CONDITION ASSESSMENT REPORT

GOLD RIVER MULTI-USE BRIDGE

GOLD RIVER, NOVA SCOTIA

September 26, 2021 (**REVISION No. 1**)

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EXECUTIVE SUMMARY

ABLE Engineering Services Inc. (ABLE) has been engaged by the Municipality of the District of Chester (MODC) to carry out a detailed inspection and condition assessment of the Gold River Multi-Use Bridge which forms a part of the Trans Canada Trail network.

Originally constructed 100 or more years ago as an elevated railway crossing over the Gold River just west of Chester, the bridge was removed from rail service and abandoned in the 1990`s.

The bridge structure was assessed for re-purposing as a primarily pedestrian structure in 2001 and at that time it was determined that in spite of its overall condition, remnant structural capacity appeared to be ample for reduced live loadings associated with planned pedestrian service. At that time, though possibly in excess of 80 years old, the timber trestle approach structures, steel main span girder assemblies and supporting stone block masonry pier towers were deemed to be in good condition. Following that assessment the bridge was modified, re-purposed and opened for pedestrian traffic.

A subsequent engineering inspection and condition assessment report for the structure was issued in 2013. That report concluded that in the years since the 2001 condition assessment several primary and secondary structural components were beginning to visibly exhibit progressive deterioration. As a result of that report a construction tender was issued which was intended to address and correct reported damage and deficiencies. However, that work was not awarded or carried out. The structure has not had the benefit of significant repairs since then.

As of 2021 the structure has seen about 20 years of renewed service as a multi-use bridge. In those ensuing 20 years structural damage and deteriorations have steadily progressed and are readily visible in several areas.

Although some primary structural components such as some of the timber trestle components and the main steel girders appear to be able to continue to provide adequate operational service (for the short term), other primary structural components such as several timber trestle piles, related parts and fasteners have reached or exceeded their expected service lives. Of particular note is the observed very poor condition of steel girder bearings including non-functional expansion sliders. Also found to be in poor condition are the main stone masonry support piers.

Note that structural damage at the timber trestles can be repaired, but such refurbishment work will likely leave a structure in place that is comprised partially of 100+ year old weathered and otherwise deteriorated timbers. Under such conditions, especially in the Nova Scotia climate, such timber repairs and limited component replacements would not significantly improve the existing remaining service life expectations for the trestle structures. It will be necessary to replace the timber trestle structures if any significant service life extension is to be achieved.

Structural steel components associated with the main span girders are no longer in good condition. Local deteriorations have become significant in recent years and damage associated with those deteriorations has become visible. However, it is probable that the steel girders can be adequately repaired and that the expected remaining service life of the girders can be improved.

Of greatest immediate concern is the condition of the girder support bearing assemblies located at the tops of the stone masonry support towers, and their effect on the stone masonry piers. With non-functioning bearing sliders cyclical expansion and contraction of the main girders cannot be accommodated by the structure. Instead, the tops of the stone masonry piers must resist significant horizontal loads associated with thermal expansion and contraction of the steel girders. This is a load condition for which the rigid and brittle stone piers were never intended to bear and if left un-checked will eventually lead to structural failure and collapse of the bridge.

Therefore, in order to extend the existing remaining service life for the bridge significant replacements and reconstructions are necessary. Note that for every year that such work is delayed the repair scope and costs will likely grow.

If it is desired by MODC that this structure should remain in service, timber trestles can be replaced, structural steel components can be repaired, bearings can be replaced and stone masonry repaired and strengthened, but implementation of such repairs will not be an inexpensive undertaking. Alternatively, the bridge can be removed from service and decommissioned/disassembled.

Note that the condition of the structure has now reached a point where doing nothing is not an option. The bridge must be either repaired/reconstructed/replaced or removed from service and decommissioned (demolished). However, even abandonment and decommissioning will be at a significant cost.

This report examines the condition and recommends repairs for components vital to maintaining adequate structural integrity at the Gold River Multi-Use Bridge. Alternatives to repair are also presented herein. Estimated costs for those repairs and other recommended alternatives are also presented herein.

In the meantime, because of the risk and the potential consequences of bridge failure it is recommended that the bridge be taken out of service and closed to the public until such time that recommended repairs, refurbishments, reconstructions and improvements can be completed.

1 INTRODUCTION

1.1 PROJECT DESCRIPTION AND OBJECTIVES

The Gold River Multi-Use Bridge in the Municipality of the District of Chester is a former railway bridge which appears to have been constructed in the early part of the 20th century. The structure has been in use primarily as a pedestrian bridge since the railway was converted to a recreational trail system.

The bridge is a combination timber trestle and riveted steel plate-girder two-span structure which stands about 55' above the normal water surface of the Gold River.

Approaches at each side of the river are elevated timber trestle structures consisting of timber piles and bolted timber struts and bracing. The trestle approach at the east side of the river is 81 feet long, and the trestle approach structure at the west side of the structure is a length of 140 feet.

Two existing steel main span girders are located directly over the Gold River. Each span is about 73 feet, and the girders are supported by three large masonry pier towers. The west pier is located along the river's edge, the east pier is above the riverbank and the middle pier is at about the centre of the river.

The Bridge incorporates a horizontal curve on its alignment with a radius of about 500 feet. Existing pier caps and bearings arrangement demonstrate that the horizontal geometry exhibits a slight super-elevation on that curve.

Original creosoted timber trestle piles, beams, braces struts, stringers, and rail ties as well as structural steel plate girder assemblies, and stone masonry support piers remain an integral part of the bridge structure. However, steel train rails have been replaced with a treated lumber deck and treated wood pedestrian guards at each side of the deck.

Timber trestle approach structures consist of groups of driven timber piles in braced and interconnected "pile bents". (At this structure such a feature is comprised of an assembly of six piles aligned laterally to the longitudinal axis of the former tracks and topped with a timber pile cap forming a timber pier structure commonly referred to as a "pile bent"). Each pile bent is interconnected with longitudinal top stringers. Closely spaced rail ties are installed laterally atop the stringers. Pile bents are spaced at about 12 feet at the deck level and are interconnected through their height with bolted horizontal (lateral and longitudinal) struts at about 20 foot vertical increments. The trestles are cross-braced at paired pile bent framing panel points. Note that piles are installed with a prescribed batter (angle from vertical) depending on pile location to resist horizontal dynamic rail service loads and reactions.

The steel plate girders are constructed from plate and angle components typically available at the time of construction. Structural steel components in the early 20th century were normally high in carbon content and not intended for welding. (Steel structural welding did not become a common and widespread means for steel fastening in construction until about the mid-20th century). At the time of the construction of the bridge, steel components for various structural fabrications were assembled and riveted into common "I`- girder", or "box" and other miscellaneous sectional shapes, and were usually reinforced with "doubler plates" and stiffeners and otherwise braced and reinforced with angles as and where required. Interconnection of these steel parts was normally via hot rivets. This method of steel component manufacture and component assembly was utilized in the construction of the Gold River Bridge. (See Appendix A - Tacten Report photos.) Other steel structures constructed by this method in Nova Scotia in the early to mid-20th century include Halifax`s Pier 21 buildings, the Halifax Forum and the Angus L. Macdonald Bridge.

Supporting the steel span components at Gold River are three stone masonry pier towers. The towers are constructed of cut granite ashlar blocks which were originally mortared in place with a lime-based mortar compound. Each of the masonry stone pier structures (and associated rubble cores) is a gravity structure which transfers bridge main span loads and reactions into the foundation sub-structure. The masonry piers are capped with granite blocks that are intended to seal the tops of the piers from moisture intrusion while providing structural seats for steel girder bearing assemblies. Those bearings are anchored into top granite cap blocks and not only spread girder reaction loads into the caps of the pier towers, but at the centre pier sliders also provide a means for accommodating thermal expansion and contraction of the steel girders.

Each of the main structural component systems of the bridge, including the timber trestles, steel girder main span assemblies and the masonry support towers are beginning to show their age by way of significant visible deterioration. Timber components have remained serviceable since original construction, and that is a testament to the effectiveness of creosote wood preservative treatment (and its corrosion inhibiting properties). However, many metal connectors as well as the timber itself are now visibly exhibiting extensive degradation.

In addition to pedestrian traffic, the bridge frequently sees light off-road vehicle (all-terrain 4x4) crossings. Highway vehicles generally do not operate on the structure.

The bridge was removed from active rail service in September of 1991 and was apparently abandoned at that time. A structural inspection and assessment of the abandoned bridge was completed in 2001 by Waugh Associates (Waugh). Bridge renovations were completed subsequent to that inspection and assessment which featured installation of new treated lumber decking and side guards.

More recently a follow-up inspection and assessment of the bridge structure was completed in 2013 by SNC Lavalin Inc. (SNC). A construction tender package for recommended repairs and maintenance derived from the SNC report was subsequently issued for pricing. However, the work associated with that tender was not awarded or executed.

ABLE was retained by the MODC in the winter of 2021 to oversee a new updated visual inspection of the Gold River Bridge, and to produce an updated bridge condition assessment report. The prime objectives of this work was to identify any areas of deterioration that may be of immediate or future structural concern, and to examine the existing structure with the goal of deriving, prioritizing and estimating costs for necessary repairs and maintenance tasks. To this end structural components of the bridge have been inspected by ABLE in-house experienced professionals and specialized sub-consultant experts.

Previous Gold River bridge engineering inspection reports produced in 2001 and 2013 have been reviewed and referenced to provide background and historical reference for condition of the bridge structure.

ABLE has been responsible for general project planning, inspection coordination, client liaison and reporting as well as evaluations and interpretation of site inspection data collected during timber, structural steel and stone masonry component inspections. ABLE is responsible for derivation of and/or confirmation of suitability of specific structural repairs recommendations provided by engaged sub-consultants:

- Sub-consultant Tacten Industrial Inc. (Tacten) of Burnside has been engaged to provide all required site high angle access condition inspections (timber, structural steel and stone masonry), compile observations, to determine and record various component dimensions and thicknesses and to identify chemical composition of previously installed protective coatings.
- Masontech Inc. (Masontech) of Halifax has been engaged to collect and review stone masonry data collected at site, to evaluate the existing condition of the stone masonry piers and to prioritize and provide budget pricing for required masonry repairs and rehabilitation of stone masonry components.

Reporting on the observed condition of structural components and recommended necessary repairs is summarized in this report. See Section 4.0 and Appendix C of this report for construction cost estimate information.

Since there is a lack of original design drawings for the bridge structure, for reference we have included drawings produced for and included in the 2001 Waugh condition assessment report. See Appendix D.

1.2 PROJECT CONTACT INFORMATION

The project team for the Condition Assessment Inspection consists of:

- Lead Project Engineer and Timber Expert: Jamie Yates
- Project Manager: Marco Visentin, ABLE
- Project Coordinator: Neil McCallum, ABLE
- 1st Remote Access Technician: Brett Webster, Tacten
- Steel Expert: Wesley Albert, Tacten
- Masonry Expert: Mark Fougere, Masontech

2 INSPECTION

2.1 DESCRIPTION OF TASK

Due to the geometric configuration of the bridge structure, much of the critical structural components are relatively inaccessible to conventional inspections/inspectors and normal structural inspection and access techniques. Therefore two options for inspection were considered:

1. Erection of an extensive and costly scaffolding system to not only provide access for an inspection team but to also address fall protection requirements when working at and under an elevated bridge deck; or,
2. Engagement of high angle access technicians from Tacten (formerly Remote Access Technology, commonly formerly known as "RAT") to perform climbing access inspections with ropes and harnesses.

Because of the very significant cost differential between the two access alternatives (installation of scaffold was conservatively estimated to be in excess of 10x the cost of rope and climbing access), Tacten was engaged to provide access to and inspection at difficult to access components such as trestle stringers, timber piles, pile caps, bracing and struts; structural steel, bearings and related components; and masonry pier caps and cut stone and mortar.

Therefore, on April 14th, 15th, 16th, & 19th and again on June 4th, 2021 industrial rope access technicians from Tacten Industrial visited the Gold River bridge to perform visual inspections of the bridge and its structural components. The inspection of the bridge was completed using rope access techniques in accordance with the Industrial Rope Access Trades Association (IRATA) International Code of Practice (ICOP) and Acuren ACU-ROPE procedures.

Tacten inspections areas of interest included timber trestles and related components; structural steel girders, bearings and related components; and stone masonry piers and related parts. During this work timber core samples were collected from piles, struts, braces, stringers and ties and were assessed by ABLE. Remnant protective coating flake samples were collected from structural steel and were examined at Tacten/Acuren laboratory facilities. Input on component repairs costs was received from Tacten and utilized in development of overall project repair cost estimates.

A visual photographic record of components inspected during the high angle inspection exercise was assembled as part of that task. (See Appendix A for Tacten report and Appendix B for inspection photographic record)

Part-time stone masonry inspection at ground and deck level was provided by Mark Fougere of Masontech on the 14th to 19th of April. Mr. Fougere provided interpretation of Tacten collected high angle data from masonry towers and was able to probe and sample selected masonry joints in the tower structures. Once masonry information was collected and collated, an effective masonry repair methodology for the towers was derived and a component construction cost estimate was completed. (See Appendices A and B for Masontech report and photographic record)

Neil McCallum, a representative for ABLE was on site to provide coordination and liaison between sub-consultants and to assist in determination of which areas of the structure would require prioritized inspection attention.

Jamie Yates was at site on a part-time basis during inspections to provide direction to high angle inspectors and sub-consultants on areas of interest and desired material sampling and photo record assembly, as well as interpretations of initial findings at site and scope for required subsequent supplemental site inspections and sub-consultant reporting.

For reference, the East end of the bridge is nearest Croft Rd, in the direction towards Chester and the west end of the structure is furthest from Croft Road and in the direction away from Chester. The North face of the bridge is the upstream side, the south the downstream. See Appendix D for the site layout and structural details (Waugh 2001).

2.2 INSPECTION METHODOLOGY AND TECHNIQUES

2.2.1 TIMBER INSPECTIONS

Condition of timber components was assessed by three main methods:

1. Visual inspection with identification/notation of visible anomalies identifying cracking, checking, brooming, rot, hollowness, growth of moss or other biological or inorganic surface abnormalities, and any other anomalous observations. A very wide area of a structure can usually be visually inspected in a relatively short period of time, from which a reasonably representative general condition of the timber can be determined;
2. Hammer Sounding is a process of tapping the exterior of the timber components with a hammer and subjectively assessing the quality of the sound report produced by the taps. A wide area sampling of a structure is usually possible for such an inspection (usually a representative sample) over a relatively short period of time:
    a. Firm noise report typically suggests a solid cross section and good structural integrity.
    b. A dull or hollow noise resulting from hammer tapping typically indicates hollowness, delamination of growth rings, and/or interior/exterior rot.
3. Retrieval of component timber core samples in selected locations. This sampling and resting technique can usually confirm or dispel hypotheses derived from the other two testing methods described above. Conditions to note during a core sampling exercise are as follows:
    a. A solid or nearly solid retrieved core sample is normally characteristic of good structural integrity.
    b. Significant coring tool rotational resistance in the sampled timber is usually an indication of good structural integrity.

c. Poor coring tool rotational resistance in the sampled timber is usually an indication of poor structural integrity.
d. Poor sample cohesion, blackened colour and high moisture content of retrieved core usually results from rot and/or delamination of sample at growth rings which is usually indicative of poor structural integrity.

The combination of these sampling and testing techniques allowed Tacten to identify areas where timber is in relatively good condition and locations where varying levels of rot and other degradation are evident. See Appendix B for photos of timber core samples with source member type and location of retrieval described.

2.2.2 STUCTURAL STEEL INSPECTIONS

Steel sections of girder pairs for each span were visually assessed and areas exhibiting deformations and damage such as corrosion or cracking or missing fasteners or other hardware have been identified.

The former surface/paint condition was examined and samples were retrieved. See Appendix A – Tacten report for paint flake testing results.

Calipers and Ultrasonic Thickness (U.T.) measurement tools were used to gauge thicknesses of steel member components and to assist in estimation of how much loss of material thickness has been experienced in selected locations.

2.2.3 BEARINGS INSPECTIONS

Girder bearings are intended to support and distribute steel girders' vertical end reactions at the tops of the stone masonry pier towers via the cap bearing stones found at that location.

Original bearings in place at the Gold River bridge include stacked shims of varying thicknesses at each set of paired girders in order to accommodate horizontal curve super-elevation.

Each of the girders in each span pairing was originally fitted with dedicated bearing supports. At one end of each of the girder pairs fixed steel bearings and steel shims are fitted to the ends of the girders and to cap stones of the masonry pier assemblies (at east and west pier towers). At the opposite end of each girder pair, sliding steel/metal bearings have been installed to accommodate daily/seasonal thermal longitudinal expansion and contraction of the steel girders (at opposite sides of the central pier tower).

Although the sliding bearings are attached to the girders to prevent lateral movements, they are not intended to resist longitudinal girder movements relative to the bearings and the tops of the masonry pier caps which are generated by thermal expansion/contraction of the steel girders.

2.2.4 STONE MASONRY INSPECTIONS

The Masonry Piers were visually assessed for;

- Straightness, plumb and general symmetry;
- Local deformations;
- Non-uniform settlement;
- Physical condition of the ashlar granite blocks;
- Alignment and spacing of the granite blocks;
- Condition of mortar, cracking of mortar joints;
- Assessment of core of stone masonry towers.

A cordless hammer drill was used to drill into the pier core at a mortar joint near the base of one of the stone masonry towers to determine the make-up and condition of the masonry core. A more detailed inspection of the masonry core may be possible through temporary removal of a carefully chosen granite block.

2.3 OBSERVATIONS

2.3.1 TIMBER TRESTLES

The East and West approach trestles are constructed of timber pile bents, struts and bracing and are topped with longitudinal stringers and rail ties. The timber piles are installed with a prescribed slight longitudinal and/or lateral batter (vertical angle departure from vertical), depending on the location of the pile bent, to accommodate incidental longitudinal and lateral loadings and accelerations imparted to the structure when it was still utilized as part of local rail transport infrastructure.

East approach bents have been labelled 1-7 with bent 1 being the easternmost trestle abutment. The West approach bents have been labelled 1-12 with 1 being the westernmost abutment. The timber piles numbers within each bent were labelled numerically with 1 being the Northernmost pile and 6 being the southernmost pile. See Appendix A – Tacten report and Appendix D - Site Layout for clarification.

Included in this section are the observations on the condition of rail ties and bridge pedestrian lumber decking.

Piles, struts, bracing, stringers and ties were all originally treated with creosote preservative. As a result, many of the treated timber components and steel fasteners were found to be in fair to good condition. However, the following items were noted:

a) Several timber pilings were observed to have animal/insect holes and local areas of decay. At some of the piles the outer layers/growth rings have delaminated and/or there is significant spitting at the timber surface. At most piles that deterioration may not penetrate deeply into the heartwood. See Photos 1-5.
b) Decay is advancing at sawn ends of several cross-bracing members. This deterioration appears to be extending into areas where bolted connections are located, bringing the integrity of those connections into question. See Photos 6-10.
c) Areas of damage resulting from vandalism were observed. Photos 11-12.
d) Several steel fasteners and related connecting hardware were observed to be heavily corroded and in need of replacement. Photos 13-16

e) Some rail ties, in particular a grouping near the junction of the eastern trestle approach and the eastern steel girder span have been replaced in recent years. However, those ties were not treated with creosote and have suffered serious rot related deterioration. Those rotted ties, although appearing to have originally been treated with a chromated copper arsenate (CCA) or similar preservative solution are no longer serviceable in spite of being installed long after original construction of the bridge. Remaining original rail ties, though exhibiting some splits and some sawn end deterioration tend to be in much better condition than those recent replacements. See Photos 17-18

f) CCA treated Bridge decking installed over the original timber rail ties and related side safety treated lumber guards appear to be in fair to good condition, except for occasional loose, worn or otherwise damaged boards.

g) Most timber core samples at creosoted ties, stringers and pile caps exhibited fair to good internal cross section integrity, even if some of the cores were broken during the extraction process. However, several cores taken from trestle piles and braces exhibited delamination and evidence of internal rot. See Photos 59-65

2.3.2 STEEL PLATE GIRDERS

Two pairs of steel plate girders (four girders in total) were used to span between the stone masonry piers over the river. Each span was constructed of two 73’ long girders spaced 9’ apart. The spans were labelled East and West and the two girders in each span were labelled North and South. See Appendix A.

The plate girders are built-up sections fabricated from riveted plate and angles to form substantial typical steel “I-sections”. The girders have incorporated deeper sections at mid-span where bending moment is greater and have incorporated web doubler-plates at the supports where shear forces are higher. Girder section height varies from about four feet at the masonry supports to about seven feet at mid-span.

As well, girder flange widths increase from 12 ½” at the supports to 24” at mid-span and bottom flange thicknesses increases from about ¾” to 2” at mid-span by the incremental addition of added flange plates. The addition of these reinforcing plates has had the effect of strengthening and stiffening the utilized girder sections, while also making these components less prone to vibration induced damage such as metal fatigue in spite of having been exposed to heavy service loads and significant vibration while in service as rail infrastructure.

Connecting angles between the ½” web and the flanges are L 6” x 6” x ¾”. The girder webs are stiffened by “T” sections and angles riveted to each side. The plate girders are fully braced, for wind and lateral loading at the top and bottom flanges. Vertical cross braces between paired girders are provided at 9’ centres. Bracing members are usually L 3 ½” x 3 ½” x 3/8” with 3/8” thick gusset plates.

Considering the age and the service conditions (the site is located close enough to salt water to be considered a marine environment) the plate girders and related parts are in overall fair condition, with some exceptions:

- There is a relatively uniform layer of surface corrosion on all exposed surfaces of the girders, and only small remnants of a former protective coating remain on the structure. However, except in some locations where corrosion is noted as being more severe, many steel components appear to exhibit original or near original thicknesses. This observation was a bit of an initial surprise, but it appears that the tight-grained surface oxidation layer may be sealing the exposed metal and protecting it from further or rapid deterioration. More modern Grades of steel have been available that have been designed to withstand corrosion by generating a layer of sealing surface oxidation. Although a similar process may be taking place on the exposed metal surfaces at the Gold River bridge it appears that this may be a matter of happenstance rather than planned design.

- Some heavier amounts of corrosion were found along the top flanges of the girders where dirt and debris can collect and retain moisture, which has resulted in pack rust and deterioration of the angle bracing and gusset plates in these areas.

Remnant paint chips were removed from the girders and sent to a lab for analysis which indicated high levels of lead and other toxins. See appendix A for test results.

Thickness readings were taken periodically throughout the girder inspection. See below for tabulated results for average readings as provided by Tacten (see also Appendix A):

TYP. CROSS MEMBER CONNECTING PLATE-CENTER	0.453"	PHOTO 39
TYP. CROSS MEMBER CONNECTING PLATE-CORNER	0.413"	PHOTO 40
TYPICAL CROSS MEMBER	0.374"	PHOTO 41
TYPICAL GIRDER WEB STIFFENERS	0.606"	PHOTO 42
TYPICAL GIRDER WEB	0.510"	PHOTO 43

See photos 24-43 and supplemental photos 1-7.

2.3.3 BEARINGS

Plate steel bearings girder support points are located at the tops of all three masonry piers and provide the connection of the pier caps to the bottom flange ends of the main span steel plate girders. The East and West piers of the bridge, located approximately at each river bank, support the fixed girder bearings points while the center pier supports non-fixed, sliding bearings for each girder. The non-fixed sliding bearings allow for longitudinal movement of the girders at their supports which allows for longitudinal thermal expansion and contraction of the girders while not inducing horizontal longitudinal and thermally induced girder end reactions at the tops of the masonry piers.

A fixed bearing connection is created using 1 ½” diameter anchor bolts connecting the layered and shimmed plate steel bearings to the masonry pier tower stone caps. The non-fixed bearing uses 1 ½” anchor bolts in combination with slotted holes and a bearing slide to allow for longitudinal thermal expansion movements of the girders while resisting lateral forces.

It has been observed and reported by our sub-consultant high angle inspection crew that the girder bearing assemblies exhibit significant visible corrosion throughout and that the sliding bearings are no longer functioning. The sliding bearings are therefore considered to have failed. As a result of this failure, the tops of the stone masonry piers are exposed to and forced to resist steel girder horizontal longitudinal thermal expansion and contraction forces. See photos 55-58

There is some visible evidence that the ashlar stone blocks at the tops of the stone masonry piers supporting the sliding girder bearings at the centre pier have broken free from the pier caps and have been inadvertently providing the function of a sliding bearing. If this is actually taking place, then multiple one to two ton granite blocks are in motion and sliding on themselves in an uncontrolled fashion as dictated by forces induced by thermal expansion and contraction of the steel girder pairs.

2.3.4 STONE MASONRY PIERS

Stone masonry piers are approximately 12’ long x 22’ wide at the base and 6’ x 16’ at the caps and are constructed of rock-faced granite ashlar stones which vary slightly in size, maximum 2’ x 4’ with uniform bed heights of about 20 to 26”(+/-). Stones were originally mortared together in a typical stretcher course arrangement. The piers vary in height depending on ground and foundation elevation, but nominal height is about 55 feet above the normal river water surface level. The pier interior is likely mostly rubble based filler material.

The bridge is located on a horizontal curve on the railway alignment and as a result there is a slight super-elevation (lateral slope to the travelled surface) at the pier caps and at the bearing assemblies.

Most of the mortared joints throughout all three masonry piers have either completely deteriorated or are providing minimal function. The pier cap joints were also found to be in poor condition, allowing water infiltration into the interior of the pier. This moisture will freeze in winter conditions and can damage the piers if the moisture cannot adequately evacuate the structure.

Note that the center pier cap and nearby stones show evidence movement. This could be the result of failed girder sliding bearings.

Other undesirable observed conditions include vegetation growth at stone joints.

Several stone blocks throughout the pier structures have been observed to be cracked and otherwise damaged. A description of observed damages is included in Appendix A – Masontech site inspection report.

See photos 44-54.

3 EVALUATION

3.1 DETERIORATION & CONTEXT OF CONDITION

A structure such as the Gold River Multi-Use Bridge would normally be expected to provide approximately 50 to 75 years of service life following initial construction so long as regular maintenance and repairs are carried out. This means that the bridge is already about 25 to 50 years past its expected service life.

The 2013 report by SNC indicated that known deficiencies will continue to become worse with time unless efforts are made to correct, control and prevent deterioration and degradation of structural elements of the bridge. ABLE agrees with that statement. Unfortunately, no repairs have been initiated since publication of the SNC document. Therefore, deteriorated conditions observed at site and described in that report can be expected to have worsened since 2013.

However, it should be noted that the bridge's original construction and operational service conditions differ greatly from current service demands placed on the structure:

As indicated, the bridge was originally built as a part of the Nova Scotia south shore railway network linking the city of Halifax to Liverpool in the Region of Queens Municipality (formerly Queens County) and other more inland communities. Upon closure of that rail line, existing related infrastructure such as railway bridges were not demolished, but were left in place and permitted to deteriorate for many years without on-going planned maintenance.

Recent revival of the rail line system and related infrastructure as primarily a pedestrian and bicycle trail exposes the bridge structures on the trail to vastly reduced live loadings. Because service load requirements for these structures have been so significantly reduced, it has been considered in recent years and by way of recent condition assessments (2001 and 2013) that even though the Gold River bridge has been subjected to significant deterioration since abandonment, that it has appeared to remain robust enough in its existing deteriorated condition to provide adequate structural capacities for its repurposed service.

Although that consideration/assumption has been generally correct when applied to assessments of many of the structures incorporated into the trail network, and in particular for lighter, shorter span bridges, there are cases where remnant structures have been deemed inadequate for planned re-purposing and as a result have not been put back into re-purposed service without first receiving significant repairs and/or reconstruction. Larger and more complex structures such as the Gold River Multi-Use Bridge are constructed of more complex and inter-dependent structural systems than some smaller structures, and therefore are more dependent upon functionality of many specific components. With the Gold River bridge those components include timber trestles and related parts, girder bearing assemblies, and relatively high (and unreinforced) masonry pier supports. At larger structures, due to their overall size and component mass, and depending on overall condition of the remnant components, self-weight loads rather than reduced live service loads can dictate whether continued safe operability of that infrastructure is possible. Therefore, deficiencies in certain critical components in those larger structures can present significant risk to the structural integrity of that infrastructure.

Previous inspections and condition assessment reports for the Gold River Multi-Use Bridge have reported that remnant live load service capacities of deteriorating components have at the time of those inspections appeared to be adequate for the structure to remain in service as a trail bridge. However, on-going repairs and planned maintenance is critically necessary if remnant structural capacities are to remain adequate. Very little on-going maintenance and repairs appear to have been carried out at the Gold River bridge since about the early to mid-2000's.

