

Controlling Rating Factor = RF_f = ***2.04***

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BY	BAW	DATE	12/14/17	SHEET	1	OF	1
CHKD BY	TPL	DATE	12/14/17	PROJECT	Simsbury		
SUBJECT	Bottom Chord Tension - RF						

				Pedestrian		H10	
Bay	DL (kips)	LL, Ped (kips)	LL, H10 (kips)	RF_T	RF_{TF}	RF_T	RF_{TF}
1	36.1	56.3	15.0	3.72	3.52	13.93	13.19
2	36.1	56.3	15.0	3.72	3.52	13.93	13.19
3	54.1	83.8	22.1	2.35	2.21	8.90	8.40
4	64.3	99.4	26.0	1.91	1.79	7.28	6.85
5	71.8	110.7	28.9	1.66	1.56	6.37	5.98
6	76.5	117.9	30.7	1.53	1.44	5.89	5.52
7	76.5	117.9	30.7	1.53	1.44	5.89	5.52
8	71.8	110.7	28.9	1.66	1.56	6.37	5.98
9	64.3	99.4	26.0	1.91	1.79	7.28	6.85
10	54.1	83.8	22.1	2.35	2.21	8.90	8.39
11	36.1	56.3	15.0	3.72	3.52	13.93	13.18
12	36.1	56.3	15.0	3.72	3.52	13.93	13.18

Tension	Area of Element =	A_g =	13.34 in^2	0.5x15 Plate + 4 2.5x2.5x0.3125 L
	Yield Strength =	F_y =	38 ksi	
	Condition Factor =	ϕ_C =	0.95	MBE Table 6A.4.2.3-1
	System Factor =	ϕ_S =	0.9	MBE Table 6A.4.2.4-1 (riveted member)
		ϕ_{check} =	0.86	
		ϕ_{C_S} =	0.86	
	DL Load Factor =	γ_{DL} =	1.25	
	Resistance Factor, Tension =	ϕ_f =	0.95	LRFD 6.5.4.2
	LL Load Factor =	γ_{LL} =	1.75	
Tension & Fracture	Area of Holes in Element =	A_H =	1.88 in^2	=0.75"*0.375*4+0.75"*0.5*2 (3/4" dia. holes thru Ls and web)
	Net Area of Element =	A_n =	11.47 in^2	
	Tensile Strength =	F_u =	50 ksi	
	Reduction Factor for Holes =	R_p =	1.00	
	Resistance Factor, Fracture =	ϕ_u =	0.80	
	Reduction Factor, Shaer Lag =	U =	1.00	

Controlling Rating Factor = RF_t = **1.44**

Bay	DL (lbf)	LL, Ped (lbf)	LL, H10 (lbf)	RF (Ped)	RF (H10)
1	2842.2	6251.4	13058.6	12.59	6.03
2	2679.2	5402.0	12976.7	14.60	6.08
3	2613.1	5319.8	12997.3	14.83	6.07
4	2640.9	5371.4	12893.6	14.68	6.12
5	2551.1	5322.6	12955.7	14.83	6.09
6	2605.9	5298.0	12937.5	14.89	6.10
7	2555.6	5298.0	12937.5	14.90	6.10
8	2610.6	5322.6	12955.7	14.82	6.09
9	2554.0	5371.4	12893.6	14.70	6.12
10	2552.0	5319.8	12997.3	14.84	6.07
11	2543.6	5402.0	12976.7	14.61	6.08
12	3037.9	6251.4	13058.6	12.57	6.02

	C_{v1} =	*1.0*	
Depth of Beam =	d_b =	*15.0 in*	
Web Thickness =	t_w =	*0.5 in*	
Yield Strength =	F_y =	*38000 psi*	
Plastic Shear Force =	V_p =	*165300 lb*	LRFD Eq. 6.10.9.2-2
Nominal Shear Resistance =	V_n =	*165300 lb*	LRFD Eq. 6.10.9.2-1
	ϕ_{C_S} =	*0.86*	
Resistance Factor, Shear =	ϕ_v =	*1.0*	LRFD 6.5.4.2
Shear Capacity =	C_V =	*141332 lb*	MBE Eq. 6A.4.2.1-2
DL Load Factor =	γ_{DL} =	*1.25*	
LL Load Factor =	γ_{LL} =	*1.75*	

Controlling Rating Factor = RF_v = ***6.02***

*** 32% SL, Per Inspection Report *** (Bay 5)

Bay	DL (kips)	LL, Ped (kips)	LL, H10 (kips)	A_g (in^2)	RF (Ped)	RF (H10)
2	13.0	19.9	6.8	2.438	1.46	4.32
3	8.3	12.6	5.6	2.187	2.59	5.86
4	6.6	9.9	5.6	1.750	2.64	4.68
5	4.4	6.6	5.5	1.625	3.87	4.67
6	2.1	3.1	5.3	2.390	12.99	7.65
7	2.1	3.1	5.3	2.390	12.99	7.65
8	4.4	6.6	5.5	2.390	5.89	7.11
9	6.6	9.9	5.6	1.750	2.64	4.68
10	8.3	12.6	5.6	2.187	2.59	5.86
11	13.0	19.9	6.8	2.438	1.46	4.32

Yield Strength =	F_y =	38 ksi	
DL Load Factor =	γ_{DL} =	1.25	
LL Load Factor =	γ_{LL} =	1.75	
Reduction factor for holes	R_P =	1.00	LRFD 6.13.4
Resistance Factor =	ϕ_f =	0.95	LRFD 6.5.4.2
	ϕ_{bs} =	0.80	LRFD 6.5.4.2
System Factor =	ϕ_f =	0.90	MBE Table 6A.4.2.4-1 (multiple eyebar)
Condition Factor =	ϕ_c =	0.95	
	ϕ_{check} =	0.86	
	ϕ_{C_S} =	0.86	

Controlling Rating Factor = RF_t = **1.46**

Bay	$A_{g,t}$ (in^2)	$A_{g,bs}$ (in^2)	Tensile Resistance (kips)	Block Shear Resistance (kips)	Governing (kips)
2	2.438	4.469	88.0	78.8	78.793
3	2.187	4.813	79.0	84.9	78.969
4	1.750	4.813	63.2	84.9	63.175
5	1.625	4.875	58.7	86.0	58.670
6	2.390	4.875	86.3	86.0	85.956
7	2.390	4.875	86.3	86.0	85.956
8	2.390	4.875	86.3	86.0	85.956
9	1.750	4.813	63.2	84.9	63.175
10	2.187	4.813	79.0	84.9	78.969
11	2.438	4.469	88.0	78.8	78.793

Bay	DL (kips)	LL, Ped (kips)	LL, H10 (kips)	RF (Ped)		RF (H10)	
1-2	2.9	5.8	6.8	5.09	Tension	4.34	Tension
2-3	-6.7	-9.2	-4.5	3.52	Compression	7.15	Compression
3-4	-3.7	-4.6	4.3	6.79	Compression	12.15	Tension
4-5	-2.7	-3.0	4.3	10.26	Compression	12.17	Tension
5-6	-0.8	-0.2	4.4	146.74	Compression	11.67	Tension
6-7	-0.5	0.0	0.0	n/a	n/a	n/a	n/a
7-8	-0.8	-0.2	4.4	146.74	Compression	11.67	Tension
8-9	-2.6	-3.0	4.3	10.27	Compression	12.17	Tension
9-10	-3.7	-4.6	4.3	6.79	Compression	12.15	Tension
10-11	-6.7	-9.2	-4.5	3.52	Compression	7.15	Compression
11-12	2.9	5.8	6.8	5.08	Tension	4.33	Tension

LRFD Section 6.9.4.1 - Nominal Compressive Resistance:

Area of Element (1-2 & 11-12) =	A_{g1} =	*1.80 in^2*	MIDAS Section Properties	
Area of Element (2-3 to 10-11) =	A_{g2} =	*2.87 in^2*	MIDAS Section Properties	
Yield Strength =	F_y =	*38 ksi*		
Resistance Factor, Tension =	ϕ_f =	*0.95*	LRFD 6.5.4.2	
Condition Factor =	ϕ_c =	*0.95*	LRFD 6.5.4.2	
System Factor =	ϕ_S =	*0.90*	MBE Table 6A.4.2.4-1	(riveted member)
	ϕ_{check} =	*0.86*		
	ϕ_{C_S} =	*0.86*		
Elastic Modulus =	E =	*29000 ksi*		
Effective Length Factor =	K =	0.875	LRFD Article 4.6.2.5	
	I_{x1} =	*4.95 in^4*	MIDAS Section Properties	
	I_{x2} =	*21.20 in^4*	MIDAS Section Properties	
Radius of Gyration (1-2 & 11-12) =	r_{s1} =	*1.66 in*		
Radius of Gyration (2-3 to 10-11) =	r_{s2} =	*2.72 in*		
Slender Element Reduction Factor =	Q =	*1.0*	LRFD Article 6.9.4.2	
Equiv. Nominal Yield Resist. (1-2 & 11-12) =	P_{0-1} =	*68.28 kips*	LRFD Article 6.9.4.1.1	
Equiv. Nominal Yield Resist. (2-3 to 10-11) =	P_{0-2} =	*109.06 kips*	LRFD Article 6.9.4.1.1	
Compression Resistance Factor =	ϕ_c =	0.95	LRFD Article 6.5.4.2	
DL Load Factor (max) =	$\gamma_{DL,max}$ =	*1.25*		
DL Load Factor (min) =	$\gamma_{DL,min}$ =	*0.90*		
LL Load Factor =	γ_{LL} =	*1.75*		

Controlling Rating Factor = RF_t = **3.52** Elastic Flexural Buckling Resistance

			Article 6.9.4.1.2			Article 6.9.2.1	Article 6.9.3
Bay	Length (in)	$P_{r,tension}$	P_e	P_e/P_0	P_n	P_r	Kl/r
1-2	192	64.87	n/a				
2-3	232.5	103.61	146.6	1.34	79.88	75.89	OK
3-4	264	103.61	113.7	1.04	73.00	69.35	N.G.
4-5	280.5	103.61	100.7	0.92	69.32	65.85	N.G.
5-6	288	103.61	95.6	0.88	67.64	64.26	N.G.
6-7	288	103.61	95.6	0.88	67.64	64.26	N.G.
7-8	288	103.61	95.6	0.88	67.64	64.26	N.G.
8-9	280.5	103.61	100.7	0.92	69.32	65.85	N.G.
9-10	264	103.61	113.7	1.04	73.00	69.35	N.G.
10-11	232.5	103.61	146.6	1.34	79.88	75.89	OK
11-12	192	64.87	n/a				

*** Vertical Struts are closed sections connected with lacing bars, therefore per C6.9.4.1.3, they need not be considered for torsional buckling and flexural-torsional buckling. Other, non-laced members are in tension.

LRFD Section 6.9.4.2 - Slender Element Check:

Width of Channel Flange =	b_f =	***2.090 in***	Table 6.9.4.2.1-1, AISC Table 1-5
Width of Channel Web =	b_w =	***5.604 in***	Table 6.9.4.2.1-1, AISC Table 1-5
Plate Buckling Coefficient (Flange) =	k_f =	***0.56***	Table 6.9.4.2.1-1
Plate Buckling Coefficient (Web) =	k_w =	***1.49***	Table 6.9.4.2.1-1
Plate Thickness =	t =	***0.288 in***	AISC Table 1-5, per Eq. 6.9.4.2.1-1
Flange Slenderness Check =	r_{s1} =	***OK***	Eq. 6.9.4.2.1-1
Web Slenderness Check =	r_{s1} =	***OK***	Eq. 6.9.4.2.1-1

Bay	DL (kips)	LL, Ped (kips)	LL, H10 (kips)	RF, Ped.	RF, H10
1	-52.3	-81.6	-21.8	2.23	8.36
2	-55.4	-85.8	-22.6	2.24	8.48
3	-65.2	-100.8	-26.4	1.83	7.01
4	-72.0	-111.2	-29.0	1.62	6.22
5	-76.6	-118.0	-30.7	1.50	5.77
6	-78.7	-121.3	-31.5	1.45	5.58
7	-78.7	-121.3	-31.5	1.45	5.58
8	-76.6	-118.0	-30.7	1.50	5.77
9	-72.1	-111.2	-29.0	1.62	6.22
10	-65.2	-100.8	-26.4	1.83	7.01
11	-55.4	-85.8	-22.6	2.24	8.48
12	-52.3	-81.6	-21.8	2.23	8.36

LRFD Section 6.9.4.1 - Nominal Compressive Resistance:

Area of Element =	A_g =	15.57 in^2	(4 L2.5x2.5x0.25 + 2*1/4"*14.125 + 1/4"*15)
Yield Strength =	F_y =	38 ksi	
Condition Factor =	ϕ_C =	0.95	MBE Table 6A.4.2.3-1
System Factor =	ϕ_S =	0.90	MBE Table 6A.4.2.4-1 (riveted member)
	ϕ_{check} =	0.86	
	ϕ_{C_S} =	0.86	
Elastic Modulus =	E =	29000 ksi	
Effective Length Factor =	K =	0.875	LRFD Article 4.6.2.5
Strong Axis Moment of Inertia=	I_x =	459.36 in^4	(see appended calculations)
Weak Axis Moment of Inertia=	Iy=	396.10 in^4	(see appended calculations)
Strong Axis radius of gyration=	$r_{s,x}$ =	5.43 in	
Weak Axis radius of gyration=	$r_{s,y}$ =	5.04 in	
Radius of Gyration =	r_s =	5.04 in	
Slender Element Reduction Factor =	Q =	0.89	LRFD Article 6.9.4.2
Equiv. Nominal Yield Resist. =	P_0 =	525.26 kips	LRFD Article 6.9.4.1.1
Compression Resistance Factor =	ϕ_c =	0.95	LRFD Article 6.5.4.2
DL Load Factor =	γ_{DL} =	1.25	
LL Load Factor =	γ_{LL} =	1.75	
Controlling Rating Factor =	RF_t =	1.45	Elastic Flexural Buckling Resistance

		Article 6.9.4.1.2			Article 6.9.2.1	Article 6.9.3
Bay	Length (in)	P_e	P_e/P_0	P_n	P_r	Kl/r
1	265.2	2104.8	4.01	473.16	449.50	N.G.
2	187.4	4215.2	8.03	498.56	473.63	N.G.
3	185.7	4294.4	8.18	499.04	474.09	N.G.
4	183.7	4386.0	8.35	499.58	474.60	N.G.
5	183.2	4414.2	8.40	499.74	474.75	N.G.
6	183.0	4421.6	8.42	499.78	474.79	N.G.
7	183.0	4421.6	8.42	499.78	474.79	N.G.
8	183.2	4414.2	8.40	499.74	474.75	N.G.
9	183.7	4386.0	8.35	499.58	474.60	N.G.
10	185.7	4294.4	8.18	499.04	474.09	N.G.
11	187.4	4215.2	8.03	498.56	473.63	N.G.
12	265.2	2104.8	4.01	473.16	449.50	N.G.

*** Top Chord members are closed sections connected with lacing bars, therefore per C6.9.4.1.3, they need not be considered for torsional buckling and flexural-torsional buckling. Other, non-laced members are in tension.

LRFD Section 6.9.4.2 - Slender Element Check:

Width of Top Plate =	b_t =	12.750 in		LRFD Table 6.9.4.2.1-1
Width of Side Plates =	b_s =	11.750 in		LRFD Table 6.9.4.2.1-1
Plate Buckling Coefficient =	k =	1.40		LRFD Table 6.9.4.2.1-1 (Rect. Built-up)
Top Flange Plate Thickness =	t_t =	0.250 in		
Web Plate Thickness =	t_w =	0.250 in		
Top Flange Plate Slenderness Check =	r_{T1} =	N.G.	Slender	Eq. 6.9.4.2.1-1
Web Plates Slenderness Check =	r_{W1} =	N.G.	Slender	Eq. 6.9.4.2.1-1

Top Flange Plate:

Unstiff. Slender Elem. Red. Factor=	$Q_{s,T}$ =	0.16		Eqs. 6.9.4.2.2-5 & 6.9.4.2.2-6
	f=	6 ksi	$=Q_{s,T}*F_Y$	(LRFD 6.9.4.2.2)
Effective width=	$b_{e,T}$=	12.75 in		Eq. 6.9.4.2.2-10
Area=	A_T=	3.19 in^2		
Effective Area=	$A_{eff,T}$=	3.19 in^2	$=A_T-(b-b_{e,T})*t_f$	(LRFD 6.9.4.2.2)

Web Plate:

Unstiff. Slender Elem. Red. Factor=	$Q_{s,W}$ =	0.18		Eqs. 6.9.4.2.2-5 & 6.9.4.2.2-6
	f=	7 ksi	$=Q_{s,W}*F_Y$	(LRFD 6.9.4.2.2)
Effective width=	$b_{e,W}$=	11.75 in		Eq. 6.9.4.2.2-10
Area=	A_W=	2.94 in^2		
Effective Area=	$A_{eff,W}$=	2.94 in^2	$=A_W-(b-b_{e,W})*t_w$	(LRFD 6.9.4.2.2)

Total Area=	A=	15.57 in^2		
Total Effective Area=	A_{eff}=	13.82 in^2		
Stiff. Slender Elem. Red. Factor=	Q_a=	0.89	$=A_{eff}/A$	Eq. 6.9.4.2.2-9

LRFD Section 6.9.4.3 - Built Up Member:

*** Although the Top Chord is a built up member, no shear force is generated while loading as this is an axial element. Therefore, any modification to the slenderness ratio per Section 6.9.4.3 is not necessary.

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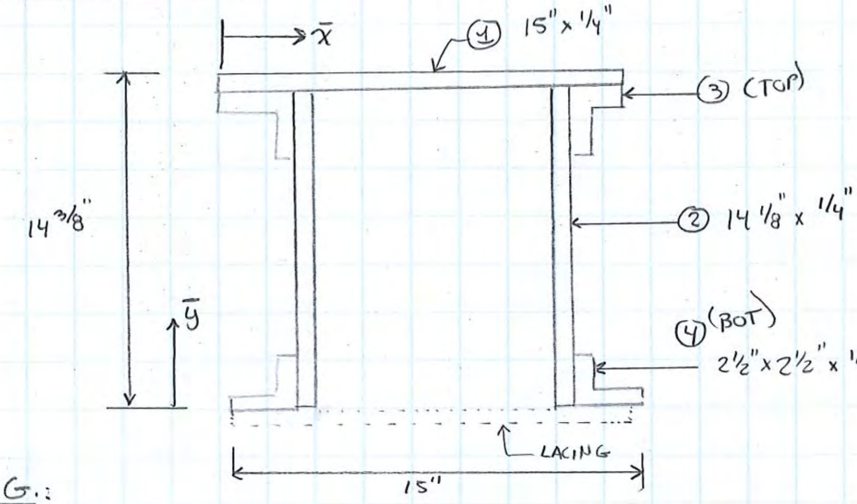
Job: Flower Bridge LR | Computed By: LV | Date: 1/16/2019
Description: Top. Chord Sect. Prop. | Checked By: | Date:
Sheet 1 of 2

Top Chord Section Properties:



Area + C.G.:

$A_1 = (15)(0.25) = 3.75\ in^2$

$\bar{y}_1 = 14.375 - 0.25/2 = 14.25\ in$

$\bar{y} = \dfrac{\Sigma A_i \bar{y}_i}{\Sigma A_i}$

$A_2 = (14.125)(0.25) = 3.53\ in^2$

$\bar{y}_2 = (14.125/2) = 7.06\ in$

$$\bar{y} = \frac{(3.75)(14.25) + (3.53)(7.06)(2) + (1.19)(13.414)(2) + (1.19)(0.711)(2)}{(3.75) + 2(3.53) + 4(1.19)}$$

$A_3 = 1.19\ in^2$

$\bar{y}_3 = 14.375 - 0.25 - 0.711 = 13.414\ in$

$\bar{y} = \dfrac{136.90}{15.57} = \underline{\underline{8.79\ in}}$

$A_4 = 1.19\ in^2$

$\bar{y}_4 = 0.711\ in$

$\bar{x} = 15/2 = \underline{\underline{7.5\ in}}$

Moment of inertia:

Strong Axis: $I_{1x} = \left(\frac{1}{12}\right)(15)(0.25)^3 = 0.0195\ in^4$; $I_{2x} = \left(\frac{1}{12}\right)(0.25)(14.125)^3 = 58.71\ in^4$

