Metrolinx Manual of Practice for the Maintenance of Railway Structures

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PREFACE

This is the first edition of the Metrolinx Manual of Practice for the Maintenance of Railway Structures (RC-0506-04STR-02). It is adapted from CN Manual of Practice for the Maintenance of Railway Structures as per the agreement between Metrolinx and CN on March 28, 2013. In accordance with the agreement, METROLINX is authorized to affix the name of Metrolinx to the CN Standards, shall remove all references to CN and update / modify the standards to Metrolinx Standards.

The purpose of Metrolinx Manual of Practice for the Maintenance of Railway Structures is to provide guidelines for the design and preparation of drawings for the maintenance of railway structures. It will be useful to design personnel as it provides basic technical background and typical design solutions to the most common maintenance problems. It will also be useful to field personnel for planning and execution purposes.

In addition to the eleven chapters, which provide very basic technical background on bridge maintenance, a library of figures and standard drawings is also included to assist in the preparation of drawings. Some of them can also be used by supervisors for job planning and even for the execution of the work without the use of any other drawings.

The drawings of the MF series, which cover fixed fall protection equipment, also include instructions on the use of this equipment that should be incorporated into fall protection procedures.

The instructions and guidelines in this manual apply to situations that often occur at Metrolinx; however, it should be noted that they might not be applicable in certain specific cases. Proper judgment is required at all times when using this manual. If any doubts, the Senior Manager of Track and Structures of the Corridor Maintenance should be contacted.
A consistent approach in the application of Metrolinx Manual of Practice for the Maintenance of Railway Structures will reduce disputes during the design and construction phases of a project, enhance the long term safety, reliability and extend the useful service life of the infrastructure.

Note
This manual is not intended to modify existing company policies or standard practices. We believe that the content complies with all Standard Practice Circulars, Standard Drawings and other official documentations. However, if any information included in this manual were to contradict Metrolinx official documentation, the official documentation shall prevail.

Suggestions for revisions and improvement
Suggestions for revision or improvement can be sent to the Senior Manager of Track and Structures, Corridor Maintenance. Please include a description of the proposed change, background of the application and any other useful rationale or justification. Please include your name, company affiliation (if applicable), email address and phone number.

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1 INTRODUCTION

1.1 INTRODUCTION

1.1.1 This manual contains eleven chapters and a library of standard figures and drawings dedicated to the maintenance of railway structures. The information given in this manual has been selected in order to facilitate the design and preparation of drawings for maintenance work; moreover, it is presented in a very simple and concise form. This manual can therefore be useful to both design and field personnel.

1.1.2 The topics covered in this manual are:

(a) General instructions for the preparation of drawings
(b) Maintenance of steel structures (MS drawing series)
(c) Maintenance of concrete structures (MC drawing series)
(d) Maintenance of timber structures (MT drawing series)
(e) Maintenance of masonry structures (MM drawing series)
(f) Maintenance of bridge decks and walkways (MD drawing series)
(g) Maintenance of bearings
(h) Maintenance of bridge approaches
(i) Maintenance of culverts
(j) Fall protection and scaffolding (MF drawing series)

1.1.3 Standard figures include useful information such as detailing standards, weight of spans, information on interpretation of inspection reports, and required clearances for equipment.

1.1.4 Standard drawings include pre-designed solutions to some of the most common problems that must be resolved in bridge maintenance.

1.1.5 Included for design personnel are very basic and brief summaries of the technical background which forms the basis of maintenance standards and practices. Other references will have to be consulted for more details. Standard figures will be useful for design and drafting purposes. Standard drawings are intended to reduce the time required to produce design solutions.

1.1.6 Sufficient information has been included for field personnel to understand the technical background which forms the basis of maintenance standards and practices. Some of the standard figures can assist in job planning and in the examination of drawings. Some of the standard drawings can be used as is to execute some of the maintenance work without any other further or additional drawings.

1.2 ENVIRONMENTAL ASSESSMENT

1.2.1 Once the proposed works are determined, appropriate environmental due diligence studies shall be undertaken to ensure environmental impacts and mitigation measures are identified. These studies may include, but are not limited to:

(a) Air Quality

(b) Natural Heritage

(c) Noise and Vibration

(d) Spill Prevention and Contingency

(e) Cultural Heritage

(f) Designated Substances and Hazardous Materials Surveys

(g) Archaeology

(h) Soil Management

(i) Landscaping

2 GENERAL INSTRUCTIONS FOR THE PREPARATION OF DRAWINGS

2.1 LETTERING

2.1.1 The complete set of drawings shall be in two different formats – ADOBE ACROBAT “PDF” and AUTOCAD “DWG” format, with CTB file included, in accordance with Metrolinx CADD-BIM Manual.