Without repairs and on-going maintenance such structures, some already in excess of 100 years old, will have a very limited remaining service life expectation. Further, once significant degradation becomes clearly apparent, the state of that deterioration is usually at an accelerated rate. Such is the case at Gold River, where creosoted timber systems and metal fastening components are beginning to show significant visual deterioration, steel girder flanges are corroded in specific locations, girder support bearings have become non-functional, and masonry support piers exhibit what may be significant damage.

3.1.1 TIMBER TRESTLES

The 2001 Waugh condition assessment report found the Timber components, considering their age, to be in excellent condition. Using the same inspection techniques of visually assessing and hammer sounding used in that inspection exercise many timber components were found to be still in good condition. However, although creosote has proven to be a very effective preservative treatment for timber components and an effective corrosion inhibitor for steel and other metal components such as fasteners, many of the existing piles, braces and struts have begun to show their age with a slightly mottled surface appearance, surface splitting, delamination and brooming at saw-cut ends exposed to weather.

Unfortunately, creosote treated timber used in infrastructure projects has a normal and finite expected service lifespan which is normally approximately 50 years. Longer service life is possible in some applications, such as structures which remain fully submerged or those in fully dry service and which are exposed to good air-flow around structural components. The Gold River bridge timber approach trestles are structures which have been exposed to service conditions which are conducive to extended service life. However, adequate performance in service in excess of 100 years for a timber structure without significant reconstruction or repairs is a rarity. After that much time in service even creosote treated wood in the Nova Scotia climate can start to become soft, and begin to lose strength as the wood fibres undergo a natural organic degradation. When such deterioration sets in the deterioration rate usually accelerates and structural components quickly lose integrity. Similarly, steel fasteners and related components in service in a marine climate quickly deteriorate once protective coatings wear away. This is the condition in which the timber trestle components are now found. Therefore, the trestle structures have significantly exceeded their practical service lives.

Note that not all timber components are in a poor condition. Existing stringers and rail ties (with some noted exceptions) appear to remain in very good condition. Some of those components, sheltered from rain and poor weather have remained so sound that hand-boring equipment meant for core sample retrievals was damaged in the coring process, as the wood was so solid that it resisted cutting by that specialized tool.

Although decaying of the ends of the timber cross bracing members at the trestle structures was identified in 2001, it was noted that such local degradation was not particularly extensive or serious at that time and that observed damage was at that time unlikely to detrimentally affect the load carrying capacity of those components. However, the structure has seen an additional 20 years of service since that report was produced, and timber conditions appear to have generally deteriorated over that period. Much of the sawn end deterioration has extended into locations where metal connectors are utilized. Shear resistance of the wood fibres at many of those connections appears to have significantly degraded. Therefore, and in consideration of the age of the trestles, the timber trestles would now be classified as being in poor condition. The trestles are exhibiting only limited remaining expected effective service life without extensive repair/reconstruction or replacement.

Note however that undertaking a trestle repair project is now no longer practical, the structures must be replaced. It is not practical to repair the trestle structures and leave some 100+ year-old partially deteriorated timber components and metal fasteners (with limited remaining service expected life) in place. Therefore, and due to the nature of the timber trestle structure, and the expected extent of required disassembly needed during a repair exercise, it will be more expedient and much more worthwhile to replace those timber trestle structures in their entirety rather than attempt to repair or reconstruct.

Although alternative methods of repairing piles have been explored, such work would not result in any expected service life improvements. See photo 22 which identifies a previous pile splice repair. Such a repair technique should be avoided unless adequate supplemental struts and bracing are installed to provide improved lateral rigidity at the joint. Such repairs should not be considered to be permanent and should not be implemented in a wide-spread manner. The layout and details of the timber trestles can be found in Appendix D.

Replacement of some deteriorated timber rail ties was recommended in 2001. It is unknown whether/when that work was carried out. Photos 17-18 show severe deterioration of replacement rail ties. Those newer ties have decayed at a much faster rate than adjacent original creosote treated timber parts and are now unserviceable.

Creosote treatment has proven itself over many years to be one of the most effective means of protecting timber structures from rot and decay. However, creosote and related coal-tar based products are extremely toxic and are known carcinogens. Therefore, very little timber is so treated nowadays. Even creosote protection does not last forever, and rot based deterioration has occurred at the porous ends of the various timbers which may have been cut to size at site during original construction, and after factory creosote treatment.

Evaluation of timber core retrieved samples generally confirmed findings of visual examinations and hammer-sounding assessments. (See photos 59-65) Core samples proved rot existed where suspected and existence of good quality wood beneath the creosoted exterior is found elsewhere. However, all examinations revealed some degree of visible deterioration in most timber components of the trestle structures.

The timber trestle structure would benefit from the removal of natural vegetation (trees and shrubs) nearby the structure as vegetation growth as observed can impede air flow around timber components and can thereby promote accelerated timber decay. Enhanced free-flow of air around above-ground structures usually improves service longevity.

Note that the overall findings of the current timber inspection broadly resemble what was found by SNC in 2013, except that the extent and severity of degradation appears to now be worse.

3.1.2 STEEL PLATE GIRDERS

Steel plate girders were determined to be in mostly very good condition in 2001 with only limited evidence of significant corrosion. In 2013 the steel plate girders and angle bracing were deemed to be in good condition with some minor surface rusting, while the top flange plates and gusset plates exhibited visible evidence of surface rusting at a few locations.

In the current examination it was noted that the top flange of the plate girders was experiencing some decay in several areas due to moisture being trapped between the flange and the rail ties where dust and debris has been able to accumulate.

Heavy amounts of pack rust were found in locations at girder bottom flange stiffener connections where the stiffener ties into the flange. The areas of note were located along the flat portion of the girders and on the internal sides of the girders. Approximately 40% of the connections were affected. See Appendix A – supplemental photos 6-7 for reference and associated markup sketch indicating location.

Note that along the girder top flanges approximately 50% of the vertical cross bracing upper corner connection plates had heavy amounts of pitting and corrosion. Some of these plates will require replacement. See Appendix A - supplemental photos 4-5 for reference and associated markup sketch indicating location.

Approximately 90-95% of the top chord horizontal diagonal bracing connection gusset plates had significant amounts of pitting and corrosion (heaviest on the top side) throughout the steel assemblies. Some of these bracing plates will require replacement. See Appendix A - supplemental photos 1-3 for reference and associated markup sketch indicating location.

Extent and amounts of damage and corrosion observed in this inspection exercise appear to have progressed since 2013. This should come as no surprise since the degradation process is one that continues if not abated. As well, there are areas of significant corrosion at the girders that appear to have not been identified in previous reports. One of these locations is identified in Appendix A - Photo 36 where there is relatively heavy pitting along the bottom flange.

Where significant metal degradation has been observed and it is determined that repairs are necessary, it is recommended that replacement pieces be engineered and installed as full moment bolted connection splices (allowing full bending stress and tension/compression transfer). These repair pieces will have to be bolted into the girder sections as and where necessary to provide near like-for-like repairs to the existing riveted structure. As previously indicated, the existing structural steel likely has a high carbon content which will make provision of effective structural steel repairs by way of welding impractical.

A protective coating (paint) was at one time applied to the steel girder assemblies. That coating appears to have failed several decades ago, sometime prior to 2001. However, the steel utilized in primary girder components in construction of this bridge, produced in the early 1900's appears to have the ability to surface oxidize and seal itself, reducing its exposure to corrosive conditions and compounds. A light but uniform layer of surface corrosion was observed on most exposed steel surfaces. That surface oxidation is comparable, to what was described in the 2013 inspection report.

As a result, and given the limited structural steel corrosion damage observed at the bridge, it appears that blasting and recoating is not necessary. Further, costs associated with environmental protection associated with steel surface preparation/media blasting of high-lead remnant paint can be avoided. (Note that similar conditions were recently discovered at the Liverpool Mersey River rail crossing bridge. Due to the condition of the existing steel at that location it was decided to not recoat that structure.)

However, remnants of the previous protective coating will hold moisture and contribute to further unwanted corrosion in those areas and should therefore be mechanically removed and disposed of off-site.

This inspection confirmed the plate girders and associated stiffeners and bracing generally remain in fair condition, and that local repairs will be necessary in order to maintain steel girder structural integrity.

3.1.3 BEARINGS

The expansion bearings specifically were noted to be rusty in 2001. The 2013 inspection found all bearings to be in poor condition and in need of replacement. This inspection confirmed there is heavy corrosion in all the bearing plates and that there has been no visible free movement at the sliding bearings for some time. As a result, it is concluded that that the sliding bearings (at least) have failed. See photo 56.

The bearing plates have been deteriorating for many years. As they have corroded the coefficient of friction between the sliding bearing plates has steadily increased. That means that although vertical bridge girder loads and reactions may still be adequately distributed to bridge masonry support piers, effectiveness of sliding bearings to accommodate longitudinal girder movements has been becoming less and less efficient. In their current condition the sliding bearings are non-functional.

Based on bridge code calculations the expected total thermally induced longitudinal movement for the 73' girders is expected to be about 0.75 to 1.0 inches. Without adequately operating sliding support bearings at one end of each pair of girders the tops of the rigid masonry piers will be forced to resist longitudinal forces associated with that movement. Those forces will be considerable and will be measured in tons. The stone piers have not been constructed with the intent of being able to withstand such horizontal loadings.

Although the thermal expansion and contraction of the girders is not a large dimension in terms of the existing bridge height and span, this is a significant distance when considering the inherent rigidity and brittle nature of the supporting vertical stone masonry piers. Such induced loadings at the pier tops will force the rigid and unreinforced stone masonry piers into flexure and to act as vertical cantilevers. Such loading will eventually result in significant damage to the masonry structures, rendering the piers unstable in service.

There is some visible evidence that stone blocks supporting the bearings at the masonry pier caps may have worked loose and are now shifting and inadvertently providing allowance for expansion and contraction of the steel girders. This condition represents instability in the bridge structure. See photos 47-51.

Although visual examinations suggest that the bridge masonry piers are not in imminent danger of collapse, such conditions represents a structural instability that must be mitigated as soon as practical. However, due to recent pandemic restrictions and manufacturing slow-downs many replacement bridge bearings have become relatively long delivery items and therefore cannot be replaced at short notice.

Therefore, as temperatures cool and the main span steel girders contract, implementation of a masonry pier cap and bearing monitoring program to observe and measure pier cap stone blocks displacements and/or pier tower wobble/deflections is warranted. Such an information gathering exercise will enable more precise determinations of whether or not and to what degree the bridge structure is experiencing structural failure.

Note that the optimum time for bearing replacement activity may be during warmer months when the bridge span is at or near its maximum expansion and night temperatures are not significantly lower than day temperatures.

Also note that it is recommended that bearing replacements be undertaken as part of a wider masonry pier tower reconstruction effort as bearings cannot be fully replaced until masonry pier caps are stabilized and yet, pier caps cannot be stabilized until bearings are again functioning adequately.

Bearings can be temporarily accessed for replacement via a partial disassembly of the bridge deck travelled way and temporary removal of rail ties in the vicinity of the bridge bearings. The bridge will have to be temporarily closed to the public while bearings replacement work is underway.

3.1.4 STONE MASONRY PIERS

The stone masonry piers that support the main span steel girders are gravity-type structures that primarily support vertical loads and reactions, and are dependent upon on their self-mass and geometry for stability and resistance of light to moderate horizontal loads. These structures are not reinforced with steel and have no significant tensile or flexural structural capacity. Such structures are considered to be rigid and inflexible, and not intended to resist significant horizontal live loads. It is especially important that the piers not be exposed to concentrated horizontal live loads applied at the tops of the piers. These structures are wholly reliant on their mass and cohesive interconnection of their parts for structural integrity and stability.

Although each of the pier tower structures has a geometry which provides accommodation (resistance) for moderate lateral loads associated with wind or earthquakes, the tops of the stone masonry piers are not intended to ever be exposed to concentrated horizontal longitudinal loadings associated with thermal expansion/contraction of the steel girder pairs. If the tops of the piers are subjected to such forces the piers will be prone to significant structural damage. Such damage and continued unintended loadings can eventually result in an uncontrolled collapse of the structure. Therefore, it is imperative that the girder bearings always be maintained in serviceable condition.

It is critically important that the integrity of all the components of these constructed pier supports (including girder bearings) be kept well maintained for so long as the masonry pier towers remain in service. Other components that must be maintained in good condition include, but are not limited to: foundation material; stone blocks; mortar; drains and vents; cap stones; mechanical fastenings; and, joints and seals.

Severe mortar joint deterioration in all three piers and pier caps was observed in 2013. Similar observations have been made as part of this inspection, although it appears that mortar deterioration at the piers is more extensive in 2021 than was reported in 2013.

Note that loss of mortar in stone masonry structures over a period of at least 100 years is not surprising. Some of the older mortar mixes didn't have the long-term durability that more modern mixes now exhibit. Older lime-based mortar mixes are easily damaged by movement and/or vibration. Probable causes for mortar failure at Gold River are:

- Long-term exposure to weathering;
- Exposure to induced vibrations from normal railway transport loadings;
- Freeze thaw cycle damage and exposure to moisture that may enter the piers via the joints in the stone pier caps;
- Unintended movement of component stones in the pier structure;
- Unintended flexure (wobble) of the rigid pier structures due to exposure to forces related to expansion and contraction of the steel girder spans.

Long term exposure to wind and rain will eventually degrade and damage cementitious mortars and grouts. This damage is usually characterized by the grout eroding from the joints between stones, or having pieces of grout literally fall out of the joints as they de-bond from the granite stone.

Rail cars and locomotives induce heavy loadings and high frequency vibrations into structures such as rail bridges. Stone masonry construction is relatively rigid and brittle and is therefore prone to damage when exposed to such cyclical and high impact loadings. Such vibrations will tend to crack mortar and stone. Since older masonry mortar is not usually reinforced, pieces of the mortar will fall out of the joints as they flex and fracture.

Cementitious mortar is prone to freeze and thaw damage in winter months especially in the presence of moisture. The mortar will absorb moisture, especially at cracks, and when that moisture freezes it will expand and cause further cracking in those cementitious components. This can become a more severe problem if moisture is permitted to enter the core of the structure and cannot evacuate via installed drains or vents at the mortar joints. Under such conditions that moisture can exert hydrostatic pressure on the interior surface of the stone blocks and mortar joints. Such pressure will tend to move the blocks and push the mortar or mortar remnants out of the joint. If stone blocks experience non-uniform movement, pieces of the mortar joint will break and fall from the masonry pier structures.

Stone cap joints in their current condition are recognized as a source for water ingress to the pier cores in the 2013 SNC report, and correction of this condition was identified therein as a high priority repair item. As indicated, those repairs were not carried out subsequent to the 2013 inspection report.

Girder sliding bearings no longer function. Therefore it is probable that forces associated with thermal expansion/contraction of the steel girders may be inducing loads which cause local movement in stone blocks. Any non-uniform movement of stone blocks in the masonry piers will cause a mechanical failure of the mortar.

Because girder sliding bearings no longer function, it is probable that forces associated with thermal expansion/contraction of the steel girders may be inducing a wobble to the pier towers. If the pier towers flex, or blocks move in a non-uniform manner a mechanical failure of the mortar will result.

Missing and cracked mortar within the masonry stone joints is widespread. Block mortar is not only essential to preserving structural integrity of pier tower stone blocks, it also provides resistance to water infiltration at the masonry core material, and thereby reduces the effects of detrimental freeze and thaw cycles. Therefore mortar repair remains a top repair priority for the masonry structures. Failure to repair the mortar will lead to more serious and more visible structural deterioration in the stone masonry.

Mortar appears to have originally extended into the stone joints at least several inches. High quality replacement mortar should extend similarly into the joints to form tight and secure bonds between stone blocks. A repointing effort should be carried out to adequately seal the structure from on-going moisture infiltration while allowing remnant core moisture a means for draining and venting from the structure. Repaired mortar should be inspected regularly post repair, and further repairs periodically implemented upon discovery of new damage.

Stone damage will normally occur as a result of mechanical impact loadings (sometimes these impacts are repetitive), exposure to other concentrated stresses and/or expansion of existing cracking via freeze-thaw activity and/or non-uniform movement in the piers. Stone blocks can chip, spall, break and split depending on source and type of loading responsible for observed damage. If internal hydrostatic loads develop, those forces can displace unanchored stone blocks. As well, failure at sliding bearing components can also result in block damage and displacement.

Light damage to blocks such as corner cracks and spalling can sometimes be accommodated by the structure without significant repair efforts. A mortar repair may remedy a corner or edge crack, while a small surface spall may not require any attention.

Replacement is usually the most effective means for correcting severe block damage. Such damage would include breaking, splitting, lateral or angular fracture or crushing damage. However, depending on location and the amount of disassembly or shoring of the structure that might be necessary, block replacement may not be a feasible alternative. In that case, local in-situ repairs might be more practical. Such repairs may involve one or more of the following repair techniques and materials:

- Use of epoxy adhesives to bond damaged parts;
- Tools to bore into the block to allow the installation of metal rods and other mechanical fasteners to re-join damaged block parts.
- Installation of smaller patch Dutchman blocks or simple mortar infills as appropriate.

Typically each individual damaged stone block should be independently assessed for whether damage should be repaired. If a damaged block is to be repaired, it should be determined which repair method is best to be employed in the repair of that particular block.

In 2013 the masonry stones were found to be in relatively good condition. In 2021 while most of the stone blocks remain in good condition, some stones have been found to exhibit some movements, splits and cracks.

As indicated, masonry piers probably originally incorporated a rubble filled core. Loss of mortar from the stone block joints can also result in loss of fine material from the core. The longer that joints are left open, the longer that the core may be subjected to migration of material. Photo 49 shows a tape measure inserted into a missing mortar joint of the centre pier as part of the most recent inspection exercise. It Measures approximately 7' deep until reaching firm core material. A similar picture from the 2013 report also shows a tape measure in a missing mortar joint of the centre pier. The measuring tape probe measured approximately 4' deep at that location at that time.

So, the stone masonry pier tower structures are not intended to resist significant horizontal loads, or to resist horizontal forces associated with longitudinal thermal expansion of steel girders. Functioning support bearings are intended to protect the masonry piers from being exposed to expansion/contraction related horizontal longitudinal loads from the steel girders. To correct overloading and/or structural destabilizations the stone masonry pier towers must be repaired and girder support bearings must be replaced.

Masonry piers repair scope should include the following:

- Repointing of any open joints;
- Repair of fractured granite ashlar;
- Resetting of dislodged granite blocks;
- Installation of drain and vent holes in the mortared joints;
- Caulking of joints in top of granite caps with a high quality sealer is required;
- Dutchman repairs to granite blocks should be carried out where determined necessary;
- Cleaning of granite surfaces (Optional).

See attached masonry pier inspection report from Masontech Limited in Appendix A.

4 SUMMARY & RECOMMENDATIONS

The findings and recommendations of this bridge structure inspection and condition assessment report are based on visual examination and experienced engineering judgement.

Note that this report compilation is not a detailed engineering design exercise meant to specifically derive damage mitigations for planned repairs. Rather, this document is an inspection and condition assessment of existing structural components condition with the intent to identify deficiencies and to provide conceptual repair alternatives and related Order of Magnitude budgetary construction cost estimates.

It appears, from visual examinations that damage and structural distress being experienced by the bridge is extensive and will eventually lead to general structural failure. However, there are no current definitive visual indications that the bridge is in imminent danger of collapse. However, such a finding should be confirmed by more detailed measurement and data collection at site.

More detailed measurement and data interpretation is required to enable a more precise determination of whether or not the structure is experiencing stresses and distortions that will lead to structural failure in the short term. To that end a monitoring, measurement and inspection plan that can record and compare variable distortions is recommended. Section 4.1

of this report describes the recommended inspection and monitoring requirements that can be carried out over the next several months and which will provide more detailed data enabling more accurate determination of the level of distress that the structure is experiencing.

The list of required repairs for the structure is extensive and will probably become larger as partial disassembly and repair activities are likely to reveal further deficiencies.

Gold River Multi-Use Bridge has seen limited on-going repairs and planned maintenance since the early 2000's. The structure is now in excess of 100 years old and is in need of extensive repairs and reconstructions if it is to remain in service.

Because of the extent and nature of damage and deteriorations on primary and secondary structural components it is not possible to derive existing dead load and service load capacities except by the most conservative methods and by incorporation of the most conservative assumptions. However, if necessary repairs are carried out, once they are completed a reasonable assessment of actual post reconstruction structural capacities can be derived.

Similarly, the extent of bridge damage and required repairs is such that it is not possible without detailed measurement data to accurately derive a reasonable deterioration rate or remaining service life for the structure. Further, since many of the structural components of the bridge are already at or beyond their expected service lives, deterioration rates and estimates of remaining service life are irrelevant.

As indicated, repair requirements are extensive, and a phased approach for repairs and reconstruction works may be appropriate. Recommended work includes replacement of timber trestles at each approach to the structure, repairs to main span steel plate girders assemblies, replacement of girder bearing assemblies, and repairs to stone masonry girder support piers.

No cost or scheduling allowance has been made for environmental permitting or related applications or consultations required for repair/reconstruction construction approvals.

A brief description of four possible alternatives for the existing bridge structure are listed and described in Section 4.2 below. A repairs option, two replacement options ae well as a removal from service and decommissioning option (without replacement) are considered and summarized.

The listed options are all costly, but are worthy of consideration in moving forward. Please refer to Appendix C for related construction cost estimates.

Note that to do nothing and to let the bridge continue to fall further into disrepair is not a practical option given the potential public safety risk and environmental liabilities that could result from an eventual uncontrolled structural collapse.

Breakdown of costs are included in Appendix C. All listed cost estimates are considered to be Class 4 in conformance with AACE International Cost Estimate Classification System, and are therefore exclusive of contingency amounts and applicable taxes.

Since there are no definitive visible indications of imminent failure it is difficult to precisely determine when the bridge will fail. **However, has been determined that due to the observed**

condition of the bridge and its support components it is a certainty that the bridge is at risk of structural failure. Therefore, it is recommended that MODC to immediately remove the Gold River Multi-Use Bridge from service and close it to public access until such time that required repairs and reconstructions can be completed.

4.1 PRE-REFURBISHMENT MONITORING AND MEASUREMENT INSPECTION PROGRAM

Sliding expansion bearing assemblies at the top of the central stone masonry girder support tower are no longer functional. The primary purpose of those bearings (and the non-sliding bearings located at the tops of the east and west towers) is to transfer vertical reaction loads from the steel main span girders to each of the main support piers, and to accommodate horizontal longitudinal movements associated with thermal expansion and contraction of the steel girders via a horizontal sliding connections (expansion bearings). Accommodation of those movements at the bearing assemblies prevents imposition of horizontal loads at the tops of the stone masonry pier towers which are associated with resistance of expansion and contraction of the steel girder assemblies.

The stone masonry piers are gravity structures and when in good repair have a very significant vertical load capacity (having been constructed to carry railway locomotive and freight car loadings). However, unlike modern reinforced concrete bridge support structures, the rigid stone masonry piers are unreinforced. Therefore the piers have a limited capacity to withstand imparted horizontal longitudinal and lateral loadings before experiencing fracture and possible eventual structural failure. Therefore a loss of functionality of girder support bearings and the accompanying loss of capacity to accommodate expansion and contraction of major steel components is potentially a very serious structural deficiency at the bridge.

Since the sliding expansion bearings at the central pier are no longer functioning, considerable longitudinal horizontal loads are now being imparted to the tops of the masonry support piers when the steel girders expand and when they contract.

The nominal thermal gradient (coldest normal expected winter steel temperature to warmest expected summer steel temperature, based on normal ambient temperatures) appear to induce about 0.75 to about 1.0 inches expansion and contraction in each steel girder span. Although that expansion is not of an overly large magnitude given the total span of each girder, given the overall size of the girders the force required to resist horizontal girder expansion and contraction will be measured in tons.

Under these conditions, with non-functioning sliding expansion bearings at the central pier, the girders will in warm weather go into axial compression and push against each other at the central support pier. This may result in a reaction where the expanding girders thus push the tops of the east and west masonry piers further from the central pier. In this loading condition the east and west piers will act as vertical cantilevers to resist steel girder horizontal expansion. Such horizontal loading will be expected to cause some longitudinal horizontal deflection at the tops of the east and west pier towers.

In cold conditions pier horizontal loadings opposite to those experienced in warm conditions will occur. When cold, the girders will go into tension as they contract. Under cold horizontal loading conditions longitudinal horizontal forces at the tops of the east and west piers will reverse, drawing the tops of those piers closer to the central pier tower. In addition, a significant shear or vertical tearing stress will be imparted to the top and approximate lateral centre line of the central pier to resist girder tensile forces from both directions when the girders contract. The contraction induced forces at the top of the central pier will literally be trying to tear the top of that pier apart in opposite directions while the tops of the east and west piers are pulled in a direction opposite to how they are loaded in warm weather.

Since the pier girder expansion bearing assemblies are no longer functional, it should be noted that the bridge is located on a horizontal curve in the former railway alignment. Therefore opposing horizontal loads at the top of the central pier are not equal and opposite, are not balanced and will not cancel each other. Instead, an unbalanced circumferential resultant load will be experienced at the top of the central pier under those conditions. In warm weather that reaction will be induced by girder compression in a direction normal (at a right angle) to the outside of the curve alignment at the top of the central pier. In cooler weather, a similar but opposite net lateral unbalanced load reaction will be produced by girder contraction (tension) at the inside of the curve at the top of the central pier. Left unmitigated, these loads will eventually destabilize the central masonry pier tower. These loads and reactions may induce a measurable wobble at each of the masonry pier towers.

The failed bearings may not only induce lateral movements in the pier towers. As indicated, since the un-mortared stone blocks may now be only held in place by self-weight and friction, stone blocks which are rigidly attached to existing bearing assemblies may be unstable and disconnected from the pier caps. Such stones may have inadvertently assumed the function of the sliding expansion bearing assemblies. This phenomenon may be occurring at the east and west piers caps as well as at the top of the central pier.

When weather is warm and the sun is shining on the steel girders they may experience higher than normal theoretical temperature gradients and may experience even greater expansion rates than that suggested above.

Therefore, depending on how the piers actually react to each other in warm and cold conditions, and how well the masonry structures resist these horizontal longitudinal loadings, the east and west structures could each be subjected to as much as an inch of net sway (or more), while the central pier stone masonry tower cap and bearing seats could consist of stone blocks which are floating on top of the pier structure, no longer effectively rigidly interconnected with the other blocks.

If it is intended to return the Gold River Multi-Use Bridge to service, establishment of a comprehensive monitoring and measurement program is worthwhile and is recommended. Such a program should span the next several months at least, and should collect precise and detailed geometric data on bridge deflections/distortions and measurable bridge pier non-uniform movements.

A comprehensive monitoring and measurement program will be one that provides precise monitoring and measuring of longitudinal and lateral pier sway and deflections as well as providing data on possible individual granite block instabilities and relative movements of those blocks. Engineering interpretation of collected site data will be a necessary part of such a program. Through data interpretation it should be possible to make determination as to whether or not the bridge structure is at risk of imminent collapse.

An effective monitoring program would involve repeated visits to site to measure and re-measure the locations of points on the bridge in three dimensions over a period of time to identify any changes in relative dimensions and geometry at varying ambient temperatures.

Measurements taken in warm and in cold conditions would be compared. Changes in measured distances and/or locations of reference points on the bridge relative to established independent control points would be indicative of movement, flexure or displacement at the bridge structure. From that data the amount of movement in the tops of bridge piers, for instance, can be accurately determined.

It may at some time become apparent during such a monitoring program that keeping a record of the precise length of the main girder bridge spans through warm and cold weather may also be useful. However, that may add cost to a monitoring program. The focus of information collection in this program must be on how much the rigid pier structures actually move, and whether there is any relative measurable individual stone block component movement.

A Total Station (TS) survey is a method of electronic surveying of structures with Electronic Distance Measuring (EDM) devices to accurately determine straight line distances as well as horizontal and vertical angles to selected points on the structure relative to established stationary control locations. When measured distances and angles are located by triangulation and are plotted in three dimensions, measured points on the structure can be located in space relative to established stationary control points. Small changes in measured dimensions and angles from one site visit to the next would represent structural displacements. A TS survey method is probably the easiest and most effective means for establishing and monitoring overall bridge and pier geometry and general structural displacements.

Monitoring of selected points on the bridge structure would be carried out by the installation of reflective offset prisms rather than metal pins. Prisms will allow accurate measurements to be taken with the total station base unit without having to place a movable target on various fixed pins at hard to get at locations. Note that many prism installations will have to be carried out in hard to get at and high angle locations. Therefore it is likely that a contractor such as Tacten would be required to carry out those prism installs.

It may be necessary to carry out some pre-construction vegetation and tree cutting and clearing at site to ensure adequate site lines for the TS survey to be effective.