$d_{1x} = \bar{y}_1 - \bar{y} = 14.25 - 8.79 = 5.46\ in$; $d_{2x} = 1.73\ in$

$I_{3x} = I_{4x} = 0.692\ in^4$; $d_{3x} = 4.624\ in$; $d_{4x} = 8.079\ in$

$$I_{T_x} = (0.0195) + (3.75)(5.46)^2 + 2\left[58.71 + (3.53)(1.73)^2\right] + 2\left[0.692 + (1.19)(4.624)^2\right] + 2\left[0.692 + (1.19)(8.079)^2\right]$$

$$I_{T_x} = 111.813 + 138.55 + 52.27 + 156.73 = \underline{\underline{459.36\ in^4}}$$

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Job Flower Bridge LR Computed By LV Date 1/16/2019

Description Top Chord Sect. Prop. Checked By ______ Date ______

Sheet 2 of 2

Weak Axis:

$I_{1_y} = (\frac{1}{12})(0.25)(15)^3 = 70.3\ in^4$; $d_{1_y} = 0$

$I_{2_y} = (\frac{1}{12})(14.125)(0.25)^3 = 0.018\ in^4$; $d_{2_y} = 7.5 - 2.5 - 0.25/2 = 4.875"$

$I_{3_y} = I_{4_y} = 0.692\ in^4$; $d_{3_y} = d_{4_y} = 7.5 - 2.5 + 0.711 = 5.711"$

$I_{T_y} = 70.3 + 2[0.018 + (3.53)(4.875)^2] + 4[0.692 + (1.19)(5.711)^2]$

$I_{T_y} = \underline{396.1\ in^4}$

Designed By: BAW
Date: 11/13//17
Checked By: TPL
Date: 11/14/17

FLOOR BEAM ANALYSIS

FLOWER BRIDGE, SIMSBURY, CT
AASHTO LRFD Bridge Design Specifications 7th Edition -2014 w/ Interims thru 2016(LRFD), LRFD Guide Specs for the Design of Pedestrian Bridges, AASHTO Manual for Bridge Evaluation 2nd Edition -2011 w/Interims thru 2016(MBE)

Notes:
- The loading to be used in the analysis of the floor beam is 90 PSF for Pedestrian Load and H10 Load for maintenance vehicle.
- Floor Beam assumed to be S12x31.8 (taken from "Engineering Study to Determine Live Load April 1990")

1. Initial Design Specifications

Span length: $L_{span} := 17.3ft$

Contributing load width: $W_c := 3.6666666ft$

Deck Thickness: $t := 0.25ft$

Timber weight: $w_{timber} := 60pcf$

Floor Beam DL: $DL_b := 31.8\,\frac{lb}{ft}$

2. Section Properties:

Calculated Deck Area: $A_{panel} := W_c \cdot t = 0.917 \cdot ft^2$

Moment of Inertia: $I_x := 164in^4$ — AISC Steel Construction Manual 13th Edition Table 1-3

Section Loss Properties: (TF w/ FW by 0.3125" deep SL) $A_{SL} := (0.3125 \cdot 5)in^2 = 1.563\ in^2$

$$I_{SL} := \left[\left(\frac{1}{12}\right)(5)(0.3125)^3\right]in^4 + A_{SL}\cdot(5.84in)^2 = 53.303\ in^4$$

Section Modulus: $$S_x := 36.2in^3 - \frac{I_{SL}}{(6 - 0.3125)in} = 26.828\ in^3$$

Beam depth: $d_b := 12in$

Web thickness: $t_w := 0.35in$

3. Panel Dead Load:

Dead Load of Deck Only: $DL_{deck} := w_{timber} \cdot A_{panel} = 55 \frac{lb}{ft}$

Dead Load of Wearing Surface:

Thickness of Wearing Surface: $t_{ws} := 0in$

$DL_{ws} := t_{ws} \cdot W_c \cdot w_{timber} = 0$

Dead Load of Nailer Beam: $A_{nb} := 0.5625ft^2$ nailer 9x9

$DL_{nb} := A_{nb} \cdot w_{timber} = 33.75 \frac{lb}{ft}$

Panel Dead Load of Entire System: $DL_{panel} := DL_{deck} + DL_{ws} + DL_{nb}$

$DL_{panel} = 88.75 \frac{lb}{ft}$

4. Dead and Live Load Moments

Dead Load Maxmium Moment: $M_{DL} := \frac{(DL_b + DL_{panel}) \cdot L_{span}^2}{8} = 4509.9 \, ft \cdot lb$

Pedestrian Live Load: $LL_{ped} := 90psf$

Pedestrian Live Load per Floor Beam: $LL_{panel} := LL_{ped} \cdot W_c = 330 \frac{lb}{ft}$

The moment produced by the Pedestrian Loading: $M_{LL\_p} := \frac{LL_{panel} \cdot L_{span}^2}{8} = 12345.7 \, ft \cdot lb$

H10 Live Load: $LL_H := 8000lb$ per wheel

(The maximum loading is one axle on the floor beam)

The moment produced by the H10 Loading:

$$R_1 := LL_H \cdot \frac{\left[\left[L_{span} - \left(\frac{L_{span}}{2} + 4.5ft\right)\right] + \left(\frac{L_{span}}{2} + 1.5ft\right)\right]}{L_{span}} = 6612.7 \, lb$$

$$M_{LL\_H} := R_1 \cdot \left(\frac{L_{span}}{2} - 1.5ft\right) = 47280.9 \, ft \cdot lb$$

5. Bending Capacity

Bending Yield Strength: $F_y := 38000 \frac{lb}{in^2}$

Condition Factor: $\phi_c := 0.85$ Assumed poor condition MBE Table 6A.4.2.3-1

System Factor: $\phi_s := 1.0$ MBE Table 6A.4.2.4-1

$$\phi_{check} := \phi_c \cdot \phi_s = 0.85$$

$$\phi_{c\_s} := \text{if}\left(\phi_{check} < 0.85, 0.85, \phi_{check}\right) = 0.85$$ MBE Eq. 6A.4.2.1-3

Resistance Factor, Flexure: $\phi_f := 1.0$ LRFD 6.5.4.2

Nominal Resistance of Floor Beam, Flexure: $R_{n\_f} := F_y \cdot S_x = 84955.63\ ft \cdot lb$

Flexural Capacity of Floor Beam: $C_f := \phi_{c\_s} \cdot \phi_f \cdot R_{n\_f} = 72212.29\ ft \cdot lb$ MBE Eq. 6A.4.2.1-2

Load factor for DL: $\gamma_{DL} := 1.25$ MBE Table 6A.4.2.2-1

Load factor for LL: $\gamma_{LLinv} := 1.75$ MBE Table 6A.4.2.2-1

$\gamma_{LLoper} := 1.35$ MBE Table 6A.4.2.2-1

Rating Factor for Flexure:

Inventory

Pedestrian $$RF_{f\_p} := \frac{C_f - \gamma_{DL} \cdot M_{DL}}{\gamma_{LLinv} \cdot M_{LL\_p}} = 3.081$$ MBE Eq. 6A.4.2.1-1

H10 $$RF_{f\_H} := \frac{C_f - \gamma_{DL} \cdot M_{DL}}{\gamma_{LLinv} \cdot M_{LL\_H}} = 0.805$$ MBE Eq. 6A.4.2.1-1

Operating

Pedestrian $$RF_{f\_p\_oper} := \frac{C_f - \gamma_{DL} \cdot M_{DL}}{\gamma_{LLoper} \cdot M_{LL\_p}} = 3.994$$ MBE Eq. 6A.4.2.1-1

H10 $$RF_{f\_H\_oper} := \frac{C_f - \gamma_{DL} \cdot M_{DL}}{\gamma_{LLoper} \cdot M_{LL\_H}} = 1.043$$ MBE Eq. 6A.4.2.1-1

6. Vertical Shear Capacity:

Dead load Vertical Shear: $V_{DL} := DL_{panel} \cdot \left(\frac{L_{span}}{2}\right) = 767.69\,lb$

Live Load Vertical Shear: Pedestrian $V_{LL\_p} := LL_{panel} \cdot \left(\frac{L_{span}}{2}\right) = 2854.5\,lb$

H10 $V_{LL\_H} := LL_H \cdot \frac{\left[\left(L_{span} - 1ft\right) + \left(L_{span} - 7ft\right)\right]}{L_{span}} = 12300.6\,lb$

$C_v := 1.0$

Plastic Shear Force: $V_p := 0.58 \cdot F_y \cdot d_b \cdot t_w = 92568\,lb$ LRFD Eq. 6.10.9.2-2

Nominal Shear Resistance: $V_n := C_v \cdot V_p = 92568\,lb$ LRFD Eq. 6.10.9.2-1

Resistance Factor, Shear: $\phi_v := 1.0$ LRFD 6.5.4.2

Shear Capacity for Floor Beam: $C_V := \phi_{c\_s} \cdot \phi_v \cdot V_n = 78682.8\,lb$ MBE Eq. 6A.4.2.1-2

Rating Factor for Shear:

Inventory

Pedestrian $RF_{V\_p} := \frac{C_V - \gamma_{DL} \cdot V_{DL}}{\gamma_{LLinv} \cdot V_{LL\_p}} = 15.559$ MBE Eq. 6A.4.2.1-1

H10 $RF_{V\_H} := \frac{C_V - \gamma_{DL} \cdot V_{DL}}{\gamma_{LLinv} \cdot V_{LL\_H}} = 3.611$ MBE Eq. 6A.4.2.1-1

Operating

Pedestrian $RF_{V\_p\_oper} := \frac{C_V - \gamma_{DL} \cdot V_{DL}}{\gamma_{LLoper} \cdot V_{LL\_p}} = 20.169$ MBE Eq. 6A.4.2.1-1

H10 $RF_{V\_H\_oper} := \frac{C_V - \gamma_{DL} \cdot V_{DL}}{\gamma_{LLoper} \cdot V_{LL\_H}} = 4.68$ MBE Eq. 6A.4.2.1-1

TIMBER DECK ANALYSIS

FLOWER BRIDGE, SIMSBURY, CT
(AASHTO Standard Specification for Highway Bridges 17th Edition -2002), LRFD Guide Specs for the Design of Pedestrian Bridges

Notes:
- The loading to be used in the analysis of the timber decking: 90 PSF for Pedestrian Load
  H10 Loading for Maintenance Vehicle .

1. Initial Design Specifications

Span length: $L_{span} := 44in$

Bridge Width: $W_{br} := 16ft$

Number of Deck Panels: $n_{panels} := 24$

Panel Width: $W_p := \frac{W_{br}}{n_{panels}} = 8 \cdot in$

Deck Thickness: $t := 3in$

Timber weight: $w_{timber} := 60pcf$

2. Section Properties:

Calculated Panel Area: $A_{panel} := W_p \cdot t = 24 \cdot in^2$

Calculated Section Modulus: $S_y := \frac{7.25in \cdot (2.5in)^2}{6} = 7.552 \cdot in^3$

Calculated Moment of Inertia: $I_y := \frac{7.25in \cdot (2.5in)^3}{12} = 9.44 \cdot in^4$

Note: Dimensions used for section properties are based on actual (i.e. not nominal) dimensions

3. Panel Dead Load:

Dead Load of Deck Only: $DL_{deck} := w_{timber} \cdot A_{panel} = 10 \frac{lb}{ft}$

Dead Load of Wearing Surface:

Thickness of Wearing Surface: $t_{ws} := 0in$

$DL_{ws} := t_{ws} \cdot W_p \cdot w_{timber} = 0$

Weight of Individual Rail System: $w_{rail} := 0 \frac{lb}{ft^2}$ (USDA Standard Plans)

Dead Load of Individual Rail System per deck panel: $DL_{rail} := \frac{w_{rail} \cdot W_p}{n_{panels}} = 0$

Number of Rail Systems: $n_{rail} := 0$

Panel Dead Load of Entire System: $DL_{panel} := DL_{deck} + DL_{ws} + n_{rail} \cdot DL_{rail}$

$DL_{panel} = 10 \frac{lb}{ft}$

4. Dead and Live Load Moments

Moment is computed by assuming each panel acts as a simply supported beam.

Dead Load Maxmium Moment: $M_{DL} := \frac{DL_{panel} \cdot L_{span}^2}{8} = 16.81 \, lb \cdot ft$

Pedestrian Live Load: $LL_{ped} := 90psf$

$LL_{ped\_d} := 0.625psi$

Pedestrian Live Load per Panel: $LL_{panel} := LL_{ped} \cdot W_p = 60 \frac{lb}{ft}$

$LL_{panel\_d} := LL_{ped\_d} \cdot W_p$

The moment produced by the Pedestrian Loading: $M_{LL\_p} := \frac{LL_{panel} \cdot L_{span}^2}{8} = 100.833 \, lb \cdot ft$

H10 Live Load per Panel: $LL_H := 8000lb$

The moment produced by the H10 Loading: $M_{LL\_H} := \frac{LL_H \cdot L_{span}}{4} = 7333.333\, lb \cdot ft$

Total Moment: Pedestrian $M_{T\_p} := M_{DL} + M_{LL\_p} = 117.6\, lb \cdot ft$

H10 $M_{T\_H} := M_{DL} + M_{LL\_H} = 7350.139\, lb \cdot ft$

5. Bending Stress and Deck Combination Selection

Bending Stress: Pedestrian $f_{b\_p} := \frac{M_{T\_p}}{S_y} = 186.9 \cdot \frac{lb}{in^2}$

H10 $f_{b\_H} := \frac{M_{T\_H}}{S_y} = 11679.1 \cdot \frac{lb}{in^2}$

Timber Species is assumed to be SPRUCE-PINE-FIR (SOUTH) No. 2 Grade

Bending Yield Strength: $F_{by} := 750 \frac{lb}{in^2}$ (AASHTO Table 13.5.1A)

A size factor needs to be implemented for species other than Southern Pine

Size Factor: $C_F := 1.2$

When timber is used where moisture content may surpass 19%, a Wet Service Factor, C_m, needs to be applied to design values:

Is a Wet Service Factor necessary?
(1 for Yes, 0 for No) $Necessary := 1$

$C_{m\_bending1} := if(Necessary > 0, 0.85, 1.0) = 0.85$ (AASHTO Table 13.5.1A)

$C_{m\_check} := F_{by} \cdot C_F = 0.9 \cdot ksi$

$C_{m\_bending} := if(C_{m\_check} > 1.15ksi, C_{m\_bending1}, 1.0) = 1$

Flat use factor: $C_{fu} := 1.15$ (AASHTO Table 13.5.1A)

Allowable Bending Strength: $F_b := F_{by} \cdot C_{m\_bending} \cdot C_F \cdot C_{fu} = 1035 \cdot \frac{lb}{in^2}$

Pedestrian $Checkf_{b\_p} := \begin{cases} \text{"O.K."} & \text{if } f_{b\_p} \le F_b \\ \text{"N.G."} & \text{otherwise} \end{cases}$ $\boxed{Checkf_{b\_p} = \text{"O.K."}}$

H10 $Checkf_{b\_H} := \begin{cases} \text{"O.K."} & \text{if } f_{b\_H} \le F_b \\ \text{"N.G."} & \text{otherwise} \end{cases}$ $\boxed{Checkf_{b\_H} = \text{"N.G."}}$

6. Live Load Deflection:

Modulus of Elasticity: $E := 1100000 psi$ (AASHTO Table 13.5.1A)

$C_{m\_LL} := \text{if}(Necessary > 0, 0.9, 1) = 0.9$ (AASHTO Table 13.5.1A)

Corrected Modulus of Elasticity: $E_{LL} := E \cdot C_{m\_LL} = 9.9 \times 10^5 \text{ psi}$

Live Load Panel Deflection: $\Delta_{LL} := \frac{5 \cdot LL_{panel\_d} \cdot L_{span}^4}{384 \cdot E_{LL} \cdot I_y} = 0.0261 \cdot in$

The maximum panel deflection is recommended to be equal to L_{span}/360. (AASHTO Ped Guide Spec Section 5)

Maximum Panel Deflection: $\Delta_{max} := \frac{L_{span}}{360} = 0.122 \cdot in$

$Check\Delta_{LL} := \begin{cases} \text{"O.K."} & \text{if } \Delta_{LL} < \Delta_{max} \\ \text{"N.G."} & \text{otherwise} \end{cases}$ $\boxed{Check\Delta_{LL} = \text{"O.K."}}$

7. Vertical Shear:

Effective Area of Panel for Shear: $A_{panel} := 7.25in \cdot 2.5in = 18.125\,in^2$

Note: Dimensions used for shear capacity are based on actual sawn dimensions.

Dead load Vertical Shear: $V_{DL} := DL_{panel} \cdot \left(\frac{L_{span}}{2} - t\right) = 15.833\,lb$

Live Load Vertical Shear: Pedestrian $V_{LL\_p} := LL_{panel} \cdot \left(\frac{L_{span}}{2} - t\right) = 95\,lb$

H10 $V_{LL\_H} := \frac{LL_H}{2} = 4000\,lb$

Vertical Shear Stress: $V\_1 := V_{DL} + V_{LL\_p} = 110.833\,lb$

Pedestrian

$f_{v\_1} := 1.5 \cdot \frac{V\_1}{A_{panel}} = 9.172 \cdot \frac{lb}{in^2}$

H10 $V\_2 := V_{DL} + V_{LL\_H} = 4.016 \times 10^3\,lb$

$f_{v\_2} := 1.5 \cdot \frac{V\_2}{A_{panel}} = 332.345 \cdot \frac{lb}{in^2}$

$C_{m\_shear} := 0.97$ (AASHTO Table 13.5.1A)

Shear Strength Parallel to Grain: $F_{vy} := 70 \frac{lb}{in^2}$ (AASHTO Table 13.5.1A)

Allowable Shear Stress: $F_{V\_1} := C_{m\_shear} \cdot F_{vy} = 67.9 \cdot \frac{lb}{in^2}$

$Checkf_{v\_1} := \begin{cases} \text{"O.K."} & \text{if } f_{v\_1} < F_{V\_1} \\ \text{"N.G."} & \text{otherwise} \end{cases}$ $\boxed{Checkf_{v\_1} = \text{"O.K."}}$

$Checkf_{v\_2} := \begin{cases} \text{"O.K."} & \text{if } f_{v\_2} < F_{V\_1} \\ \text{"N.G."} & \text{otherwise} \end{cases}$ $\boxed{Checkf_{v\_2} = \text{"N.G."}}$

8. Load Rating:

AASHTO Manual for Bridge Evaluation, 2nd Edition, 2011 with interims through 2016 (MBE)

Moment :

Inventory Moment Capacity: $M_{R\_inv} := F_b \cdot S_y = 0.65 \cdot ft \cdot kip$

MBE Section 6B.5.2.7

Iventory Rating Factor for Moment:

Pedestrian

$$RF_{M\_inv\_p} := \frac{M_{R\_inv} - M_{DL}}{M_{LL\_p}} = 6.293$$

H10

$$RF_{M\_inv\_H} := \frac{M_{R\_inv} - M_{DL}}{M_{LL\_H}} = 0.087$$

Shear :

Inventory Shear Capacity: $V_{R\_inv} := \frac{2}{3} \cdot F_{V\_1} \cdot A_{panel} = 820.458\ lb$

MBE Section 6B.5.2.7

Iventory Rating Factor for Shear:

Pedestrian

$$RF_{V\_inv\_p} := \frac{V_{R\_inv} - V_{DL}}{V_{LL\_p}} = 8.47$$

H10

$$RF_{V\_inv\_H} := \frac{V_{R\_inv} - V_{DL}}{V_{LL\_H}} = 0.201$$

Connections to Bottom Chord: Rating Factor Summary:

	Diag. Welded Conn.		Conn. Plate		Plate Welded Conn.		Controlling
Panel Point	Pedestrian	H10	Pedestrian	H10	Pedestrian	H10	
L2 & L10	1.99	5.86	0.93	2.74	1.08	2.93	0.93
L3 & L9	3.20	7.23	1.56	3.52	2.07	3.99	1.56
L4 & L8	4.46	7.92	2.12	3.76	2.85	4.20	2.12
L5 & L7	4.88	5.89	3.77	4.55	5.17	4.53	3.77
L6	9.99	5.88	8.13	4.79	11.68	6.87	4.79
							0.93

Member Forces:

Vertical Struts			
Panel Point	DL (kips)	LL, Ped (kips)	LL, H10 (kips)
L1	2.9	5.8	6.8
L2	-6.7	-9.2	-4.5
L3	-3.7	-4.6	4.3
L4	-2.7	-3.0	4.3
L5	-0.8	-0.2	4.4
L6	-0.5	0.0	0.0
L7	-0.8	-0.2	4.4
L8	-2.6	-3.0	4.3
L9	-3.7	-4.6	4.3
L10	-6.7	-9.2	-4.5
L11	2.9	5.8	6.8

Diagonal Struts			
Panel Point	DL (kips)	LL, Ped (kips)	LL, H10 (kips)
L2 & L10	13.0	19.9	6.8
L3 & L9	8.3	12.6	5.6
L4 & L8	6.6	9.9	5.6
L5 & L7	4.4	6.6	5.5
L6	2.1	3.1	5.3

Note: Loads shown above are per diagonal strut.