2.1.2 Capital letters (uppercase) shall be used at all times except for assembly marks on steel drawings, where minuscules (lowercase) must be used.

2.1.3 Size of text shall be 3.5 mm when printed.

2.1.4 As far as practicable, dimensions and reference lines are to be placed so that they do not come too close or intersect other dimensions, reference lines, details or text.

2.1.5 All dimensions are to have a foot mark and a decimal point when expressed in decimal feet or a foot mark and a dash when expressed in feet and inches. Do not use inch marks (Ex: 24.5', 24'-6).

2.1.6 Use an equal sign in giving elevations (Ex: ELEV = 100.18). No foot mark is necessary on elevations.

2.1.7 As far as practicable, metric units are to be used. Metric units shall be in millimeters, unless otherwise noted on the drawings.

2.2 LINE WEIGHTS

2.2.1 Existing material shall be shown with faint lines (Ex: line weight = 0 or 1) and new material shall be shown with heavy lines (Ex: line weight = 2, 3 or 4).

2.3 DRAWING SCALES

2.3.1 As far as practicable, metric units are to be used. However, steel repair drawings shall be prepared using the same unit system as the original shop drawings. Drawings for timber bridges and timber decks usually employ the imperial system.

2.3.2 The following scales are recommended:

Table 2-01: Recommended Drawing Scales

IMPERIAL		METRIC	
Steel details	1" = 1'0	Steel details	1:10
Concrete details	¼” = 1'-0	Concrete details	1:50
Reinforcing steel details	¼” = 1'-0	Reinforcing steel details	1:50

2.3.3 The scales above apply to the main views on drawings. No smaller scales shall be used. However, larger scales may be used when required to clearly illustrate small or intricate details.

2.4 TRACK DIRECTIONS

2.4.1 In referring to tracks carrying main-line traffic on drawings, "Eastward Main Track" and "Westward Main Track" are no longer used. Simply refer to "North Main Track" or "South Main Track" on an east-west line or to "West Main Track" or "East Main Track" on a north-south line.

2.4.2 Do not use "Mainline Track," use "Main Track."

2.4.3 The direction of track shall be indicated on drawings as follows:

Westward to Aldershot ← **Eastward to Oshawa** →

Do not say:

From Aldershot ← **From Oshawa** →

2.4.4 The direction of increasing mileage shall always be shown on the left side of the drawing. When showing direction of track, indicate the last station at each end of the subdivision. If the drawings are to be used by contractors who are not familiar with the territory, it might be preferable to use city names rather than station names.

2.5 STANDARD ABBREVIATIONS FOR RAILWAY SPANS

As per SPC 4000 Commentary

Table 2-02: Timber Bridge Abbreviations

Ballast Deck Frame Trestle	BDFT
Ballast Deck, Masonry Walls, Wood Stringers	BDMW
Ballast Deck Pile Trestle	BDPT
Ballast Deck, Wood Walls, Wood Stringers	BDWW
Concrete Piers, Wood Stringers	CPWS
Frame Bent Trestle on Piles	FPT
Frame Bent Trestle on Concrete Pedestals	FTCP
Frame Bent Trestle on Mud Sills	FTMS
Frame Trestle on Wood Cribs	FTWC
Masonry Walls, Wood Stringers	MWWS
Open Deck Pile Trestle	PT
Timber Bridge	T
Wood Cribs, Wood Stringers	WCWS
Wood Walls, Wood Stringers	WWWS

Table 2-03: Concrete Bridges Abbreviations

Concrete Bridge	C
Concrete Arch (not reinforced)	CA
Continuous Prestressed Concrete Slabs	CPCS
Continuous Post-Tensioned Concrete Slabs	CPTCS
Continuous Reinforced Concrete Beams	CRCB
Continuous Reinforced Concrete Frames	CRCF
Continuous Reinforced Concrete Slabs	CRCS
Masonry Arch	MA
Prestressed Concrete Girders	PCG
Prestressed Concrete Slabs	PCS
Prestressed Double-Voided Box Girders	PDVBG
Prestressed Single-Voided Box Girders	PSVBG
Post-Tensioned Concrete Girders	PTCG
Post-Tensioned Concrete Slabs	PTCS
Post-Tensioned Double-Voided Box Girders	PTDVBG
Post-Tensioned Single-Voided Box Girders	PTSVBG
Reinforced Concrete Arch	RCA
Reinforced Concrete Beams	RCB
Reinforced Concrete Frames	RCF
Reinforced Concrete Slabs	RCS
Reinforced Concrete Trestle	RCT