Such a monitoring program should be arranged to easily collect as much pertinent bridge structural and geometric data as possible while being able to illustrate with collected data the existing bridge condition/geometry and relative displacements at various ambient temperatures. A comprehensive TS type survey with established local control points (and a tie-in with other technology such as Lidar) can be used to collect data, establish geometry and monitor dimensional changes in the structure. However, a TS survey will in all likelihood not be a practical means for identifying or monitoring possible movement of granite blocks relative to the position of masonry pier support structure in which those blocks reside.

Collection of detailed cut ashlar stone block location/movement data will require the introduction of a 3D laser scan survey. By adding the laser scan data collection enhancement, a much more complete picture of movements of structural components in the stone masonry piers can be determined.

Regardless of the level of precision which is available with a TS or laser scan survey data collection exercise, each site visit will only provide a snap-shot in time of the geometric conditions of the bridge at the time of a particular visit to site. However, accumulated geometric changes over time can determine whether or not structural displacements are excessive, and to what degree such displacements are indicative of structural distress.

Therefore it is recommended that site data collection exercises not necessarily be pre-scheduled, but rather they should be planned to be carried out to coincide with weather conditions which might be at approximate warm and cold extremes, if/when possible. In addition, special inspections should also be carried out after significant storm or environmental events. Observed damage or deformations discovered during those inspections should be referred for engineering assessment and repaired without delay.

Although once prisms are in place and any required laser targets are installed much of the site surveying can be carried out by a single technician, it is recommended that for safety reasons there should always be a second technician (or helper) on site during data collection activities in case of injury or exposure to unexpected environmental conditions. MODC should be always aware of when a data collection survey is underway at site.

For budgetary purposes, it is expected that once prisms are in place and an initial site control survey and the initial bridge spatial survey are completed a further eight to ten site visits for follow-ups through autumn, winter and spring may be required.

Some scope adjustment may be required during the execution of a monitoring program at the Gold River Multi-Use Bridge, therefore a budget price range estimate for establishing and carrying out that work has been derived and is expected to be about $75,000 (plus HST).

A basic cost breakdown for Monitoring Program work is as follows:

Cost of Prisms (allowance - 20 prisms)	$10,000
Prism Installations by High-Angle Team (allow two days)	$ 8,500
Initial Site Set-up, Establishment of Control Points (allowance)	$ 5,000
Initial TS Survey and 3D Laser Scan of Structure (allowance)	$10,000
Trips to Site for Collection and Assembly of Geometric Data (allow 10 trips)	$30,000
Structural Interpretation of Surveying Results (allowance)	$ 5,000
Overhead and Insurance Costs (allowance)	$ 6,500

4.2 BRIDGE REFURBISHMENT OPTIONS AND RELATED COST ESTIMATES

The following is a list of estimated costs and allowances for bridge repair/replacement and abandonment options.

- **Option 1**: Make all necessary repairs to the existing structure, including timber trestle approaches replacements, repairs to steel girder main spans, replacement of bearing assemblies, refurbishment of stone masonry piers. Estimated Cost: $3,100,000.
- **Option 2**: Replace entire structure at site (option to refurbish, reconfigure and maintain existing stone masonry piers. Estimated cost: $4,034,000.
- **Option 3**: Replace entire structure at a new site such as adjacent the Trunk Rte. No. 3 bridge crossing Gold River within the highway right of way (if possible). Estimated cost includes allowances for extending trails/bike lanes and or sidewalks from the existing trail to the new Gold River crossing location, an allowance for expropriations and costs which may be required to incorporate new trail components within the existing highway rights of way and an estimate of costs for abandoning and decommissioning the existing Gold River bridge. Estimated Cost: $3,500,000.
- **Option 4**: Abandon and decommission existing structure without replacement. Estimated Cost: $1,000,000.

A summary of work scopes and costs for Options 1 through 4 is as follows in sections 4.2.1, through 4.2.4:

4.2.1 OPTION 1 – MAKE ALL NECESSARY REPAIRS TO EXISTING BRIDGE

4.2.1.1 Replacement of East and West Timber Trestle Approaches:

- Disassemble, demolish, remove and dispose of two existing Timber Trestles Approach Structures
- Replace Timber Trestles Approach Structures with new marine grade treated timber and galvanized fastening components
- Replace deteriorated rail ties with like-size marine grade treated hemlock.
- Loose or damaged decking boards at main span and approaches to be re-fastened or replaced.

Approximate Expected Service Life Extension 25 years

Component Cost Estimate Allowance **$1,400,000**

4.2.1.2 Steel Plate Girders Repairs:

- Removal of deteriorated gusset plates and bracing and replacement with galvanized or otherwise protected components.
- Replacement of deteriorated portions of top and bottom flanges.

- **NO WELDING**. Due to probable high carbon content in the existing steel main span girders it is unlikely that existing steel is readily weldable. Therefore, all replacement components to be bolted with high-strength fasteners. Where steel is to be replaced, only bolted connections are to be permitted.
- Loose and remnant existing protective coating and grit accumulations to be mechanically removed (where practical) and collected, disposed of off-site to reduce moisture collection and reduce potential for further corrosion in those locations.
- **NO SAND or MEDIA BLASTING.** Sand or media blasting to be avoided for environmental protection purposes, also to avoiding a reduction in steel thickness and costs associated with proper removal and disposal.

Approximate Expected Service Life Extension 25 years

Component Construction Cost Estimate Allowance: **$350,000**

4.2.1.3 Bearings Assembly Replacements:

- Removal and reinstatement/repairs of bridge decking and former rail ties at bearings locations to allow access to tops of pier towers and bearing assemblies.
- Replacement of fixed steel bearings at shoreline masonry pier towers.
- Replacement of expansion sliding bearings at centre masonry pier tower with elastomeric bearing pads and steel plate components that will accommodate longitudinal movements of steel plate girders associated with thermal expansion and contraction of girder assemblies.
- Bearing seats at masonry piers will require re-anchorage and/or reconstruction.

Approximate Expected Service Life Extension 25 years

Component Construction Cost Estimate Allowance: **$250,000**

4.2.1.4 Stone Masonry Piers Repairs:

- A 100% repointing of the mortar joints. Including installation of vents and drain holes.
- All vegetation rooted in masonry joints to be removed.
- Re-install pier caps blocks to original position.
- Pier cap stones re-pointed and -sealed.
- Resetting dislodged granite and repairing fractured granite and joints. This may include the provision of cut granite inserts (dutchman pieces).
- Bearing seats to be repaired or reconstructed.
  - Since we do not have access to any original design drawings, the appropriate bearing seat repair methodology may not be known until the existing bearings components are disassembled and removed.
- It may be appropriate to grout the masonry pier cores, but more engineering is required before a final determination can be made on that repair alternative.
- Cleaning of the stones for aesthetic purposes (OPTIONAL).

Approximate Expected Service Life Extension 25 years

Component Construction Cost Allowance: $**1,100,000** (add $150,000 for cleaning masonry stone surfaces.)

4.2.1.5 Total Estimate of Costs OPTION 1 – Make All Necessary Repairs:

Approximate Expected Service Life Extension 25 years

Total Construction Cost Allowance for **OPTION 1: $3,100,000** (add $150,000 for cleaning masonry stone surfaces.)

4.2.2 OPTION 2 – REPLACE ENTIRE STRUCTURE AT EXISTING SITE

- Existing steel plate girders and bearings removed and disposed of $700,000, includes cost for new crane access road on east side of the river to access east span.
- Remove and dispose of existing timber trestle structures $100,000.
- Replacement bridge structure cost (4 spans) about $734,000 (fabrication only).
- Replacement bridge Installation and bearings $800,000.
- Stone masonry piers must be repaired or replaced as part of this option in order for new bridge spans to be installed (1,100,000).
- East and west stone masonry piers must be strengthened or enhanced to provide support for new spans which will replace timber trestle structure (allowance $400,000).
- Construct new bridge concrete abutments at each end of bridge to support new steel spans which will replace timber trestle structures. ($200,000).

Approximate Expected Service Life Extension: 35 years

Total Construction Cost Allowance for **OPTION 2: $4,034,000**

4.2.3 OPTION 3 – REMOVE FROM SERVICE AND DECOMMISSION EXISTING STRUCTURE AND PROVIDE REPLACEMENT STRUCTURE AT A NEW LOCATION

- Existing timber trestles, steel plate girders, bearings and masonry piers removed and disposed of $1,000,000, includes cost for new crane access road on east side of the river to access east span and approach.
- Replacement bridge structure cost (single span) (allowance $350,000 - fabrication only).
- Construct new bridge concrete abutments at each end of bridge to support new steel span ($250,000).
- Replacement bridge Installation (allowance $400,000.)
- Construct new bike path/sidewalks from existing trails along paved roadway right of way (allowance $500,000)
- Costs associated with land acquisition/agreement with provincial Highways Department for construction in or near the highway Right of Way. (allowance $500,000)
- Expropriation Costs (allowance $500,000).

Approximate Expected Service Life Extension: 35 years

Total Construction Cost Allowance for **OPTION 3: $3,500,000**

4.2.4 OPTION 4 – REMOVE FROM SERVICE AND DECOMMISSION (REMOVE) EXISTING STRUCTURE WITHOUT REPLACEMENT

- Construct equipment/crane access roads and provide siltation protection.
- Remove and dispose of entire existing bridge structure off site.
- Access road removal and environmental reinstatements at the water course.

Approximate Expected Service Life Extension: N/A

Approximate Construction Cost Estimate Allowance: **$1,000,000**

Prepared by:

Jamie Yates, P.Eng.
Sr. Civil Project Engineering Consultant

APPENDIX A

- TACTEN INDUSTRIAL INC. 2021 INSPECTION REPORT (Inc. PAINT SAMPLE ANALYSIS)
- MASONTECH INC. 2021 INSPECTION REPORT

Tacten
61 Raddall Avenue, Unit 0
Dartmouth, NS, Canada B3B 1T4
www.tacten.ca
Phone: 902.434.4405

Integrated Industrial Services

INSPECTION REPORT

CLIENT: Able Engineering
SECTION:

DATE: Aug 8, 2021
TACTEN JOB #: 801-10ACU004-J010575
REPORT #: VT-CD-J010575-R04
CONTRACT/PO: NA WO: NA
ATTENTION: **Neil McCallum**
WORK LOCATION: Chester Basin
PROJECT: Gold River Bridge Inspection
ITEM(S) EXAMINED: Timber Trestles, Steel Girder & Stone Piers

PART #: NA MATERIAL: NA THICKNESS: NA
SCOPE: See below
TYPE OF INSPECTION: Visual

TEST DETAILS:

ACCEPTANCE STANDARD: Client's Information REVISION: N/A
PROCEDURE/TECHNIQUE: Client's Information REVISION: N/A

METHOD:

EQUIPMENT TYPE: Camera MANUFACTURER: Nikon MODEL: Coolpix AW130 S/N: 50001778
LIGHT SOURCE: Flashlight/Ambient ILLUMINATION INTENSITY: >100 Foot-Candles
LIGHT METER S/N: CAL. DUE:
ADDITIONAL EQUIPMENT: NA MAGNIFICATION POWER: NA
SUPPLEMENTAL NDT REPORT ATTACHED?: NA PROCEDURE DEMONSTRATION REQUIRED?: NA
TEST SURFACE CONDITION: NA

SCOPE:

At the request Able Engineering, Tacten Industrial Inc. conducted a visual inspection on the Gold River Multi-Use Bridge between April 14th and 19th, 2021. All components that were not reasonably accessible from the ground were inspected within arms reach via rope access. The inspection took place at the direction of the Able Engineering on site representative and subject matter experts.

TECHNIQUES:

The inspection of the bridge was completed using rope access techniques in accordance with the Industrial Rope Access Trades Association (IRATA) International Code of Practice (ICOP) and Acuren ACU-ROPE procedures.

For referencing, the East side of the bridge was nearest Croft Rd and the North face of the bridge was the upstream side.

CLIENT: Neil McCallum
CLIENT PRINTED NAME
CLIENT SIGNATURE
ACCEPTED & ACKNOWLEDGED BY

DTR NO.: N/A

TACTEN TECHNICIAN: Brett Webster — 1st Technician; Nelson Seniuk — 2nd Technician

REVIEWER: Cory Dearman

(Generated Using: CAN-QUA-02F007 R09 - 02/26/2020)

TIMBER TRESTLES:

The East and West approach trestles consist of timber bents. The East approach bents were labelled 1-7 with bent 1 being nearest the abutment. The West approach bents were labelled 1-12 with 1 being nearest the abutment. The post numbers within each bent were labelled numerically with 1 being the Northmost post. Included in this section are the observations on the rail ties.

Generally, the treated timber members were found to be in fair to poor condition however, the following items were noted:

a) Several posts were found to have animal/insect holes and local decay. Photos 1-5.

b) Decay on many ends of the cross-bracing members. Photos 6-10.

c) Area of vandalism. Photos 11-12.

d) Several heavily corroded and deteriorating connectors. Photos 13-16

e) Some newer rail ties were heavily deteriorated. Photos 17-18

f) Two posts were spliced. Photos 22-23

STEEL PLATE GIRDERS:

Four steel plate girders were used to span between the stone masonry piers over the river. Each span was constructed of two 73' long girders. The spans were labelled East and West and the two girders in each span were labelled North and South.

The plate girders and bracing were in good condition overall. There was, however, at least a mild degree of surface corrosion found throughout the girders and bracing due to the failed coating throughout the steel. Heavier amounts of corrosion were found along the top flanges of the girders which has resulted in pack rust and deterioration of the angle bracing and gusset plates in these areas. Seriously effecting approximately 20% of the connections along the top flange. This is likely caused by moisture penetration through the rail ties.

Paint chips were removed from the girders and sent to a lab for metal analysis. See page 41 for results.

See photos 19-43

Thickness readings were taken periodically throughout the girder inspection. See below results for average readings:

CROSS MEMBER CONNECTING PLATE-CENTER	0.453”	PHOTO 29
CROSS MEMBER CONNECTING PLATE-CORNER	0.413”	PHOTO 30
CROSS MEMBER	0.374”	PHOTO 31
GIRDER WEB STIFFENERS	0.606”	PHOTO 32
GIRDER WEB	0.510”	PHOTO 33

PIERS:

Three stone bridge piers are used along this bridge, East, center, and West piers. The piers are constructed with 2' x 4' granite stones and mortared together. The interior core is unknown.

The main area of concern with the piers appears to be the condition of the mortared joints. The majority of the joints throughout all three piers have either completely deteriorated or are providing minimal function. Also, the pier cap joints were found to be in poor condition, allowing water infiltration into the interior of the pier. The center pier cap also showed signs of shifting. Other areas of note include vegetation growth throughout the piers and cracking found on multiple stones throughout the piers.

See photos 44-54

BEARINGS:

The plate bearings are located on all three piers and are anchored to the pier caps. The West and East piers of the bridge support the fixed ends of the girders and the center pier supports the expansion ends of the girders.

Heavy amounts of corrosion were found throughout all bearings. It appears that the amount of corrosion has hindered the ability for expansion and contraction at the center pier as no signs of recent bearing movement were evident.

See photos 55-58

RECOMMENDATIONS

TIMBER TRESTLES:

- If practical. area of vandalism should be cleaned of rot, treated with a wood preservative, and reinforced.
- All deteriorated and missing structural support and connector components are to be replaced as directed by Able. Prior to replacement, all existing bolt holes to remain should be cleaned of rot and treated with a wood preservative.

STEEL PLATE GIRDERS:

- Remnant protective coating should be removed.
- Deteriorated gusset plates and bracing should be removed and replaced with galvanized steel members.

PIERS:

- All mortar joints should be re-pointed.
- All vegetation should be removed from joints.
- Pier cap stones should be re-pointed and re-sealed.
- Re-install pier caps to original position

BEARINGS:

- Expansion sliding plates at center piers should be removed and replaced.

Photo 1:

East approach bent 6 post 4. Animal/insect damage.

Photo 2:

Close up of previous photo.

Photo 3:

East approach bent 6 post 6. Animal/insect damage.

Photo 4:

East approach bent 3 post 1. Animal/insect damage.

Photo 5:

East approach bent 6 post 3. Animal/insect damage.

Photo 6:

East approach bent 6 post 1. Bracing end decay.

Photo 7:

East approach bent 7 post 1. Bracing end decay.

Photo 8:

East approach bent 6 post 6. Bracing end decay.

Photo 9:

East approach bent 6 post 6. Bracing end decay.

Photo 10:

East approach bent 6 post 6. Bracing end decay.

Photo 11:

West approach bent 11 post 6. Vandalism.

Photo 12:

West approach bent 11 post 6. Vandalism.

Photo 13:

East approach North side. Heavily corroded and deteriorating nut.

Photo 14:

West approach bent 9 post 4. Missing nut.

Photo 15:

West approach bent 3 post 1. Missing nut.

Photo 16:

West approach bent 1 post 1. Loose rod.

Photo 17:

East approach North side near bent 4. Newer style rail tie is decaying.

Photo 18:

East approach North side near bent 4. Newer style rail tie is decaying.

Photo 19:

East approach South side. General cracking.

Photo 20:

East approach South side. General cracking.

Photo 21:

East approach bent 7 post 1. Decay at footing.

Photo 22:

Spliced post. West approach bent 6 post 2.

Photo 23:

Spliced post. East approach bent 6 post 3.

Photo 24:

Bottom flange West girder.

2.13”

Photo 25:

Top flange West girder.

2.32”

Photo 26:

Bottom flange West girder.

1.38”

Photo 27:

Bottom flange West girder.

2.21”

Photo 28:

2.14”

Photo 29:

Bottom flange West girder.

2.17”

Photo 30:

Top flange East girder.

2.01

Photo 31:

Bottom flange East girder.

2.14”

Photo 32:

Girder East span. Pack rust and deterioration at a top lateral bracing-girder gusset plate.

Photo 33:

Girder East span South side. Coating failure throughout.

Photo 34:

General photo looking through the West span.

Photo 35:

Girder West span. Pack rust and deterioration at a top lateral bracing-girder gusset plate.

Photo 36:

Girder West span. Heavy pitting along the bottom flange.

Photo 37:

General upper corner connection.

Photo 38:

General lower corner connection.

Photo 39:

Cross bracing center connection plate. Thickness reading location.

Photo 40:

Cross bracing corner connection plate. Thickness reading location.

Photo 41:

Cross bracing. Thickness reading location.

Photo 42:

Girder web stiffener thickness reading.

Photo 43:

Girder web thickness readings.

Photo 44:

East pier. Cracked granite stone.

Photo 45:

East Pier. Missing mortar 6" depth.

Photo 46:

East pier. General photo.

Photo 47:

Center pier South side. Cap has shifted.

Photo 48:

Center pier. Missing mortar throughout the pier cap.

Photo 49:

Center pier. Missing mortar. Tape measure inserted approx. 7”.

Photo 50:

Center pier. Cracked granite stone.

Photo 51:

Center pier. Cracked granite stone.

Photo 52:

Center pier. Vegetation growth throughout.

Photo 53:

West pier. Missing mortar at the pier cap.

Photo 54:

West pier. Missing mortar.

Photo 55:

East pier bearing. Heavy corrosion throughout.

Photo 56:

Center pier bearing. Heavy corrosion and no sign of expansion.

Photo 57:

Center pier bearing. Heavy corrosion and no sign of expansion.

Photo 58:

West pier bearing. Heavy corrosion throughout.

APPENDIX A

At the request of Able Engineering, Tacten Industrial Inc. conducted an additional visual inspection as well as ultrasonic thickness readings on the Gold River Multi-Use Bridge on June 4, 2021. The additional inspection was requested to obtain thickness information throughout the steel girders and associated components, quantify the components that require replacement and obtain bearing details. All steel girder components were inspected within arms reach via rope access. The inspection took place at the direction of the Able Engineering on site representative. All field photos and bearing details were submitted to Able outside of this report.

The majority of the members requiring replacement are along the top chord. Approximately 90-95% of the top chord lateral diagonal bracing connection plates had heavy amounts of pitting and corrosion (heaviest on the top side) throughout the plates. See photos 1-3 for reference. See page 38 for drawing mark up to detail the plate locations.

As well along the top chord, approximately 50% of the vertical cross bracing upper corner connection plates had heavy amounts of pitting and corrosion throughout the plates. See photos 4-5 for reference. See page 39 for drawing mark up to detail the plate locations.

Heavy amounts of pack rust were found on the bottom flange stiffener connection where the stiffener ties into the flange. The areas of note were located along the flat portion of the girders and on the internal sides of the girders. Approximately 40% of the connections were heavily effected. See photos 6-7 for reference. See page 40 for drawing mark up to detail the location.

All thickness readings taken on areas with multiple layers of plating, only the thickness of one outside plate was obtained and recorded on the member with chalk. The overall thickness was obtained using a caliper. All thickness readings and caliper measurements are recorded in the field photos sent separately to Able.

Photo 1:

Top chord lateral diagonal bracing connection plates. Pack rust on topside between plate and tie.

Photo 2:

Top chord lateral diagonal bracing connection plates. Heavy corrosion and missing rivets.

Photo 3:

Top chord lateral diagonal bracing connection plates. Pack rust on topside between plate and tie.

Photo 4:

Vertical cross bracing upper corner connection plates. Heavy amount of pack rust between members.

Photo 5:

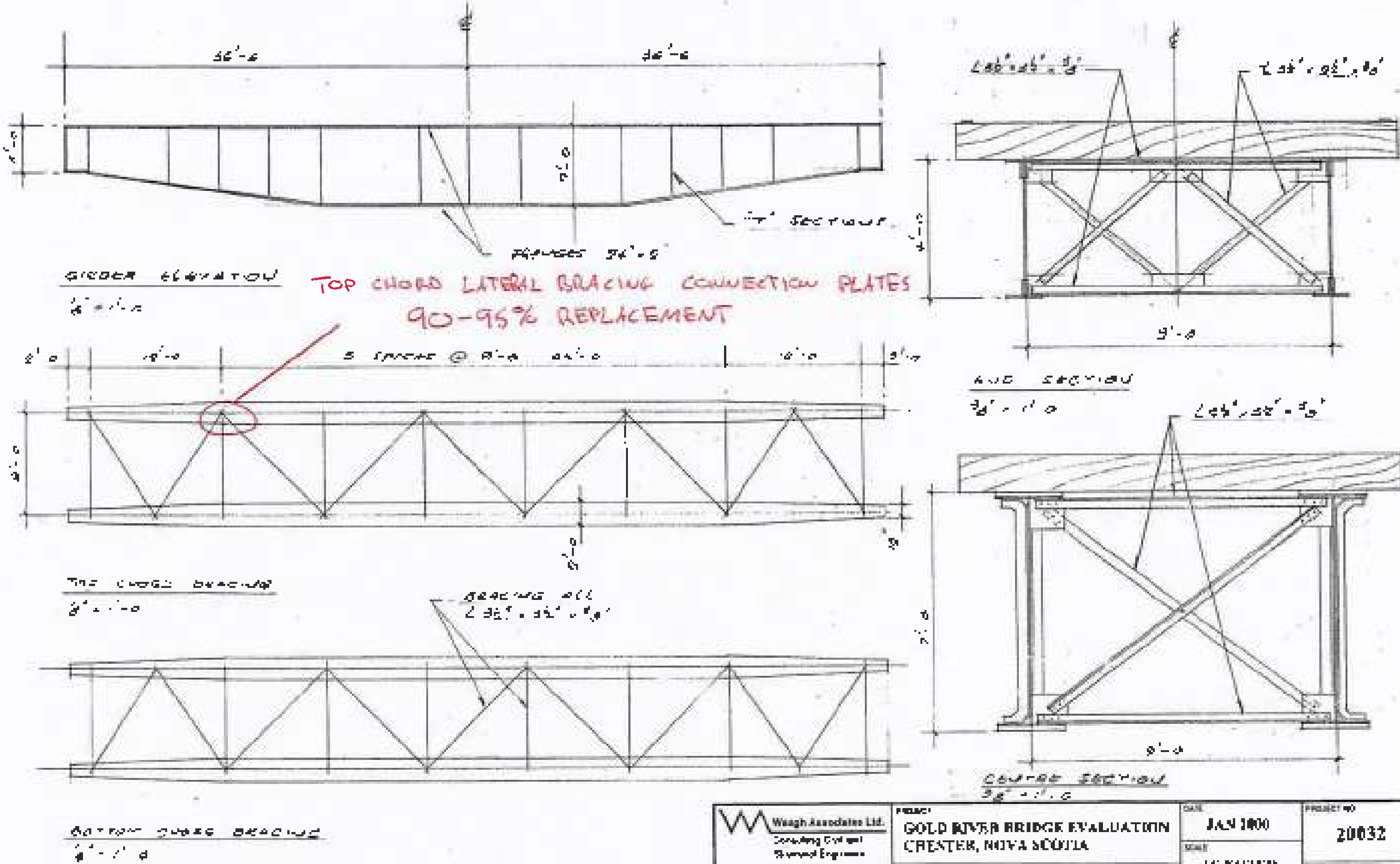
Vertical cross bracing upper corner connection plates. Heavy amount of pack rust between members.

Photo 6:

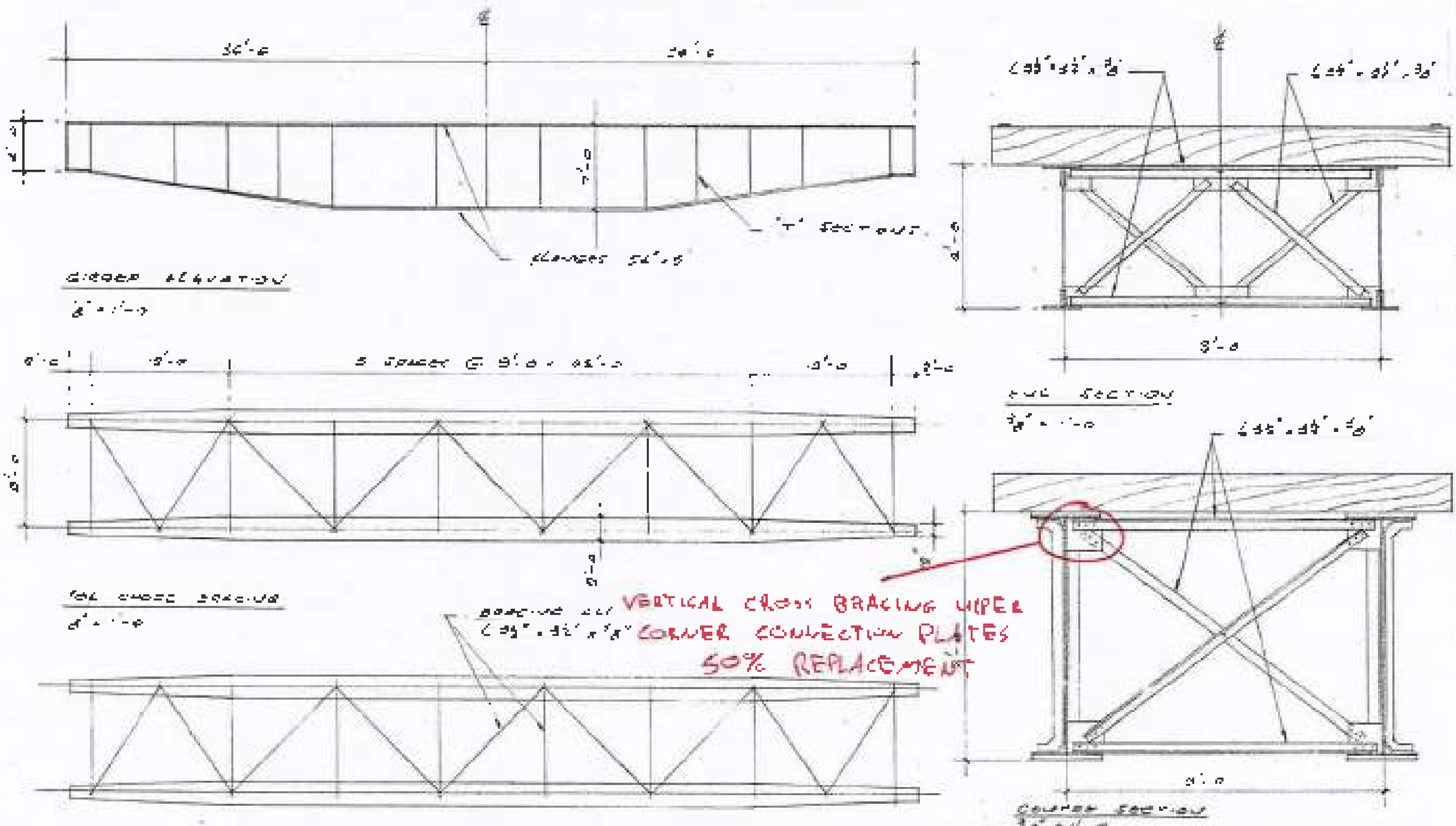
Bottom flange stiffener connection where the stiffener ties into the flange. Heavy amount of pack rust between members.