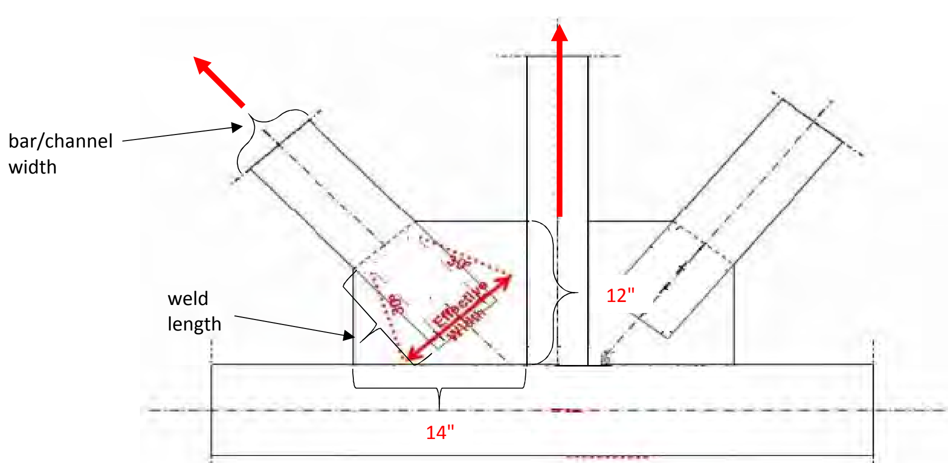


Plate thickness=	t=	0.25	in	
Yield Strength=	F_Y=	36.00	ksi	
Resistance Factor =	ϕ_Y =	0.95		LRFD 6.5.4.2
Load Factors:	γ_{DL} =	1.25		
	γ_{LL} =	1.75		
Condition Factor =	ϕ_C =	1.00		MBE Table 6A.4.2.3-1
System Factor =	ϕ_S =	0.85		MBE Table 6A.4.2.3-1
	ϕ_{check} =	0.85		
	ϕ_{C_S} =	0.85		

Diagonal Struts Welded Connection Load Rating:

Weld Size=	0.25	in	(assumed)
Weld Capacity=	5.60	kip/in	(1.4 kip/in per 1/16 of weld)

Panel Point	Bar/Channel Width	Weld Length (in)	Eff. Plate Width (in)	Eff. Plate Area (in^2)	Capacity (kips)	Rating Factor	
						Pedestrian	H10
L2 & L10	3.0	18.0	23.8	5.95	100.80	1.99	5.86
L3 & L9	2.5	17.0	22.1	5.53	95.20	3.20	7.23
L4 & L8	2.0	18.0	22.8	5.70	100.80	4.46	7.92
L5 & L7	6.0	13.0	21.0	5.25	72.80	4.88	5.89
L6	6.0	12.0	19.9	4.96	67.20	9.99	5.88

Plate Load Rating:

Panel Point	Bar/Channel Width	Weld Length (in)	Eff. Plate Width (in)	Eff. Plate Area (in^2)	Capacity (kips)	Rating Factor	
						Pedestrian	H10
L2 & L10	3.0	9.0	13.4	3.35	114.50	0.93	2.74
L3 & L9	2.5	8.5	12.3	3.08	105.29	1.56	3.52
L4 & L8	2.0	9.0	12.4	3.10	105.95	2.12	3.76
L5 & L7	6.0	6.5	13.5	3.38	115.47	3.77	4.55
L6	6.0	6.0	12.9	3.23	110.54	8.13	4.79

Note: Two diagonal struts (one on each side) acting on the gusset plate

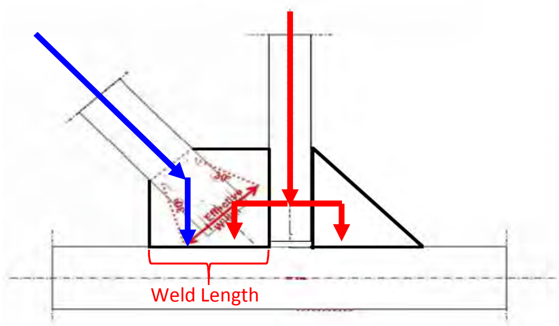
Plate Welded Connection Load Rating:

Weld Size=	0.25	in	(assumed)
Weld Unit Capacity=	5.60	kip/in	(1.4 kip/in per 1/16 of weld)

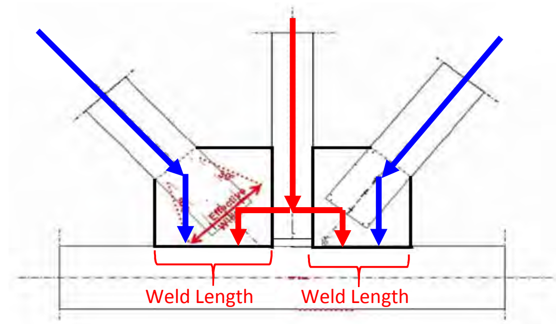
			Diagonal			Vertical			Rating Factor	
Panel Poinr	Weld Length (in)	Capacity (kips)	DL (kips)	LL, Ped (kips)	LL, H10 (kips)	DL (kips)	LL, Ped (kips)	LL, H10 (kips)	Pedestrian	H10
L2 & L10	28	156.8	26.1	39.9	13.5	6.66	9.19	4.52	1.08	2.93
L3 & L9	28	156.8	16.5	25.2	11.2	3.69	4.60	4.32	2.07	3.99
L4 & L8	28	156.8	13.1	19.9	11.2	2.65	2.95	4.27	2.85	4.20
L5 & L7	28	156.8	8.8	13.2	10.9	0.81	0.21	4.37	5.17	4.53
L6	56	313.6	8.2	12.5	21.2	1.04	0.00	0.00	11.68	6.87

Note: Forces shown above reflect the number of members per panel point (i.e. two diagonals for Panel Points L2 to L5 and L7 to L10; 4 diagonals for Panel Point L6)

Assumed Load Path for Plate Weld Load Rating



Panel Pt. L2 to L5 & Panel Pt. L7 to L10



Panel Pt. L6

Floorbeams Connections Load Rating

DL Shear=	V_{DL}=	*0.77*	*kip*	
LL, Ped. Shear=	$V_{LL,Ped}$=	*2.85*	*kip*	
LL, H10 Shear=	$V_{LL,H10}$=	*12.30*	*kip*	
Load Factors:	γ_{DL} =	*1.25*		
	γ_{LL} =	*1.75*		
Condition Factor =	ϕ_C =	*1.00*		MBE Table 6A.4.2.3-1

Riveted Connection:

Rivet Diameter=	D_{Rivet}=	*0.75*	*in*	
Rivet Area=	A_{rivet}=	*0.44*	*in^2*	
Number of faying surfaces=	m=	*1.00*		
Tensile Strength=	F_U=	*50.00*	*ksi*	(MBE Table 6A.6.12.5.1-1)
Resistance Factor=	ϕ_s=	*0.80*		
	R1=	*0.67*		
	R2=	*0.97*	*(L = 6" assumed)*	
	R3=	*1.00*		
Factored Resistance=	R_n=	*11.48*	*kip/rivet*	(MBE Eq. 6A.6.12.5.1-1)
Number of Rivets=	n_{rivets}=	*4.00*		

RF, Ped=	*9.00*
RF, H10=	*2.09*

Top Chord Pin Load Rating Summary: Pedestrian Load

Panel Point	Shear	Bearing			Controlling
		Vertical Strut	Diagonal Strut	Top Chord	
U1 & U11	3.94	4.26	3.04	4.17	3.04
U2 & U10	11.22	2.40	5.50	13.96	2.40
				Min.	**2.40**

Resultant Factored Loads on Pin:

Panel Point	Shear (kips) DL	Shear (kips) LL, Ped	Moment (k-ft) DL	Moment (k-ft) LL, Ped
U1 & U11	19.18	42.84	0.73	1.58
U2 & U10	10.43	22.07	0.30	0.67

Reactions on Top Pin:

	DL (kips) Horiz.	DL (kips) Vertical	LL, Ped (kips) Horiz.	LL, Ped (kips) Vertical
U1 & U11	9	12.4	13.7	20.3
U2 & U10	0.2	5.2	0.7	7.8

Diameter of Pin= *2.5* *in*
Fy= *47.0* *ksi*
ϕ_f= *1.0*
ϕ_v= *1.0*
ϕ_b= *1.0*
Load Factors: γ_{DL} = *1.25*
γ_{LL} = *1.75*
Condition Factor= *0.95* *(fair condition)*
System Factor= *0.9*
Cond.&Sys. Factor= *0.86*

Shear Load Rating:

RF_U1 & U11: RF= ***3.94***

RF_U2 & U10: RF= ***11.22***

Bearing Load Rating

U1 & U11
Vertical Strut Dead Load= *2.9* *kips*
Vertical Strut Live Load, Ped= *5.8* *kips*
Vertical Strut Web Thickness= *0.3125* *in* *(5/16")*
RF= ***4.26***

U2 & U10
Vertical Strut Dead Load= *6.8* *kips*
Vertical Strut Live Load, Ped= *9.2* *kips*
Vertical Strut Web Thickness= *0.3125* *in* *(assumed as 5/16")*
RF= ***2.40***

U1 & U11
Diagonal Strut Dead Load= *13.1* *kips*
Diagonal Strut Live Load, Ped= *19.9* *kips*
Diagonal Strut thickness= *0.8125* *in* *(13/16")*
RF= ***3.04***

U2 & U10
Diagonal Strut Dead Load= *8.4* *kips*
Diagonal Strut Live Load, Ped= *12.6* *kips*
Diagonal Strut thickness= *0.8750* *in* *(7/8")*
RF= ***5.50***

U1 & U11
Dead Load Reaction= *15.3* *kips*
Live Load Reaction, Ped= *24.5* *kips*
Top Chord Web Thickness= *1.3125* *in*
RF= ***4.17***

U2 & U10
Dead Load Reaction= *5.2* *kips*
Live Load Reaction, Ped= *7.8* *kips*
Top Chord Web Thickness= *1.3125* *in*
RF= ***13.96***

Note: Thickness of Top chord web taken as 5/16" web plate + 1/4" filler plate + 3/4" additional plate.

RATING EQUATIONS FOR PIN ELEMENTS

Design equation for the interaction of shear and flexure: LRFD 6.7.6.2.1

$$\frac{6.0M_u}{\phi_f D^3 F_y} + \left(\frac{2.2V_u}{\phi_v D^2 F_y}\right)^3 \leq 0.95$$

Modification of design equation was made to obtain the following rating equation

$$\frac{1}{0.95}\left[\frac{6.0M_u}{\phi_f D^3 F_y} + \left(\frac{2.2V_u}{\phi_v D^2 F_y}\right)^3\right] \leq 1$$

$$RF = \frac{1 - \frac{1}{0.95}\left[\frac{6.0M_{DL}}{\phi_f D^3 F_y} + \left(\frac{2.2V_{DL}}{\phi_v D^2 F_y}\right)^3\right]}{\frac{1}{0.95}\left[\frac{6.0M_{LL}}{\phi_f D^3 F_y} + \left(\frac{2.2V_{LL}}{\phi_v D^2 F_y}\right)^3\right]}$$

Design Equation for Bearing Resistance of Pin: LRFD 6.7.6.2.2

$$(R_{pB})_r = \phi_b (R_{pB})_n = \phi_b (1.5tDF_y)$$

$$RF = \frac{\phi_b (1.5tDF_y) - V_{DL}}{V_{LL}}$$

GM2 ASSOCIATES	BY	LV	DATE	01/08/18	SHEET	1	OF	1
	CHKD BY	JG	DATE	01/09/18	PROJECT	Simsbury		
	SUBJECT	Top Chord Pin Load Rating - H10						

Top Chord Pin Load Rating Summary: H10

Panel Point	Shear	Bearing			Controlling
		Vertical Strut	Diagonal Strut	Top Chord	
U1 & U11	10.24	3.64	8.96	8.10	3.64
U2 & U10	27.03	4.89	12.43	19.17	4.89
				Min.	**3.64**

Top Chord Pin Loads:

Resultant Factored Loads on Pin:

	Shear (kips)		Moment (k-ft)	
Panel Point	DL	LL, H10	DL	LL, H10
U1 & U11	19.18	22.09	0.73	0.69
U2 & U10	10.43	9.76	0.30	0.29

Reactions on Top Pin:

	DL (kips)		LL, H10 (kips)	
	Horiz.	Vertical	Horiz.	Vertical
U1 & U11	9	12.4	4.7	11.7
U2 & U10	0.2	5.2	3.5	4.5

	Diameter of Pin=	2.5	in	
	Fy=	47.0	ksi	
	ϕ_f=	1.0		
	ϕ_v=	1.0		
	ϕ_b=	1.0		
Load Factors:	γ_{DL} =	1.25		
	γ_{LL} =	1.75		
	Condition Factor=	0.95		(fair condition)
	System Factor=	0.9		
	Cond.&Sys. Factor=	0.86		

Shear Load Rating:

RF_U1 & U11: RF= 10.24

RF_U2 & U10: RF= 27.03

Bearing Load Rating

U1 & U11				U2 & U10			
Vertical Strut Dead Load=	2.9	kips		Vertical Strut Dead Load=	6.8	kips	
Vertical Strut Live Load H10=	6.8	kips		Vertical Strut Live Load H10=	4.5	kips	
Vertical Strut Web Thickness=	0.3125	in	(5/16")	Vertical Strut Web Thickness=	0.3125	in	(assumed as 5/16")
	RF= 3.64				RF= 4.89		

U1 & U11				U2 & U10			
Diagonal Strut Dead Load=	13.1	kips		Diagonal Strut Dead Load=	8.4	kips	
Diagonal Strut Live Load H10=	6.8	kips		Diagonal Strut Live Load H10=	5.6	kips	
Diagonal Strut thickness=	0.8125	in	(13/16")	Diagonal Strut thickness=	0.8750	in	(7/8")
	RF= 8.96				RF= 12.43		

U1 & U11			U2 & U10		
Dead Load Reaction=	15.3	kips	Dead Load Reaction=	5.2	kips
Live Load Reaction H10=	12.6	kips	Live Load Reaction H10=	5.7	kips
Top Chord Web Thickness=	1.3125	in	Top Chord Web Thickness=	1.3125	in
	RF= 8.10			RF= 19.17	

Note: Thickness of Top chord web taken as 5/16" web plate + 1/4" filler plate + 3/4" additional plate.

RATING EQUATIONS FOR PIN ELEMENTS

Design equation for the interaction of shear and flexure: LRFD 6.7.6.2.1

$$\frac{6.0M_u}{\phi_f D^3 F_y} + \left(\frac{2.2V_u}{\phi_v D^2 F_y}\right)^3 \leq 0.95$$

Modification of design equation was made to obtain the following rating equation

$$\frac{1}{0.95}\left[\frac{6.0M_u}{\phi_f D^3 F_y} + \left(\frac{2.2V_u}{\phi_v D^2 F_y}\right)^3\right] \leq 1$$

$$RF = \frac{1 - \frac{1}{0.95}\left[\frac{6.0M_{DL}}{\phi_f D^3 F_y} + \left(\frac{2.2V_{DL}}{\phi_v D^2 F_y}\right)^3\right]}{\frac{1}{0.95}\left[\frac{6.0M_{LL}}{\phi_f D^3 F_y} + \left(\frac{2.2V_{LL}}{\phi_v D^2 F_y}\right)^3\right]}$$

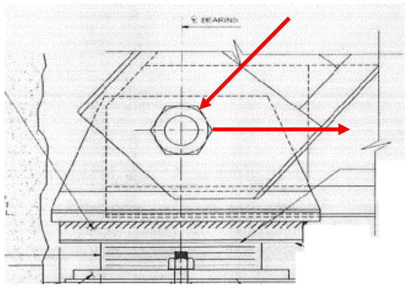
Design Equation for Bearing Resistance of Pin: LRFD 6.7.6.2.2

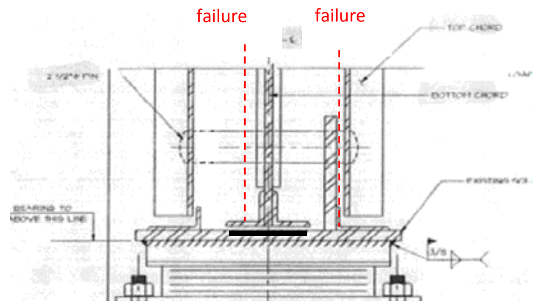
$$(R_{pB})_r = \phi_b (R_{pB})_n \qquad = \phi_b(1.5tDF_y)$$

$$RF = \frac{\phi_b(1.5tDF_y) - V_{DL}}{V_{LL}}$$

Pin Load Rating Summary: Support Pin at Northwest Support (missing bearing plate; Rating @ location of controlling member forces)

Support Pin: **Note: Controlling member forces for failure plane 2 are outside of SL area. Therefore, no SL is considered for this case.**





Loads on Support Pin:

	DL (kips)	LL, Ped (kips)	LL, H10 (kips)
Top Chord	52.67	81.58	21.8
Bottom Chord	36.05	56.28	15.04

Member Forces:

	Failure Plane 1						Failure Plane 2					
	Moment			Shear			Moment			Shear		
	DL (kip-ft)	LL, Ped (kip-ft)	LL, H10 (kip-ft)	DL (kip)	LL, Ped (kip)	LL, H10 (kip)	DL (kip-ft)	LL, Ped (kip-ft)	LL, H10 (kip-ft)	DL (kip)	LL, Ped (kip)	LL, H10 (kip)
My, Vy				18.19	28.1	7.49	6.57	10.14	2.72	18.19	28.1	7.49
Mz, Vz				19.1	29.49	7.9	6.26	9.66	2.58	19.1	29.49	7.9
Pin (service)	0.00	0.00	0.00	26.38	40.73	10.89	9.07	14.00	3.75	26.38	40.73	10.89
Pin (factored)	0.00	0.00	0.00	32.97	71.28	19.05	11.34	24.51	6.56	32.97	71.28	19.05

Reactions on Support Pin:

DL (kips)		LL, Ped (kips)		LL, H10 (kips)	
Horiz.	Vertical	Horiz.	Vertical	Horiz.	Vertical
18.19	38.19	28.1	58.98	7.49	15.8

Original Pin Dia. @ threads=	*2.50*	*in*	
Original Pin Dia. Inside, D_{in}=	*3.00*	*in*	
Depth of pin Section Loss Plane 1, d_{SL1}=	*0.50*	*in*	*(section loss on bottom half of pin)*
Depth of pin Section Loss Plane 2, d_{SL2}=	*0.00*	*in*	*(No SL @ location of controlling forces near bottom chord web)*
Area of top half of pin Plane 1, $A_{pin,top1}$=	*3.53*	*in^2*	$=0.5*\pi*D_{in}^2/4$
Area of top half of pin Plane 2, $A_{pin,top2}$=	*3.53*	*in^2*	$=0.5*\pi*D_{in}^2/4$
Area of bot. half of pin Plane 1, $A_{pin,bot1}$=	*2.45*	*in^2*	$=0.5*\pi*(D_{in}-d_{SL1})^2/4$
Area of bot. half of pin Plane 2, $A_{pin,bot2}$=	*3.53*	*in^2*	$=0.5*\pi*(D_{in}-d_{SL2})^2/4$
Total Area of deteriorated Pin Plane 1, $A_{eff,pin1}$=	*5.99*	*in^2*	$=A_{pin,top1}+A_{pin,bot1}$
Total Area of deteriorated Pin Plane 2, $A_{eff,pin2}$=	*7.07*	*in^2*	$=A_{pin,top2}+A_{pin,bot2}$
Effective Diameter of Pin Plane 1, D_1=	*2.76*	*in*	$=(4*A_{eff,pin1}/\pi)^{1/2}$
Effective Diameter of Pin Plpane 2, D_2=	*3.00*	*in*	$=(4*A_{eff,pin2}/\pi)^{1/2}$
Fy=	*47.0*	*ksi*	
ϕ_f=	*1.0*		
ϕ_v=	*1.0*		
ϕ_b=	*1.0*		
Load Factors: γ_{DL} =	*1.25*		
γ_{LL} =	*1.75*		
Condition Factor=	*0.85*	*(poor condition)*	
System Factor=	*0.9*		
Cond.&Sys. Factor=	*0.85*		

Shear Load Rating:

	Pedestrian Load	H10
RF Failure Plane 1:	RF= ***9.54***	RF= ***499.64***
RF Failure Plane 2:	RF= ***0.11***	RF= ***0.43***

BY	LV	DATE	02/06/19	SHEET 1	OF 1
CHKD BY	JG	DATE	02/08/19	PROJECT	Simsbury
SUBJECT	Support Pins Load Rating				

Bearing Load Rating

Pedestrian Load			H10		
Dead Load Reaction=	42.3	kips	Bottom Chord Dead Load=	42.3	kips
Live Load Reaction Ped=	65.3	kips	Bottom Chord Live Load H10=	17.5	kips
Bottom Chord web thickness=	1.75	in	(1/2" web plate + 2*(5/16") filler plate + 2*(5/16") additional plates)		
			(Note: pin supported by bottom chord at NW support due to missing plate)		
RF=	2.07		RF=	7.74	

Pedestrian Load			H10		
Top Chord Dead Load=	26.3	kips	Top Chord Dead Load=	26.3	kips
Top Chord Live Load Ped=	40.8	kips	Top Chord Live Load H10=	10.9	kips
Top Chord web thickness=	1.3125	in	(5/16" web plate + 1/4" filler plate + 3/4" additional plate)		
RF=	2.58		RF=	9.66	

Pedestrian Load			H10		
Dead Load Reaction=	18.2	kips	Dead Load Reaction=	18.2	kips
Live Load Reaction Ped=	28.1	kips	Live Load Reaction H10=	7.5	kips
Bearing Plate Thickness=	0.75	in	(estimated thickness of bearing plate)		
RF=	2.06		RF=	7.73	

RATING EQUATIONS FOR PIN ELEMENTS

Design equation for the interaction of shear and flexure: LRFD 6.7.6.2.1

$$\frac{6.0M_u}{\phi_f D^3 F_y} + \left(\frac{2.2V_u}{\phi_v D^2 F_y}\right)^3 \leq 0.95$$

Modification of design equation was made to obtain the following rating equation

$$\frac{1}{0.95}\left[\frac{6.0M_u}{\phi_f D^3 F_y} + \left(\frac{2.2V_u}{\phi_v D^2 F_y}\right)^3\right] \leq 1$$

$$RF = \frac{1 - \frac{1}{0.95}\left[\frac{6.0M_{DL}}{\phi_f D^3 F_y} + \left(\frac{2.2V_{DL}}{\phi_v D^2 F_y}\right)^3\right]}{\frac{1}{0.95}\left[\frac{6.0M_{LL}}{\phi_f D^3 F_y} + \left(\frac{2.2V_{LL}}{\phi_v D^2 F_y}\right)^3\right]}$$

Design Equation for Bearing Resistance of Pin: LRFD 6.7.6.2.2

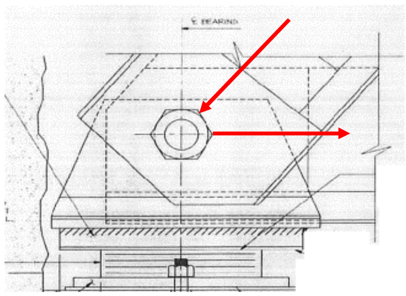
$$(R_{pB})_r = \phi_b (R_{pB})_n = \phi_b (1.5tDF_y)$$

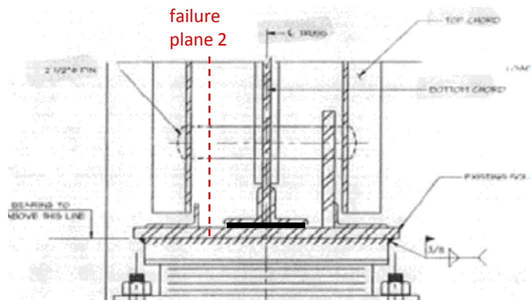
$$RF = \frac{\phi_b(1.5tDF_y) - V_{DL}}{V_{LL}}$$

BY	*LV*	DATE	*02/06/19*	SHEET 1	OF 1
CHKD BY	*JG*	DATE	*02/08/19*	PROJECT	*Simsbury*
SUBJECT	*Support Pins Load Rating*				

Pin Load Rating Summary: Support Pin at Northwest Support (missing bearing plate; Rating @ location with section loss)

Support Pin: **Note: Member forces evalulated at location of missing bearing plate, where existing SL occurs. SL included in load rating analysis**





Loads on Support Pin:

	DL (kips)	LL, Ped (kips)	LL, H10 (kips)
Top Chord	52.67	81.58	21.8
Bottom Chord	36.05	56.28	15.04

Member Forces:

	Failure Plane 1						Failure Plane 2					
	Moment			Shear			Moment			Shear		
	DL (kip-ft)	LL, Ped (kip-ft)	LL, H10 (kip-ft)	DL (kip)	LL, Ped (kip)	LL, H10 (kip)	DL (kip-ft)	LL, Ped (kip-ft)	LL, H10 (kip-ft)	DL (kip)	LL, Ped (kip)	LL, H10 (kip)
My, Vy							4.03	6.22	1.67	18.19	28.1	7.49
Mz, Vz							3.84	5.93	1.58	19.1	29.49	7.9
Pin (service)	0.00	0.00	0.00	0.00	0.00	0.00	5.57	8.59	2.30	26.38	40.73	10.89
Pin (factored)	0.00	0.00	0.00	0.00	0.00	0.00	6.96	15.04	4.02	32.97	71.28	19.05

Reactions on Support Pin:

DL (kips)		LL, Ped (kips)		LL, H10 (kips)	
Horiz.	Vertical	Horiz.	Vertical	Horiz.	Vertical
18.19	38.19	28.1	58.98	7.49	15.8

Original Pin Dia. @ threads=	*2.50*	*in*	
Original Pin Dia. Inside, D_{in}=	*3.00*	*in*	
Depth of pin Section Loss Plane 1, d_{SL1}=	*0.50*	*in*	*(section loss on bottom half of pin)*
Depth of pin Section Loss Plane 2, d_{SL2}=	*0.50*	*in*	*(section loss on bottom half of pin)*
Area of top half of pin Plane 1, $A_{pin,top1}$=	*3.53*	*in²*	$=0.5*\pi*D_{in}^2/4$
Area of top half of pin Plane 2, $A_{pin,top2}$=	*3.53*	*in²*	$=0.5*\pi*D_{in}^2/4$
Area of bot. half of pin Plane 1, $A_{pin,bot1}$=	*2.45*	*in²*	$=0.5*\pi*(D_{in}-d_{SL1})^2/4$
Area of bot. half of pin Plane 2, $A_{pin,bot2}$=	*2.45*	*in²*	$=0.5*\pi*(D_{in}-d_{SL2})^2/4$
Total Area of deteriorated Pin Plane 1, $A_{eff,pin1}$=	*5.99*	*in²*	$=A_{pin,top1}+A_{pin,bot1}$
Total Area of deteriorated Pin Plane 2, $A_{eff,pin2}$=	*5.99*	*in²*	$=A_{pin,top2}+A_{pin,bot2}$
Effective Diameter of Pin Plane 1, D_1=	*2.76*	*in*	$=(4*A_{eff,pin1}/\pi)^{1/2}$
Effective Diameter of Pin Plpane 2, D_2=	*2.76*	*in*	$=(4*A_{eff,pin2}/\pi)^{1/2}$
Fy=	*47.0*	*ksi*	
ϕ_f=	*1.0*		
ϕ_v=	*1.0*		
ϕ_b=	*1.0*		
Load Factors: γ_{DL} =	*1.25*		
γ_{LL} =	*1.75*		
Condition Factor=	*0.85*	*(poor condition)*	
System Factor=	*0.9*		
Cond.&Sys. Factor=	*0.85*		

Shear Load Rating:

	Pedestrian Load	H10
RF Failure Plane 1:	RF= -	RF= -
RF Failure Plane 2:	RF= ***0.25***	RF= ***1.00***

Bearing Load Rating

	Pedestrian Load			H10	
Dead Load Reaction=	*42.3*	*kips*	Bottom Chord Dead Load=	*42.3*	*kips*
Live Load Reaction Ped=	*65.3*	*kips*	Bottom Chord Live Load H10=	*17.5*	*kips*
Bottom Chord web thickness=	*1.75*	*in*	*(1/2" web plate + 2*(5/16") filler plate + 2*(5/16") additional plates)*		
			(Note: pin supported by bottom chord at NW support due to missing plate)		
	RF= **2.07**			RF= **7.74**	

	Pedestrian Load			H10	
Top Chord Dead Load=	*26.3*	*kips*	Top Chord Dead Load=	*26.3*	*kips*
Top Chord Live Load Ped=	*40.8*	*kips*	Top Chord Live Load H10=	*10.9*	*kips*
Top Chord web thickness=	*1.3125*	*in*	*(5/16" web plate + 1/4" filler plate + 3/4" additional plate)*		
	RF= **2.58**			RF= **9.66**	

	Pedestrian Load			H10	
Dead Load Reaction=	*18.2*	*kips*	Dead Load Reaction=	*18.2*	*kips*
Live Load Reaction Ped=	*28.1*	*kips*	Live Load Reaction H10=	*7.5*	*kips*
Bearing Plate Thickness=	*0.75*	*in*	*(estimated thickness of bearing plate)*		
	RF= **2.06**			RF= **7.73**	

RATING EQUATIONS FOR PIN ELEMENTS

Design equation for the interaction of shear and flexure: LRFD 6.7.6.2.1

$$\frac{6.0M_u}{\phi_f D^3 F_y} + \left(\frac{2.2V_u}{\phi_v D^2 F_y}\right)^3 \le 0.95$$

Modification of design equation was made to obtain the following rating equation

$$\frac{1}{0.95}\left[\frac{6.0M_u}{\phi_f D^3 F_y} + \left(\frac{2.2V_u}{\phi_v D^2 F_y}\right)^3\right] \le 1$$

$$RF = \frac{1 - \frac{1}{0.95}\left[\frac{6.0M_{DL}}{\phi_f D^3 F_y} + \left(\frac{2.2V_{DL}}{\phi_v D^2 F_y}\right)^3\right]}{\frac{1}{0.95}\left[\frac{6.0M_{LL}}{\phi_f D^3 F_y} + \left(\frac{2.2V_{LL}}{\phi_v D^2 F_y}\right)^3\right]}$$

Design Equation for Bearing Resistance of Pin: LRFD 6.7.6.2.2

$$(R_{pB})_r = \phi_b (R_{pB})_n = \phi_b(1.5tDF_y)$$

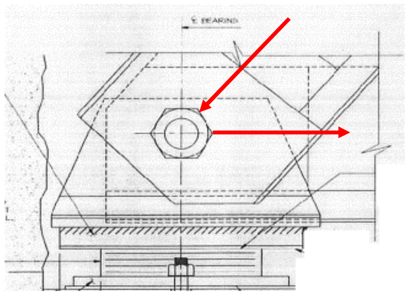
$$RF = \frac{\phi_b(1.5tDF_y) - V_{DL}}{V_{LL}}$$

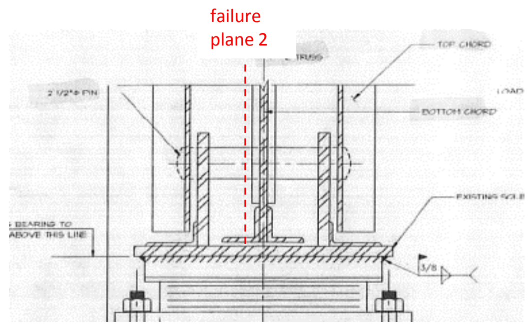
GM2 ASSOCIATES

BY	*LV*	DATE	*02/06/19*	SHEET	1 OF 1
CHKD BY	*JG*	DATE	*02/08/19*	PROJECT	*Simsbury*
SUBJECT	*Support Pins Load Rating*				

Pin Load Rating Summary: Support Pins at Southwest and East Supports (Rating @ location of controlling member forces)

Support Pin:





Loads on Support Pin:

	DL (kips)	LL, Ped (kips)	LL, H10 (kips)
Top Chord	52.67	81.58	21.8
Bottom Chord	36.05	56.28	15.04

Member Forces:

	Failure Plane 1						Failure Plane 2					
	Moment			Shear			Moment			Shear		
	DL (kip-ft)	LL, Ped (kip-ft)	LL, H10 (kip-ft)	DL (kip)	LL, Ped (kip)	LL, H10 (kip)	DL (kip-ft)	LL, Ped (kip-ft)	LL, H10 (kip-ft)	DL (kip)	LL, Ped (kip)	LL, H10 (kip)
My, Vy							0	0	0	18	28.15	7.5
Mz, Vz							4.69	7.33	1.95	0	0	0
Pin (service)	0.00	0.00	0.00	0.00	0.00	0.00	4.69	7.33	1.95	18.00	28.15	7.50
Pin (factored)	0.00	0.00	0.00	0.00	0.00	0.00	5.86	12.83	3.41	22.50	49.26	13.13

Reactions on Support Pin:

DL (kips)		LL, Ped (kips)		LL, H10 (kips)	
Horiz.	Vertical	Horiz.	Vertical	Horiz.	Vertical
0.1	19.1	0	29.5	0	7.9

Original Pin Dia. @ threads=	*2.50*	*in*	
Original Pin Dia. Inside, D_{in}=	*3.00*	*in*	
Depth of pin Section Loss Plane 1, d_{SL1}=	*0.50*	*in*	*(section loss on bottom half of pin)*
Depth of pin Section Loss Plane 2, d_{SL2}=	*0.00*	*in*	*(section loss on bottom half of pin)*
Area of top half of pin Plane 1, $A_{pin,top1}$=	*3.53*	*in²*	$=0.5*\pi*D_{in}^2/4$
Area of top half of pin Plane 2, $A_{pin,top2}$=	*3.53*	*in²*	$=0.5*\pi*D_{in}^2/4$
Area of bot. half of pin Plane 1, $A_{pin,bot1}$=	*2.45*	*in²*	$=0.5*\pi*(D_{in}-d_{SL1})^2/4$
Area of bot. half of pin Plane 2, $A_{pin,bot2}$=	*3.53*	*in²*	$=0.5*\pi*(D_{in}-d_{SL2})^2/4$
Total Area of deteriorated Pin Plane 1, $A_{eff,pin1}$=	*5.99*	*in²*	$=A_{pin,top1}+A_{pin,bot1}$
Total Area of deteriorated Pin Plane 2, $A_{eff,pin2}$=	*7.07*	*in²*	$=A_{pin,top2}+A_{pin,bot2}$
Effective Diameter of Pin Plane 1, D_1=	*2.76*	*in*	$=(4*A_{eff,pin1}/\pi)^{1/2}$
Effective Diameter of Pin Plpane 2, D_2=	*3.00*	*in*	$=(4*A_{eff,pin2}/\pi)^{1/2}$
Fy=	*47.0*	*ksi*	
ϕ_f=	*1.0*		
ϕ_v=	*1.0*		
ϕ_b=	*1.0*		
Load Factors: γ_{DL} =	*1.25*		
γ_{LL} =	*1.75*		
Condition Factor=	*0.85*	*(poor condition)*	
System Factor=	*0.9*		
Cond.&Sys. Factor=	*0.85*		

Shear Load Rating:

	Pedestrian Load	H10
RF Failure Plane 1:	RF= -	RF= -
RF Failure Plane 2:	RF= **0.64**	RF= **2.44**

BY	LV	DATE	02/06/19	SHEET	1 OF 1
CHKD BY	JG	DATE	02/08/19	PROJECT	Simsbury
SUBJECT	Support Pins Load Rating				

Bearing Load Rating

Pedestrian Load			H10		
Bottom Chord Dead Load=	36.1	kips	Bottom Chord Dead Load=	36.1	kips
Bottom Chord Live Load Ped=	56.3	kips	Bottom Chord Live Load H10=	15.0	kips
Bottom Chord web thickness=	1.75	in	(1/2" web plate + 2*(5/16") filler plate + 2*(5/16") additional plates)		
RF=	2.48		RF=	9.29	

Pedestrian Load			H10		
Top Chord Dead Load=	26.3	kips	Top Chord Dead Load=	26.3	kips
Top Chord Live Load Ped=	40.8	kips	Top Chord Live Load H10=	10.9	kips
Top Chord web thickness=	1.3125	in	(5/16" web plate + 1/4" filler plate + 3/4" additional plate)		
RF=	2.58		RF=	9.66	

Pedestrian Load			H10		
Dead Load Reaction=	19.1	kips	Dead Load Reaction=	19.1	kips
Live Load Reaction Ped=	29.5	kips	Live Load Reaction H10=	7.9	kips
Bearing Plate Thickness=	0.75	in	(estimated thickness of bearing plate)		
RF=	1.94		RF=	7.25	

RATING EQUATIONS FOR PIN ELEMENTS

Design equation for the interaction of shear and flexure: LRFD 6.7.6.2.1

$$\frac{6.0M_u}{\phi_f D^3 F_y} + \left(\frac{2.2V_u}{\phi_v D^2 F_y}\right)^3 \leq 0.95$$

Modification of design equation was made to obtain the following rating equation

$$\frac{1}{0.95}\left[\frac{6.0M_u}{\phi_f D^3 F_y} + \left(\frac{2.2V_u}{\phi_v D^2 F_y}\right)^3\right] \leq 1$$

$$RF = \frac{1 - \frac{1}{0.95}\left[\frac{6.0M_{DL}}{\phi_f D^3 F_y} + \left(\frac{2.2V_{DL}}{\phi_v D^2 F_y}\right)^3\right]}{\frac{1}{0.95}\left[\frac{6.0M_{LL}}{\phi_f D^3 F_y} + \left(\frac{2.2V_{LL}}{\phi_v D^2 F_y}\right)^3\right]}$$

Design Equation for Bearing Resistance of Pin: LRFD 6.7.6.2.2

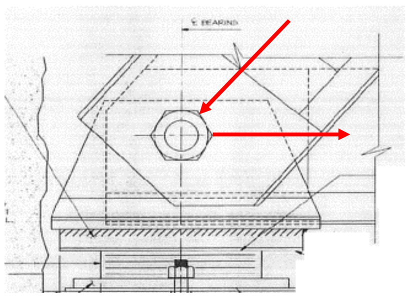
$$(R_{pB})_r = \phi_b (R_{pB})_n = \phi_b(1.5tDF_y)$$

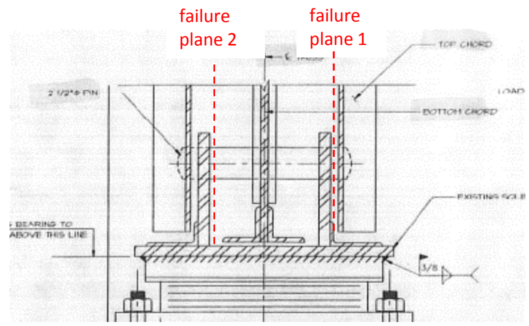
$$RF = \frac{\phi_b(1.5tDF_y) - V_{DL}}{V_{LL}}$$

GM2 ASSOCIATES						
	BY	*LV*	DATE	*02/06/19*	SHEET 1	OF 1
	CHKD BY	*JG*	DATE	*02/08/19*	PROJECT	*Simsbury*
	SUBJECT	*Support Pins Load Rating*				

Pin Load Rating Summary: Support Pins at Southwest and East Supports (Rating @ location with section loss)

Support Pin:





Loads on Support Pin:

	DL (kips)	LL, Ped (kips)	LL, H10 (kips)
Top Chord	52.67	81.58	21.8
Bottom Chord	36.05	56.28	15.04

Member Forces:

	Failure Plane 1						Failure Plane 2					
	Moment			Shear			Moment			Shear		
	DL (kip-ft)	LL, Ped (kip-ft)	LL, H10 (kip-ft)	DL (kip)	LL, Ped (kip)	LL, H10 (kip)	DL (kip-ft)	LL, Ped (kip-ft)	LL, H10 (kip-ft)	DL (kip)	LL, Ped (kip)	LL, H10 (kip)
My, Vy				18.19	28.1	7.49	0	0	0	18	28.15	7.5
Mz, Vz				19.1	29.49	7.9	1.5	2.35	0.62	0	0	0
Pin (service)	0.00	0.00	0.00	26.38	40.73	10.89	1.50	2.35	0.62	18.00	28.15	7.50
Pin (factored)	0.00	0.00	0.00	32.97	71.28	19.05	1.88	4.11	1.09	22.50	49.26	13.13

Reactions on Support Pin:

DL (kips)		LL, Ped (kips)		LL, H10 (kips)	
Horiz.	Vertical	Horiz.	Vertical	Horiz.	Vertical
0.1	19.1	0	29.5	0	7.9