Table 2-04: Steel Bridge Abbreviations

Arch	ARCH
Beam Span	BM
Continuous Deck Plate Girder Spans	CDPG
Deck Plate Girder, Composite Concrete Deck	DGCD
Deck Plate Girder, Timber Pile Piers	DGPP
Deck Plate Girder	DPG
Deck Plate Girder, Lift Span	DPGL
Deck Plate Girder, Swing Span	DPGS
Deck Plate Girder, Steel Viaduct	DPGV
Deck Truss Span	DT
Deck Truss, Lift Span	DTL
Deck Truss, Swing Span	DTS
Deck Truss, Steel Viaduct	DTV
Half Through Plate Girder Span	HTPG
Pony Truss	PYT
Steel Stringers, Timber Frame Bents on Piles	SSFP
Steel Stringers, Timber Frame Bents on Sills	SSFS
Steel Stringers, Timber Pile Bents	SSPT
Steel Stringers, Wood Cribs	SSWC
Steel Bridge	STL
Steel Trestle, Steel Plate Ballast Trough	STSD
Through Plate Girder Span	TPG
Through Plate Girder, Lift Span	TPGL
Through Plate Girder, Swing Span	TPGS
Through Plate Girder Span, Steel Viaduct	TPGV
Through Truss Span	TT
Through Truss, Lift Span	TTL
Through Truss, Swing Span	TTS

For culvert abbreviations see Standard Practice Circular 4402.

3 MAINTENANCE OF STEEL STRUCTURES

3.1 DESIGN OF STEEL REPAIRS

3.1.1 The following steps are to be followed when designing a steel repair.

(a) Determine the scope of the work to be done on the bridge
(b) Evaluate the inspection report to determine what has to be repaired
(c) Select the appropriate repair detail for the deteriorated members
(d) Prepare drawings

3.2 SCOPE OF THE WORK

3.2.1 It is important to determine what the scope of the work is before any attempt is made at designing the repair. It shall be agreed upon by the individuals responsible for maintaining the bridge plant, such as the Bridges & Structures Manager and Assistant Manager. The scope of the work can usually be classified in one of the following categories:

(a) Minimal repairs that prolong the life of the bridge by less than 5 years
(b) Normal repairs that prolong the life of the bridge by approximately 15 years
(c) Thorough repairs that prolong the life of the bridge by 25 years or more

3.2.2 The scope of the work is affected by factors such as the traffic on the line, the future of the line, the future of the bridge, the overall condition of the bridge plant and all other factors given in AREMA Chapter 15, Section 7.4.1. The additional cost of upgrading the scope shall also be considered. For example, on certain bridges, the cost of thorough repairs might not be much more than the cost of normal repairs.

In general, where repairs are carried out, a thorough repair shall be undertaken.

3.3 EVALUATION OF INSPECTION REPORTS

3.3.1 Steel bridges are inspected as per Standard Practice Circular 4000. The condition of the bridge is noted on the inspection report. The most common problems found on steel bridges are corrosion and cracking of steel members. Other problems are also given in the SPC 4000 Commentary.

3.3.2 To determine required repairs, it is important to differentiate between primary and secondary members. Primary members are the ones which carry the train load, such as trusses, girders, floor beams and stringers. Flanges and webs which form these members are considered to be primary members. These members can be evaluated through simple structural analysis. Bracing, gusset plates and stiffeners fall into the category of secondary members. These members cannot be evaluated accurately by simple structural analysis. For the purpose of designing steel repairs, flange angles are sometimes considered to be secondary members when evaluating the capacity of bottom flanges over the bearings to resist cracking in the fillet, as well as the capacity of top flanges under the ties to resist moon-shape cracking of the horizontal leg. This is justified by the fact that these cracks cannot be explained by simple structural analysis.

3.3.3 The capacity of the primary members is evaluated by the Rating Consultant. Their recommendations are given in a report that indicates precisely which members need to be repaired or strengthened.