Photo 7:

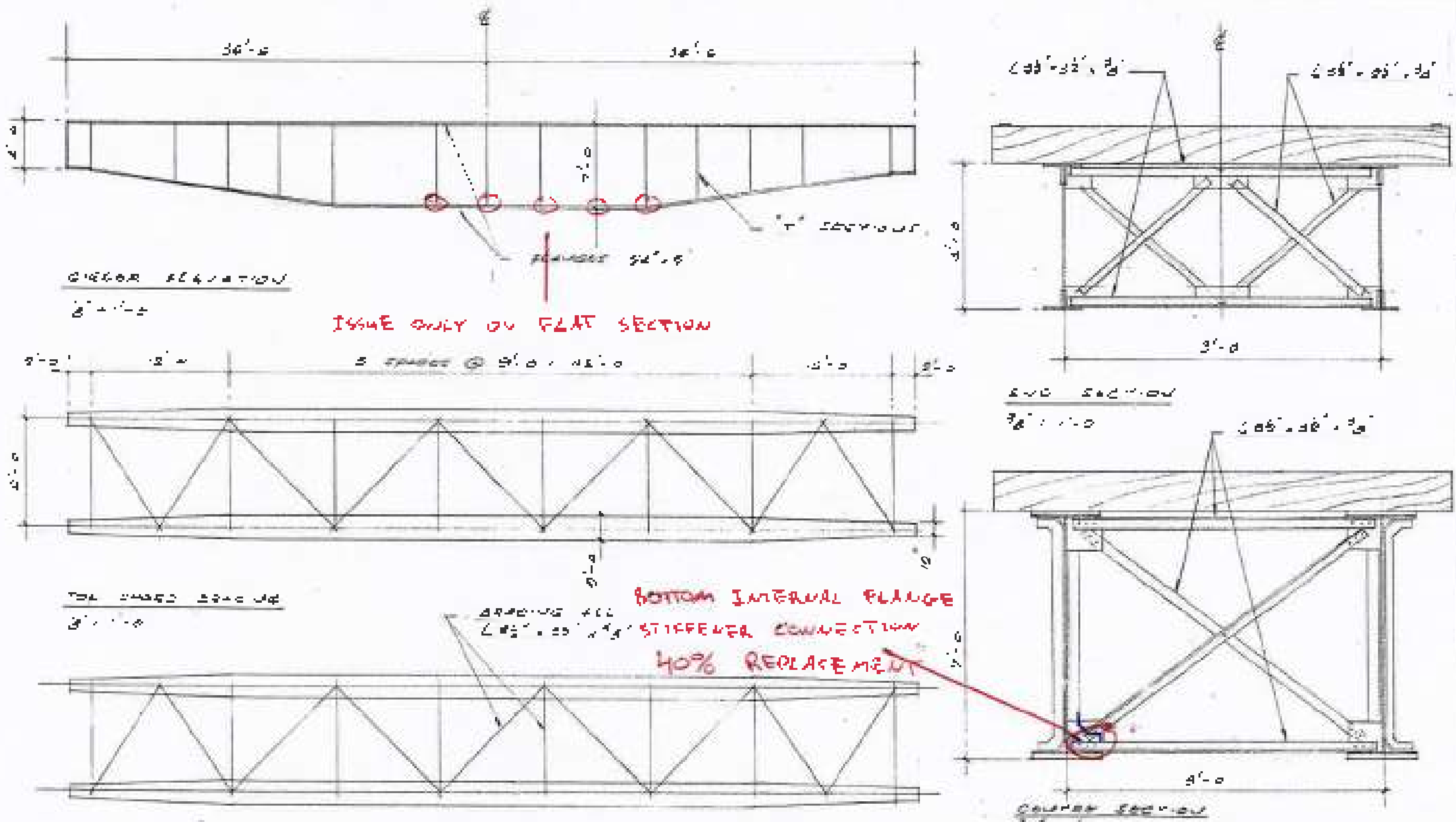
Bottom flange stiffener connection where the stiffener ties into the flange. Heavy amount of pack rust between members.



Waugh Associates Ltd. Consulting Civil and Structural Engineers	PROJECT: GOLD RIVER BRIDGE EVALUATION CHESTER, NOVA SCOTIA	DATE: JAN 2000	PROJECT NO: 20032
		SCALE: AS NOTED	
	SHEET TITLE: GIRDER ELEVATION, END AND CENTRE SECTIONS AND BRACING	DRAWN/CHECKED: WAW	SKETCH NO: SK-GRB-4



Waugh Associates Ltd. Consulting Civil and Structural Engineers	PROJECT: GOLD RIVER BRIDGE EVALUATION CHESTER, NOVA SCOTIA	DATE: JAN 2000	PROJECT NO: 20032
		SCALE: AS NOTED	
	SHEET TITLE: GIRDER ELEVATION, END AND CENTRE SECTIONS AND BRACING	DRAWN/CHECKED: WAW	SKETCH NO: SK-GRB-4



Waugh Associates Ltd. Consulting Civil and Structural Engineers	PROJECT: GOLD RIVER BRIDGE EVALUATION CHESTER, NOVA SCOTIA	DATE: JAN 2000	PROJECT NO: 20032
		SCALE: AS NOTED	
	SHEET TITLE: GIRDER ELEVATION, END AND CENTRE SECTIONS AND BRACING	DRAWN/CHECKED: WAW	SKETCH NO: SK-GRB-4

GOLD RIVER BRIDGE: MASONRY CONDITION REPORT

REPORT BY:
Mark Fougere
MASONTECH INC

May 20, 2021
ATTN: Able Engineering Services Inc

Gold River Bridge, Chester N.S.: Masonry Condition Report

Overview:

Further to your request for the above-mentioned property, a visual, non-intrusive, inspection was performed in April 2021.

The bridge was constructed near the beginning of the 20th century and was in use as a railway bridge until September 19, 1991.

The main focus was to determine the overall condition of the three masonry piers that support the bridge. These piers consist of rock-faced granite ashlar with bed heights of +/-26", entail measurements of +/-20" and varying lengths. The composition of the core is unknown but the piers are assumed to have originally been built by laying a full course of the granite ashlar and then infilling the cores using rubble and mortar.

During the visual inspection we were able to slide a measuring tape 7' deep in one of the open joints and up to 4' deep in another location. This evidence combined with age of the structure and lack of maintenance leads us to believe that the rubble core has likely been subject to extensive water infiltration and freeze/thaw cycles which, in turn, likely means that the core has lost any structural capacity that it may have once had.

It was also evident from the visual inspection that the vast majority of the mortar joints require repointing. Determining exact quantities for various depths of repointing is difficult but based on the visual inspection and some minor exploratory work (drilling into a few joints using a rotary hammer drill) we believe that, at a minimum, all mortar joints should be cut and repointed to depth of 5 inches.

As with the restoration/conservation of any historic structure, minimizing the extent of intervention is the guiding principle and, in this instance, we believe the scope of work should include:

- Repointing of any open or degraded mortar joints;
- Repair of fractured granite ashlar;
- Resetting of any dislodged granite;
- Installation of weep and vent holes;
- Caulking joints on top of the granite caps with a high-quality sealant.

Optional items that would address some aesthetic issues include:

- Cleaning of granite;
- Dutchmen repairs to broken granite.

Recommended Quantities & Methods for Scope of Work:

Repointing:

The total quantity of repointing is approximately 6,900 lineal feet which I believe should all be cut out and repointed to a depth of 5”. This repointing amounts to approximately one quarter of the overall depth of the stones and will help solidify the structure. In most exterior, above grade applications a Type “N” mortar (1:1:6 – Portland Cement:Lime:Sand) is usually adequate – however, due to the environmental exposure and high-moisture environment, we believe that it would be more prudent to use a Type “S” (2:1:9) (see attached data sheets). The Type “S” will be slightly more prone to cracking but it will also be more resistant to water infiltration, which we believe is the biggest factor to the degradation of the piers. There will also be many areas that will require deeper repointing but the quantities for these is quite difficult. For tendering purposes, I have created a unit rate table and have allotted various percentages of the 6,900 lineal feet total:

Depth	**Quantity**	**Percentage to Repoint**	**Total Quantity**
5”	6,900 lin.ft.	100%	6,900 lin.ft.
5”-7”	6,900 lin. ft.	50%	3,450 lin.ft.
7”-9”	6,900 lin.ft.	25%	1,725 lin.ft.
9”-11”	6,900 lin.ft.	~10%	862 lin.ft.
11”-13”	6,900 lin.ft	~5%	431 lin.ft.

Considering that the depth of the stones appears to be +/-20”, any repointing that is required beyond a depth of 7” would likely require some shoring of the stones to ensure that they do not shift after the old mortar has been removed. This could be achieved using small pieces of granite or a high-density plastic shim.

Ideally, any repointing beyond a depth of the 5” minimum would be achieved using non-pressurized grouting – however, considering that the composition of the core is unknown and that we have environmental concerns regarding the river, it is not likely a feasible approach. It may be useful to do some exploratory investigative work to try and determine if grouting could be an option as this would likely reduce costs for the stabilization of the piers.

Repair of Fractured Ashlar

While on site, it was noted, that there are a few pieces of granite that have vertical cracks that likely extend through the entire depth of the bed. The recommended intervention in these situations would typically be to remove the stone and insert stainless steel threaded rod and epoxy in a manner that would not be visible when the stone is reset in the wall. However, in this instance, you could pin them in situ by drilling diagonally through the face so that the drilled hole would span both pieces of the fractured stone. You would then clean the hole, pump in some epoxy (e.g., Epcon A7+ by Redhead – see attached data sheet) and insert the stainless-steel threaded rod. The hole in the face of the stone could either be repaired with a granite dutchmen or a repair mortar.

It appeared as though at least five pieces of granite were cracked but it was difficult to be sure of the exact total. We recommend that there be an allowance to repair ten stones.

Resetting of Dislodged Granite

It was observed that a few pieces of granite ashlar have been dislodged over the years – potentially from the vibrations caused by the trains or maybe due to water infiltration that could have froze and moved the stones.

They may or may not pose a structural issue but we believe it would be prudent to remove these from the wall and have them reset in proper alignment. This could also give you a chance to observe the conditions of the pier cores and determine if any further interventions could be required.

Installation of Weep and Vent Holes:

With many open joints and an unknown core structure that has been absorbing water for many years, it could be beneficial to install some weep and vent holes to allow the piers to breathe and hopefully reduce their moisture content to help minimize the damage caused by freeze/thaw cycles.

Something on the order of 36 holes per pier (10 on each large elevation and 8 on each small elevation) should help with air flow. In a perfect world you would remove and reset a granite unit adjacent to each hole, however, you could choose to simply drill some inclined holes through the joints (where the bed joints meet the perpendicular joints).

Caulking

The joints between the granite caps were covered in what appeared to be a mastic based product. With the extra exposure to the elements, we typically recommend that any horizontal joints be sealed using a good quality sealant (e.g., Dymonic 100 by Tremco – see attached data sheet).

Masonry Cleaning

Quite a bit of the granite could benefit from some cleaning, though it would only be for aesthetic purposes. Obviously, any type of chemical cleaner would be difficult to use due to the potential environmental exposure but you could just use a pressure washer. Another option would be a low pressure micro-abrasive system such as the “Rotec Vortex” cleaning system by Quintek. The Rotec Votex system is typically used when you are dealing with softer stone, such as sandstone, and would be a bit of overkill for this application, unless you are looking to get the stone very clean.

Please note that the budget in the unit rate table is for the Quintek system. Cost for pressure washing would be significantly less.

Dutcmen Repairs

There are a few corners that have fractured or broken off completely that could be repaired using dutchmen repairs. For tendering purposes, we have included for ten repairs and the unit rate provided would cover the cost for a 6”x6”x4” granite dutchmen. These repairs are both for aesthetics and for practical reasons – if you opt to not proceed with these repairs, you will need to fill them with mortar during the repointing and large areas like this would be more prone to cracking and premature failure.

Conclusion:

As with any masonry restoration project, the full extent of work and the methods to achieve that work can vary significantly once work has begun.

It would be prudent to carry a contingency fund to help cover any unforeseen issues. The size of this contingency is tough to pinpoint but something on the order of 20-30% wouldn’t be out of the question. Without having done any real exploratory work and not having been able to access the majority of the stonework, there is also a chance that the budgets we have provided could end up falling significantly short of what could actually be required to ensure the structural stability of the piers and bridge.

It should also be noted that this report is comprised of personal opinions and that we are not able to confirm that the interventions recommended will be adequate for the intended loads on the bridge.

Trust the above meets your approval. Should you have any questions or concerns please do not hesitate to contact me.

Cheers,

Mark Fougere
Masontech Inc.

MODC - Gold River Bridge [Masonry]

May 1, 2021

Unit Price Table

Item	Class of Labour, Plant or Material	Unit	Quantity	Price per Unit	Extended Amount
1	Scaffolding	lump	N/A	N/A	$ 250,000.00
2a	Repointing up to 5" depth	lin. ft.	6900	$ 40.00	$ 276,000.00
2b	Repointing from 5" to 7"	lin. ft.	3450	$ 52.00	$ 179,400.00
2c	Repointing from 7" to 9"	lin. ft.	1725	$ 67.60	$ 116,610.00
2d	Repointing from 9" to 11"	lin. ft.	862.5	$ 87.88	$ 75,796.50
2e	Repointing from 11" to 13"	lin. ft.	431.25	$ 114.24	$ 49,267.73
3	Repair Broken Granite	per stone	10	$ 200.00	$ 2,000.00
4	Reset Dislodged Granite	per stone	5	$ 2,500.00	$ 12,500.00
5	Caulking	lin. ft.	100	$ 20.00	$ 2,000.00
6	Dutchmen Repair (Granite)	ea	20	$ 500.00	$ 10,000.00
7	Masonry Cleaning	sq.ft.	9,900	$ 15.00	$ 148,500.00
8	General Conditions	lump	N/A	N/A	$ 100,000.00
				Total Extended Amount (TEA):	**$ 1,222,074.23**

TYPE S MORTAR
DIVISION 04

KING 2-1-9 GREY

All KING products are manufactured using ISO 9001:2008 Certified Processes

FEATURES & BENEFITS

- High compressive strength
- Superior adhesion
- Superior workability
- Good resistance to freeze-thaw cycles
- Self-healing property

USES

- Laying brick, natural stone or concrete blocks where greater compressive strength is required
- Plastering
- Repointing work where very high compressive strength is required (Contact your KING Technical Representative)

CAUTION

Colour variations on the hardened mortar can be observed even if the mortar in-place has been previously coloured in the factory and complies with the project specifications.

These colour variations are mainly attributed to various implementation conditions such as delay between mixing and tooling of the joints, lack of protection against the weather during implementation, or rate of absorption/humidity variability. In order to avoid an undesirable result, we recommend that you pay particular attention to these points.

KING 2-1-9 GREY is a pre-mixed, pre-bagged, Type S mortar specially formulated to be used laying brick, natural stone, concrete blocks and other masonry products, when a higher compressive strength is required. This mortar is a blend of grey Type GU Portland Cement, Type S hydrated lime, an air entraining agent, and sand with controlled grain size. KING 2-1-9 GREY mortar complies with Table 6 of CSA-A179-14 for Type S mortar with addition of water on-site. This product is grey in colour, but may be coloured in the factory or field using KING's exclusive **Colour Plus System**.

EXECUTION

- The application of the mortar must comply with the requirements of Sections 6 and 7 of CSA A371-14
- Never spread mortar on frozen surfaces

MIXING

Mix KING 2-1-9 GREY with a maximum of 5.0 L (1.3 US gallons) of potable water per 30 KG (66 lb) of mortar in a clean mortar mixer. Pour 4.5 L (1.2 US gallons) of water into the mixer and add 30 KG (66 lb) of KING 2-1-9 GREY mortar. Mix for 3 to 5 minutes, or 5 to 10 minutes when a colourant is added on-site. Allow the mortar to rest for a short period of time. Using the remaining water, adjust the mortar to obtain the desired consistency.

PLACEMENT OF MORTAR

The placement of the mortar must be done in the period of time stipulated in article 6.3.1. of CSA A179-14.

JOINT CLEANING

The tooling of joints exposed to rain is an important step that contributes to the waterproofing of the masonry system, and must be done using a jointer. The amount of water present in the mortar joint at the time of tooling will determinate the final colour of the cured mortar. To avoid colour variation, ensure that the mortar joint always contains the same amount of water when it is tooled. As a general rule, the joint is considered ready to be tooled when the mortar has hardened sufficiently, such that a fingerprint mark remains. Unless otherwise stated, a concave joint is preferred.

CLEANING

Using a little water, a piece of jute or a small piece of wood, make sure to remove as much splash or mortar stains as possible before the mortar has hardened to prevent the use of cleaning agents. If the use of cleaning products is necessary, be sure to contact the manufacturer of the product to validate the compatibility of the product and the procedure to follow.

Regardless of the technique, or product selected, it is essential to preserve the integrity of the mortar.

TYPE S MORTAR
DIVISION 04

KING 2-1-9 GREY

LIMITATIONS

- Do not use KING 2-1-9 when Type N mortar is specified. In this case, it is recommended to use KING 1-1-6
- Never add admixtures on-site to modify set time, handling or any other properties of the plastic or hardened mortar
- Use only the recommended amount of water to obtain the desired plastic or hardened properties

PACKAGING

This product is packaged in 30 KG (66 lb), triple-lined bags or bulk bags, wrapped on wooden pallets.

STORAGE AND SHELF LIFE

Always store in a dry area, protected from the weather. On-site, an additional tarpaulin must be used to cover the product to prevent water infiltration. Unopened, properly stored bags have a shelf life of 12 months.

SAFETY PROCEDURES

This product is made of Portland Cement. Wearing safety equipment used for the handling of cement-based products is therefore recommended: rubber gloves, dust mask and safety glasses. Safety Data Sheets can be provided upon request.

TECHNICAL DATA*

	REQUIREMENT OF CSA A179-14 STANDARD	AVERAGE VALUE OF KING 2-1-9 GREY
COMPRESSIVE STRENGTH		
ASTM C 109		
7 Days	7.5 MPa (1088 psi)	8.5 MPa (1233 psi)
28 Days	12.5 MPa (1813 psi)	15 MPa (2175 psi)
FLOW	110% +/- 5%	110% +/- 5%
AIR CONTENT		
CSA A 3004	18% Maximum	10%-12%
WATER RETENTION		
ASTM C 1506	70% Minimum	70%
VAPOUR TRANSMISSION		
ASTM E 96	N/A	15 Perms
WITHDRAWAL		
ASTM C 596 - 91 Day	N/A	0.119%
FREEZE-THAW RESISTANCE		
ASTM C 666M	N/A	Excellent after 100 cycles
YIELD PER 30 KG (66 LB) BAG	N/A	0.018 m^3 (0.65 ft^3) of fresh mortar

*All values required by the CSA A-179-14 Standard, as well as the average values of the KING product, are obtained under laboratory conditions. The average values of the KING product are applicable when the product is used as a bedding mortar; if the product is used as a repointing or parging mortar, the average values will be different.

Note: The contents of this Technical Data Sheet are updated regularly. To ensure that you have the most recent version, please visit our website at the following address: www.king-masonry.com

V0119

KING PACKAGED MATERIALS COMPANY

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Brantford, ON N3T 5L8
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Fax: 519-756-7490

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Dowling, ON P0M 1R0
Phone: 705-855-1155
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WRITERS' PROFESSIONAL LIABILITY

This document is published by *KING – A SIKA Co.* It contains information for the sole purpose of helping you make informed decisions. <u>It is not our intention and we cannot assume in anyway</u> the role and professional liability of the architect who executed, signed and sealed these plans and specifications.

As such, this document was diligently drafted by experienced professionals and therefore must not be copied integrally; rather you must adapt or even modify it according to your project, which our technical representatives and engineering service would be more than happy to help you with.

Additional information available:

Although this document covers a wide variety of applications, we also invite you to refer to our electronic catalog of recommendations. Thus, following a proposal for use specific to your project, we will recommend one or more products. The electronic catalog and all of the technical sheets for our products can be found on our website at the following link: <u>www.king-masonry.com</u>

PART 1 – GENERAL

1.1 REFERENCES

.1 CSA Standards
.1 CSA A-179 Mortar and grout for unit masonry
.2 CSA A-371 Masonry for buildings

.2 ASTM Standards
.1 ASTM C 207 Standard Specification for Hydrated Lime for Masonry Purposes
.2 ASTM C 270 Standard Specification for Mortar for Unit Masonry
.3 ASTM C 979 Standard Specification for Pigments for Integrally Colored Concrete

.3 National Building Code (Quebec)
.1 Section 9.20 (Load-bearing and non-load-bearing masonry)
.2 Sections 9.20 and 9.22 (chimney and fireplace)

1.2 DOCUMENTS/SAMPLES/INFORMATION TO SUBMIT FOR APPROVAL

.1 Submit the required technical data sheets and the samples conforming to section 01 33 00 –Documents and samples to submit.

.2 Submit 3 samples of each mortar used by presenting them in the U-shaped plastic extrusions measuring 10 mm X 10 mm X 100 mm in length. The samples must be correctly identified.

.3 Submit the technical data sheet of each mortar or grout used. The technical data sheet must include the product's characteristics, performance criteria and limits.

.4 Submit two copies of the material safety data sheet of each mortar or grout used.

.5 No requests for equivalency will be accepted after the bid closing date.

1.3 HANDLING AND STORAGE

.1 The bags of mortar and grout must be delivered in their original packaging with the legible identification of the manufacture

.2 The mortar and grout product bags must be stored on wooden pallets and protected against inclement weather.

1.4 WALL Mock-up

.1 Erect a wall mock-up with a minimum height and length of 1000 mm X 1000 mm.

.2 Erect a wall mock-up for each mortar and grout specified.

.3 The wall mock-up should display what the final colour and texture of the joint will look like.

.4 The wall mock-up must form an integral part of the works.

.5 Do not start work until the wall mock-up has been approved by the professional in charge of the project.

1.5 PLACEMENT CONDITIONS

.1 Cold weather placement during construction:

.1 -4°C to 4°C: The mortar shall have a minimum temperature of 4°C and a maximum, temperature of 50°C.

.2 -7°C to -4°C:

1.5.2.1 The mortar shall have a minimum temperature of 4°C and a maximum, temperature of 50°C.

1.5.2.2 Source heat shall be provided on both sides of the walls

1.5.2.3 Windbreaks shall be employed when the wind speed exceeds 25 km/h

.3 -7°C and below:

1.5.3.1 The mortar shall have a minimum temperature of 4°C and a maximum, temperature of 50°C.

1.5.3.2 Enclosures and supplementary heat shall be provided to maintain an air temperature above 0°C

.2 Cold weather protection for completed masonry or section not in progress

.1 0°C to 4°C: Masonry shall be protected from rain or snow for 48 hours

.2 -4°C to 0°C: Masonry should be completely covered for 48 hours

.3 -7°C to -4°C: Masonry shall be completely covered with insulating blankets for 48 hours

.4 -7°C and below: The masonry temperature shall be maintained above 0°C for 48 hours by enclosure and supplementary heat.

.3 Hot weather placement:

.1 Cover the works with a waterproof tarpaulin to prevent them from drying too quickly. Make sure to use a tarpaulin that does not stain.

.2 Never wet the masonry units, unless otherwise indicated by the professional in charge of the project.

1.6 PROTECTIVE MEASURES

.1 Unfinished masonry works must be wrapped with waterproof tarpaulins that do not stain. The tarpaulins must cover the walls and extend them by 600 mm on each side to protect the works against gusts of rain caused by wind.

.2 Finished masonry works must be protected from mortar spatter by covering them with stain-free tarpaulins or polyethylene.

.3 Protect the windows, frames, doors and sills from spatter or other damaging elements.

PART 2 – PRODUCTS

2.1 MATERIALS

.1 Mortar and grout materials must be provided by the same supplier.

.2 All mortar and grout must be manufactured in a plant where processes are certified ISO 9001:2008.

.3 Portland Type GU Cement, conforming to standard CSA A-3000.

.4 Hydrated lime Type “S”, conforming to standard ASTM C-207.

.5 Sand: Fine-grain sand particle size conforming to table 1 of standard CSA A-179.

.6 Water: Only use clean potable water free of harmful substances such as oils, acids, salts and organic matter.

.7 Pigments: The percentage of pigments should not exceed 10% of the binder density.

.8 It is strictly prohibited to use any type of additive to alter the setting time, workability or any other property of the plastic or cured mortar.

2.2 MORTARS

.1 All mortars described hereafter are manufactured by the company «KING – A SIKA Co.».

.2 Each type of mortar must be factory pre-blended with Portland cement, lime, sand and colouring agents, and then mixed with water at the construction site according to the manufacturer's instructions.

If pigments needs to be add on site, use only the **Colour Plus System** exclusive to « KING – A SIKA Co.»

.3 Mortar for exterior masonry work, above ground level.

.1 Mortar for load-bearing walls: As a minimum, use a Type "S" mortar such as KING 2-1-9, prepared according to the batching specifications

.2 Mortar for non-load-bearing walls: As a minimum, use a Type "N" mortar like KING 1-1-6, prepared according to the batching specifications

.3 Mortar used in the case of parapets and masonry exposed to a high level of saturation such as chimneys and self-supporting exterior walls: Use a Type "S" mortar, prepared according to dosage specifications such as KING 2-1 -9 mortar.

.4 Mortar used for laying bricks and glass blocks: Use a Type "S" mortar with waterproofing agent, such as MasonGlass mortar.

.4 Mortar for exterior masonry work at ground level or below.

.1 Mortar used for foundation walls, retaining walls, manholes, sewers, pavements, aisles and patios: Minimally use a Type "S" mortar such as the KING BLOCK or a mortar prepared according to the specifications relating to the dosage, such as KING 2-1-9 mortar.

.5 Mortar for interior masonry works

.1 Mortar for load-bearing walls: As a minimum, use a Type "S" mortar like KING Block or KING 2-1-9.

.2 Mortars for non-load-bearing walls requiring low compressive strength resistances or non-load-bearing walls: Minimally use a Type "N" mortar prepared according to the dosage specifications such as KING 1-1-6.

.3 Mortar used when laying glass blocks: Use a Type “S” mortar with waterproofing agent, such as MasonGlass mortar or a Type “N” mortar, prepared according to dosage specifications such as KING 1-1-6 mortar.

2.3 GROUTS

.1 All grouts described hereafter are manufactured by the company «KING – A SIKA Co.».

.2 It is strictly prohibited to use mortar as grout.

.3 Each type of grout must be factory pre-blended with the raw materials, and then mixed with water on the construction site according to the manufacturer’s instructions.

.4 The grouts must conform to table 7 of standard CSA A179.

.5 Grout should be an expansive type. Expansion shall be less than 2%.

.6 Unless otherwise indicated, to fill the cells of the block, use a grout with 15 MPa at 28 days, such as KING CellFiller E-15.

PART 3 – EXECUTION

3.1 MIXING

Important: In order to avoid segregation issues, always mix the total content of one bag. If less than 30 kg is required, dry mix - without water – the total contents of the bag in a clean container, take the required amount, and then add water to the amount withdrawn from the mixture.

.1 Always use a clean mixer for each type of mortar and colour.

.2 Conformity: Comply with the requirements, recommendations and specifications on the manufacturer’s technical data sheet.

3.2 PLACEMENT

.1 Unless otherwise indicated by the architect, place the masonry mortar and grout in compliance with standards CSA A-179 and CSA A-371.

3.3 JOINTS

.1 Unless otherwise indicated by the architect, the joints must be 10-mm thick.

.2 The joints must be smoothed so that they have a concave profile.

3.4 PLACEMENT TIMEFRAME FOR MORTAR AND GROUT

.1 Mortar

.1 If room temperature is equal to or greater than 25°C, mortar must be placed in under 1.5 hours after mixing. If room temperature is less than 25°C, mortar must be placed in under 2.5 hours after mixing.

.2 Grout

.1 Expansive grout must be placed at the latest 20 minutes after mixing.
Regular grout must be placed in under 1.5 hours after mixing.

3.5 REMIXING

.1 Remixing is a criteria of placing mortar and grout. It is done to ensure the necessary workability.

.2 Once the desired consistency is obtained, it is not recommended to add water to the coloured mortars in order to compensate for the loss of water caused by evaporation. Adding water could affect the final colour of the product.

3.6 COLOUR UNIFORMITY

.1 In order to ensure colour uniformity of the mortar, the contractor must:

.1 Use the same supplier for all mortar and grout.

.2 Once the desired consistency is obtained, it is not recommended to add water to the coloured mortars in order to compensate for the loss of water caused by evaporation. Adding water could affect the final colour of the product.

.3 Process of tooling joints when the mortar has hardened sufficiently such that a fingerprint mark remains

.4 Ensure that the quantity of water in the mortar joints remains the same while smoothing them.