Original Pin Dia. @ threads=	*2.50*	*in*	
Original Pin Dia. Inside, D_{in}=	*3.00*	*in*	
Depth of pin Section Loss Plane 1, d_{SL1}=	*0.50*	*in*	*(section loss on bottom half of pin)*
Depth of pin Section Loss Plane 2, d_{SL2}=	*0.50*	*in*	*(section loss on bottom half of pin)*
Area of top half of pin Plane 1, $A_{pin,top1}$=	*3.53*	*in^2*	$=0.5*\pi*D_{in}^2/4$
Area of top half of pin Plane 2, $A_{pin,top2}$=	*3.53*	*in^2*	$=0.5*\pi*D_{in}^2/4$
Area of bot. half of pin Plane 1, $A_{pin,bot1}$=	*2.45*	*in^2*	$=0.5*\pi*(D_{in}-d_{SL1})^2/4$
Area of bot. half of pin Plane 2, $A_{pin,bot2}$=	*2.45*	*in^2*	$=0.5*\pi*(D_{in}-d_{SL2})^2/4$
Total Area of deteriorated Pin Plane 1, $A_{eff,pin1}$=	*5.99*	*in^2*	$=A_{pin,top1}+A_{pin,bot1}$
Total Area of deteriorated Pin Plane 2, $A_{eff,pin2}$=	*5.99*	*in^2*	$=A_{pin,top2}+A_{pin,bot2}$
Effective Diameter of Pin Plane 1, D_1=	*2.76*	*in*	$=(4*A_{eff,pin1}/\pi)^{1/2}$
Effective Diameter of Pin Plpane 2, D_2=	*2.76*	*in*	$=(4*A_{eff,pin2}/\pi)^{1/2}$
Fy=	*47.0*	*ksi*	
ϕ_f=	*1.0*		
ϕ_v=	*1.0*		
ϕ_b=	*1.0*		
Load Factors: γ_{DL} =	*1.25*		
γ_{LL} =	*1.75*		
Condition Factor=	*0.85*	*(poor condition)*	
System Factor=	*0.9*		
Cond.&Sys. Factor=	*0.85*		

Shear Load Rating:

	Pedestrian Load	H10
RF Failure Plane 1:	RF= ***9.54***	RF= ***499.64***
RF Failure Plane 2:	RF= ***2.04***	RF= ***8.41***

Bearing Load Rating

Pedestrian Load			H10		
Bottom Chord Dead Load=	36.1	kips	Bottom Chord Dead Load=	36.1	kips
Bottom Chord Live Load Ped=	56.3	kips	Bottom Chord Live Load H10=	15.0	kips
Bottom Chord web thickness=	1.75	in	(1/2" web plate + 2*(5/16") filler plate + 2*(5/16") additional plates)		
RF=	2.48		RF=	9.29	

Pedestrian Load			H10		
Top Chord Dead Load=	26.3	kips	Top Chord Dead Load=	26.3	kips
Top Chord Live Load Ped=	40.8	kips	Top Chord Live Load H10=	10.9	kips
Top Chord web thickness=	1.3125	in	(5/16" web plate + 1/4" filler plate + 3/4" additional plate)		
RF=	2.58		RF=	9.66	

Pedestrian Load			H10		
Dead Load Reaction=	19.1	kips	Dead Load Reaction=	19.1	kips
Live Load Reaction Ped=	29.5	kips	Live Load Reaction H10=	7.9	kips
Bearing Plate Thickness=	0.75	in	(estimated thickness of bearing plate)		
RF=	1.94		RF=	7.25	

RATING EQUATIONS FOR PIN ELEMENTS

Design equation for the interaction of shear and flexure: LRFD 6.7.6.2.1

$$\frac{6.0M_u}{\phi_f D^3 F_y} + \left(\frac{2.2V_u}{\phi_v D^2 F_y}\right)^3 \leq 0.95$$

Modification of design equation was made to obtain the following rating equation

$$\frac{1}{0.95}\left[\frac{6.0M_u}{\phi_f D^3 F_y} + \left(\frac{2.2V_u}{\phi_v D^2 F_y}\right)^3\right] \leq 1$$

$$RF = \frac{1 - \frac{1}{0.95}\left[\frac{6.0M_{DL}}{\phi_f D^3 F_y} + \left(\frac{2.2V_{DL}}{\phi_v D^2 F_y}\right)^3\right]}{\frac{1}{0.95}\left[\frac{6.0M_{LL}}{\phi_f D^3 F_y} + \left(\frac{2.2V_{LL}}{\phi_v D^2 F_y}\right)^3\right]}$$

Design Equation for Bearing Resistance of Pin: LRFD 6.7.6.2.2

$$(R_{pB})_r = \phi_b (R_{pB})_n = \phi_b(1.5tDF_y)$$

$$RF = \frac{\phi_b(1.5tDF_y) - V_{DL}}{V_{LL}}$$

Steel Beam Ends Load Rating - UnStiffened Web

v1.1 3/10/2017

Description: The purpose of this worksheet is to compute rating factors for Steel Beams without bearing stiffeners, and provide a sample calculation for the approval of the CTDOT Beam End SpreadSheet v2.

References:

MBE - AASHTO The Manual for Bridge Evaluation 2nd ed. 2014 with 2016 Interim Revisions
LRFD - AASHTO LRFD Bridge Design Specifications 7th ed. with 2016 Interim Revisions
BLRM - CTDOT Bridge Load Rating Manual v1.0

Orange backgrounds signifies input regions

Bridge: Flower Bridge
Span: 1
Girder: Bottom Chord
Location: Northwest Support

Section Depth	D	6	in
Web Thickness	t.w	1.75	in
Web Yield Strength	F.yw	38	ksi
E of Steel	E	29000	ksi
Flange Thickness	t.f	0.5	in
Flange + Fillet Thickness	K	0.5	in
Length of Bearing	N	3	in
Minimum End Length	L.OH	6	in
Web Thickness Loss	SL.w	0	%
Flange + Fillet Loss	SL.K	0	%
Flange Loss	SL.tf	0	%

Note: This CTDOT Beam End Rating Spreadsheet is being used to load rate the bottom chord web, which is acting as a support at the northwest corner of the bridge.

Units

$D := D \cdot in$

$t_w := t_w \cdot in$

$F_{yw} := F_{yw} \cdot ksi$

$E := E \cdot ksi$

$t_f := t_f \cdot in$

$K := K \cdot in$

$N := N \cdot in$

$L_{OH} := L_{OH} \cdot in$

$SL_w := SL_w \cdot \%$

$SL_K := SL_K \cdot \%$

$SL_{tf} := SL_{tf} \cdot \%$

As-Inspected Girder Section Properties

Web Thickness $t_w := t_w \cdot (1 - SL_w) = 1.75 \cdot in$

Flange + Fillet $K := K \cdot (1 - SL_K) = 0.5 \cdot in$

Flange $t_f := t_f \cdot (1 - SL_{tf}) = 0.5\,in$

LRFD Resistance Factors, MBE 6A.6.3 & LRFD 6.5.4.2

For Bearing On Milled Surfaces

$\phi_b := 1.0$

For Web Crippling

$\phi_w := 0.80$

LRFR Factors

System Factor, MBE 6A.4.2.4 & MBE Table 6A.4.2.4-1

$\phi_s := 0.90$

For All Other Girder Bridges and Slab Bridges

Condition Factor, MBE 6A.4.2.3 & MBE Table 6A.4.2.3-1

$\phi_c := 0.85$

Poor Condition + Increased by 0.05 for field measured losses, MBE C6A.4.2.3

Beam Ends Without Bearing Stiffeners, LRFD D6.5.2
The following calculations are applicable only for UnStiffened Beam Ends

Web Local Yielding, LRFD D6.5.2

Nominal Resistance to the Concentrated Loading, LRFD D6.5.2-2 or D6.2.2-3

$$R_{nb} := \begin{cases} (5 \cdot K + N) \cdot F_{yw} \cdot t_w & \text{if } L_{OH} > D \\ \left(2.5 \cdot K + N + \min\left(2.5 \cdot K, \max\left(0, L_{OH} - \frac{N}{2}\right)\right)\right) \cdot F_{yw} \cdot t_w & \text{otherwise} \end{cases}$$

$$R_{nb} = 365.75 \cdot kip$$

$R_{ub} := \phi_b \cdot R_{nb} = 365.75 \cdot kip$

Web Crippling, LRFD D6.5.3

Nominal Resistance to the Concentrated Loading, LRFD D6.5.3-2, D6.5.3-3, or D6.5.3-4

$$R_{nw} := \begin{cases} \left[0.8 \cdot t_w^2 \cdot \left[1 + 3 \cdot \left(\frac{N}{D}\right) \cdot \left(\frac{t_w}{t_f}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot F_{yw} \cdot t_f}{t_w}}\right] & \text{if } L_{OH} \ge \frac{D}{2} \\ \left[0.4 \cdot t_w^2 \cdot \left[1 + 3 \cdot \left(\frac{N}{D}\right) \cdot \left(\frac{t_w}{t_f}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot F_{yw} \cdot t_f}{t_w}}\right] & \text{if } \frac{N}{D} \le 0.2 \\ \left[0.4 \cdot t_w^2 \cdot \left[1 + \left(\frac{4N}{D} - 0.2\right) \cdot \left(\frac{t_w}{t_f}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot F_{yw} \cdot t_f}{t_w}}\right] & \text{otherwise} \end{cases}$$

$$R_{nw} = 1.488 \times 10^4 \cdot kip$$

$R_{uw} := \phi_w \cdot R_{nw} = 1.19 \times 10^4 \cdot kip$

Loading

DC Load Factor $\gamma_{DC} := 1.25$

DW Load Factor $\gamma_{DW} := 1.50$

DC Load $DC := 38.19kip$

DW Load $DW := 0kip$

Vehicle	Class	Load Factor	Load (kip)
Pedestrian	-	1.75	58.98
H10	-	1.75	15.8

$i := 0 .. 1$

Rating

Determine Minimum Capacity

$R_n := \min(R_{ub}, R_{uw}) = 365.75 \cdot kip$

$C := \max(0.85, \phi_s \cdot \phi_c) \cdot R_n = 310.887 \cdot kip$

Compute Ratings

$$RF_i := \frac{C - \gamma_{DC} \cdot DC - \gamma_{DW} \cdot DW}{\gamma_{LL_i} \cdot LL_i \cdot kip}$$

Vehicle	Class	Rating
Pedestrian	-	2.54
H10	-	9.51

APPENDIX C: LOAD RATING ANALYSIS (1990)

DRAKE HILL ROAD TRUSS BRIDGE OVER FARMINGTON RIVER

IN THE TOWN OF

SIMSBURY, CONNECTICUT

ENGINEERING STUDY TO DETERMINE

LIVE LOAD CAPACITY OF THE BRIDGE DECK

AS A VIEWING PLATFORM

SUBMITTED TO

MR. FRANK ROSSI, P.E.

DIRECTOR OF PUBLIC WORKS

TOWN OF SIMSBURY, CONNECTICUT

APRIL 1990

MACCHI ENGINEERS

A. JOHN MACCHI
JAMES BROCKMAN

MACCHI ENGINEERS

44 GILLETT STREET
HARTFORD, CT 06105-2694

203/549-6190 — FAX 203/524-5088

April 18, 1990

Mr. Frank Rossi, P.E.
Director of Public Works
Town of Simsbury
760 Hopmeadow Road
Simsbury, CT 06070

Re: Load Capacity of the Drake Hill Road
Truss Bridge as Viewing Platform

Dear Mr. Rossi,

Pursuant to your request we have evaluated the existing conditions of the Drake Hill Road truss bridge for use as a viewing platform during sporting events on the Farmington River.

It is our recommendation that the occupancy of the bridge deck be limited to 300 persons, until the scheduled repairs can be made. The 300 onlookers may be concentrated at any location or on either side of the bridge.

Enclosed for your review are 2 copies of the engineering study for this load determination.

Please contact us if you need additional information.

Very truly yours,

MACCHI ENGINEERS

James Brockman

JAMES BROCKMAN, P.E.
PARTNER

encl.

DETERMINATION OF LIVE LOAD CAPACITY
FOR THE
DRAKE HILL ROAD BRIDGE AS A VIEWING PLATFORM

INTRODUCTION

The historic truss bridge over the Farmington River, now permanently closed to vehicular traffic, serves as a pedestrian and bicycle path for the residents of Simsbury.

The bridge, built in 1892, has a span of 183 feet. The two trusses are spaced at 17.3 feet. The truss height at the midpoint of the span is about 24 feet. Steel floorbeams, 12" deep and spaced at about 3'-10" frame into the truss bottom chords. These floorbeams support timber stringers and timber planks for the bridge deck.

Macchi Engineers was retained by the Town of Simsbury in 1977 to design the reconstruction of the lower vertical truss panel points, because of extensive section loss and rust delaminations. The loads between diagonals, struts and bottom chords are now transferred through new rigid connections, thereby bypassing most of the load transfer through the original hinge points, which are extensively deteriorated. Macchi Engineers inspected this bridge again in 1983 and submitted an inspection report with load ratings to the Town of Simsbury.

The Connecticut Department of Transportation performed and in depth inspection in 1988. Their report included the following deficiencies at this bridge:

a) Lateral bracing under the bridge deck, severed in numerous places.
b) Paint is flaking and the steel is rusted throughout.
c) Frozen roller bearings.
d) Mortar voids and cracking of stone and some settlement at the east abutment.

On March 16, 1990 Macchi Engineers inspected most of the lateral bracing connections at the truss panel points for steel loss due to corrosion, and performed a cursory inspection of the entire bridge to determine the live load capacity of the bridge as a viewing platform.

FINDINGS

Bridge Deck:

The bituminous overlay and 1.5" thick longitudinal timber planking are only in fair condition. The timber curbs are worn and split.

The 3" x 12" creosoted transverse planking and 3" x 8 creosoted stringers are generally in very good condition, with a load rating in excess of 500 pounds per square foot (psf.)

The iron lattice railing has impact damage, and is bent and broken at several locations. The railing is also extensively rusted.

Lower Truss Panel Points:

Vertical struts, truss diagonals, and floor beams are framed into the bottom chords at each panel point. The framing details include an 8" wide horizontal gusset plate for the lateral "X" bracing. Extensive deterioration occurred over the years at these lower truss panel points which are located below the bridge deck, and where salt laden sand accumulations caused considerable loss of steel sections.

The 1977 panel point repairs are in sound condition. However, extensive rust delaminations on the original construction have to be removed by sandblasting to determine the extent of required repairs.

The horizontal bracing is extremely deteriorated. At 6 of the 26 panel points, the gusset plate is severed due to rusting and at another 8 panel points, the section loss of the gusset is up to 75%. In addition, the section loss of the bracing angles at these locations is as high as 60%.

The deteriorated condition essentially eliminates the effectiveness of the lateral bracing system. As a result, the bridge is at risk of damage during high wind velocities. The loss of bracing has also reduced the torsional stiffness of the bridge. This could cause noticeable swaying, should an entire group of onlookers during sporting events move in unison from one side to the other.

The panel points have to be sandblasted, before the extent of required repairs can be determined.

Floor Beams:

The 12 inch deep floor beams (12Ix31.8) have a load capacity in excess of 200 psf. Loss of flange and web steel has occurred at many floor beams adjacent to the panel points where the floor beams support the horizontal gusset plate for the lateral bracing. However, this loss is not critical and need not be repaired.

Painting:

The bridge paint is worn and peeling. It appears that the bridge has not been painted for decades. Rusting is evident all over the trusses, with heavy rusting and steel delamination generally located in the splash zone and at the panel points. Complete sandblasting and a three coat paint application should be considered.

Bridge Trusses:

The outstanding leg of the inside bottom angle of the bottom chord has some section loss at the downstream panel points L8 - L10, as reported by DOT in 1988.

There is some minor impact damage on the vertical struts. The remaining truss members are generally in sound condition.

Bridge Bearings:

The roller bearings are frozen and have rust laminations, as reported by DOT in 1988.

Abutments:

The abutments are constructed of stone masonry. Mortar joint voids and vertical cracks occur at the east abutment and south east wingwall. The south east bearing area has settled, as reported by DOT in 1988.

LOAD RATINGS

Allowable Stresses:

When this bridge was built during the 1890's, published catalogs from rolling mills recommended 16,000 psi allowable stresses for buildings and 12,500 psi for bridges.

The AASHTO Manual for Maintenance Inspection of Bridges limits allowable stresses for unknown steel from prior to 1905 to 14,000 psi for normal inventory loading and allows up to 19,500 psi stresses for infrequent loadings.

Tensile tests were performed in 1977 by Henry Souther Laboratories on small coupons (3/16" thick x 1/2" wide), cut from the vertical struts on this bridge. These tests indicated a yield strength of 51,500 psi (pounds per square inch) for 2 coupons from channel webs and 47,400 psi for a lacing bar. Even though these tests may not be representative of all the steel on this bridge, a yield strength of 36,000 psi was used as the basis for load rating computations, after the bridge has been repaired.

Bridge Loadings:

As tabulated on page 6, the maximum number persons, weighing an average of 160 lbs, were determined for combined dead load and live load stresses limits from 14,000 to 20,000 psi.

The bottom chord dead load stresses at the center of the bridge are 9,700 psi, based on an estimated section loss of 8%. The live load stresses consist of 75%-80% direct tension due to truss action and 20%-25% bending of the bottom chords due to floor beam loads.

The maximum occupancy was established for the following three conditions:

A. Uniform occupancy along the bridge.
B. Occupancy on 8 of 12 truss bays.
C. Occupancy on 6 of 12 truss bays.

Tables "A" through "C" show the results of or findings. As an example: 730 persons uniformly spaced on one side of the bridge would cause a stress of 18,000 psi in the bottom chord, as shown in Table "A".

However, only 494 persons would cause the same stress range of 18,000 psi stress when densely packed at the center half of the bridge, as shown in Table "C".

Recommendations:

It is recommended to limit the maximum occupancy at this time to 300 persons, or to a stress limit of 16,000 psi.

After the lateral bracing system has been reconstructed and other deficiencies have been repaired, the occupancy load could be increased to 500 or 600 persons.