3.3.4 The 50% rule is used as a guideline in determining which members need to be repaired. This rule only applies to secondary (non-structural) members. This rule states that if a member is cracked or has corroded to half of its original thickness, consideration shall be given to repairing it. This is not a rule that has to be strictly enforced. It is a guideline for making a preliminary evaluation of each member. The Consultant/designer will be responsible in making the final decision on whether or not to repair a member. The 50% rule shall be adjusted either up or down to account for the following factors:

(a) the scope of the work
(b) the environment (mild or corrosive)
(c) the traffic on the bridge
(d) the condition of the paint
(e) the other work done in the vicinity

3.3.5 Before evaluating the inspection report, the designer shall ensure they possess the latest inspection report and determine with the Bridge Specialist how to interpret the 50% rule. Additional information on interpretation of steel inspection reports are given on Figures 19 and 20.

3.4 SELECTION OF REPAIR DETAILS

3.4.1 Repairs to steel bridges consist largely in replacing secondary members in kind. Some more complicated details which are often used, such as splicing flange angles, web repairs, stiffener repairs, jacking spans, shimming, replacing anchor bolts, etc., have been standardized in the MS drawing series and are included in this document.

3.4.2 Before repairing or replacing cracked members, it is important to identify what has caused the crack. Additional retrofitting might be required in order to prevent the new member from cracking in the near future. If the crack was caused by section loss due to corrosion, it is usually adequate to replace the member in kind.

3.4.3 Repairs that are not typical shall be designed according to Chapter 15 of the American Railway Engineering and Maintenance of Way Association (AREMA) Manual.

3.4.4 The size of strengthening materials is usually given in the report produced by the Rating Consultant. Connections and other details shall be designed according to the AREMA Manual.

3.5 PREPARATION OF REPAIR DRAWINGS

3.5.1 Before any drafting is attempted, it is important to verify that the original existing, as-built and shop drawings that will be used for the design actually represent what is on site. Check all sources of information (inspection report, bridge list, photos, plan files, bridge folder and correspondence file if necessary) to make sure that no work has been carried out, such as strengthening, alterations or replacements, since the bridge was first built. If any records are misleading or incorrect, make a note of correction or explanation wherever the misinformation occurs with date and initial.

3.5.2 Standard size of drawings (559 mm X 864 mm (22 in x 34 in)) METROLINX Title Block, in accordance with the design requirements manual (CI-0705), shall be used at all times. All drawings shall be numbered and dated unless otherwise instructed. Drawing numbers are to be given by the Bridge Specialist at some time during the preparation of the drawings. All drawings issued prior to signing by the Engineer shall be stamped "PRELIMINARY."

3.5.3 The original issue of the drawings is not identified with a revision letter. A comment can still be indicated in the revision block such as "PRELIMINARY" or "ISSUED FOR TENDER." Revisions of the drawings are then identified by the letters A, B, C, etc., and are indicated in the bottom right corner of the drawing. Revised items shall be identified with a revision mark to be located as close as possible to the revision. A description of the revision such as "GAUGE ON A2" is given in the revision block. Alternatively, a comment such as "ISSUED FOR FABRICATION" can be indicated. If too many items are revised and the description does not fit in the revision block, then the comment "GENERAL" can be used instead.

3.5.4 An erection diagram shall always be provided. When several drawings have to be prepared for steel repairs, the erection diagram shall always be shown on the first sheet. If the bridge is very long or many repairs are required, it may be preferable to use two drawings for the erection diagram. Refer to Figure 13 for additional information.

3.5.5 Steel details shall be made according to the standard figures. The members shall be marked as shown on Figure 12.

3.5.6 When preparing repair drawings, attention shall always be given to maintaining the structural integrity (see also Sections 3.20 and 3.21) of this manual. Pay particular attention to the removal of main members and their connections, splices and bracing. Often, splice plates will not be identified as such on the inspection report; however, they must be recognized by the designer. Construction procedures shall be given when required to identify the work that must be completed under work blocks or slow orders.

3.5.7 When the drawings are completed, the detailer gives checking prints and all other information used to prepare the drawings to the checker. The checker must verify that they have the most recent inspection report and all pertinent drawings, including strengthening drawings or other modifications if any, of the existing structure. The checker reviews all the information on the drawing and notes what they believe is incorrect. When the checked drawings are returned to the detailer, the latter must confirm that all information noted as incorrect by the checker is indeed so. The correctness of the checker's notes shall not be taken for granted. Once corrected the drawings and checking prints are resubmitted to the checker, who will once again review the drawings. The checker is responsible for the ultimate correctness of anything they mark correct, and the detailer is responsible for anything the checker marks incorrect.