.5 Always use a clean water container

.6 Always use a clean mixer.

3.7 CLEANING

.1 Once finished the work, remove the excess mortar using a wooden blade. Once the mortar has sufficiently cured, the contractor must:

.1 Moisten the wall surface with clean water, starting from the bottom.

.2 Scour the wall surface using water and a brush with nylon bristles.

.3 Do NOT use any form of acid, unless otherwise indicated by the professional in charge of the project.

.4 If the use of cleaning product is necessary, contact the product manufacturer to validate the compatibility of the product and the procedure to follow. If the colour ONYX is used, be sure to mention to the cleaning product manufacturer that the mortar contains Carbon Oxides pigments. Generally used cleaning agents are not compatible with Carbon Oxides. Apart from colour Onyx, all KING – A SIKA Co. coloured mortars contain iron or titanium oxides.

.5 Regardless of the technique or product selected, it is essential to preserve the integrity of the mortar.

.6 Proceed with a witness section of 2000 mm high X 2000 mm long minimum.

.7 Wait for approval of the cleaning control zone by the professional in charge of the project before proceeding with the entire building.

END OF SECTION

RED HEAD®

Technical Datasheet - A7+ Adhesive Anchor

Description: Quick-Cure Adhesive Anchor for Concrete and Masonry Applications

Product Description

- High-Strength, quick-cure structural concrete and masonry adhesive anchoring system,

Adhesive Type

2-part injectable hybrid epoxy (10:1 ratio)

Cartridge Types & Sizes

Durable and re-sealable cartridges available in 3 sizes:
9.5oz coaxial cartridge (standard caulking tube)
28oz. dual cartridge
5oz coaxial cartridge

Approvals

ICC-ES ESR 3903 (Concrete Report)

ICC-ES ESR 3951 (Masonry Report)

2015, 2012, 2009, 2006 International Building Code (IBC) Compliant

Florida Building Code (FBC)

City of Los Angeles (COLA)

Extensive Department of Transportation (DOT) Listings (visit itwredhead.com for more info)

NSF/ANSI 61 Approval for use in Drinking Water System Components

ASTM C881, Types I, II, IV, and V, Grade 3, Classes A, B, & C (meets Type III except elongation)

Anchor Sizes & Types

Threaded Rod: ¼" – 1-1/2"

Rebar: #3-#11 Rebar

Load Types

Suitable for use in applications subject to short- and long-term sustained loads, including static, seismic and wind loads in tension or shear.

Water Resistance

100% hydrophobic, suitable for use in saturated concrete and water-filled or submerged holes

Hole Orientation

Suitable for use with vertical down, horizontal and overhead anchors

Hole Size

1/16" to 1/8" larger than diameter of rod / rebar, contact technical support at (800) 848-5611 for more detail

Drill Types

Hammer or standard rotary drill using carbide drill bits. For instructions for use with diamond core drills, call Technical Service at (800) 848-5611

In-Service Temperatures

-41° through 176°F (-41° through 80°C)

Working (Gel Time)

5 minutes at 70°F (21°C)

Full Cure Time

45 minutes at 70°F (21°C)

Adhesive Color

Gray when properly mixed

Storage Life & Temperature

18 months from date of manufacture when stored in 32° through 95°F (0° through 35°C)

Country of Origin

Made in France (28oz & 5oz Kits Packaged in the US)

For additional information, please visit www.itwredhead.com

RED HEAD®
ADHESIVE ANCHORING SPECIALISTS

A7+

The Most Versatile Quick Cure Adhesive

A7P-10 A7P-28

APPLICATIONS / USES

- Concrete dowelling (slabs, walls, columns)
- Steel framing (columns, beams, ledgers)
- Brick pinning and CMU reinforcement
- Architectural metal fastening (railings, signage)
- Mechanical, electrical, and plumbing attachment
- Vibratory equipment anchoring
- Overhead and horizontal anchors

DESCRIPTION

Quick Curing Hybrid Epoxy Adhesive

RED HEAD A7+ is a high-strength, fast-cure adhesive that is designed to securely anchor threaded rod and rebar to cured concrete and masonry. A7+ is one of the most versatile achoring solutions on the market, suitable for use in an extremely wide range of applications and environmental conditions.

- The only quick-cure ICC-ES listed for use in all wet conditions
- Qualified for use in concrete, block, brick, and clay tile. Solid or hollow base materials
- Cures in only 45 minutes (at substrate temperature of 70°F/21°C)
- ICC-ES listed for cracked concrete and seismic applications (ICC-ES ESR 3903)
- ICC-ES listed for masonry applications (ICC-ES ESR 3951)
- No drip formula that allows direct-injection overhead installation
- Low odor - suitable for use indoors and in occupied buildings
- 18-month storage life minimizes waste and risk of using expired product
- Rugged cartridge resists breakage due to rough handling or cold temperatures
- Store between 32ºF and 95ºF in a cool, dry place.

ADVANTAGES

- All weather formula
- Works in damp holes and underwater applications
- Fast curing time, 45 minutes at 70°F
- ICC-ES Evaluation Report ESR-3903 (Concrete) and ESR-3951 (Masonry)
- NSF 61 Listed, certified for use in conjunction with drinking water systems
- Fast & easy dispensing, even 28 ounce cartridge can be hand dispensed
- Formula for use in solid and hollow base materials

Curing Times

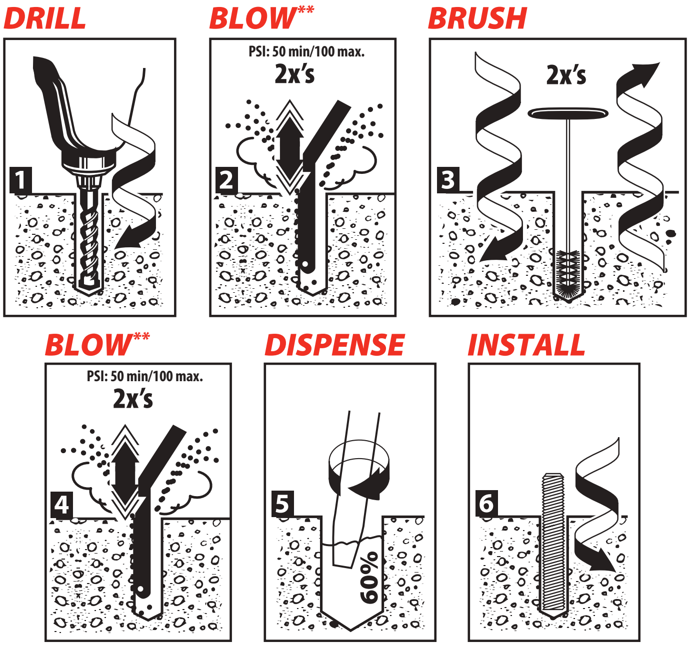
CONCRETE (F°)	CONCRETE (C°)	ADHESIVE (F°)	ADHESIVE (C°)	GEL TIME	FULL CURE TIME
110	43	110	43	1.5 minutes	45 minutes
90	32	90	32	3 minutes	45 minutes
70	21	70	21	5 minutes	45 minutes
50	10	50	10	15 minutes	90 minutes
32	0	32	0	35 minutes	4 hours
14	-10	32	0	35 minutes	24 hours

Most Competitive Spacing and Edge Distance

NOMINAL ANCHOR DIAMETER (IN.)	MINIMUM SPACING (IN.)	MINIMUM EDGE DISTANCE (IN.)
3/8	15/16	15/16
1/2	1-1/2	1-1/2
5/8	2-1/2	2-1/2
3/4	3	3
7/8	3-1/2	3-1/2
1	4	4
1-1/4	5	5

A7+

INSTALLATION STEPS



* Damp, submerged and underwater applications require 4x's air, 4x's brushing and 4x's air

** Dust is shown for diagram purposes only. To help mitigate airborne dust and comply with OSHA requirements, we recommend that you either wet the concrete before blowing out the hole, or use a drill dust extractor with your pneumatic air nozzle. We recommend vacuum assisted dust extractors like Milwaukee part numbers 5261-DE or 5317-DE. Call our technical services at (800) 848-5611 for more information."

APPROVALS/LISTINGS

ICC-ES ESR 3903 (Concrete Report)

ICC-ES ESR 3951 (Masonry Report)

2018, 2015, 2012, 2009, 2006 International Building Code (IBC) Compliant

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City of Los Angeles (COLA)

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ASTM C881, Types I, II, IV, and V, Grade 3, Classes A, B, & C (meets Type III except elongation)

For most current approvals and listings please visit: www.itwredhead.com

APPLICATIONS

Water Treatment Facilities

The best-in-class in edge and spacing distance of Red Head A7+ and its ability to work in water have make it a great fit for waste water treatment plants.

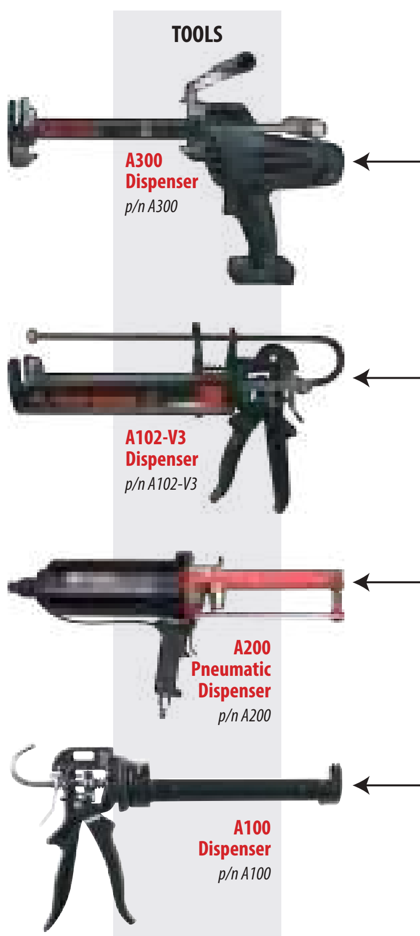
Roadway Doweling

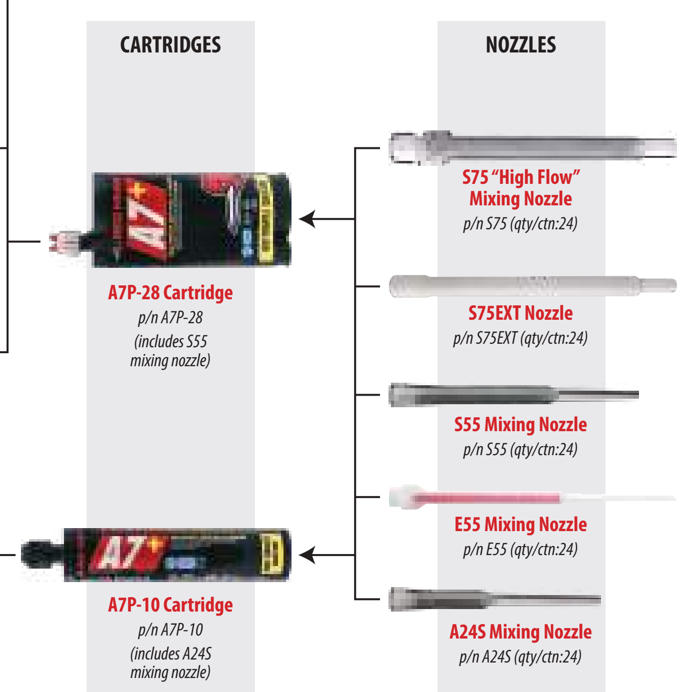
A7+ dispenses so quickly and rebar inserts so easily that contractors find installed costs are lower than many other products including grout for doweling.

PACKAGING

1. Disposable, self-contained cartridge system capable of dispensing both components in the proper mixing ratio
2. Acrylic components dispensed through a static mixing nozzle that thoroughly mixes the material and places the material at the base of the pre-drilled hole
3. Cartridge markings: Include manufacturer's name, batch number and best-used-by date, mix ratio by volume, ANSI hazard classification, and appropriate ANSI handling precautions

All Red Head nozzles can be used with all A7+ cartridges. Choose the nozzle that is right for you depending on hole depth and dispensing needs.





A7P–10 fl. oz. Ordering Information

PART NUMBER	DESCRIPTION	BOX QTY
 A7P-10	9.5 Fluid Ounce Cartridge with A24S Nozzle	6
 A24S	Mixing Nozzle for A7P-10 Cartridge Nozzle diameter fits 3/8" to 5/8" holes (overall length of nozzle 6-3/8")	24
 A100	Hand Dispenser Designed for A7P-10 Cartridge Contractor Quality 26:1 Thrust Ratio	1

ESTIMATING TABLES

A7+ 9.5 Fluid Ounce Cartridge

Number of Anchoring Installations per Cartridge using Threaded Rod with A7+ in Solid Concrete*

ROD (In.)	DRILL HOLE DIA. INCHES	EMBEDMENT DEPTH IN INCHES 1	2	3	4	5	6	7	8	9	10
1/4	5/16	371.3	185.6	123.8	92.8	74.3	61.9	53.0	46.4	41.3	37.1
3/8	7/16	189.4	94.7	63.1	47.4	37.9	31.6	27.1	23.7	21.0	18.9
1/2	9/16	114.6	57.3	38.2	28.6	22.9	19.1	16.4	14.3	12.7	11.5
5/8	3/4	64.5	32.2	21.5	16.1	12.9	10.7	9.2	8.1	7.2	6.4
3/4	7/8	47.4	23.7	15.8	11.8	9.5	7.9	6.8	5.9	5.3	4.7
7/8	1	36.3	18.1	12.1	9.1	7.3	6.0	5.2	4.5	4.0	3.6
1	1-1/8	28.6	14.3	9.5	7.2	5.7	4.8	4.1	3.6	3.2	2.9
1-1/4	1-3/8	19.2	9.6	6.4	4.8	3.8	3.2	2.7	2.4	2.1	1.9
1-1/2	1-5/8	13.7	6.9	4.6	3.4	2.7	2.3	2.0	1.7	1.5	1.4

*The estimated number of anchoring installations per cartridge is based upon calculations of filling the hole 60% full of adhesive per the recommendation in our installation instructions. Hole volumes are calculated using ANSI tolerance carbide tipped drill bits. These estimates do not account for any waste.

ESTIMATING TABLE

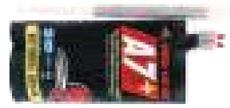
A7+ 9.5 Fluid Ounce Cartridge

Number of Anchoring Installations per Cartridge using Rebar with A7+ in Solid Concrete*

REBAR	DRILL HOLE DIA. INCHES	EMBEDMENT DEPTH IN INCHES 1	2	3	4	5	6	7	8	9	10
#3	7/16	189.4	94.7	63.1	47.4	37.9	31.6	27.1	23.7	21.0	18.9
#4	5/8	92.8	46.4	30.9	23.2	18.6	15.5	13.3	11.6	10.3	9.3
#5	3/4	64.5	32.2	21.5	16.1	12.9	10.7	9.2	8.1	7.2	6.4
#6	7/8	47.4	23.7	15.8	11.8	9.5	7.9	6.8	5.9	5.3	4.7
#7	1	36.3	18.1	12.1	9.1	7.3	6.0	5.2	4.5	4.0	3.6
#8	1-1/8	28.6	14.3	9.5	7.2	5.7	4.8	4.1	3.6	3.2	2.9
#9	1-1/4	23.2	11.6	7.7	5.8	4.6	3.9	3.3	2.9	2.6	2.3
#10	1-1/2	16.1	8.1	5.4	4.0	3.2	2.7	2.3	2.0	1.8	1.6
#11	1-3/4	11.8	5.9	3.9	3.0	2.4	2.0	1.7	1.5	1.3	1.2

*The estimated number of anchoring installations per cartridge is based upon calculations of filling the hole 60% full of adhesive per the recommendation in our installation instructions. Hole volumes are calculated using ANSI tolerance carbide tipped drill bits. These estimates do not account for any waste.

A7P–28 fl. oz. Ordering Information

PART NUMBER	DESCRIPTION	BOX QTY
 A7P-28	28 Fluid Ounce Cartridge A7+ Each catirdge comes with a S55 Nozzle	4
E55	Mixing Nozzle for A7P-28 and G5-22 Cartridge Nozzle diameter fits 3/8" to 5/8" holes. (overall length of nozzle 14")	24
 A102-V3	Heavy-Duty 34:1 thrust ratio hand dispenser for A7P-28 cartridge	1

PART NUMBER	DESCRIPTION	BOX QTY
 S55	Mixing Nozzle for A7P-28 Cartridge Nozzle diameter fits holes for 3/8" diameter & larger anchors (overall length of nozzle 10")	6
 A200	Pneumatic Dispenser for A7P-28 Cartridge	1
E25-6	6-Foot Straight Tubing (Used when holes are deeper) (can cut to proper size) (.39 in I.D. x .43 in. O.D.)	24
 A300	Cordless Battery Dispenser for A7P-28, C6P-30 and G5P-30 Cartridge. Includes one battery and charger. Works with all Milwaukee® M18™ batteries	1

*See page 65 for nozzle extension tubes and other accessories

ESTIMATING TABLE

A7+
28 Fluid Ounce Cartridge

Number of Anchoring Installations per Cartridge using Threaded Rod with A7+ in Solid Concrete*

Rod (in.)	DRILL HOLE DIA. INCHES	EMBEDMENT DEPTH IN INCHES 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1/4	5/16	1094.0	547.0	364.7	273.5	218.8	182.3	156.3	136.7	121.6	109.4	99.5	91.2	84.2	78.1	72.9
3/8	7/16	558.2	279.1	186.1	139.5	111.6	93.0	79.7	69.8	62.0	55.8	50.7	46.5	42.9	39.9	37.2
1/2	9/16	337.7	168.8	112.6	84.4	67.5	56.3	48.2	42.2	37.5	33.8	30.7	28.1	26.0	24.1	22.5
5/8	3/4	189.9	95.0	63.3	47.5	38.0	31.7	27.1	23.7	21.1	19.0	17.3	15.8	14.6	13.6	12.7
3/4	7/8	139.5	69.8	46.5	34.9	27.9	23.3	19.9	17.4	15.5	14.0	12.7	11.6	10.7	10.0	9.3
7/8	1	106.8	53.4	35.6	26.7	21.4	17.8	15.3	13.4	11.9	10.7	9.7	8.9	8.2	7.6	7.1
1	1-1/8	84.4	42.2	28.1	21.1	16.9	14.1	12.1	10.6	9.4	8.4	7.7	7.0	6.5	6.0	5.6
1-1/4	1-3/8	56.5	28.3	18.8	14.1	11.3	9.4	8.1	7.1	6.3	5.7	5.1	4.7	4.3	4.0	3.8
1-1/2	1-5/8	40.5	20.2	13.5	10.1	8.1	6.7	5.8	5.1	4.5	4.0	3.7	3.4	3.1	2.9	2.7

*The estimated number of anchoring installations per cartridge is based upon calculations of filling the hole 60% full of adhesive per the recommendation in our installation instructions. Hole volumes are calculated using ANSI tolerance carbide tipped drill bits. These estimates do not account for any waste.

ESTIMATING TABLE

A7+
28 Fluid Ounce Cartridge

Number of Anchoring Installations per Cartridge using Rebar with A7+ in Solid Concrete*

REBAR	DRILL HOLE DIA. INCHES	EMBEDMENT DEPTH IN INCHES 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
#3	7/16	558.2	279.1	186.1	139.5	111.6	93.0	79.7	69.8	62.0	55.8	50.7	46.5	42.9	39.9	37.2
#4	5/8	273.5	136.7	91.2	68.4	54.7	45.6	39.1	34.2	30.4	27.3	24.9	22.8	21.0	19.5	18.2
#5	3/4	189.9	95.0	63.3	47.5	38.0	31.7	27.1	23.7	21.1	19.0	17.3	15.8	14.6	13.6	12.7
#6	7/8	139.5	69.8	46.5	34.9	27.9	23.3	19.9	17.4	15.5	14.0	12.7	11.6	10.7	10.0	9.3
#7	1	106.8	53.4	35.6	26.7	21.4	17.8	15.3	13.4	11.9	10.7	9.7	8.9	8.2	7.6	7.1
#8	1-1/8	84.4	42.2	28.1	21.1	16.9	14.1	12.1	10.6	9.4	8.4	7.7	7.0	6.5	6.0	5.6
#9	1-1/4	68.4	34.2	22.8	17.1	13.7	11.4	9.8	8.5	7.6	6.8	6.2	5.7	5.3	4.9	4.6
#10	1-1/2	47.5	23.7	15.8	11.9	9.5	7.9	6.8	5.9	5.3	4.7	4.3	4.0	3.7	3.4	3.2
#11	1-3/4	34.9	17.4	11.6	8.7	7.0	5.8	5.0	4.4	3.9	3.5	3.2	2.9	2.7	2.5	2.3

*The estimated number of anchoring installations per cartridge is based upon calculations of filling the hole 60% full of adhesive per the recommendation in our installation instructions. Hole volumes are calculated using ANSI tolerance carbide tipped drill bits. These estimates do not account for any waste.

A7P–5 fl. oz. Ordering Information

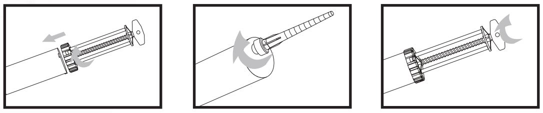
PART NUMBER	DESCRIPTION	BOX QTY	PART NUMBER	DESCRIPTION	BOX QTY
 A7P-500KIT	Kit with Dispenser Included (1) Cartridge (1) Dispenser (plastic) (1) Nozzle Nozzle diameter fits 3/8″ to 5/8″ holes	8	 A7P-501KIT	Kit for Standard Caulk Gun (1) Cartridge (1) Sleeve for Caulk Gun (1) Nozzle Nozzle diameter fits 3/8″ to 5/8″ holes	8

AVAILABLE WITH YOUR CHOICE OF TWO, EASY DISPENSING SYSTEMS

A500 PLASTIC DISPENSER

Attaches directly to cartridge allowing for easy hand dispensing. **No extra tools are required.**

Simple Assembly and Dispensing



1. Twist-lock dispenser onto cartridge.
2. Thread nozzle onto cartridge.
3. Turn lever in order to dispense adhesive.

EASY PACKAGING!

A500 and A501 kits are perfect for both counter or pegboard hanging display.

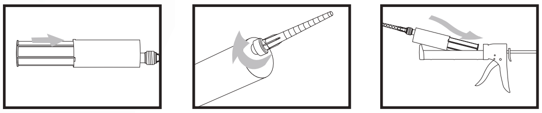
A7P-501KIT

A7P-500KIT
(not shown)

A501 CAULKINGGUN ADAPTOR

Allows cartridge to work with most standard caulking guns (caulking gun supplied by contractor)

Simple Assembly and Dispensing



1. Push adaptor tightly against back of cartridge.
2. Thread nozzle onto cartridge.
3. Place assembly in caulking gun and dispense adhesive.

ESTIMATING TABLES

A7+
5 Fluid Ounce Cartridge

Number of Anchoring Installations per Cartridge* using Threaded Rod with A7+ in Solid Concrete

ROD (in.)	DRILL HOLE DIA. INCHES	EMBEDMENT DEPTH IN INCHES 1	2	3	4	5	6	7	8
1/4	5/16	198.9	99.5	66.3	49.7	39.8	33.2	28.4	24.9
3/8	7/16	101.5	50.7	33.8	25.4	20.3	16.9	14.5	12.7
1/2	9/16	61.4	30.7	20.5	15.3	12.3	10.2	8.8	7.7
5/8	3/4	34.5	17.3	11.5	8.6	6.9	5.8	4.9	4.3
3/4	7/8	25.4	12.7	8.5	6.3	5.1	4.2	3.6	3.2
7/8	1	19.4	9.7	6.5	4.9	3.9	3.2	2.8	2.4
1	1-1/8	15.3	7.7	5.1	3.8	3.1	2.6	2.2	1.9

*The estimated number of anchoring installations per cartridge is based upon calculations of filling the hole 60% full of adhesive per the recommendation in our installation instructions. Hole volumes are calculated using ANSI tolerance carbide tipped drill bits. These estimates do not account for any waste.

A7+
5 Fluid Ounce Cartridge

Number of Anchoring Installations per Cartridge using Rebar with A7+ in Solid Concrete*

REBAR	DRILL HOLE DIA. INCHES	EMBEDMENT DEPTH IN INCHES 1	2	3	4	5	6	7	8
#3	7/16	101.5	50.7	33.8	25.4	20.3	16.9	14.5	12.7
#4	5/8	49.7	24.9	16.6	12.4	9.9	8.3	7.1	6.2
#5	3/4	34.5	17.3	11.5	8.6	6.9	5.8	4.9	4.3
#6	7/8	25.4	12.7	8.5	6.3	5.1	4.2	3.6	3.2
#7	1	19.4	9.7	6.5	4.9	3.9	3.2	2.8	2.4
#8	1-1/8	15.3	7.7	5.1	3.8	3.1	2.6	2.2	1.9
#9	1-1/4	12.4	6.2	4.1	3.1	2.5	2.1	1.8	1.6

*The estimated number of anchoring installations per cartridge is based upon calculations of filling the hole 60% full of adhesive per the recommendation in our installation instructions. Hole volumes are calculated using ANSI tolerance carbide tipped drill bits. These estimates do not account for any waste.

PERFORMANCE TABLE

A7+
The Most Versatile Quick-Cure

Threaded Rod Ultimate Tension and Shear Loads[1,2,3] Installed in Solid Concrete

THREADED ROD DIAMETER in.	(mm)	DRILL HOLE DIAMETER in	(mm)	MAX. CLAMPING FORCE AFTER PROPER CURE ft.-lbs	(Nm)	EMBEDMENT IN CONCRETE in.	(mm)	2000 PSI (13.8 MPa) CONCRETE ULTIMATE TENSION lbs.	(kN)	ULTIMATE SHEAR lbs.	(kN)	4000 PSI (27.6 MPa) CONCRETE ULTIMATE TENSION lbs.	(kN)	ULTIMATE SHEAR lbs.	(kN)
3/8	(9.5)	7/16	(11.1)	9	(12)	1-1/2	(38.1)	N/A	N/A	N/A	N/A	3,734	(16.6)	4,126	(18.3)
						3-3/8	(85.7)	5,852	(26.0)	5,220	(23.2)	10,977	(48.8)	5,220	(23.2)
						4-1/2	(114.3)	7,729	(34.4)	5,220	(23.2)	11,661	(51.9)	5,220	(23.2)
1/2	(12.7)	9/16	(14.3)	16	(21)	2	(50.8)	N/A	N/A	N/A	N/A	6,022	(26.8)	8,029	(35.7)
						4-1/2	(114.3)	10,798	(48.0)	8,029	(35.7)	17,162	(76.3)	8,029	(35.7)
						6	(152.4)	14,210	(63.2)	8,029	(35.7)	17,372	(77.3)	8,029	(35.7)
5/8	(15.9)	3/4	(19.1)	47	(63)	2-1/2	(63.5)	N/A	N/A	N/A	N/A	7,330	(32.6)	11,256	(50.1)
						5-5/8	(142.9)	16,417	(73.0)	15,967	(71.0)	26,504	(117.9)	15,967	(71.0)
						7-1/2	(190.5)	18,747	(83.4)	15,967	(71.0)	29,381	(130.7)	15,967	(71.0)
3/4	(19.1)	7/8	(22.2)	70	(95)	3	(76.2)	N/A	N/A	N/A	N/A	8,634	(38.4)	20,126	(89.5)
						6-3/4	(171.5)	18,618	(82.8)	20,126	(89.5)	29,727	(132.2)	20,126	(89.5)
						9	(228.6)	23,934	(106.5)	20,126	(89.5)	37,728	(167.8)	20,126	(89.5)
7/8	(22.2)	1	(25.4)	90	(122)	3-1/2	(88.9)	N/A	N/A	N/A	N/A	13,650	(60.7)	20,920	(92.9)
						7-7/8	(200.0)	N/A	N/A	29,866	(132.9)	44,915	(199.8)	29,866	(132.9)
						10-1/2	(266.7)	36,881	(164.1)	29,866	(132.9)	48,321	(215.0)	29,866	(132.9)
1	(25.4)	1-1/8	(28.6)	110	(149)	4	(101.6)	N/A	N/A	N/A	N/A	16,266	(72.2)	33,152	(147.5)
						9	(228.6)	32,215	(143.3)	37,538	(167.0)	48,209	(214.5)	37,538	(167.0)
						12	(304.8)	46,064	(204.9)	37,538	(167.0)	63,950	(284.5)	37,538	(167.0)
1-1/4	(31.8)	1-3/8	(34.9)	370	(501)	5	(127.0)	N/A	N/A	N/A	N/A	21,838	(97.1)	33,152	(147.5)
						11-1/4	(285.8)	45,962	(204.5)	58,412	(259.8)	56,715	(252.3)	58,412	(259.8)
						15	(381.0)	62,208	(276.7)	58,412	(259.8)	84,385	(375.4)	58,412	(259.8)

1 Allowable working loads for the single installation under static loading should not exceed 25% capacity of the ultimate load. To calculate the allowable load of the anchor, divide the ultimate load by 4.