TABLE "A" - Uniform Distribution of Occupancy on one Side of Bridge - (All 12 Bays)

Max. Stress in Truss Bottom Chord	Bridge Deck Live Load PSF	Live Load Each Truss LBS **	No. of Persons at 160 LB. Each			Rows of People	Multiplier for 2 Trusses
			Per Bay One Truss	Total One Truss	Total Persons on Bridge		
14,000 psi	33.1	48,425	25	300	339	3	1.13
16,000 psi	48.5	70,948	37	444	519	4	1.17
18,000 psi	63.8	93,471	49	588	735	6	1.25
19,500 psi	75.4	110,363	57	684	889	7	1.30
20,000 psi	79.2	115,994	60	720	972	8	1.35

TABLE "B" - Occupancy on Inner 2/3 of Bridge - (8 of 12 Truss Bays)

Max. Stress in Truss Bottom Chord	Bridge Deck Live Load PSF	Live Load Each Truss LBS **	No. of Persons at 160 LB. Each			Rows of People	Multiplier for 2 Trusses
			Per Bay One Truss	Total One Truss	Total Persons on Bridge		
14,000 psi	36.3	35,416	28	224	253	3	1.13
16,000 psi	53.2	51,889	41	328	397	5	1.21
18,000 psi	70.0	68,361	53	424	530	6	1.25
19,500 psi	82.7	80,716	63	504	655	7	1.30
20,000 psi	86.9	84,834	66	528	744	9	1.41

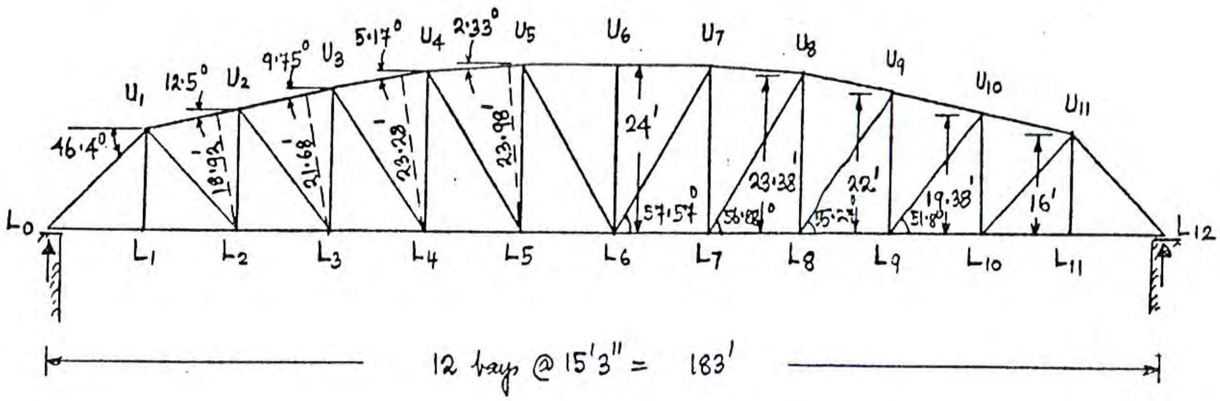
TABLE "C" - Occupancy on Inner Half of Bridge - (6 of 12 Truss Bays)

Max. Stress in Truss Bottom Chord	Bridge Deck Live Load PSF	Live Load Each Truss LBS **	No. of Persons at 160 LB. Each			Rows of People	Multiplier for 2 Trusses
			Per Bay One Truss	Total One Truss	Total Persons on Bridge		
14,000 psi	41.4	30,295	32	192	217	3	1.13
16,000 psi	60.6	44,385	46	276	334	5	1.21
18,000 psi	79.9	58,475	61	366	494	8	1.35
19,500 psi	94.3	69,043	72	432	635	10	1.47
20,000 psi	99.1	72,566	76	456	730	12	1.60

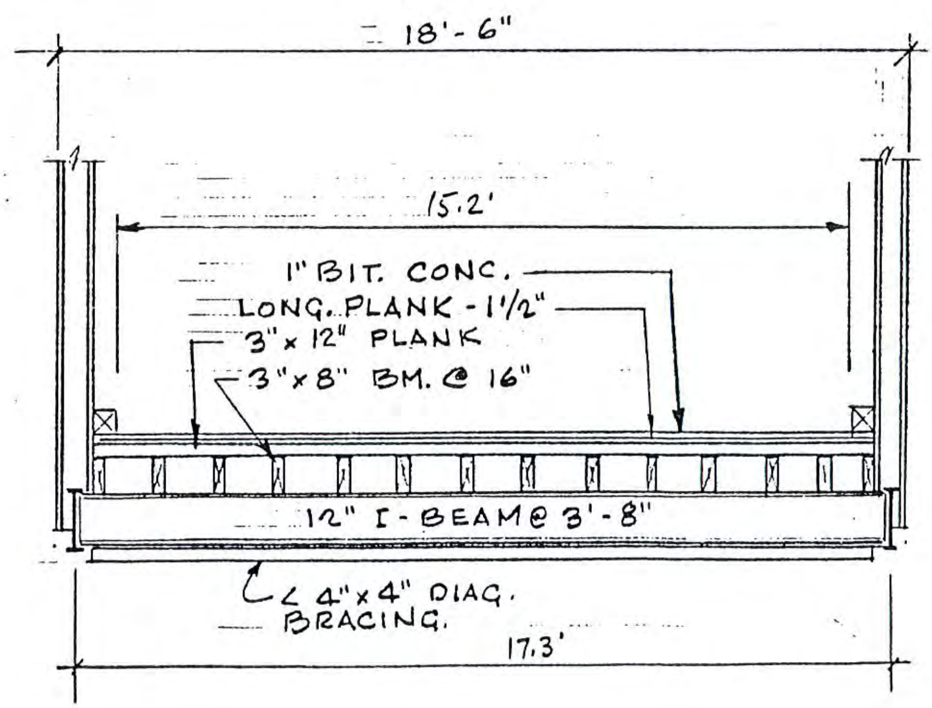
** ASSHTO H-15 Lane Loading = 70,000 Lb. per Truss
ASSHTO H-20 Lane Loading = 93,000 Lb. per Truss

4/16/90

DRAKE HILL ROAD BRIDGE OVER FARMINGTON RIVER



ELEVATION



TYPICAL SECTION

APPENDIX D: PEDESTRIAN LOAD PER AASHTO GUIDE SPECIFICATIONS

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

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

2—PHILOSOPHY

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

3—LOADS

3.1—PEDESTRIAN LOADING (PL)

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

C3.1

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

European codes appear to start with a higher nominal load (approx 105 psf), but then allow reductions based on loaded length. Additionally, the load factor applied is 1.5, resulting in a maximum factored load of (1.5)105 = 158 psf. For a long loaded length, this load can be reduced to as low as 50 psf, resulting in a factored load of (1.5)50 = 75 psf. The effect of resistance factors has not been accounted for in the above discussion of the European codes. There are, however, warnings to the designer that a reduction in the load based on loaded length may not be appropriate for structures likely to see significant crowd loadings, such as bridges near stadiums.

Consideration might be given to the maximum credible pedestrian loading. There is a physical limit on how much load can be applied to a bridge from the static weight of pedestrians. It appears that this load is around 150 psf, based on work done by Nowak (2000) from where Figures C1 through C3 were taken. Although there does not appear to be any available information relating to the probabilistic distribution of pedestrian live loading, knowing the maximum credible load helps to define the limits of the upper tail of the distribution of load. The use of a 90 psf nominal live load in combination with a load factor of 1.75 results in a loading of 158 psf, which provides a marginal, but sufficient, reserve compared with the maximum credible load of 150 psf.

Figure C3.1-1—Live Load of 50 psf

Figure C3.1-2—Live Load of 100 psf

Figure C3.1-3—Live Load of 150 psf

3.2—VEHICLE LOAD (LL)

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

C3.2

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

APPENDIX E: STRUCTURAL STEEL MATERIAL TESTING REPORT

Destructive and Non-Destructive (NDT) Material Testing Report

Bridge No. 03984

Old Drake Hill Road Bridge (Flower Bridge)

Over

Farmington River

Simsbury, Connecticut

Prepared by:

GM2 Associates, Inc.

115 Glastonbury Blvd.

Glastonbury, CT 06033

INDEX OF REPORT

EXECUTIVE SUMMARY

GM2 Associates, Inc. (GM2) was retained by the Town of Simsbury to perform destructive and non-destructive (NDT) testing on Bridge No. 03984 Old Drake Hill Road Bridge (Flower Bridge) to determine the yield strength of the structural elements of the bridge. Flower Bridge is a 183 feet long bridge consisting of two Parker trusses carrying Old Drake Hill Road over Farmington River in the Town of Simsbury, Connecticut.

Tensile tests were performed on two (2) steel coupons extracted from the top chord of the north and south trusses of the bridge. The average yield strength obtained from the tensile tests was 55 ksi. This is consistent with results from previous tensile tests performed on steel coupons obtained from vertical struts and a lacing bar in 1977, where an average yield strength of 50 ksi was obtained. A statistical analysis of the tensile stress results was performed per AASHTO Manual for Bridge Evaluation (MBE) and the recommended yield strength to be used for structural analysis was found to be 38 ksi.

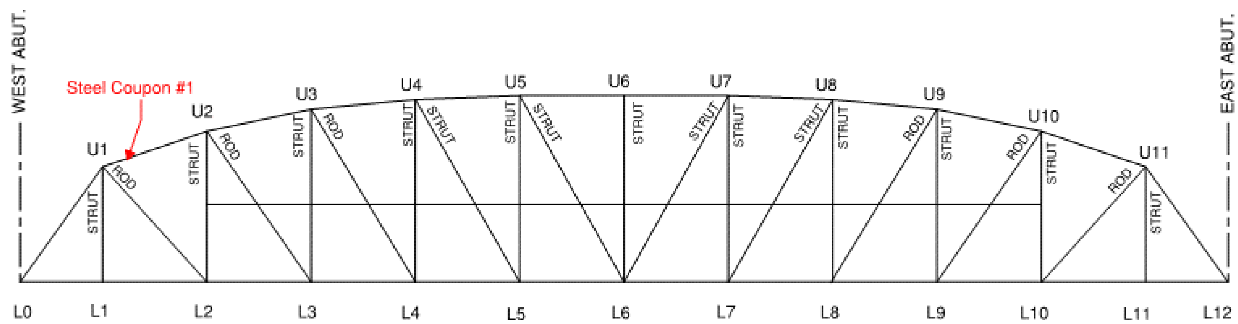
Extracting steel coupons from the tension elements of the bridge was not recommended in order to minimize the effects of the reduced area on the structural integrity of the bridge. Therefore, in-situ hardness readings were obtained on these members to correlate the results to a yield strength. The hardness to yield strength conversion was verified by performing hardness readings on the steel coupons, converting it to yield strength, and comparing with the yield strength obtained from the tensile test. A good correlation between the yield strength obtained from the tensile tests and that obtained from the hardness reading of the steel coupons was observed.

However, the in-situ hardness readings, hence yield strengths converted from hardness, were found to be higher than that obtained from the tensile test. This may be due to field conditions such as cold temperature. A correction factor for field conditions was applied to the hardness readings and a statistical analysis was performed per MBE. The results from the statistical analysis indicate a yield strength of 40 ksi for the truss members and a yield strength of 47 ksi for the pins to be used for structural analysis.

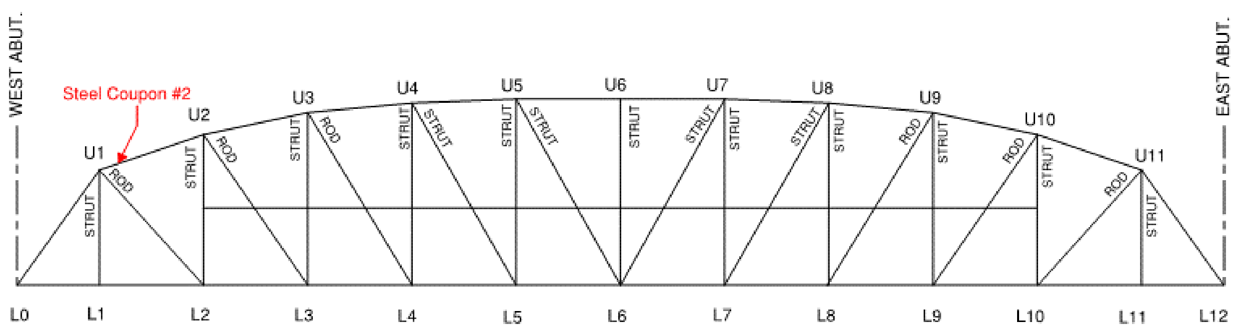
Based on the results from the material testing it is recommended that a yield strength of 38 ksi and 47 ksi be used for the truss members and the pins, respectively, in the load rating analysis.

TENSILE TEST ON STEEL COUPONS

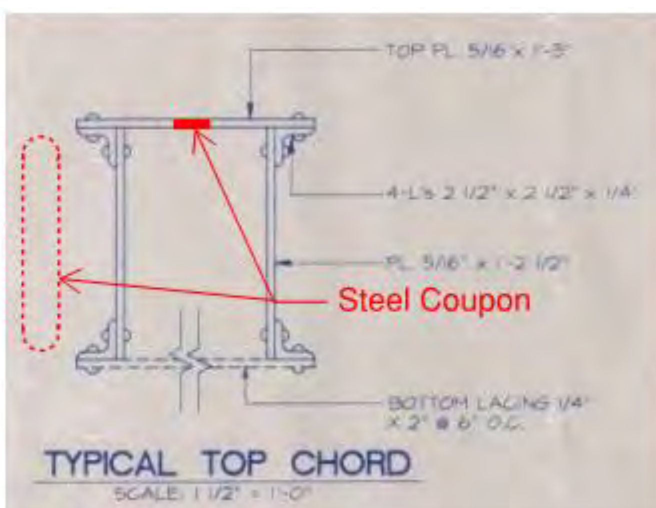
Independent Materials Testing Laboratory Testing, Inc. (IMTL) was retained to extract a total of two (2) steel coupons from the top plate of the top chord of both the north and south truss and perform tensile tests. The schematic below shows the locations of the steel coupons extraction.



SOUTH TRUSS - SOUTH ELEVATION



NORTH TRUSS - SOUTH ELEVATION



CROSS-SECTION OF TOP CHORD

Steel Coupons Extraction Photos:

Photo 1: Coupons extraction operations

Photo 2: Extracted steel coupons

Photo 3: Top plate of top chord after coupon extraction.

Tensile Test Results:

Results from the tensile test indicated a yield strength of 54 ksi and 56 ksi for the steel coupon from the north and south truss, respectively. These results are consistent with results from previous tensile tests performed in 1977. In that case, two (2) steel coupons from the channel webs of the vertical strut and a lacing bar were tested. The measured yield strength was 51.5 ksi for the channel webs and 47.4 ksi for the lacing bar. A summary of the results from the 1977 testing and detailed results from IMTL testing are included in Appendix 1 of this report.

Yield Strength for Load Rating Analysis:

In accordance with AASHTO Manual for Bridge Evaluation (MBE), the yield strength to be used for load rating analysis is calculated as the lower bound of the 95% confidence interval for the data obtained from the material testing. In addition, the yield strength measured from the tensile test was adjusted to account for the dynamic effects of the tensile test per AISC Steel Construction Manual Appendix 5, thus obtaining an equivalent static yield strength, F_{ys}. The equivalent static yield strength was obtained from the following equation:

$$F_{ys} = R(F_y - 4) \qquad \text{(AISC Eq. C-A-5-1)}$$

where

F_{ys} = static yield stress, ksi

F_y = reported yield stress from tensile tests, ksi

R = 0.95 for tests taken from web specimens

=1.00 for tests taken from flange specimens

The statistical analysis for the tensile tests results and final recommended yield strength for load rating are shown in the table below. Note that the results from the tensile tests performed in 1997 have been included in order to have a more representative sample size for the statistical analysis.

Member	F_y, ksi	*Adjust. For dyn. Effect	
		R	F_{ys}, ksi
Lacing Bar (1977)	47.4	0.95	41.23
Vertical Channel Web 1 (1977)	51.5	0.95	45.13
Vertical Channel Web 2 (1977)	51.5	0.95	45.13
Top Chord Top Plate (North Truss) (2018)	54	1	50.00
Top Chord Top Plate (South Truss) (2018)	56	1	52.00

*AISC 14th ed Appendix 5; pg. 16.1-498, Eq. C-A-5-1)

Average, μ=	46.70		
Std. Dev., σ=	5.14		=max(Std.dev, 0.11*μ)
Confidence Level (%)=	95		
z=	1.645		
Lower Limit of 95% Confidence Interval=	38.25	ksi	=μ-z*σ
Yield Strength for Load Rating Analysis=	**38.00**	**ksi**	

IN-SITU HARDNESS TEST

GM2 performed in-situ hardness readings on all structural elements of the bridge to correlate the hardness results to a yield strength, thus estimating the yield strength of members for which extracting steel coupons was not practical.

The hardness to yield strength conversion was performed using correlation equations available in the literature and was verified by performing hardness readings on the broken steel coupons obtained from the top chord of the truss. The yield strength of the steel coupons obtained from the hardness readings was found to be in close agreement with that obtained from the tensile tests. It is noted that the calibration was performed under controlled conditions, with an ambient temperature of approximately 68 °F. Results from hardness readings from steel coupons are summarized in the table below. The complete hardness test data is included in Appendix 2.

		Yield Strength, ksi	
Sample Description	Average Hardness (HB)	From Hardness	Tensile Test
North Truss Steel Coupon	128.83	61	54
South Truss Steel Coupon	118	56	56

In-situ hardness readings were found to be significantly higher than those obtained under controlled conditions. The high in-situ hardness readings may be due to field conditions such as cold ambient temperature. It is noted that the range of operation of the hardness tester used is from 14 °F to 122 °F, with expected loss of accuracy as the temperature approaches the lower and upper bounds of the range. The ambient temperature during the field test was approximately 20 °F. For this reason, a correction factor was used to account for the effects of field conditions on the hardness readings.

The average hardness measured on the north truss steel coupon under controlled conditions was 128.83 in the Brinell scale. The average field measured hardness on the same member (top chord of north truss) was 179.9. Therefore, a correction factor of 0.72 was calculated from the ratio of the hardness measured on the steel coupons and the field measured hardness (i.e. correction factor = 128.83/179.9). This correction factor was applied to all field hardness measurements since all members were tested under the same field conditions. The statistical analysis of the field testing data for the main truss members and the pins is shown in the tables below.

The lower limit of the 95% confidence interval for the main truss members based on the in-situ hardness measurements was calculated as 40 ksi. Note that this is in close agreement with the 38 ksi obtained from the statistical analysis of the tensile tests results. For load analysis it is recommended that the more conservative value of 38 ksi be used for the main truss members.

The lower limit of the 95% confidence interval for the pins was calculated as 47 ksi. Therefore, it is recommended that a value of 47 ksi be used as the yield strength of the pins in the load rating analysis.

MAIN TRUSS MEMBERS					
Member	HB (AVG.)	Fy (ksi)	Corrected Yield Strength, Fy,c (ksi)		
Top Chord (Top Plate)	179.9	85.37	61		
Top Chord (Angle)	155.9	73.98	53		
Vertical Strut (Plate)	167	79.25	57		
Vertical Strut (Channel)	145	68.81	49		
Diagonal (7/8" thick)	275	130.50	93		
Diagonal (13/16" thick)	158.7	75.31	54		
Bottom Chord (web)	213.4	101.27	73		
Bottom Chord (angle)	177.7	84.33	60		
Floor Beam (Top flange)	209.6	99.47	71		
		Average, μ =	64		
		Std. Dev., σ =	13.711		
		Confidence Level (%) =	95		
		z=	1.645		
		Lower Limit of 95% Confidence Interval=	40	ksi	=μ-z*σ
		Yield Strength for Load Rating Analysis=	**40**	**ksi**	

PINS					
Member	HB (AVG.)	Fy (ksi)	Corrected Yield Strength, Fy,c (ksi)		
Top Chord Pin (side)	251.1	119.16	85		
Support Pin 1 (Curved surf)	202.9	96.29	69		
Support Pin 2 (Curved surf)	161.9	76.83	55		
Support Pin 2 (Side)	190.6	90.45	65		
		Average, μ =	69		
		Std. Dev., σ =	12.639		
		Confidence Level (%) =	95		
		z=	1.645		
		Lower Limit of 95% Confidence Interval=	47	ksi	=μ-z*σ
		Yield Strength for Load Rating Analysis=	**47**	**ksi**	

CONCLUSIONS AND RECOMMENDATIONS

Destructive and non-destructive testing was performed on the structural members of the bridge in the form of tensile tests (destructive) and hardness tests (non-destructive). Tensile tests were performed on two (2) coupons obtained from the top chord. Results from the tensile tests indicate an average yield strength of 55 ksi. A statistical analysis was performed on the data from the tensile tests following the provisions of AASHTO Manual for Bridge Evaluation (MBE) and a recommended yield strength for structural analysis of 38 ksi was found.

In-situ hardness measurements were taken on all structural elements of the bridge. After applying a correction factor to account for field conditions, the statistical analysis of the hardness results indicated a recommended yield strength of 40 ksi for the main truss members and 47 ksi for the pins.

Based on the overall results from the material testing, a yield strength of 38 ksi and 47 ksi is recommended to be used for the main truss members and the pins, respectively, in the load rating analysis

APPENDIX 1: STEEL COUPONS TESTING REPORT

DRAKE HILL ROAD TRUSS BRIDGE OVER FARMINGTON RIVER

IN THE TOWN OF

SIMSBURY, CONNECTICUT

ENGINEERING STUDY TO DETERMINE

LIVE LOAD CAPACITY OF THE BRIDGE DECK

AS A VIEWING PLATFORM

SUBMITTED TO

MR. FRANK ROSSI, P.E.

DIRECTOR OF PUBLIC WORKS

TOWN OF SIMSBURY, CONNECTICUT

APRIL 1990

MACCHI ENGINEERS

LOAD RATINGS

Allowable Stresses:

When this bridge was built during the 1890's, published catalogs from rolling mills recommended 16,000 psi allowable stresses for buildings and 12,500 psi for bridges.

The AASHTO Manual for Maintenance Inspection of Bridges limits allowable stresses for unknown steel from prior to 1905 to 14,000 psi for normal inventory loading and allows up to 19,500 psi stresses for infrequent loadings.

Tensile tests were performed in 1977 by Henry Souther Laboratories on small coupons (3/16" thick x 1/2" wide), cut from the vertical struts on this bridge. These tests indicated a yield strength of 51,500 psi (pounds per square inch) for 2 coupons from channel webs and 47,400 psi for a lacing bar. Even though these tests may not be representative of all the steel on this bridge, a yield strength of 36,000 psi was used as the basis for load rating computations, after the bridge has been repaired.

Bridge Loadings:

As tabulated on page 6, the maximum number persons, weighing an average of 160 lbs, were determined for combined dead load and live load stresses limits from 14,000 to 20,000 psi.