2 Ultimate load values in 2000 and 4000 psi stone aggregate concrete. Ultimate loads are indicated for the embedment shown in the Embedment in Concrete column. Performance values are based on the use of high strength threaded rod (ASTM A193 Gr. B7). The use of lower strength rods will result in lower ultimate tension and shear loads.

3 Linear interpolation may be used for intermediate spacing and edge distances.

PERFORMANCE TABLE

A7+
The most Versatile Quick Cure

Threaded Rod Allowable Tension Loads[1,2] Installed in Solid Concrete

THREADED ROD DIAMETER		DRILL HOLE DIAMETER		MIN. EMBEDMENT DEPTH		ALLOWABLE TENSION LOAD BASED ON ADHESIVE BOND STRENGTH				ALLOWABLE TENSION LOAD BASED ON STEEL STRENGTH					
						2000 PSI (13.8 MPA) CONCRETE		4000 PSI (27.6 MPa) CONCRETE		ASTM A307 (SAE 1018)		ASTM A193 GR. B7 (SAE4140)		ASTM F593 AISI 304 SS	
in.	(mm)	in.	(mm)	in.	(mm)	lbs.	(kN)	lbs	(kN)	lbs	(kN)	lbs	(kN)	lbs	(kN)
3/8	(9.5)	7/16	(11.1)	1-1/2	(38.1)	N/A	N/A	934	(4.2)	2,080	(9.3)	4,340	(19.3)	3,995	(17.8)
				3-3/8	(85.7)	1,460	(6.5)	2,740	(12.2)	2,080	(9.3)	4,340	(19.3)	3,995	(17.8)
				4-1/2	(114.3)	1,930	(8.6)	2,915	(13.0)	2,080	(9.3)	4,340	(19.3)	3,995	(17.8)
1/2	(12.7)	9/16	(14.3)	2	(50.8)	N/A	N/A	1,505	(6.7)	3,730	(16.6)	7,780	(34.6)	7,155	(31.8)
				4-1/2	(114.3)	2,700	(12.0)	4,290	(19.1)	3,730	(16.6)	7,780	(34.6)	7,155	(31.8)
				6	(152.4)	3,550	(15.8)	4,340	(19.3)	3,730	(16.6)	7,780	(34.6)	7,155	(31.8)
5/8	(15.9)	3/4	(19.1)	2-1/2	(63.5)	N/A	N/A	1,832	(8.2)	5,870	(26.1)	12,230	(54.4)	11,250	(50.0)
				5-5/8	(142.9)	4,100	(18.3)	6,625	(29.5)	5,870	(26.1)	12,230	(54.4)	11,250	(50.0)
				7-1/2	(190.5)	4,685	(20.8)	7,345	(32.7)	5,870	(26.1)	12,230	(54.4)	11,250	(50.0)
3/4	(19.1)	7/8	(22.2)	3	(76.2)	N/A	N/A	2,158	(9.6)	8,490	(37.8)	17,690	(78.7)	14,860	(66.1)
				6-3/4	(171.5)	4,655	(20.7)	7,430	(33.1)	8,490	(37.8)	17,690	(78.7)	14,860	(66.1)
				9	(228.6)	5,980	(26.6)	9,430	(42.0)	8,490	(37.8)	17,690	(78.7)	14,860	(66.1)
7/8	(22.2)	1	(25.4)	3-1/2	(88.9)	N/A	N/A	3,413	(15.2)	11,600	(51.6)	25,510	(113.5)	20,835	(92.7)
				7-7/8	(200.0)	N/A	N/A	11,230	(49.9)	11,600	(51.6)	25,510	(113.5)	20,835	(92.7)
				10-1/2	(266.7)	9,220	(41.0)	12,080	(53.7)	11,600	(51.6)	25,510	(113.5)	20,834	(92.7)
1	(25.4)	1-1/8	(28.6)	4	(101.6)	N/A	N/A	4,067	(18.1)	15,180	(67.5)	31,620	(140.7)	26,560	(118.1)
				9	(228.6)	8,050	(35.8)	12,050	(53.6)	15,180	(67.5)	31,620	(140.7)	26,560	(118.1)
				12	(304.8)	11,515	(51.2)	15,985	(71.1)	15,180	(67.5)	31,620	(140.7)	26,560	(118.1)
1-1/4	(31.8)	1-3/8	(34.9)	5	(127.0)	N/A	N/A	5,460	(24.3)	23,800	(105.9)	49,580	(220.6)	34,670	(154.2)
				11-1/4	(285.8)	11,490	(51.1)	14,175	(63.1)	23,800	(105.9)	49,580	(220.6)	34,670	(154.2)
				15	(381.0)	15,550	(69.2)	21,095	(93.8)	23,800	(105.9)	49,580	(220.6)	34,670	(154.2)

1 Use lower value of either bond or steel strength for allowable tensile load.

2 Larger rods and/or deeper holes may be used. However, it may not be covered by current codes.

PERFORMANCE TABLE

A7+
The most Versatile Quick Cure

Threaded Rod Allowable Shear Loads[1,2] Installed in Solid Concrete

THREADED ROD DIAMETER		DRILL HOLE DIAMETER		MIN. EMBEDMENT DEPTH		ALLOWABLE SHEAR LOAD BASED ON CONCRETE STRENGTH				ALLOWABLE SHEAR LOAD BASED ON STEEL STRENGTH					
						2000 PSI (13.8 MPA) CONCRETE		4000 PSI (27.6 MPa) CONCRETE		ASTM A307 (SAE 1018)		ASTM A193 GR. B7 (SAE4140)		ASTM F593 AISI 304 SS	
in.	(mm)	in.	(mm)	in.	(mm)	lbs.	(kN)	lbs.	(kN)	lbs.	(kN)	lbs.	(kN)	lbs.	(kN)
3/8	(9.5)	7/16	(11.1)	1-1/2	(38.1)	N/A	N/A	1,031	(4.6)	1,040	(4.6)	2,170	(9.7)	1,995	(8.9)
				3-3/8	(85.7)	1,305	(5.8)	1,305	(5.8)	1,040	(4.6)	2,170	(9.7)	1,995	(8.9)
1/2	(12.7)	9/16	(14.3)	2	(50.8)	N/A	N/A	2,005	(8.9)	1,870	(8.3)	3,895	(17.3)	3,585	(15.9)
				4-1/2	(114.3)	2,005	(8.9)	2,005	(8.9)	1,870	(8.3)	3,895	(17.3)	3,585	(15.9)
5/8	(15.9)	3/4	(19.1)	2-1/2	(63.5)	N/A	N/A	2,814	(12.5)	2,940	(13.1)	6,125	(27.2)	5,635	(25.1)
				5-5/8	(142.9)	3,990	(17.8)	3,990	(17.8)	2,940	(13.1)	6,125	(27.2)	5,635	(25.1)
3/4	(19.1)	7/8	(22.2)	3	(76.2)	N/A	N/A	5,030	(22.4)	4,250	(18.9)	8,855	(39.4)	7,440	(33.1)
				6-3/4	(171.5)	5,030	(22.4)	5,030	(22.4)	4,250	(18.9)	8,855	(39.4)	7,440	(33.1)
7/8	(22.2)	1	(25.4)	3-1/2	(88.9)	N/A	N/A	5,230	(23.3)	5,800	(25.8)	12,760	(56.8)	10,730	(47.7)
				7-7/8	(200.0)	7,465	(33.2)	7,465	(33.2)	5,800	(25.8)	12,760	(56.8)	10,730	(47.7)
1	(25.4)	1-1/8	(28.6)	4	(101.6)	N/A	N/A	8,288	(36.9)	7,590	(33.8)	15,810	(70.3)	13,285	(59.1)
				9	(228.6)	9,385	(41.7)	9,385	(41.7)	7,590	(33.8)	15,810	(70.3)	13,285	(59.1)
1-1/4	(31.8)	1-3/8	(34.9)	5	(127.0)	N/A	N/A	8,288	(36.9)	11,900	(52.9)	24,790	(100.3)	18,840	(83.8)
				11-1/4	(285.8)	14,600	(64.9)	14,600	(64.9)	11,900	(52.9)	24,790	(100.3)	18,840	(83.8

1 Use lower value of either concrete or steel strength for allowable shear load.

2 Larger rods and/or deeper holes may be used. However, it may not be covered by current codes.

PERFORMANCE TABLE

A7+
The Most Versatile Quick-Cure

Rebar Ultimate Tension Loads[1,2,3] Installed in Solid Concrete

REINFORCING BAR DIAMETER in.	(mm)	EMBEDMENT IN CONCRETE in.	(mm)	2000 PSI (13.8 MPa) CONCRETE ULTIMATE TENSION lbs.	(kN)	4000 PSI (27.6 MPa) CONCRETE ULTIMATE TENSION lbs.	(kN)	ULTIMATE TENSILE AND YIELD STRENGTH: GRADE 60 REBAR — MINIMUM YIELD STRENGTH lbs.	(kN)	MINIMUM ULTIMATE TENSILE STRENGTH lbs.	(kN)
# 3	(9.5)	3-3/8	(85.7)	6,180	(27.5)	8,324	(37.0)	6,600	(29.4)	9,900	(44.0)
		4-1/2	(114.3)	7,560	(33.6)	11,418	(50.8)	6,600	(29.4)	9,900	(44.0)
# 4	(12.7)	4-1/2	(114.3)	9,949	(44.3)	16,657	(74.1)	12,000	(53.4)	18,000	(80.1)
		6	(152.4)	15,038	(66.9)	17,828	(79.3)	12,000	(53.4)	18,000	(80.1)
# 5	(15.9)	5-5/8	(142.9)	14,012	(62.3)	20,896	(93.0)	18,600	(82.7)	27,900	(124.1)
		7-1/2	(190.5)	16,718	(74.4)	26,072	(116.0)	18,600	(82.7)	27,900	(124.1)
# 6	(19.1)	6-3/4	(171.5)	21,247	(94.5)	26,691	(118.7)	26,400	(117.4)	39,600	(176.2)
		9	(228.6)	33,325	(148.2)	37,425	(166.5)	26,400	(117.4)	39,600	(176.2)
# 7	(22.2)	7-7/8	(200.0)	N/A	N/A	40,374	(179.6)	36,000	(160.1)	54,000	(240.2)
		10-1/2	(266.7)	38,975	(173.4)	46,050	(204.8)	36,000	(160.1)	54,000	(240.2)
# 8	(25.4)	9	(228.6)	35,600	(158.4)	47,311	(210.5)	47,400	(210.9)	71,100	(316.3)
		12	(304.8)	41,010	(182.4)	66,140	(294.2)	47,400	(210.9)	71,100	(316.3)
# 9	(28.6)	10-1/8	(257.2)	N/A	N/A	57,221	(254.5)	60,000	(266.9)	90,000	(400.4)
		13-1/2	(342.9)	N/A	N/A	79,966	(355.7)	60,000	(266.9)	90,000	(400.4)
# 10	(31.8)	11-1/4	(285.8)	49,045	(218.2)	73,091	(325.1)	76,200	(339.0)	114,300	(508.5)
		15	(381.0)	69,079	(307.3)	83,295	(370.5)	76,200	(339.0)	114,300	(508.5)
# 11	(34.9)	12-3/8	(314.3)	63,397	(282.0)	75,047	(333.8)	93,600	(416.4)	140,400	(624.6)
		16-1/2	(419.1)	81,707	(363.5)	91,989	(409.2)	93,600	(416.4)	140,400	(624.6)

1 Allowable working loads for the single installation under static loading should not exceed 25% capacity or the allowable load of the anchor rod.

2 Ultimate load values in 2000 and 4000 psi stone aggregate concrete. Ultimate loads are indicated for the embedment shown in the Embedment in Concrete column. Performance values are based on the use of minimum Grade 60 reinforcing bar. The use of lower strength rods will result in lower ultimate tension loads.

3 SHEAR DATA: Provided the distance from the rebar to the edge of the concrete member exceeds 1.25 times the embedment depth of the rebar, calculate the ultimate shear load for the rebar anchorage as 60% of the ultimate tensile strength of the rebar.

4 Larger rods and/or deeper holes may be used. However, it may not be covered by current codes.

PERFORMANCE TABLE

A7+
The Most Versatile Quick-Cure

Threaded Rod Recommended Edge Distance Requirements for Tension Loads Installed in Solid Concrete

ANCHOR DIAMETER in.	(mm)	EMBEDMENT DEPTH in.	(mm)	CRITICAL EDGE DISTANCE (100% LOAD CAPACITY) in.	(mm)	INTERPOLATED EDGE DISTANCE (90% LOAD CAPACITY) in.	(mm)	INTERPOLATED EDGE DISTANCE (80% LOAD CAPACITY) in.	(mm)	MINIMUM EDGE DISTANCE (70% LOAD CAPACITY) in.	(mm)
3/8	(9.5)	3-3/8	(85.7)	2-1/2	(63.5)	1-15/16	(49.2)	1-3/8	(34.9)	13/16	(26.2)
		4-1/2	(114.3)	3-3/8	(85.7)	2-5/8	(66.7)	1-7/8	(47.6)	1-1/8	(28.6)
1/2	(12.7)	4-1/2	(114.3)	3-3/8	(85.7)	2-5/8	(66.7)	1-7/8	(47.6)	1-1/8	(28.6)
		6	(152.4)	4-1/2	(114.3)	3-1/2	(88.9)	2-1/2	(63.5)	1-1/2	(38.1)
5/8	(15.9)	5-5/8	(142.9)	4-3/16	(106.4)	3-1/4	(82.6)	2-5/16	(58.7)	1-3/8	(34.9)
		7-1/2	(190.5)	5-5/8	(142.9)	4-3/8	(111.1)	3-1/8	(79.4)	1-7/8	(47.6)
3/4	(19.1)	6-3/4	(171.5)	5-1/16	(128.6)	3-15/16	(100.0)	2-13/16	(71.4)	1-5/8	(15.9)
		9	(228.6)	6-3/4	(171.5)	5-1/4	(133.4)	3-3/4	(95.3)	2-1/4	(57.2)
1	(25.4)	9	(228.6)	6-3/4	(171.5)	5-1/4	(133.4)	3-3/4	(95.3)	2-1/4	(57.2)
		12	(304.8)	9	(228.6)	7	(177.8)	5	(127.0)	3	(76.2)
1-1/4	(31.8)	11-1/4	(285.8)	8-7/16	(214.3)	6-9/16	(166.7)	4-3/4	(120.7)	2-7/8	(73.0)
		15	(381.0)	11-1/4	(285.8)	8-3/4	(222.2)	6-1/4	158.8)	3-3/4	(95.3)

PERFORMANCE TABLE

A7+
The Most Versatile Quick-Cure

Threaded Rod Recommended Edge Distance Requirements for Shear Loads Installed in Solid Concrete

ANCHOR DIAMETER		EMBEDMENT DEPTH (100% LOAD CAPACITY)		CRITICAL EDGE DISTANCE (80% LOAD CAPACITY)		INTERPOLATED EDGE DISTANCE (50% LOAD CAPACITY)		INTERPOLATED EDGE DISTANCE (10% LOAD CAPACITY)		MINIMUM EDGE DISTANCE	
in.	(mm)	in.	(mm)	in.	(mm)	in.	(mm)	in.	(mm)	in.	(mm)
3/8	(9.5)	3-3/8	(85.7)	4-3/16	(106.4)	3-7/16	(87.3)	2-5/16	(58.7)	13/16	(20.6)
1/2	(12.7)	4-1/2	(114.3)	5-5/8	(142.9)	4-5/8	(117.5)	3-1/8	(79.4)	1-1/8	(28.6)
5/8	(15.9)	5-5/8	(142.9)	7	(177.8)	5-3/4	(146.1)	3-1/8	(79.4)	1-3/8	(34.9)
3/4	(19.1)	6-3/4	(171.5)	8-7/16	(214.2)	6-15/16	(176.2)	4-5/8	(117.5)	1-5/8	(41.3)
1	(25.4)	9	(228.6)	11-1/4	(285.8)	9-1/4	(235.0)	6-1/4	(158.8)	2-1/4	(57.2)
1-1/4	(31.8)	11-1/4	(285.8)	14-1/16	(357.2)	11-5/8	(295.3)	7-7/8	(200.0)	2-7/8	(73.0)

PERFORMANCE REFERENCE TABLE

A7+
The Most Versatile Quick-Cure

Allowable Stress Design Reference Tables

Threaded Rod and Rebar Installation in Solid Concrete Edge / Spacing Distance Load Factor Summary[1,2]

LOAD FACTOR	DISTANCE FROM EDGE OF CONCRETE
Critical Edge Distance—Tension	
100% Tension Load	0.75 x Anchor Embedment
Minimum Edge Distance—Tension	
70% Tension Load	0.25 x Anchor Embedment
Critical Edge Distance—Shear	
100% Shear Load	1.25 x Anchor Embedment
Minimum Edge Distance—Shear	
10% Shear Load	0.25 x Anchor Embedment

LOAD FACTOR	DISTANCE FROM ANOTHER ANCHOR
Critical Spacing—Tension	
100% Tension Load	1.25 x Anchor Embedment
Minimum Spacing—Tension	
80% Tension Load	0.25 x Anchor Embedment
Critical Spacing—Shear	
100% Shear Load	1.25 x Anchor Embedment
Minimum Spacing—Shear	
25% Shear Load	0.25 x Anchor Embedment

1 Use linear interpolation for load factors at edge distances or spacing distances between critical and minimum.

2 Anchors are affected by multiple combination of spacing and/or edge distance loading and direction of the loading. Use the product of tension and shear loading factors in design.

Combined Tension and Shear Loading—for A7+/C6+/G5+ Adhesive Anchors

Allowable loads for anchors under tension and shear loading at the same time (combined loading) will be lower than the allowable loads for anchors subjected to 100% tension or 100% shear. Use the following equation to evaluate anchors in combined loading conditions:

$$\left(\frac{Na}{Ns}\right)^{5/3} + \left(\frac{Va}{Vs}\right)^{5/3} \leq 1$$

Na = Applied Service Tension Load

Ns = Allowable Tension Load

Va = Applied Service Shear Load

Vs = Allowable Shear Load

A7+

The Most Versatile Quick-Cure

Threaded Rod Tension (lbf) and Shear (lbf) Loads in Uncracked Concrete[1,2,3,4] ASTM A193 B7

Anchor Diameter (in.)	Embedment Depth (in.)	Tension (lbf)					Shear (lbf)
		2500 psi	3000 psi	4000 psi	5000 psi	6000 psi - 8000 psi	2500 psi - 8000 psi
3/8	3-3/8	3,870	3,870	3,870	3,870	3,870	3,775
	4-1/2	5,160	5,160	5,160	5,160	5,160	3,775
	7-1/2	7,265	7,265	7,265	7,265	7,265	3,775
1/2	4-1/2	6,880	6,880	6,880	6,880	6,880	6,915
	6	9,175	9,175	9,175	9,175	9,175	6,915
	10	13,305	13,305	13,305	13,305	13,305	6,915
5/8	5-5/8	10,405	10,750	10,750	10,750	10,750	11,015
	7-1/2	14,335	14,335	14,335	14,335	14,335	11,015
	12-1/2	21,185	21,185	21,185	21,185	21,185	11,015
3/4	6-3/4	13,675	14,980	15,480	15,480	15,480	16,305
	9	20,640	20,640	20,640	20,640	20,640	16,305
	15	31,355	31,355	31,355	31,355	31,355	16,305
7/8	7-7/8	17,235	17,740	17,740	17,740	17,740	22,505
	10-1/2	23,650	23,650	23,650	23,650	23,650	22,505
	17-1/2	39,420	39,420	39,420	39,420	39,420	22,505
1	9	21,060	23,070	23,170	23,170	23,170	29,525
	12	30,890	30,890	30,890	30,890	30,890	29,525
	20	51,490	51,490	51,490	51,490	51,490	29,525
1-1/4	11-1/4	29,430	32,240	37,225	41,620	42,785	47,240
	15	45,310	49,635	57,045	57,045	57,045	47,240
	25	90,855	90,855	90,855	90,855	90,855	47,240

1 Tabulated values are for estimation purposes only and should not be used for design (please use our free TruSpec anchorage design software at www.itwredhead.com)

2 Tabulated values represent strength design per ACI 318 for a single anchor in adequate concrete thickness, not near an edge nor adjacent anchorage, and not for sustained loading.

3 Bond strengths used in calculations are for dry, uncracked concrete with periodic inspection

4 Bond strengths used in calculations are for Temperature Range A (maximum long term temperature of 110F, maximum short term temperature of 142F).

A7+
The Most Versatile Quick-Cure

Threaded Rod Tension (lbf) and Shear (lbf) Loads in 4,000 psi Uncracked Concrete[1,2,3,4]

Anchor Diameter (in.)	Embedment Depth (in.)	ASTM A193 B7 Threaded Rod Tension (lbf)	ASTM A193 B7 Threaded Rod Shear (lbf)	Stainless Steel F593 Tension (lbf)	Stainless Steel F593 Shear (lbf)	Carbon Steel A36 Tension (lbf)	Carbon Steel A36 Shear (lbf)
3/8	3-3/8	3,870	3,775	3,375	1,755	3,870	2,280
	4-1/2	5,160	3,775	3,375	1,755	4,785	2,280
	7-1/2	7,265	3,775	3,375	1,755	4,785	2,280
1/2	4-1/2	6,880	6,915	6,170	3,210	6,880	4,040
	6	9,175	6,915	6,170	3,210	8,760	4,040
	10	13,305	6,915	6,170	3,210	8,760	4,040
5/8	5-5/8	10,750	11,015	9,830	5,115	10,750	6,440
	7-1/2	14,335	11,015	9,830	5,115	13,955	6,440
	12-1/2	21,185	11,015	9,830	5,115	13,955	6,440
3/4	6-3/4	15,480	16,305	14,550	7,565	15,480	7,610
	9	20,640	16,305	14,550	7,565	16,500	7,610
	15	31,355	16,305	14,550	7,565	16,500	7,610
7/8	7-7/8	17,740	22,505	17,740	10,445	17,740	10,530
	10-1/2	23,650	22,505	20,085	10,445	22,820	10,530
	17-1/2	39,420	22,505	20,085	10,445	22,820	10,530
1	9	23,170	29,525	23,170	13,700	23,170	13,815
	12	30,890	29,525	26,345	13,700	29,935	13,815
	20	51,490	29,525	26,345	13,700	29,935	13,815
1-1/4	11-1/4	37,225	47,240	37,225	21,920	37,225	22,090
	15	57,045	47,240	42,155	21,920	47,865	22,090
	25	90,855	47,240	42,155	21,920	47,865	22,090

1 Tabulated values are for estimation purposes only and should not be used for design (please use our free TruSpec anchorage design software at www.itwredhead.com)

2 Tabulated values represent strength design per ACI 318 for a single anchor in adequate concrete thickness, not near an edge nor adjacent anchorage, and not for sustained loading.

3 Bond strengths used in calculations are for dry, uncracked concrete with periodic inspection

4 Bond strengths used in calculations are for Temperature Range A (maximum long term temperature of 110F, maximum short term temperature of 142F).

STRENGTH DESIGN TABLE

A7+
The Most Versatile Quick-Cure

Threaded Rod Tension (lbf) and Shear (lbf) Loads in Cracked Concrete[1,2,3,4] ASTM A193 B7

Anchor Diameter (in.)	Embedment Depth (in.)	Tension (lbf) 2,500-8,000 psi	Shear (lbf) 2,500-8,000 psi
3/8	3-3/8	2,315	3,775
	4-1/2	3,090	3,775
	7-1/2	5,150	3,775
1/2	4-1/2	3,070	6,915
	6	4,095	6,915
	10	6,825	6,915
5/8	5-5/8	5,220	11,015
	7-1/2	6,965	11,015
	12-1/2	11,605	11,015
3/4	6-3/4	7,785	15,365
	9	10,380	16,305
	15	17,300	16,305
7/8	7-7/8	8,270	20,915
	10-1/2	11,030	22,505
	17-1/2	18,385	22,505
1	9	10,185	27,320
	12	13,580	29,525
	20	22,635	29,525
1-1/4	11-1/4	16,795	46,600
	15	22,395	47,240
	25	37,330	47,240

1 Tabulated values are for estimation purposes only and should not be used for design (please use our free TruSpec anchorage design software at www.itwredhead.com)
2 Tabulated values represent strength design per ACI 318 for a single anchor in adequate concrete thickness, not near an edge nor adjacent anchorage, and not for sustained loading.
3 Bond strengths used in calculations are for dry, cracked concrete with periodic inspection
4 Bond strengths used in calculations are for Temperature Range A (maximum long term temperature of 110F, maximum short term temperature of 142F).

STRENGTH DESIGN TABLE

A7+
The Most Versatile Quick-Cure

Threaded Rod Tension (lbf) and Shear (lbf) Loads in 4,000 psi Cracked Concrete[1,2,3,4]

Anchor Diameter (in.)	Embedment Depth (in.)	ASTM A193 B7 Threaded Rod		Stainless Steel F593		Carbon Steel A36	
		Tension (lbf)	Shear (lbf)	Tension (lbf)	Shear (lbf)	Tension (lbf)	Shear (lbf)
3/8	3-3/8	2,315	3,245	3,375	1,755	3,870	2,280
	4-1/2	3,090	3,775	3,375	1,755	4,785	2,280
	7-1/2	5,150	3,775	3,375	1,755	4,785	2,280
1/2	4-1/2	3,070	4,295	6,170	3,210	6,670	4,040
	6	4,095	5,730	6,170	3,210	8,760	4,040
	10	6,825	6,915	6,170	3,210	8,760	4,040
5/8	5-5/8	5,220	7,310	9,320	5,115	9,320	6,440
	7-1/2	6,965	9,750	9,830	5,115	13,955	6,440
	12-1/2	11,605	11,015	9,830	5,115	13,955	6,440
3/4	6-3/4	7,785	10,895	12,255	7,565	12,255	7,610
	9	10,380	14,530	14,550	7,565	16,500	7,610
	15	17,300	16,305	14,550	7,565	16,500	7,610
7/8	7-7/8	8,270	11,580	15,440	10,445	15,440	10,530
	10-1/2	11,030	15,445	20,085	10,445	22,820	10,530
	17-1/2	18,385	22,505	20,085	10,445	22,820	10,530
1	9	10,185	14,260	18,865	13,700	18,865	13,815
	12	13,580	19,010	26,345	13,700	29,050	13,815
	20	22,635	29,525	26,345	13,700	29,935	13,815
1-1/4	11-1/4	16,795	23,515	26,370	21,920	26,370	22,090
	15	22,395	31,355	40,600	21,920	40,600	22,090
	25	37,330	47,240	42,155	21,920	47,865	22,090

1 Tabulated values are for estimation purposes only and should not be used for design (please use our free TruSpec anchorage design software at www.itwredhead.com)
2 Tabulated values represent strength design per ACI 318 for a single anchor in adequate concrete thickness, not near an edge nor adjacent anchorage, and not for sustained loading.
3 Bond strengths used in calculations are for dry, cracked concrete with periodic inspection
4 Bond strengths used in calculations are for Temperature Range A (maximum long term temperature of 110F, maximum short term temperature of 142F).

A7+

The Most Versatile Quick-Cure

Rebar Tension (lbf) and Shear (lbf) Loads in Uncracked Concrete[1,2,3,4] ASTM A615 Grade 60

Rebar	Anchor Diameter (in.)	Embedment Depth (in.)	Tension (lbf)					Shear (lbf)
			2500 psi	3000 psi	4000 psi	5000 psi	6000 - 8000 psi	2500 - 8000 psi
#3	3/8	3-3/8	3,660	3,660	3,660	3,660	3,660	3,560
		4-1/2	4,880	4,880	4,880	4,880	4,880	3,560
		7-1/2	4,835	6,435	6,435	6,435	6,435	3,560
#4	1/2	4-1/2	7,445	7,520	7,520	7,520	7,520	6,480
		6	10,030	10,030	10,030	10,030	10,030	6,480
		10	11,700	11,700	11,700	11,700	11,700	6,480
#5	5/8	5-5/8	10,405	11,395	11,540	11,540	11,540	10,040
		7-1/2	15,385	15,385	15,385	15,385	15,385	10,040
		12-1/2	18,135	18,135	18,135	18,135	18,135	10,040
#6	3/4	6-3/4	13,675	14,870	14,870	14,870	14,870	14,255
		9	19,825	19,825	19,825	19,825	19,825	14,255
		15	25,740	25,740	25,740	25,740	25,740	14,255
#7	7/8	7-7/8	17,235	18,880	19,465	19,465	19,465	19,440
		10-1/2	25,955	25,955	25,955	25,955	25,955	19,440
		17-1/2	35,100	35,100	35,100	35,100	35,100	19,440
#8	1	9	21,060	23,070	25,110	25,110	25,110	25,595
		12	32,420	33,485	33,485	33,485	33,485	25,595
		20	46,215	46,215	46,215	46,215	46,215	25,595
#9	1-1/8	10-1/8	25,130	27,525	31,195	31,195	31,195	32,400
		13-1/2	38,690	41,590	41,590	41,590	41,590	32,400
		22-1/2	58,500	58,500	58,500	58,500	58,500	32,400
#10	1-1/4	11-1/4	29,430	32,240	37,225	41,620	44,505	41,145
		15	45,310	49,635	57,315	59,345	59,345	41,145
		25	74,295	74,295	74,295	74,295	74,295	41,145

1 Tabulated values are for estimation purposes only and should not be used for design (please use our free TruSpec anchorage design software at www.itwredhead.com)

2 Tabulated values represent strength design per ACI 318 for a single anchor in adequate concrete thickness, not near an edge nor adjacent anchorage, and not for sustained loading.

3 Bond strengths used in calculations are for dry, uncracked concrete with periodic inspection

4 Bond strengths used in calculations are for Temperature Range A (maximum long term temperature of 110F, maximum short term temperature of 142F).

A7+
The Most Versatile Quick-Cure

Rebar Tension (lbf) and Shear (lbf) Loads in Cracked Concrete[1,2,3,4] ASTM A615 Grade 60

Rebar	Anchor Diameter (in.)	Embedment Depth (in.)	Tension (lbf) 2500 - 8000 psi concrete	Shear (lbf) 2500 - 8000 psi concrete
#3	3/8	3-3/8	1,650	2,310
		4-1/2	2,200	3,080
		7-1/2	3,665	3,560
#4	1/2	4-1/2	2,935	4,105
		6	3,910	5,475
		10	6,520	6,480
#5	5/8	5-5/8	4,585	6,420
		7-1/2	6,115	8,560
		12-1/2	10,190	10,040
#6	3/4	6-3/4	5,115	7,160
		9	6,820	9,550
		15	11,370	14,255
#7	7/8	7-7/8	6,965	9,750
		10-1/2	9,285	13,000
		17-1/2	15,475	19,440
#8	1	9	9,095	12,735
		12	12,125	16,980
		20	20,215	25,595
#9	1-1/8	10-1/8	11,510	16,115
		13-1/2	15,350	21,490
		22-1/2	25,585	32,400
#10	1-1/4	11-1/4	16,795	23,515
		15	22,395	31,355
		25	37,330	41,145

1 Tabulated values are for estimation purposes only and should not be used for design (please use our free TruSpec anchorage design software at www.itwredhead.com)
2 Tabulated values represent strength design per ACI 318 for a single anchor in adequate concrete thickness, not near an edge nor adjacent anchorage, and not for sustained loading.
3 Bond strengths used in calculations are for dry, cracked concrete with periodic inspection
4 Bond strengths used in calculations are for Temperature Range A (maximum long term temperature of 110F, maximum short term temperature of 142F).

MASONRY DESIGN TABLE

A7+
The Most Versatile Quick-Cure

Grout-filled Concrete Block: Threaded Rod Allowable Tension and Shear Load Based on Steel Design Information for U.S. Customary Unit [1,2,3]

Anchor Diameter (in.)	Tension (lb)			Shear (lb)		
	ASTM A307 F_u = 60 ksi	ASTM A193 Grade B7 F_u = 125 ksi	ASTM F593 SS 304 F_u = 100 ksi	ASTM A307 F_u = 60 ksi	ASTM A193 Grade B7 F_u = 125 ksi	ASTM F593 SS 304 F_u = 100 ksi
3/8	2,185	4,555	3,645	1,125	2,345	1,875
1/2	3,885	8,100	6,480	2,000	4,170	3,335
5/8	6,075	12,655	10,125	3,130	6,520	5,215
3/4	8,750	18,225	12,390	4,505	9,390	6,385

For SI: 1 inch = 25.4mm, 1 lbf = 4.45N, 1ft-lbf = 1.356 N-M, 1 psi = 0.006895 MPa

1 Allowable load used in the design must be the lesser of bond values and tabulated steel element values.

2 Allowable tension and shear loads for threaded rods to resist short term loads, such as wind or seismic, must be calculated in accordance with Section 4.1 of ICC ESR 3951as applicable.

3 Allowable steel loads are based on allowable tension and shear stresses equal to 0.33X Fu and 0.17xFu, respectively.

MASONRY DESIGN TABLE

A7+
The Most Versatile Quick-Cure

Grout-filled Concrete Block: Threaded Rod Allowable Tension Loads with Reduction Factors [1,2,3,4,7,9,10,12]

Anchor Diameter (in.)	Minimum Embedment (inches)	Load at scr and c_{cr} (lb)	Spacing[5]			Edge Distance[6]		
			Critical s_{cr} (inches)	Minimum s_{min} (inches)	Load reduction factor for s_{min} [8]	Critical c_{cr} (inches)	Minimum c_{min} (inches)	Load reduction factor for c_{min} [8]
3/8	3-3/8	1,125	13.5	4	1.00	12	4	1.00
1/2	4-1/2	1,695	18	4	0.60	20	4	0.90
5/8	5-5/8	2,015	22.5	4	0.60	20	4	0.90
3/4	6-3/4	3,145	27	4	0.60	20	4	0.63

MASONRY DESIGN TABLE

A7+
The Most Versatile Quick-Cure

Grout-filled Concrete Block: Threaded Rod Allowable Shear Loads with Reduction Factors [1,2,3,4,7,9,10,12]

Anchor Diameter (in.)	Minimum Embedment (in.)	Load at s_{cr} and c_{cr} (lb.)	Spacing[5]			Edge Distance[6]		
			Critical s_{cr} (in.)	Minimum s_{min} (in.)	Load reduction factor for s_{min} [8]	Critical c_{cr} (in.)	Minimum c_{min} (in.)	Load reduction factor for c_{min} [8]
3/8	3-3/8	750	13.5	4	0.50	12	4	0.95
1/2	4-1/2	1,520	18	4	0.50	20	4	0.44
5/8	5-5/8	2,285	22.5	4	0.50	12	4	0.26
3/4	6-3/4	2,345	27	4	0.50	20	4	0.26

For SI: 1 inch = 25.4mm, 1 lbf = 0.0044 kN, 1 ksi = 6.894 MPa. (Refer to Table 4 for footnotes)

1. All values are for anchors installed in fully grouted concrete masonry with minimum masonry strength of 1500 psi (10.3 MPa). Concrete masonry units must be light-, medium, or normal-weight conforming to ASTM C 90. Allowable loads have been calculated using a safety factor of 5.0.
3. Anchors may be installed in any location in the face of the masonry wall (cell, web, bed joint).
4. A maximum of two anchors may be installed in a single masonry cell in accordance with the spacing and edge or end distance requirements. Embedment is measured from the outside surface of the concrete masonry unit to the embedded end of the anchor. See Figure 2 of ICC ESR 3951.
5. The critical spacing distance, scr, is the anchor spacing where full load values in the table may be used. The minimum spacing distance, smin, is the minimum anchor spacing for which values are available and installation is permitted. Spacing distance is measured from the centerline to centerline between two anchors.
6. The critical edge or end distance, ccr, is the distance where full load values in the table may be used. The minimum edge or end distance, cmin, is the minimum distance for which values are available and installation is permitted. Edge or end distance is measured from anchor centerline to the closest unrestrained edge.
7. The tabulated values are applicable for anchors in the ends of grout-filled concrete masonry units where minimum edge distances are maintained.
8. Load values for anchors installed less than scr and ccr must be multiplied by the appropriate load reduction factor based on actual spacing (s) or edge distance (c). Load factors are multiplicative; both spacing and edge reduction factors must be considered.
9. Linear interpolation of load values between minimum spacing (smin) and critical spacing (scr) and between minimum edge or end distance (cmin) and critical edge or end distance (ccr) is permitted.
10. Concrete masonry width (wall thickness) must be equal to or greater than 1.5 times the anchor embedment depth (e.g. 3/8-inch- and 1/2-inch-diameter anchors are permitted in minimum nominally 6-inch-thick concrete masonry). The 5/8- and 3/4-inch-diameter anchors must be installed in minimum nominally 8-inch-thick concrete masonry.
11. Allowable loads must be the lesser of the adjusted masonry or bond values tabulated above and the steel strength values given in Table 2 of ECC ESR 3951.
12. Tabulated allowable bond loads must be adjusted for increased in-service base material temperatures in accordance with Figure 1 of ECC ESR 3951.

MASONRY DESIGN TABLE

A7+
The Most Versatile Quick-Cure

Grout-filled Concrete Block: Rebar Allowable Tension and Shear Loads[1, 2, 3]

Rebar Size	Tension (lb) ASTM A615, Grade 60	Shear (lb) ASTM A615, Grade 60
No. 3	3,270	1,685
No. 4	5,940	3,060
No. 5	9,205	4,745
No. 6	13,070	6,730

For SI: 1 inch = 25.4mm, 1 lbf = 4.45N, 1ft-lbf = 1.356 N-M, 1 psi = 0.006895 MPa

1 Allowable load used in the design must be the lesser of bond values and tabulated steel element values.

2 Allowable tension and shear loads for threaded rods to resist short term loads, such as wind or seismic, must be calculated in accordance with Section 4.1 of ICC ESR 3951 as applicable.

3 Allowable steel loads are based on allowable tension and shear stresses equal to 0.33X Fu and 0.17xFu, respectively.

MASONRY DESIGN TABLE

A7+
The Most Versatile Quick-Cure

Grout-filled Concrete Block: Rebar Allowable Tension Loads with Reduction Factors[1, 2, 3, 4, 7, 9, 10, 12]

Anchor Diameter (in.)	Minimum Embedment (inches)	Load at scr and c_{cr} (lb.)	Spacing[5]			Edge Distance[6]		
			Critical s_{cr} (in.)	Minimum s_{min} (in.)	Load reduction factor for s_{min} [8]	Critical c_{cr} (in.)	Minimum c_{min} (in.)	Load reduction factor for c_{min} [8]
3/8	3-3/8	1,530	13.5	4	1.00	12	4	1.00
1/2	4-1/2	1,845	18	4	0.60	20	4	0.90
5/8	5-5/8	2,465	22.5	4	0.60	20	4	0.90
3/4	6-3/4	2,380	27	4	0.60	20	4	0.63

MASONRY DESIGN TABLE

A7+
The Most Versatile Quick-Cure

Grout-filled Concrete Block: Rebar Allowable Shear Loads with Reduction Factors [1, 2, 3, 4, 7, 9, 10, 12]

Anchor Diameter (in.)	Minimum Embedment (in.)	Load at s_{cr} and $c_{cr} \perp$ to edge (lb.)	Spacing[5]			Edge Distance[6]		
			Critical s_{cr} (in.)	Minimum s_{min} (in.)	Load reduction factor for s_{min} [8]	Critical c_{cr} (in.)	Minimum c_{min} (in.)	Load reduction factor for c_{min} [8]
3/8	3-3/8	1,410	13.5	4	0.50	12	4	0.95
1/2	4-1/2	1,680	18	4	0.50	20	4	0.44
5/8	5-5/8	3,245	22.5	4	0.50	12	4	0.26
3/4	6-3/4	4,000	27	4	0.50	20	4	0.26

For SI: 1 inch = 25.4 mm; 1 lbf = 0.0044 kN, 1 ksi = 6.894 MPa.

(The following footnotes apply to both Tables 6 and 7)

1 All values are for anchors installed in fully grouted concrete masonry with minimum masonry strength of 1500 psi (10.3 MPa). Concrete masonry units must be light-, medium, or normal-weight conforming to ASTM C 90. Allowable loads have been calculated using a safety factor of 5.0.

3 Anchors may be installed in any location in the face of the masonry wall (cell, web, bed joint).

4 A maximum of two anchors may be installed in a single masonry cell in accordance with the spacing and edge or end distance requirements. Embedment is measured from the outside surface of the concrete masonry unit to the embedded end of the anchor. See Figure 2 of ICC ESR 3951.

5 The critical spacing distance, scr, is the anchor spacing where full load values in the table may be used. The minimum spacing distance, smin, is the minimum anchor spacing for which values are available and installation is permitted. Spacing distance is measured from the centerline to centerline between two anchors.

6 The critical edge or end distance, ccr, is the distance where full load values in the table may be used. The minimum edge or end distance, cmin, is the minimum distance for which values are available and installation is permitted. Edge or end distance is measured from anchor centerline to the closest unrestrained edge.

7 The tabulated values are applicable for anchors in the ends of grout-filled concrete masonry units where minimum edge distances are maintained.

8 Load values for anchors installed less than scr and ccr must be multiplied by the appropriate load reduction factor based on actual spacing (s) or edge distance (c). Load factors are multiplicative; both spacing and edge reduction factors must be considered.

9 Linear interpolation of load values between minimum spacing (smin) and critical spacing (scr) and between minimum edge or end distance (cmin) and critical edge or end distance (ccr) is permitted.

10 Concrete masonry width (wall thickness) must be equal to or greater than 1.5 times the anchor embedment depth (e.g. No. 3 and No. 4 reinforcing bars are permitted in minimum nominally 6-inch-thick concrete masonry). No. 5 and No. 6 reinforcing bars must be installed in minimum nominally 8-inch-thick concrete masonry.

11 Allowable loads must be the lesser of the adjusted masonry or bond values tabulated above and the steel strength values given in Table 2 of ICC ESR 3951.

12 Tabulated allowable bond loads must be adjusted for increased in-service base material temperatures in accordance with Figure 1 of ICC ESR 3951 as applicable.

High Density Stone Shims

Product Information:

Use:

▪ To level and plumb stone

Dimensions:

☐ 1/16" x 2" x 2"
☐ 1/8" x 2" x 2"
☐ 1/4" x 2" x 2"
☐ 3/8" x 2" x 2"
☐ 1/2" x 2" x 2"
☐ Custom Sizes Available call 1-800-659-4731 or sales@masonpro.com

Finish:

▪ High density plastic
▪ Molded from a fire retardant engineered copolymer plastic
▪ Compressive strength of 10,000 to 12,000 psi

Advantages:

▪ Simple and economical to install
▪ Will not corrode in contact with limestone
▪ Excellent stability, eliminates rust, stained concrete, etc.
▪ Extremely long life

For technical assistance call us toll free at 1-800-659-4731.

APPENDIX B

- SUPPLEMENTAL PHOTOGRAPHS AND PAINT ANALYSIS

PHOTO 59:
TYPICAL HEALTHY TIMBER BEAM
BENEATH CREOSOTE EXTERIOR

PHOTO 60:
CORE EXAMPLE OF ROTTEN POST

PHOTO 61:
CORE EXAMPLE OF ROT IN CENTER
OF CROSS BRACING ENDS

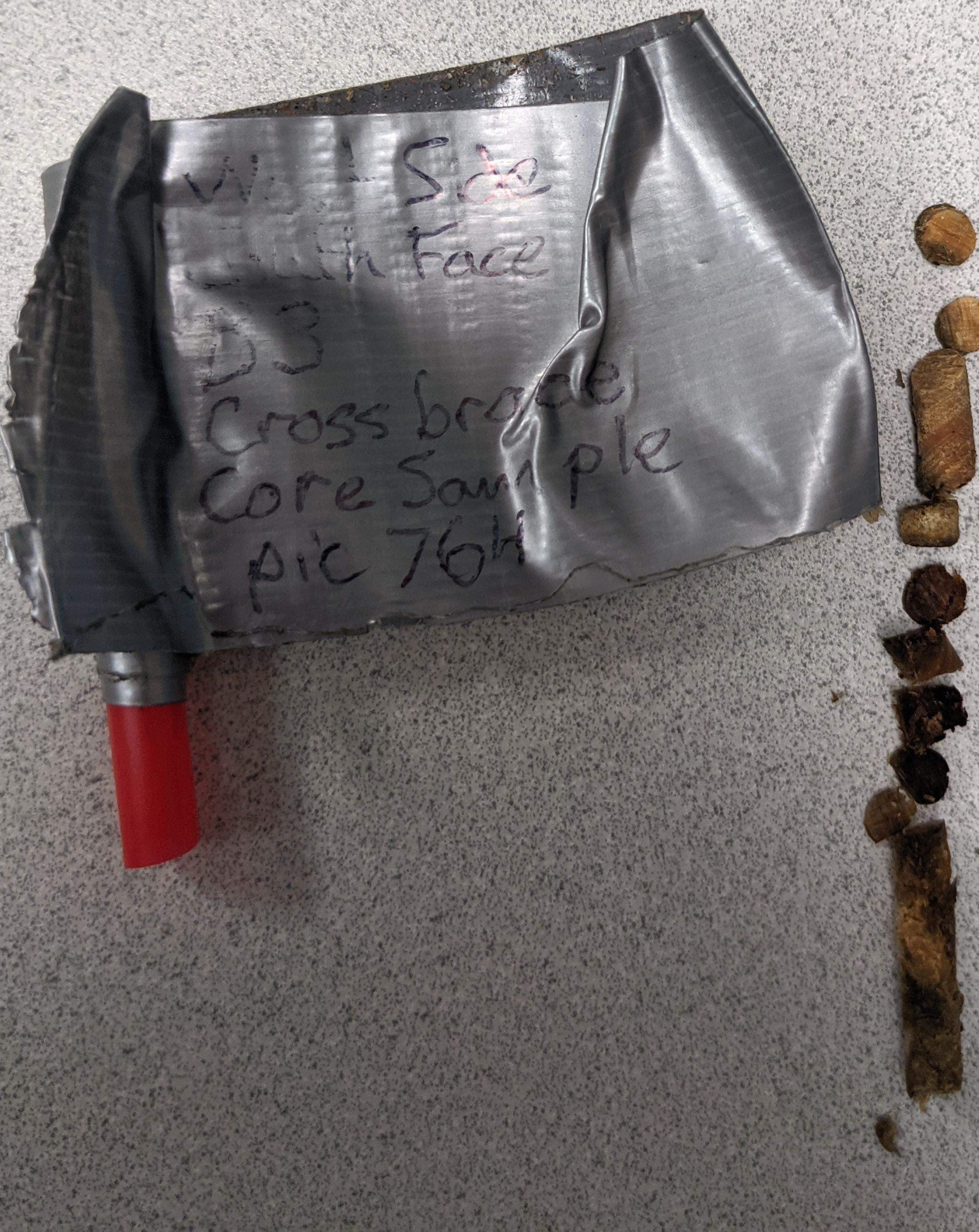


PHOTO 62:
CORE EXAMPLE OF ROTTEN POST

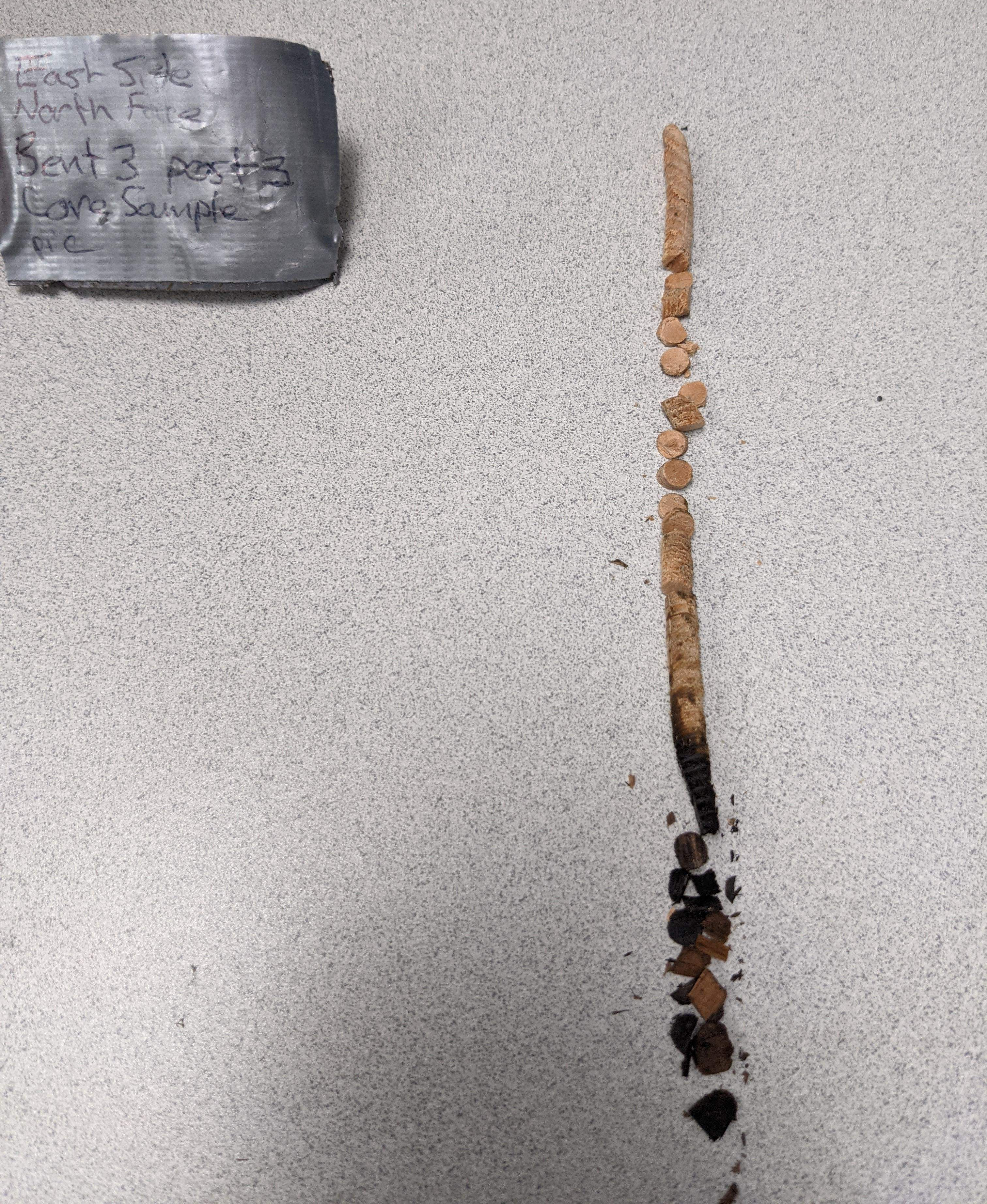


PHOTO: 63
TYPICAL RAILROAD
TIE SAMPLE

PHOTO: 64
TYPICAL RAILROAD
TIE SAMPLE

Attention: Cory Dearman

Tacten Industrial Inc.
61 Raddall Ave., Unit 0
Dartmouth, NS
Canada B3B 1T4

Report Date: 2021/04/29
Report #: R6614440
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C1A9660
Received: 2021/04/24, 13:40

Sample Matrix: Paint
Samples Received: 1

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Metals Paint Acid Extr. ICPMS	1	2021/04/28	2021/04/28	ATL SOP 00058	EPA 6020B R2 m

Remarks:

Bureau Veritas is accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Bureau Veritas are based upon recognized Provincial, Federal or US method compendia such as CCME, MELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Bureau Veritas' profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Bureau Veritas in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

Bureau Veritas liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Bureau Veritas has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Bureau Veritas, unless otherwise agreed in writing. Bureau Veritas is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested. When sampling is not conducted by Bureau Veritas, results relate to the supplied samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Bureau Veritas Laboratories 200 Bluewater Rd, Suite 105, Bedford, Nova Scotia Canada B4B 1G9 Tel: 902-420-0203 Toll-free: 800-565-7227 Fax: 902-420-8612 www.bvlabs.com

Attention: Cory Dearman

Tacten Industrial Inc.
61 Raddall Ave., Unit 0
Dartmouth, NS
Canada B3B 1T4

Report Date: 2021/04/29
Report #: R6614440
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C1A9660
Received: 2021/04/24, 13:40

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Preeti Kapadia, Project Manager
Email: Preeti.Kapadia@bureauveritas.com
Phone# (902)420-0203 Ext:252
==
This report has been generated and distributed using a secure automated process.
BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

ELEMENTS BY ATOMIC SPECTROSCOPY (PAINT)

BV Labs ID		PKE170	
Sampling Date		2021/04/15	
	UNITS	GOLD RIVER BRIDGE	RDL
Metals			
Acid Extractable Aluminum (Al)	mg/kg	1100	100
Acid Extractable Antimony (Sb)	mg/kg	ND	20
Acid Extractable Arsenic (As)	mg/kg	110	10
Acid Extractable Barium (Ba)	mg/kg	2300	50
Acid Extractable Beryllium (Be)	mg/kg	ND	20
Acid Extractable Boron (B)	mg/kg	ND	500
Acid Extractable Cadmium (Cd)	mg/kg	ND	3.0
Acid Extractable Chromium (Cr)	mg/kg	47	20
Acid Extractable Cobalt (Co)	mg/kg	ND	10
Acid Extractable Copper (Cu)	mg/kg	63	20
Acid Extractable Iron (Fe)	mg/kg	76000	500
Acid Extractable Lead (Pb)	mg/kg	73000	5.0
Acid Extractable Manganese (Mn)	mg/kg	490	20
Acid Extractable Mercury (Hg)	mg/kg	ND	1.0
Acid Extractable Molybdenum (Mo)	mg/kg	ND	20
Acid Extractable Nickel (Ni)	mg/kg	41	20
Acid Extractable Selenium (Se)	mg/kg	ND	5.0
Acid Extractable Silver (Ag)	mg/kg	ND	5.0
Acid Extractable Strontium (Sr)	mg/kg	ND	50
Acid Extractable Thallium (Tl)	mg/kg	2.3	1.0
Acid Extractable Tin (Sn)	mg/kg	ND	20
Acid Extractable Uranium (U)	mg/kg	ND	1.0
Acid Extractable Vanadium (V)	mg/kg	ND	20
Acid Extractable Zinc (Zn)	mg/kg	290	50

RDL = Reportable Detection Limit
ND = Not detected

Bureau Veritas Laboratories 200 Bluewater Rd, Suite 105, Bedford, Nova Scotia Canada B4B 1G9 Tel: 902-420-0203 Toll-free: 800-565-7227 Fax: 902-420-8612 www.bvlabs.com

APPENDIX C

- REPAIRS MANAGEMENT STRATEGY TABLE AND BUDGETARY CONSTRUCTION COST ESTIMATES

Table C.1 Bridge Repairs Management Strategy Table

ITEM	BRIDGE COMPONENT	CURRENT CONDITION	REPAIR REQUIRED	PRIORITY CODE*	ESTIMATED COST**	ESTIMATED SERVICE LIFE UNTIL REQUIRED REPAIR
1	**TIMBER TRESTLES**					
	Timber Trestle Repairs					
	Posts	Poor	Some hollow/rotten piles require replacement with kiln dried marine grade treated hemlock.	B		3 years
	Cross Bracing	Poor	Repair or replace partially rotten wooden cross bracing with kiln dried marine grade treated hemlock.	B		3 years
	Bent Caps	Good	None	D		> 5 years
	Stringers	Good	None	D		> 5 years
	Connectors	Poor	Corroded and deteriorating connectors require replacement with galvanized steel bolts	B		3 year
	Retaing Walls	Good	None	D	$1,000,000	> 5 years
	Rail Ties	Fair	Replace rotted timber rail ties with kiln dried marine grade treated hemlock	B		3 years
	Decking Repairs	Fair	Repair or replace damaged and rotted decking boards with kiln dried marine grade treated hemlock. Re-fasten boards where appriopriate with galvanized fasteners.	C		5 years
	Guards Repairs	Good	None	D		> 5 years
	Timber Trestle Replacement					
	Replace Timber Trestle Structure	N/A	Replace timber approach structures.	N/A		N/A
	Decking Repairs	Fair	Repair or replace damaged and rotted decking boards with kiln dried marine grade treated hemlock. Re-fasten boards where appriopriate with galvanized fasteners.	C	$1,400,000	5 years
	Guards Repairs	Good	None	D		> 5 years
2	**STEEL PLATE GIRDERS**					
	Top Flange	Poor	Local rusting requires replacement	A		1 year
	Girders	Good	None	D		>5 years
	Cross Bracing	poor	Deteriorated members to be removed or replaced	B	$350,000	3 years
	Gusset Plates	Fair/Poor	Deteriorated members to be removed and replaced	A		1 year
	Bottom Flange	Fair	Local rusting requires replacement	B		3 years
3	**BEARINGS**					
	Expansion	Very Poor	Removal & replacement	A	$250,000	1 year
	Fixed	Poor	Removal & replacement	A		1 year
4	**STONE MASONRY PIERS**					
	Masonry Stones	Fair	Reset dislodged granite, repair broken granite, reinstalling pier caps to original position, scaffolding & general, optional cleaning	A	$200,000	1 year
	Masonry Stones facade	Good	Cleaning of stones is optional and for aesthetic puroposes only. (Therefore cost is omitted in total sum)		$150,000	NA
	Mortar	Very Poor	Remove vegetation and perform 100% repoint, scaffolding & general	A	$900,000	1 year