The bottom chord dead load stresses at the center of the bridge are 9,700 psi, based on an estimated section loss of 8%. The live load stresses consist of 75%-80% direct tension due to truss action and 20%-25% bending of the bottom chords due to floor beam loads.

The maximum occupancy was established for the following three conditions:

A. Uniform occupancy along the bridge.
B. Occupancy on 8 of 12 truss bays.
C. Occupancy on 6 of 12 truss bays.

Tables "A" through "C" show the results of or findings. As an example: 730 persons uniformly spaced on one side of the bridge would cause a stress of 18,000 psi in the bottom chord, as shown in Table "A".

IMTL
Accurate information you can rely on.

Top Chord Thru-Truss Bridge Steel Sampling And Testing Report

Client:	GM2 Associates	Project No.:	4011
Project:	Old Drake Hill Flower Bridge – Simsbury, CT	Report No.:	001
Inspector:	Shawn Roberts	Date:	11/05/18
Subject:	Steel Sampling	Page No.:	1 of 4

This firm was scheduled by GM2 Associates to perform the following tests:

- Sampling and testing top chord thru-truss bridge steel
- This Crew cut and extracted two (2) steel coupons 12" x 1" x 5/16" on each of the two (2) top chords of the existing bridge at the location specified by GM2.

This work was done as prescribed by GM2 in their instructions to IMTL.

The steel slots were debuted and coated with zinc-based paint. Mr. Lorin Pippin, P.E., of GM2 Associates, was present during the sampling.

This Crew transported the steel coupons back to the IMTL laboratory for processing.

STATE OF CONNECTICUT RICHARD WHITE No. 10036 REGISTERED PROFESSIONAL ENGINEER

pc: Jagdeesh Gopal, P.E., Luis Vila, P.E., GM2 Associates
dr

LABORATORY TESTING INC.
2331 Topaz Drive, Hatfield, PA 19440
TEL: 800-219-9095 • FAX: 800-219-9096

Certified Test Report

IMT002-18-11-36807-1

Accredited Nadcap
Materials Testing Laboratory
Nondestructive Testing

SOLD TO
Independent Matls. Testing
57 N. Washington Street
Plainville, CT 06062

SHIP TO
Independent Matls. Testing
57 N. Washington Street
Plainville, CT 06062
ATTN: David Aiudi

CUSTOMER P.O.	CERTIFICATION DATE	SHIP VIA
Verbal/Dave	11/29/2018	EMAIL, UPS GROUND

DESCRIPTION

Quantity: 2
Size: 11 3/4" x 7/8" x 5/16"
Description: Steel Plate Samples
Reference: Old Drake Hill Rd Flower Bridge Simsbury CT

TENSILE TEST:
APPLICABLE SPECIFICATIONS: ASTM E8-16a and Customer's instructions
KEY: C - Conforms NC - Non-Conformance R-Report for Information

SAMPLE ID	(ksi) TENSILE STRENGTH	(ksi) YIELD STRESS (0.2% OFFSET)	(%) ELONGATION IN 2" (MANUAL)	(%) REDUCTION OF AREA	FRACTURE LOCATION	KEY C/NC/R
NORTH	65.0	54.0	28	49	Middle 50% of GL	R
SOUTH	65.5	56.0	26	48	Middle 50% of GL	R

Procedures/Methods: 86-TT-2, Rev. 15, Room Temp. Tensile Testing for Metallic Materials

The services performed above were done in accordance with LTI's Quality System Program Manual Revision 20 dated 12/12/12 and ISO/IEC 17025. These results relate only to the items tested and this report shall not be reproduced, except in full, without the written approval of Laboratory Testing, Inc. L.T.I. is accredited by Nadcap for NDT and Materials Testing for the test methods and specific services as listed in the Scopes of Accreditation available at www.labtesting.com and www.eAuditNet.com. The results reported on this test report represent the actual attributes of the material tested and indicate full compliance with all applicable specification and contract requirements.

MERCURY CONTAMINATION: During the testing and inspection, the product did not come in direct contact with mercury or any of its compounds nor with any mercury containing devices employing a single boundary of containment.

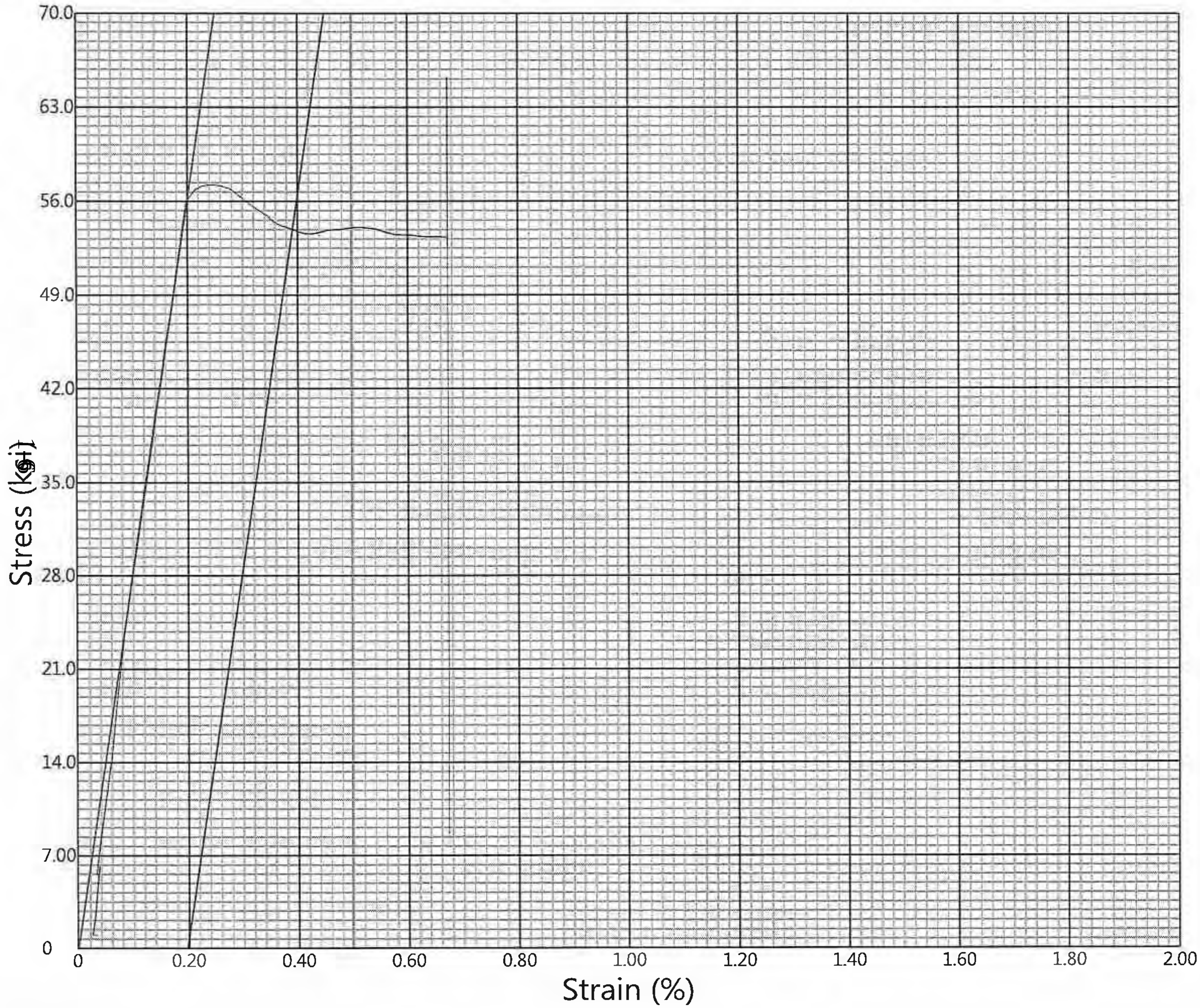
NOTE: The recording of false, fictitious or fraudulent statements or entries on this document may be punishable as a felony under Federal Statutes.

Sherri L. Scheifele
QA Specialist
Sherri L. Scheifele
Authorized Signature

Laboratory Testing Inc., 2331 Topaz Drive, Hatfield, PA, 19440
www.labtesting.com

Traveler #: IMT002181136807

Old Drake Hill Flower Bridge –
Simsbury, CT
No: 4011 Report No: 001
November 5, 2018



Technician:	NSM
Procedure:	86-TT-2
Identifier (e.g. Heat):	STEEL PLATE SAMPL

Test Date and Time:	11/29/2018 11:22 AI
Sample ID:	NORTH
Strain Rate (Initial):	0.0150 in/in/min
Position Rate (calculated):	0.844 in/min
Width:	0.5010 in
Thickness:	0.2672 in
Area:	0.1339 in²
Reduced Section Length:	2.25 in
Gage Length (Initial):	2.0000 in
Gage Length (Final):	2.5600 in
Width (Final):	0.381 in
Thickness (Final):	0.179 in
Modulus:	27.8 Mpsi
Offset Force @ 0.01%:	7604 lbf
OS @ 0.2%:	7217 lbf
EUL @ 0.5%:	7234 lbf
Offset Force @ 2.0%:	N/F
Ultimate:	8725 lbf
Offset Stress @ 0.01%:	391.6 MPa
OS @ 0.2%:	53.91 ksi
EUL @ 0.5%:	54.04 ksi
Offset Stress @ 2.0%:	N/F
Ultimate:	65.17 ksi
OS @ 0.2%:	371.7 MPa
EUL @ 0.5%:	372.6 MPa
Offset Stress @ 2.0%:	N/F
Yield Stress:	395 MPa
Ultimate:	449.4 MPa
TE (Manual):	28.0 %
Area (Reduction):	49.2 %
Fracture Location:	Middle 50% of GL
Fracture Type:	

*** All data is unrounded ***

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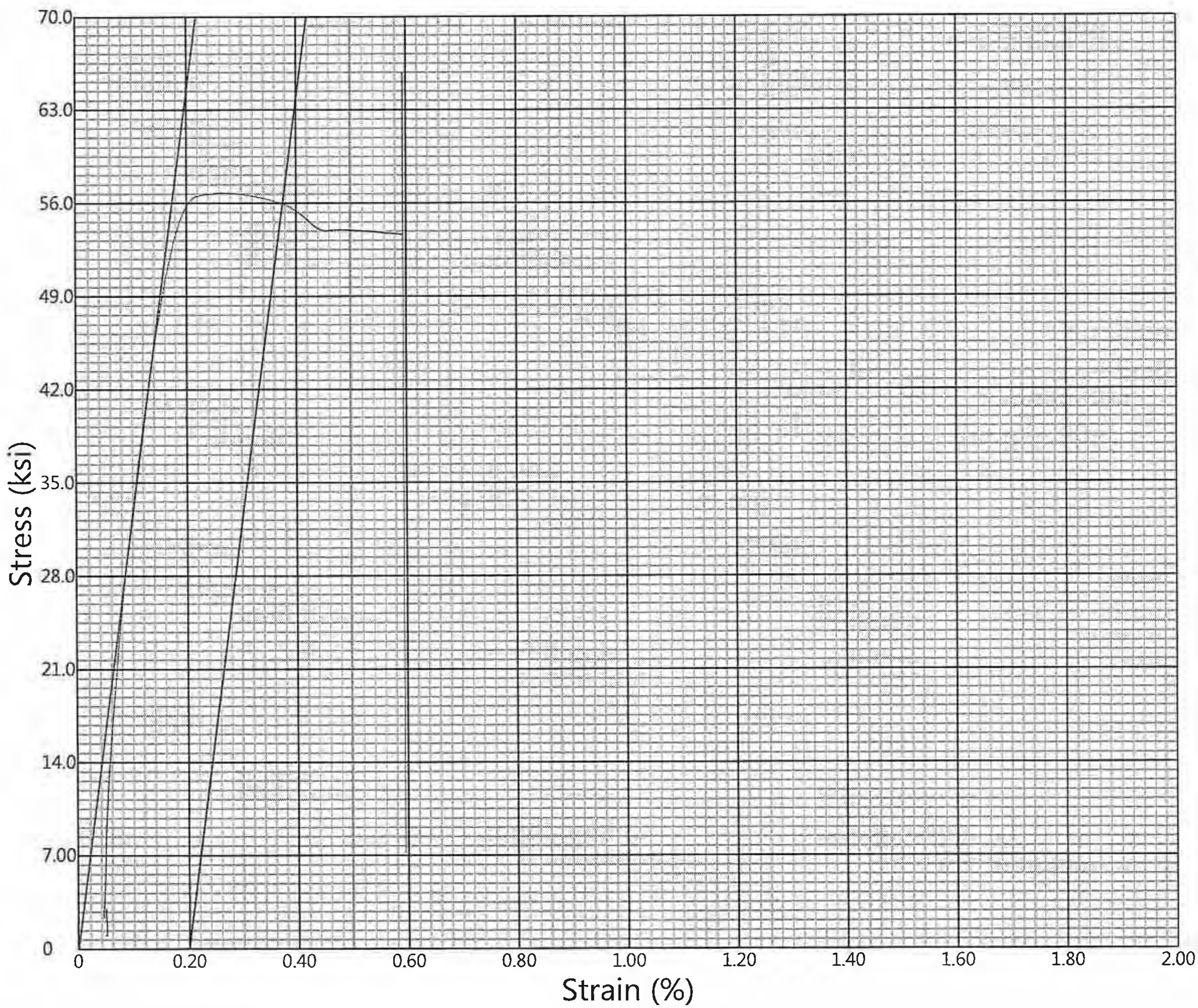
www.labtesting.com

Traveler #: IMT002181136807



Technician:	NSM
Procedure:	86-TT-2
Identifier (e.g. Heat):	STEEL PLATE SAMPL

Test Date and Time:	11/29/2018 11:26 A
Sample ID:	SOUTH
Strain Rate (Initial):	0.0150 in/in/min
Position Rate (calculated):	0.844 in/min
Width:	0.5020 in
Thickness:	0.2590 in
Area:	0.1300 in^2
Reduced Section Length:	2.25 in
Gage Length (Initial):	2.0000 in
Gage Length (Final):	2.5300 in
Width (Final):	0.388 in
Thickness (Final):	0.176 in
Modulus:	31.8 Mpsi
Offset Force @ 0.01%:	6768 lbf
OS @ 0.2%:	7276 lbf
EUL @ 0.5%:	7017 lbf
Offset Force @ 2.0%:	N/F
Ultimate:	8549 lbf
Offset Stress @ 0.01%:	358.9 MPa
OS @ 0.2%:	55.96 ksi
EUL @ 0.5%:	53.97 ksi
Offset Stress @ 2.0%:	N/F
Ultimate:	65.75 ksi
OS @ 0.2%:	385.9 MPa
EUL @ 0.5%:	372.1 MPa
Offset Stress @ 2.0%:	N/F
Yield Stress:	391 MPa
Ultimate:	453.3 MPa
TE (Manual):	26.5 %
Area (Reduction):	47.6 %
Fracture Location:	Middle 50% of GL
Fracture Type:	

1-7

*** All data is unrounded ***

APPENDIX 2: HARDNESS TEST DATA

GM2 Associates, Inc.

Consulting Engineers

Job	Flower Bridge, Simsbury, CT	**Computed By**	LV	**Date**	4-Jan-19
Description:	Hardness to Yield Conversion Verification	**Checked By**	JG	**Date**	30-Jan-19

CORRELATION EQUATION BETWEEN HARDNESS AND YIELD STRENGTH:

$$S_y = \text{BHN} \times 9.816 \ / \ 3 \qquad (5.27)$$

where S_y is the yield strength of the material in MPa and BHN is the Brinell hardness in kg/mm^2.

HARDNESS TESTER DATA:

Model: GE DynaPocket
Calibration Date: 8/30/2018
Calibration Due Date: 8/30/2019
Serial No.: 35159-1856

STEEL SAMPLE DATA:

Description: North Truss Steel Coupon
Test Location on Member: Top Chord
Yield Strength= 61 ksi (yield strength converted from hardness reading)
Measured yield strength= 54 ksi (yield strength from tensile test)

Harness Tester Position	Test No.	HB (kg/mm^2)
Vertical-Down	1	99
Vertical-Down	2	119
Vertical-Down	3	116
Vertical-Down	4	137
Vertical-Down	5	144
Vertical-Down	6	158
	Average=	128.83

STEEL SAMPLE DATA:

Description: South Truss Steel Coupon
Test Location on Member: Top Chord
Yield Strength= 56 ksi (yield strength converted from hardness reading)
Measured yield strength= 56 ksi (yield strength from tensile test)

Harness Tester Position	Test No.	HB (kg/mm^2)
Vertical-Up	1	108
Vertical-Up	2	113
Vertical-Up	3	140
Vertical-Up	4	129
Vertical-Up	5	112
Vertical-Up	6	106
	Average=	118

GM2 Associates, Inc.

Consulting Engineers

Job	Flower Bridge, Simsbury, CT	**Computed By**	LV	**Date**	11-Jan-19
Description:	Field Hardness NDT	**Checked By**	JG	**Date**	30-Jan-19

CORRELATION EQUATION BETWEEN HARDNESS AND YIELD STRENGTH:

$$S_y = \text{BHN} \times 9.816\ /\ 3 \qquad (5.27)$$

where S_y is the yield strength of the material in MPa and BHN is the Brinell hardness in kg/mm^2.

HARDNESS TESTER DATA:

Model: GE DynaPocket
Calibration Date: 8/30/2018
Calibration Due Date: 8/30/2019
Serial No.: 35159-1856

STEEL SAMPLE DATA:

Description: Top Chord - North Truss
1st node from west end
Test Location on Member: Top Plate
Yield Strength, Fy= 85.37 ksi (yield strength converted from hardness reading)
Corrected Yield Strength, Fy,c= 61 ksi *(yield strength corrected for field conditions)
***Correction Factor = 0.72**

Harness Tester Position	Test No.	HB (kg/mm^2)
Vertical-Down	1	176
Vertical-Down	2	149
Vertical-Down	3	176
Vertical-Down	4	162
Vertical-Down	5	214
Vertical-Down	6	175
Vertical-Down	7	193
Vertical-Down	8	187
Vertical-Down	9	172
Vertical-Down	10	195
	Average=	179.9

CORRECTION FACTOR CALCULATION:

Average Hardness from steel coupon, HB_{coupon}= 128.83
Correction factor for field conditions= 0.72 =128.83/179.9

GM2 Associates, Inc.

Consulting Engineers

Job	Flower Bridge, Simsbury, CT	**Computed By**	LV	**Date**	11-Jan-19
Description:	Field Hardness NDT	**Checked By**	JG	**Date**	30-Jan-19

STEEL SAMPLE DATA:

Description: Top Chord - North Truss

Test Location on Member: Bottom of Top-Outside Angle

Yield Strength, Fy= 73.98 ksi (yield strength converted from hardness reading)

Corrected Yield Strength, Fy,c= 53 ksi *(yield strength corrected for field conditions)

***Correction Factor = 0.72**

Harness Tester Position	Test No.	HB (kg/mm^2)
Vertical-Up	1	150
Vertical-Up	2	180
Vertical-Up	3	182
Vertical-Up	4	202
Vertical-Up	5	173
Vertical-Up	6	130
Vertical-Up	7	113
Vertical-Up	8	101
Vertical-Up	9	162
Vertical-Up	10	166
	Average=	155.9

STEEL SAMPLE DATA:

Description: Vertical Strut - South Truss (Two 5 3/4" by 5/16" thick plates)
1st vertical member from W. end

Test Location on Member: Outside face of outer plate

Yield Strength, Fy= 79.25 ksi (yield strength converted from hardness reading)

Corrected Yield Strength, Fy,c= 57 ksi *(yield strength corrected for field conditions)

***Correction Factor = 0.72**

Harness Tester Position	Test No.	HB (kg/mm^2)
Horizontal	1	168
Horizontal	2	176
Horizontal	3	170
Horizontal	4	169
Horizontal	5	167
Horizontal	6	175
Horizontal	7	165
Horizontal	8	155
Horizontal	9	172
Horizontal	10	153
	Average=	167

GM2 Associates, Inc.

Consulting Engineers

Job	Flower Bridge, Simsbury, CT	**Computed By**	LV	**Date**	11-Jan-19
Description:	Field Hardness NDT	**Checked By**	JG	**Date**	30-Jan-19

STEEL SAMPLE DATA:

Description: Vertical Strut - South Truss (Two C7x9.8)
2nd vertical member from W. end

Test Location on Member: Outside face of inside channel (between channels)

Yield Strength, Fy= 68.81 ksi (yield strength converted from hardness reading)

Corrected Yield Strength, Fy,c= 49 ksi *(yield strength corrected for field conditions)

***Correction Factor = 0.72**

Harness Tester Position	Test No.	HB (kg/mm^2)
Horizontal	1	149
Horizontal	2	144
Horizontal	3	160
Horizontal	4	97
Horizontal	5	144
Horizontal	6	109
Horizontal	7	157
Horizontal	8	150
Horizontal	9	170
Horizontal	10	170
	Average=	145

STEEL SAMPLE DATA:

Description: Diagonal - North Truss (Two 2" by 7/8" thick plates)
3rd Diagonal from West end

Test Location on Member: Outside face of outer plate

Yield Strength, Fy= 130.50 ksi (yield strength converted from hardness reading)

Corrected Yield Strength, Fy,c= 93 ksi *(yield strength corrected for field conditions)

***Correction Factor = 0.72**

Harness Tester Position	Test No.	HB (kg/mm^2)
Horizontal	1	255
Horizontal	2	233
Horizontal	3	314
Horizontal	4	314
Horizontal	5	260
Horizontal	6	272
Horizontal	7	279
Horizontal	8	253
Horizontal	9	278
Horizontal	10	292
	Average=	275

GM2 Associates, Inc.

Consulting Engineers

Job	Flower Bridge, Simsbury, CT	**Computed By**	LV	**Date**	11-Jan-19
Description:	Field Hardness NDT	**Checked By**	JG	**Date**	30-Jan-19

STEEL SAMPLE DATA:

Description: Diagonal - South Truss (Two 3" by 13/16" plates)
Test Location on Member: Inside face of inside plate (unpainted area w/ surface rust; surface rust cleaned prior to testing)
Yield Strength, Fy= 75.31 ksi (yield strength converted from hardness reading)
Corrected Yield Strength, Fy,c= 54 ksi *(yield strength corrected for field conditions)

***Correction Factor = 0.72**

Harness Tester Position	Test No.	HB (kg/mm^2)
Horizontal	1	96
Horizontal	2	192
Horizontal	3	166
Horizontal	4	155
Horizontal	5	152
Horizontal	6	143
Horizontal	7	106
Horizontal	8	201
Horizontal	9	227
Horizontal	10	149
	Average=	158.7

STEEL SAMPLE DATA:

Description: Bottom Chord - North Truss near West support
Test Location on Member: Outside face of web
Yield Strength, Fy= 101.27 ksi (yield strength converted from hardness reading)
Corrected Yield Strength, Fy,c= 73 ksi *(yield strength corrected for field conditions)

***Correction Factor = 0.72**

Harness Tester Position	Test No.	HB (kg/mm^2)
Horizontal	1	228
Horizontal	2	211
Horizontal	3	202
Horizontal	4	229
Horizontal	5	209
Horizontal	6	204
Horizontal	7	179
Horizontal	8	227
Horizontal	9	214
Horizontal	10	231
	Average=	213.4

GM2 Associates, Inc.

Consulting Engineers

Job	Flower Bridge, Simsbury, CT	**Computed By**	LV	**Date**	11-Jan-19
Description:	Field Hardness NDT	**Checked By**	JG	**Date**	30-Jan-19

STEEL SAMPLE DATA:

Description: Bottom Chord - North Truss near West support

Test Location on Member: Bottom-Outside Angle

Yield Strength, Fy= 84.33 ksi (yield strength converted from hardness reading)

Corrected Yield Strength, Fy,c= 60 ksi *(yield strength corrected for field conditions)

***Correction Factor = 0.72**

Harness Tester Position	Test No.	HB (kg/mm^2)
Vertical-Down	1	175
Vertical-Down	2	171
Vertical-Down	3	189
Vertical-Down	4	160
Vertical-Down	5	160
Vertical-Down	6	193
Vertical-Down	7	199
Vertical-Down	8	206
Vertical-Down	9	177
Vertical-Down	10	147
	Average=	177.7

STEEL SAMPLE DATA:

Description: Floorbeam (1st floorbeam from west end)

Test Location on Member: Top Flange

Yield Strength, Fy= 99.47 ksi (yield strength converted from hardness reading)

Corrected Yield Strength, Fy,c= 71 ksi *(yield strength corrected for field conditions)

***Correction Factor = 0.72**

Harness Tester Position	Test No.	HB (kg/mm^2)
Vertical-Down	1	270
Vertical-Down	2	252
Vertical-Down	3	145
Vertical-Down	4	219
Vertical-Down	5	200
Vertical-Down	6	166
Vertical-Down	7	221
Vertical-Down	8	200
Vertical-Down	9	201
Vertical-Down	10	222
	Average=	209.6

GM2 Associates, Inc.

Consulting Engineers

Job	Flower Bridge, Simsbury, CT	**Computed By**	LV	**Date**	11-Jan-19
Description:	Field Hardness NDT	**Checked By**	JG	**Date**	30-Jan-19

STEEL SAMPLE DATA:

Description: Top Chord Pin - North Truss
1st node at west end

Test Location on Member: Inside face of pin

Yield Strength, Fy= 119.16 ksi (yield strength converted from hardness reading)

Corrected Yield Strength, Fy,c= 85 ksi *(yield strength corrected for field conditions)

***Correction Factor = 0.72**

Harness Tester Position	Test No.	HB (kg/mm^2)
Horizontal	1	250
Horizontal	2	243
Horizontal	3	252
Horizontal	4	255
Horizontal	5	243
Horizontal	6	275
Horizontal	7	228
Horizontal	8	235
Horizontal	9	266
Horizontal	10	264
	Average=	251.1

STEEL SAMPLE DATA:

Description: Support Pin - North Truss, West End

Test Location on Member: Curved surface inside of top chord member

Yield Strength, Fy= 96.29 ksi (yield strength converted from hardness reading)

Corrected Yield Strength, Fy,c= 69 ksi *(yield strength corrected for field conditions)

***Correction Factor = 0.72**

Harness Tester Position	Test No.	HB (kg/mm^2)
Diag.-Down	1	173
Diag.-Down	2	222
Diag.-Down	3	234
Diag.-Down	4	229
Diag.-Down	5	243
Diag.-Down	6	183
Diag.-Down	7	113
Diag.-Down	8	234
Diag.-Down	9	186
Diag.-Down	10	212
	Average=	202.9

GM2 Associates, Inc.

Consulting Engineers

Job	Flower Bridge, Simsbury, CT	**Computed By**	LV	**Date**	11-Jan-19
Description:	Field Hardness NDT	**Checked By**	JG	**Date**	30-Jan-19

STEEL SAMPLE DATA:

Description: Support Pin - North Truss - East Support
Test Location on Member: Curved surface inside of top chord member
Yield Strength, Fy= 76.83 ksi (yield strength converted from hardness reading)
Corrected Yield Strength, Fy,c= 55 ksi *(yield strength corrected for field conditions)
***Correction Factor = 0.72**

Harness Tester Position	Test No.	HB (kg/mm^2)
Horizontal	1	140
Horizontal	2	158
Horizontal	3	137
Horizontal	4	200
Horizontal	5	122
Horizontal	6	166
Horizontal	7	160
Horizontal	8	216
Horizontal	9	182
Horizontal	10	138
	Average=	161.9

STEEL SAMPLE DATA:

Description: Support Pin - North Truss - East Support
Test Location on Member: Flat surface on outside face of Pin
Yield Strength, Fy= 90.45 (yield strength converted from hardness reading)
Corrected Yield Strength, Fy,c= 65 ksi *(yield strength corrected for field conditions)
***Correction Factor = 0.72**

Harness Tester Position	Test No.	HB (kg/mm^2)
Horizontal	1	210
Horizontal	2	173
Horizontal	3	187
Horizontal	4	196
Horizontal	5	184
Horizontal	6	200
Horizontal	7	169
Horizontal	8	193
Horizontal	9	209
Horizontal	10	185
	Average=	190.6

Exhibit C
SAMPLE CONTRACT

ENGINEERING SERVICES AGREEMENT
BY AND BETWEEN THE
TOWN OF SIMSBURY, CONNECTICUT
AND
NAME

THIS AGREEMENT is made on the date last signed below, by and between the Town of Simsbury, Connecticut, acting herein by and through its First Selectman, hereinafter called OWNER, and NAME, with offices at ADRESS, hereinafter called ENGINEER.

WITNESSETH, for the consideration hereinafter set forth, the parties hereto agree as follows:

ARTICLE 1 - ENGAGEMENT OF ENGINEER

1.1 OWNER hereby engages ENGINEER, and ENGINEER hereby accepts the engagement to perform certain professional engineering services on an on-call basis as requested by OWNER.

1.2 ENGINEER's services shall be performed in a manner consistent with that degree of skill and care ordinarily exercised by practicing design professionals performing similar services in the same locality, at the same site and under the same or similar circumstances and conditions. ENGINEER makes no other representations or warranties, whether expressed or implied, with respect to the services rendered hereunder.

ARTICLE 2 - SCOPE OF SERVICES

2.1 General On-Call Services:

ENGINEER shall provide services as requested by OWNER on an as-needed / as-requested basis. General On-Call Services shall include general engineering consultation and services as requested by OWNER, which services are provided during the course of normal business hours (generally 8:00 AM to 5:00 PM) or as may be otherwise scheduled and agreed upon in advance (such as scheduled evening Board and Commission Meetings). General On-Call Services include any services that are not otherwise included under a specific Task Order (per 2.2 below) or provided as Emergency Services (per 2.3 below).

2.2 Task Order On-Call Services:

ENGINEER shall provide services as requested by OWNER for which a specific Scope of Services and associated lump sum fee are negotiated at the request of OWNER. Prior to commencement of Task Order On-Call Services, OWNER and ENGINEER will negotiate and agree upon the Task Order's Scope of Services, Fee, and Schedule for the requested assignment.

2.3 Emergency On-Call Services:

ENGINEER shall provide services as requested by OWNER on an as-needed / as-requested emergency basis, for which immediate response is required, and for which the essence of time precludes the typical documentation and clarifications otherwise required under the above described classifications.

2.4 For each assignment, Engineer shall identify a project representative for day-to-day administrative and technical conduct of services for that assignment. In addition, ENGINEER's prime contact shall be:

NAME

ARTICLE 3 - RESPONSIBILITIES OF OWNER

OWNER, without cost to ENGINEER, shall do the following in a timely manner so as not to delay the services of ENGINEER:

3.1 Designate in writing a person or persons to act as OWNER 's representative with respect to work to be performed under this AGREEMENT, such person to have complete authority to transmit instructions, receive information, interpret and define OWNER'S policies and decisions with respect to materials, equipment elements and systems pertinent to the work covered by assignments under the various classifications of this AGREEMENT.

3.2 Through its officials and other employees who have knowledge of pertinent conditions, confer with ENGINEER regarding both general and special considerations relating to assignments.

3.3 Assist ENGINEER by placing at the disposal of ENGINEER, all available information pertinent to the Task Order(s) including previous reports and any other data relative to assignments.

3.4 Pay all application and permit fees associated with approvals and permits from all governmental authorities having jurisdiction over assignments and such approvals and consents from others as may be necessary for completion of assignments.

3.5 Arrange for access to and make all provisions for ENGINEER to enter upon public and private lands as required for ENGINEER to perform its work.

3.6 Furnish ENGINEER all needed property, boundary and right-of-way maps.

3.7 Cooperate with and assist ENGINEER in all additional work that is mutually agreed upon.

3.8 Pay ENGINEER for work performed in accordance with the terms specified herein.

ARTICLE 4 - PAYMENTS TO ENGINEER

4.1 For services performed under this AGREEMENT, OWNER agrees to pay ENGINEER within thirty (30) days of the invoice date for the various service classifications as follows:

4.1.1 For General On-Call Services, ENGINEER shall invoice OWNER monthly on a time charged plus expense basis at the hourly rates indicated in Attachment A. The hourly rates applicable for each calendar year shall be provided in writing by ENGINEER, and shall be adjusted only through written amendment to this Agreement. Compensation shall be payable monthly, as earned.

4.1.2 For Task Order On-Call Services, ENGINEER shall invoice OWNER monthly on a percent complete basis, or on any other basis as described by an approved Task Order. Compensation shall be payable monthly, as earned.

4.1.3 For Emergency On-Call Services, ENGINEER shall invoice OWNER monthly on a time charged plus expense basis at 1.25 times the hourly rates indicated in Attachment A. The hourly rates applicable for each calendar year shall be provided in writing by ENGINEER, and shall be adjusted only through written amendment to this Agreement. Compensation shall be payable monthly, as earned.

4.2 If OWNER fails to make any payment due ENGINEER for services and expenses within thirty (30) days after receipt of ENGINEER'S invoice therefore, ENGINEER may, after giving seven (7) days written notice to OWNER, suspend services under this AGREEMENT. Unless ENGINEER receives payment within seven (7) days of the date of the notice, the suspension may take effect without further notice. In the event of a suspension of services, ENGINEER shall have no liability to OWNER for delay or damage caused OWNER because of such suspension of services.

ARTICLE 5 - INSURANCE

5.1 General Liability Insurance

ENGINEER shall secure and maintain, for the duration of this Agreement, the following General Liability Insurance policy or policies at no cost to OWNER. With respect to the operations ENGINEER performs, ENGINEER shall carry Commercial General Liability Insurance providing for a combined single limit of One Million Dollars ($1,000,000) for bodily injury, death, and property damage. Provide certificates indicating insurance coverage as indicated herein, and include the Town of Simsbury, its employees and agents, and their successors and assigns as additional named insured on the insurance certificates.

5.2 Automobile Liability Insurance

ENGINEER shall secure and maintain, for the duration of this Agreement, Automobile Liability Insurance covering the operation of all motor vehicles, including those hired or borrowed, used by ENGINEER in connection with this Agreement, in the following amount:

5.2.1 Not less than Five Hundred Thousand Dollars ($500,000) for all damages arising out of bodily injuries to or death of one person and subject to that limit for each person, a total limit of Five Hundred Thousand Dollars ($500,000) for all damages arising out of bodily injuries to or death of two or more persons in any one accident or occurrence, and

5.2.2 Not less than One Hundred Thousand Dollars ($100,000) for all damages arising out of injury to or destruction of property in any one accident or occurrence.

5.3 Umbrella Liability Insurance

In addition to the above-mentioned coverage, ENGINEER shall carry a minimum of One Million Dollars ($1,000,000) umbrella liability policy for the duration of the PROJECT.

5.4 Workers Compensation Coverage

5.4.1 ENGINEER shall maintain statutory Worker's Compensation insurance coverage for all of its employees working under this Agreement as required by the State of Connecticut.

ARTICLE 6 - INDEMNIFICATION

6.1 To the fullest extent permitted by law, ENGINEER agrees to indemnify and hold harmless OWNER and its officers, directors, employees, agents, and independent professional associates, and any of them, from any claims, losses, damages or expense (including reasonable attorneys' fees) arising out of the death of, injuries, or damages to any person, or damage or destruction of any property, in connection with ENGINEER'S services under this Agreement to the extent caused by the negligent acts, errors, or omissions of ENGINEER or its officers, directors, employees, agents or independent professional associates, or any of them.

ARTICLE 7 - EXTENSION OF SERVICES

7.1 Additional Work

In the event ENGINEER, as requested by OWNER, is to make investigations or reports on matters not covered by the scope of services for a particular Task Order assignment, or is to perform other services not included herein, additional compensation shall be paid

ENGINEER as is mutually agreed upon by and between OWNER and ENGINEER. Such services shall be incorporated into written amendments to the individual Task Order assignment(s) or as a new Task Order assignment. Litigation support services, if requested by OWNER, shall be performed as a separate Task Order.

7.2 Changes in Work

OWNER, from time to time, may require changes or extensions in the Scope of Services to be performed under a particular Task Order assignment. Such changes or extensions, including any increase or decrease in the amount of compensation, to be mutually agreed upon by and between OWNER and ENGINEER, shall be incorporated into written amendments to the Task Order.

7.3 Hazardous Materials Encountered

If, in the performance of the work, hazardous materials are encountered and are judged by ENGINEER to be an imminent threat to on-site personnel and/or the general public, ENGINEER shall inform the Local and State Emergency Personnel of the release. OWNER agrees to compensate ENGINEER for any time spent and/or reasonable expenses incurred by ENGINEER to mitigate the threat. Such services shall be considered General On-Call Services paid at the then-current hourly rates.

ARTICLE 8 - OWNERSHIP AND USE OF DOCUMENTS

8.1 OWNER shall retain ownership of documents submitted to OWNER by ENGINEER pursuant to this AGREEMENT. However, such documents are neither intended nor represented to be suitable for reuse by OWNER or others on extensions of the assignment(s) or on any other project or for any other purpose. Any reuse without written verification or adaptation by OWNER for the specific purpose intended shall be at OWNER'S sole risk and without liability or legal exposure to ENGINEER or to ENGINEER'S independent sub-consultants, and OWNER shall indemnify and hold harmless ENGINEER and ENGINEER'S sub-consultants from all claims, damages, losses and expenses, including reasonable attorneys' fees arising out of or resulting there from. Any such verification or adaptation shall entitle ENGINEER to further compensation at rates to be agreed upon by OWNER and ENGINEER.

ARTICLE 9 – TERMINATION

9.1 The obligation to provide further services for any work under this Agreement may be terminated by either party upon thirty (30) days' written notice.

9.2 If an assignment is suspended or abandoned in whole or in part for more than three (3) months, ENGINEER shall be compensated for all services performed prior to receipt of written notice from OWNER of such suspension or abandonment, together with other direct costs then due and all Termination Expenses as defined in Article 9.4. If the assignment is resumed after being suspended for more than three (3) months, ENGINEER'S compensation shall be equitably adjusted.

9.3 In the event of termination by OWNER under Article 9.1, ENGINEER shall be paid for all unpaid services and unpaid other direct costs incurred to the date of receipt of written notice of termination, including sub-consultants, and for the services necessary to affect termination, in accordance with the provisions of Article 4 of this Agreement.

9.4 In the event of termination by ENGINEER under Article 9.1, or termination by OWNER for OWNER'S convenience, ENGINEER shall be paid for all unpaid services and unpaid other direct costs incurred to the date of receipt of written notice of termination, including sub-consultants, for the services necessary to affect termination, plus termination expenses. Payment for services will be in accordance with the provisions of Article 4 of this Agreement. Termination expenses include additional costs of services directly attributable to termination, which shall include an additional amount computed as the costs ENGINEER reasonably incurs relating to commitments, which had become firm before the termination.

ARTICLE 10- GENERAL PROVISIONS

10.1 Precedence

The terms and conditions in this Agreement shall take precedence over any inconsistent or contradictory provisions contained in any proposal, contract, purchase order, requisition, notice to proceed, or like document regarding ENGINEER'S services.

10.2 Severability

If any of the terms and conditions in this Agreement shall be finally determined to be invalid or unenforceable in whole or part, the remaining provisions hereof shall remain in full force and effect, and be binding upon the parties hereto. The parties agree to reform this Agreement to replace any such invalid or unenforceable provision with a valid enforceable provision that comes as close as possible to the intention of the stricken provision.

10.3 Mediation

All claims, disputes or controversies arising between OWNER and ENGINEER shall be submitted to non-binding mediation prior to and as a condition precedent to the commencement of any litigation between those parties. The American Arbitration Association, or such other person or mediation service shall conduct the non-binding mediation as the parties mutually agree upon. The party seeking to initiate mediation shall do so by submitting a formal written request to the other party to this Agreement and the American Arbitration Association or such other person or mediation service as the parties mutually agree upon. The costs of mediation shall be borne equally by the parties. All statements of any nature made in connection with the non-binding mediation shall be privileged and will be inadmissible in any subsequent court or other proceeding involving or relating to the same claim.

10.4 Subrogation

OWNER and ENGINEER waive all rights against each other and against the contractors, consultants, agents and employees of the other for damages, but only to the extent covered by any property or other insurance in effect whether during or after the assignment. OWNER and ENGINEER shall each require similar waivers from their contractors, consultants and agents.

10.5 Statute of Limitations

Causes of action between the parties to this Agreement pertaining to acts or failures to act shall be deemed to have accrued and the applicable statutes of limitations shall commence to run not later than either the date of completion of services performed for acts or failures to act occurring prior to the date of completion of services performed or the completion date contained in this Agreement for acts or failures to acts occurring after the date of completion of services performed. In no event shall such statutes of limitations commence to run any later than the date when ENGINEER's services are substantially completed.

IN WITNESS WHEREOF, the parties hereto have executed this AGREEMENT the day and year first above written.

ACCEPTED FOR:

TOWN OF SIMSBURY, CT	**COMPANY**
By: Town Manager	By Its: President/Owner
______________________ Signature	______________________ Signature
______________________ Printed Name	______________________ Printed Name
______________________ Date	______________________ Date