Summary:		
Option 1: MAKE ALL NECESSARY REPAIRS TO EXISTING BRIDGE		
Timber Trestle Replacement (repair deemed impractical)		$1,400,000
Steel Girder Repairs		$350,000
Bearings Replacement		$250,000
Stone Masonry Piers Refurbishment		$1,100,000
	Total:	**$3,100,000**
Option 2: REPLACE ENTIRE STRUCTURE AT EXISTING SITE		
Removal & Disposal of Existing Steel Girders		$700,000
Removal & Disposal of Existing Timber Trestle Structure		$100,000
Replacement Bridge Structure (4 spans)		$734,000
Replacement Bridge Installation and Bearings		$800,000
Stone Masonry Piers Refurbishment or Replacement		$1,100,000
Strengthening of east and west piers as they support two spans rather than one span one trestle		$400,000
New Bridge Abutments to support new spans in place of trestles		$200,000
	Total:	**$4,034,000**
Option 3: ABANDON AND DECOMISSION EXISTING STRUCURE AND PROVIDE REPLACEMENT STRUCTURE AT A NEW LOCATION		
Removal & Disposal of Entire Structure		$1,000,000
Replacement Bridge Structure (Fabrication Only)		$350,000
New Bridge Abutments		$250,000
Replacement Bridge Installation		$400,000
Construction of new pedestrian paths to accommodate new pedestrian bridge location		$500,000
Costs associated with land acquisition/agreement with provincial highways for construction in or near highway ROW		$500,000
Expropriation Costs		$500,000
	Total:	**$3,500,000**
Option 4: ABANDON AND DECOMISSION EXISTING STRUCTURE WITHOUT REPLACEMENT		
	Total:	**$1,000,000**

***Priority Code for Component Repair of a Structure:**

Bridge Inspection Manual, 2008, by Public Works and Government Services Canada - Section 2.3

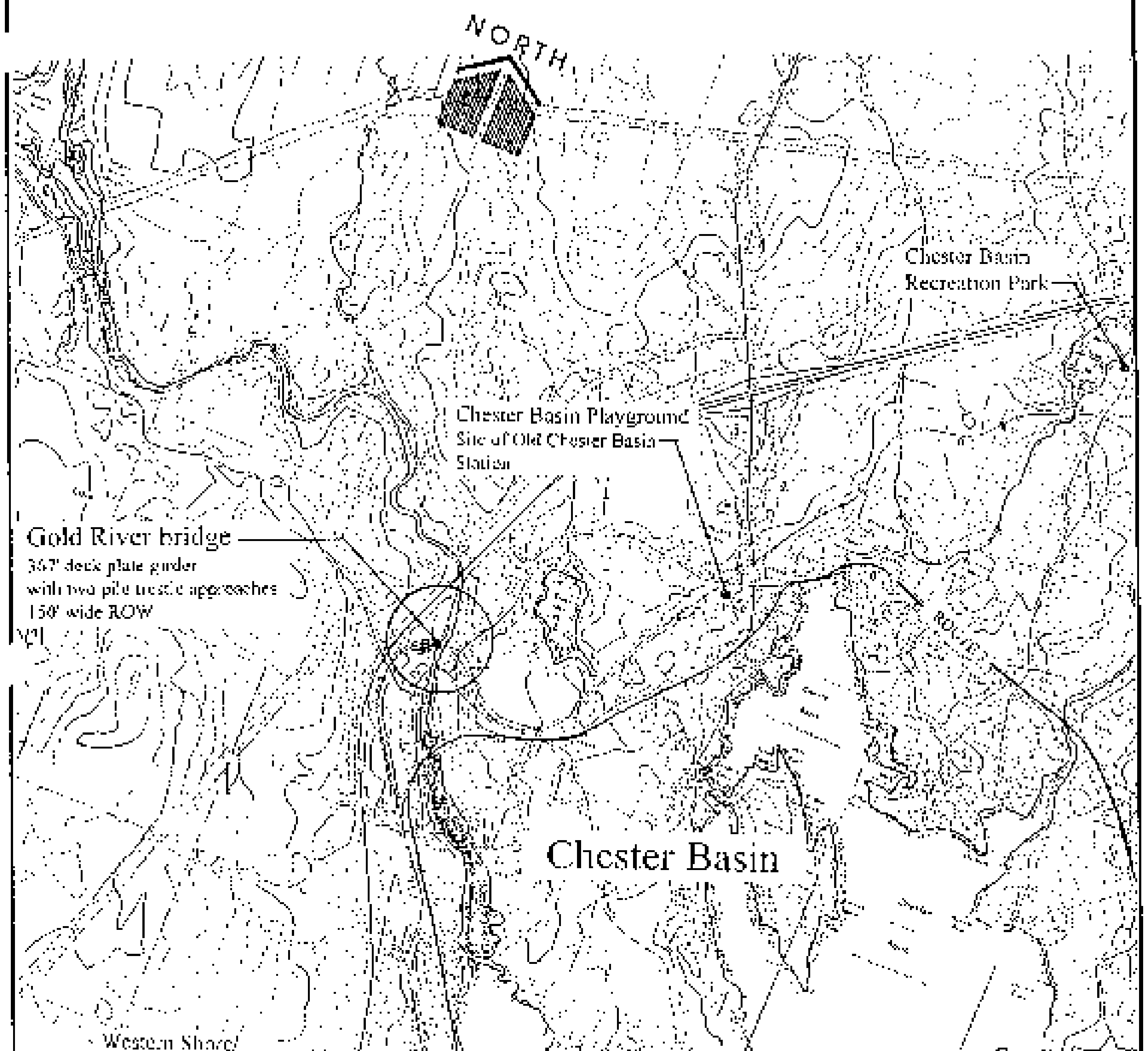
U Urgent requires immediate attention and remedial measures to ensure public safety
M Required work to be done as part of routine annual maintenance
S Further sutdy/investigations/surveys required prior to initiating repair programme
A Repair and/or replacement to be done in less than 1 year
B Repair and/or replacement to be done in less than 3 year
C Repair and/or replacement to be done in less than 5 year
D Conditions to be re-assessed at the next inspection

****Estimated Cost:**

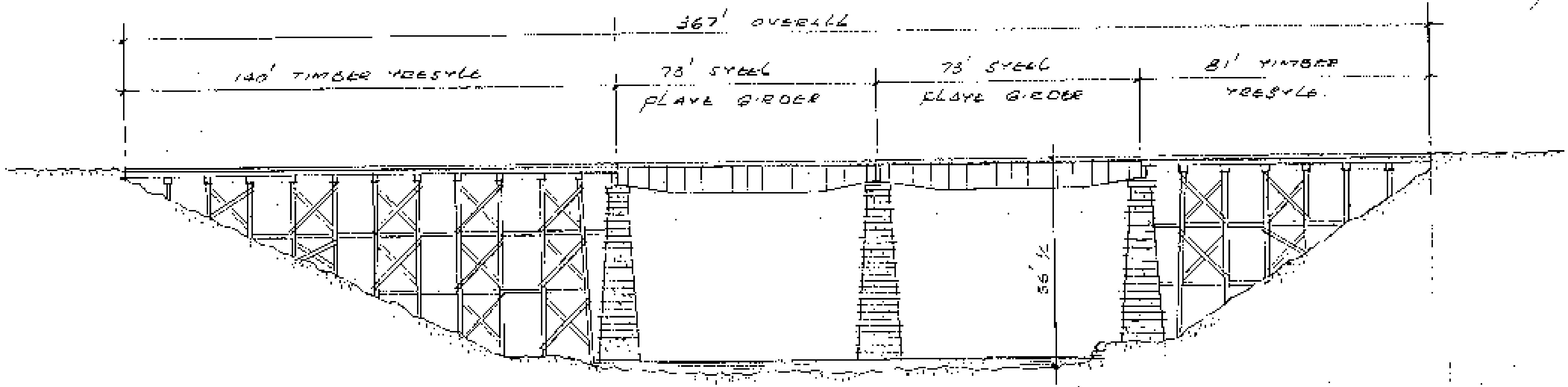
1. Costs indicated are rough order of magnitude only

APPENDIX D

- **WAUGH ASSOCIATES LTD. 2001 GOLD RIVER BRIDGE DRAWINGS**

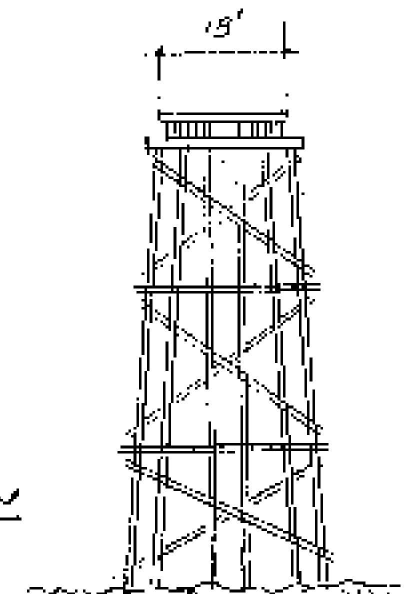


Waugh Associates Ltd. Consulting Civil and Structural Engineers Phone: (902) 466-0997 Fax (902) 465 4892	PROJECT: GOLD RIVER BRIDGE EVALUATION CHESTER, NOVA SCOTIA	DATE: JAN 2000 SCALE: 1:30,000	PROJECT NO: 20032
	SHEET TITLE: LOCATION PLAN	DRAWN/CHECKED: WAW	SKETCH NO: SK-GRB-1

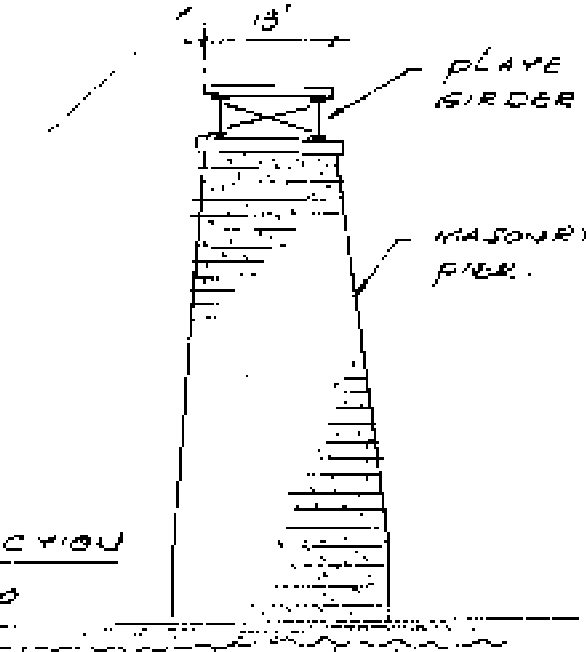


SOUTH ELEVATION
1" = 30'-0

BRIDGE PLAN
1" = 30'-0



TRESTLE SECTION
1" = 20'-0



PIER SECTION
1" = 20'-0

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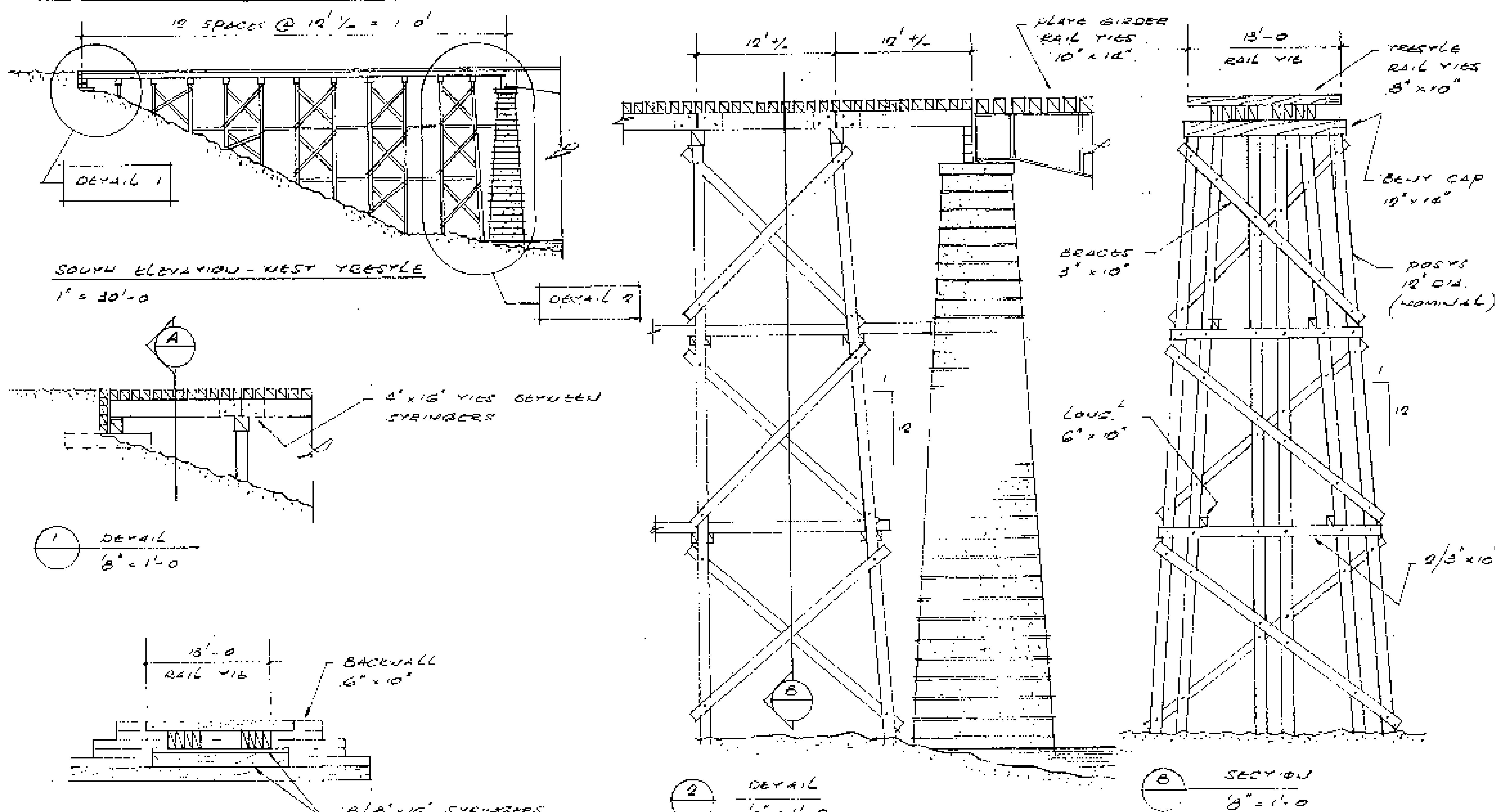
PROJECT	GOLD RIVER BRIDGE EVALUATION CHESTER, NOVA SCOTIA
SHEET TITLE	SOUTH ELEVATION, PLAN, TRESTLE AND PIER SECTIONS
DATE	JAN 2000
SCALE	AS NOTED
DRAWN/CHECKED	WAW
PROJECT NO.	20032
SKETCH NO.	SK-GRB-2



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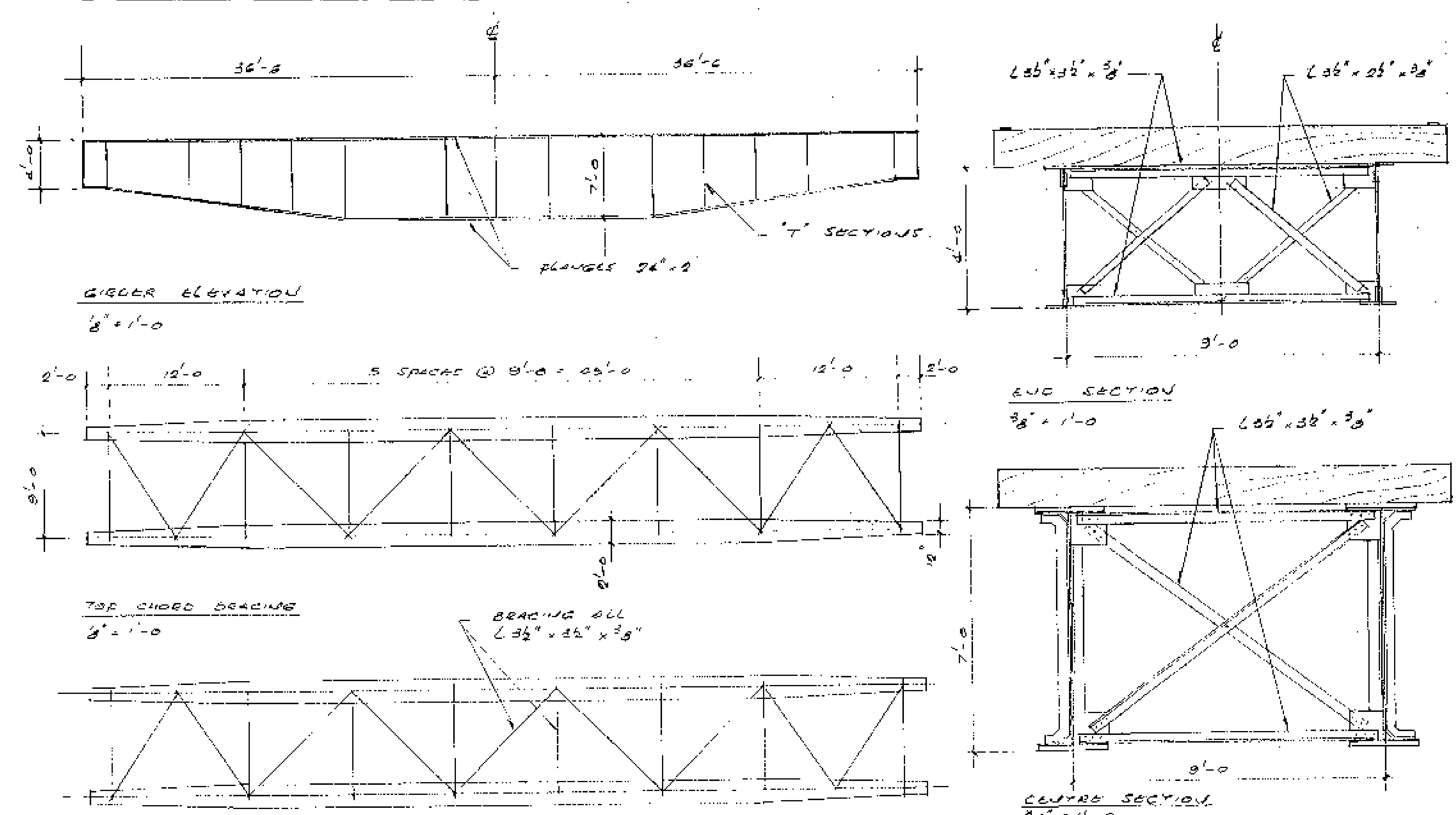
PROJECT: GOLD RIVER BRIDGE EVALUATION CHESTER, NOVA SCOTIA	DATE: JAN 2000		PROJECT NO: 20032
	SCALE: AS NOTED		
SHEET TITLE: WEST TRESTLE ELEVATION, SECTIONS AND DETAILS	DRAWN/CHECKED: WAW		SKETCH NO: SK-GRB-3



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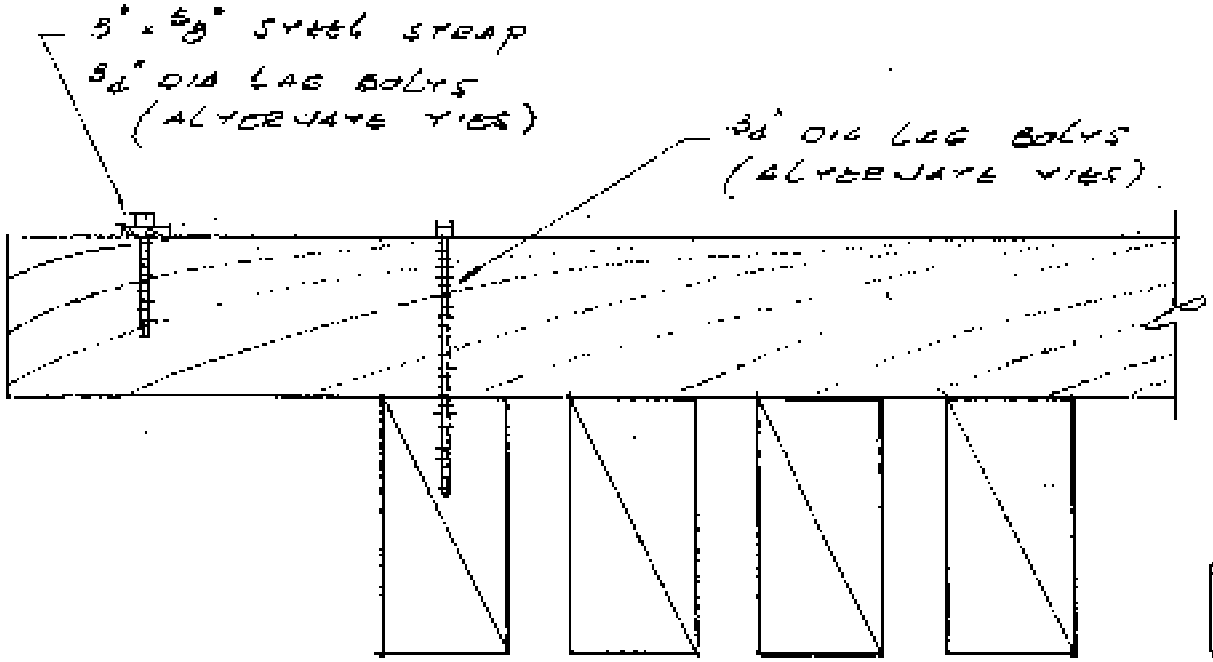
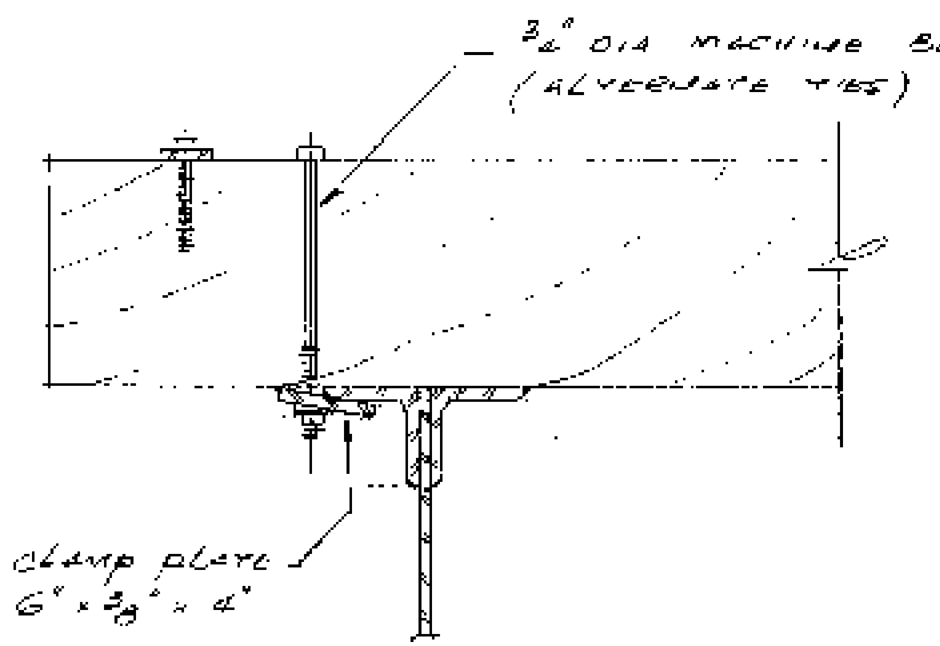
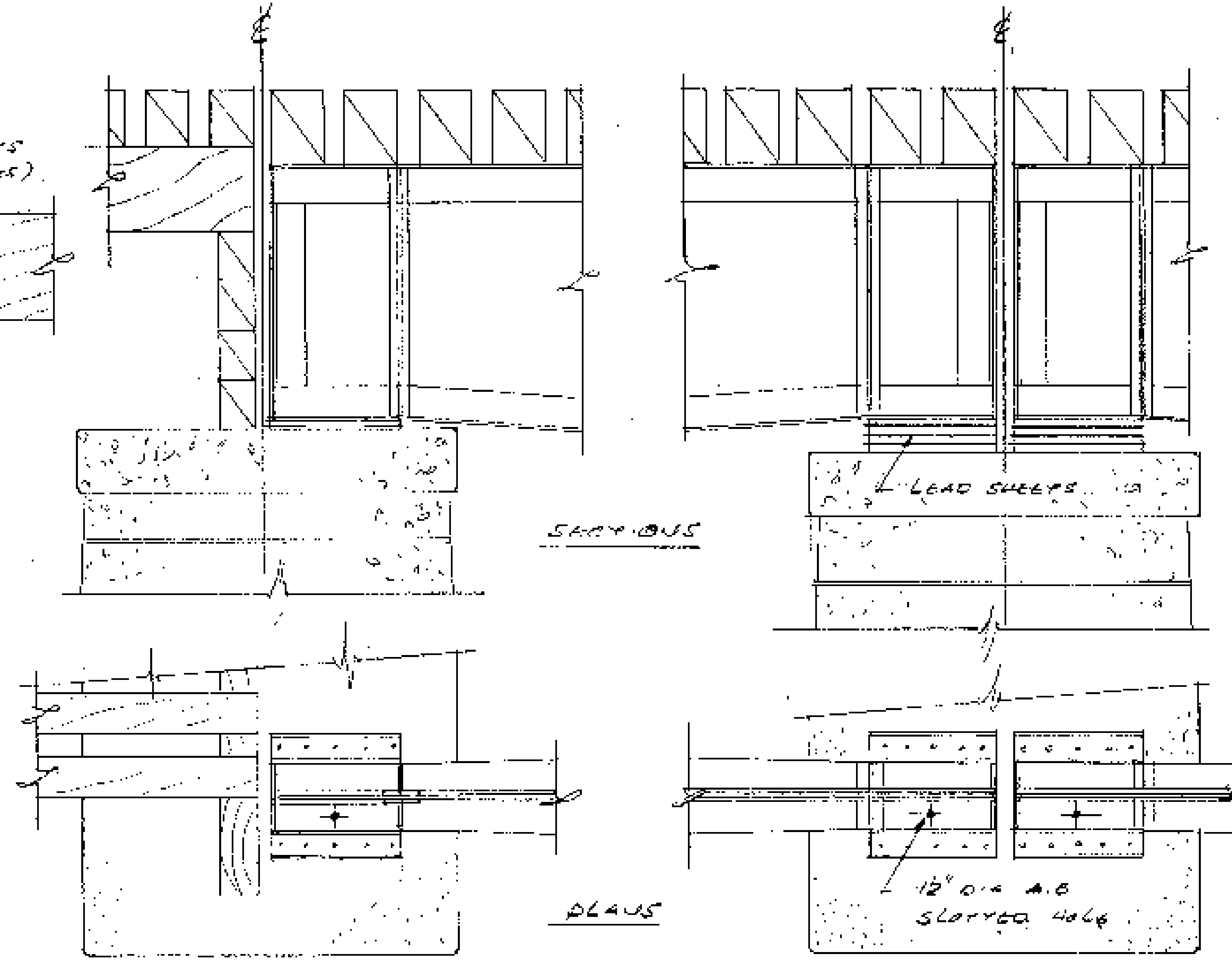
PROJECT: GOLD RIVER BRIDGE EVALUATION CHESTER, NOVA SCOTIA	DATE: JAN 2000	PROJECT NO: 20032
SHEET TITLE: GIRDER ELEVATION, END AND CENTRE SECTIONS AND BRACING	SCALE: AS NOTED	SHEET NO: SK-GRB-4
	DRAWN/CHECKED: WAW	



RAIL TIE / STRINGER CONNECTION
1" = 1'-0"



RAIL TIE / PLATE GIRDER CONNECTION
1" = 1'-0"



BEARING DETAILS
1/2" = 1'-0"

Waugh Associates Ltd. Consulting Civil and Structural Engineers Phone: (902) 466-0997 Fax: (902) 466-4692	PROJECT GOLD RIVER BRIDGE EVALUATION CHESTER, NOVA SCOTIA	DATE JAN 2000	PROJECT NO 20032
	SHEET TITLE RAIL TIE CONNECTIONS BEARING DETAILS	SCALE AS NOTED DRAWN/CHECKED WAW	SKETCH NO SK-GRB-5