3.5.8 When the project is completed in the field, an appropriate stamp is put on the erection diagram drawing. The stamp shall indicate if the project has been completed in whole or in part along with the completion date. Any work that has been done differently than as indicated on the drawing shall be clearly identified as such. A revision of the erection diagram drawing shall be issued with the comment "AS CONSTRUCTED." Revisions shall also be made to the detail drawings if any changes have occurred during construction. Work completion comments and revision notes can be printed by hand. The drawings shall be given to the Manager Bridges and Structures for filing.

3.5.9 The following information shall be provided on drawings for steel repairs, strengthening or shimming:

(a) Erection diagram
(b) Standard notes and specifications
(c) Details of new material and weights
(d) Profile of proposed and existing base of rail when applicable
(e) Field details
(f) References to standard drawings
(g) Construction procedures
(h) Estimated quantities
(i) Weight of spans to be jacked

3.6 ASSEMBLY, SHIPPING AND ERECTION MARKS

3.6.1 The following system is to be adopted for detailing structural steel members. An example is given on Figure 12 showing how to indicate assembly, shipping and erection marks.

3.6.2 ASSEMBLY MARKS

(a) Only the following MINUSCULE (lowercase) letters shall be used:
  (i) For angles: a1, a2, a3, a4, a5, etc.
  (ii) For plates: p1, p2, p3, p4, p5, etc.
  (iii) For beams: b1, b2, b3, b4, b5, etc.
  (iv) For all others: m1, m2, m3, m4, m5, etc.

(b) Assembly parts on a steel member shall be billed as shown in following examples:
  (i) 1-L 6 x 6 x ½ x 8 - a1
  (ii) 1-PL 12 x ½ x 2'-3 - p1
  (iii) 1-W 12 x 27 x 20'-0 - b1

(c) The quantity is shown first, the member size is shown next and the assembly mark is indicated last. To indicate Right or Left assembly part, simply add the letter R or L to the assembly mark as shown in the following examples:
  (i) 1-L 6 x 6 x ½ x 8 - a1R
  (ii) 1-L 6 x 6 x ½ x 8 - a1L
  (iii) 2-L 6 x 6 x ½ x 8 - a1RL

(d) An example is given on Figure 12 showing how to indicate assembly, shipping and erection marks.

3.6.3 SHIPPING AND ERECTION MARKS

(a) The fabricator shall clearly indicate all shipping and erection marks on all pieces. They have two functions:
  (i) They are used to identify each piece of steel, and
  (ii) They are shown on the erection diagram to indicate the location of the member on the bridge.

(b) The shipping and erection marks also identify the drawing on which the member is detailed. For example, on Drawing S-125, the marks shall be A6, 86, C6, etc.

(c) Only the following <u>CAPITAL</u> letters shall be used:
  (i) A, B, C, D, E, F, G, H, K, M, N, P, S, T, W, X, Y
  (ii) If additional marks are required, use AA, AB, AC, AD, etc.

(d) To indicate Right or Left erection marks, use the following examples:
  (i) When two or more members are exactly right and left, use
  <u>ONE – A1^{R} - RIGHT</u>
  <u>ONE - A1^{L} - LEFT</u>
  (ii) When two or more members are practically (but not exactly) one opposite of the other, they can be detailed together on the same view. The shipping marks shall then read:
  <u>ONE - F2 - RIGHT</u>
  <u>ONE - G2 - LEFT</u>
  (iii) In this case, the R and L notation is not used after the marks F2 and G2.

3.7 UNIT WEIGHTS

3.7.1 The unit weight of all steel pieces identified by a shipping and erection mark must be indicated on the drawings. An example is given on Figure 12 showing how to indicate the weight.

3.7.2 Two different weights are used in the steel fabrication industry

(a) Gross weight: The weight of steel required by the fabricator to fabricate the piece

(b) Shipping weight: The actual weight of the fabricated piece

3.7.3 The shipping weight is equal to the gross weight minus the weight of the scrap steel in cuts or copes. Theoretically, the weight of the bolt holes should be subtracted in order to get the shipping weight, however this is not done in practice. The weight of the paint is also neglected in calculating the shipping weight.

3.7.4 The shipping weight is the one indicated on the drawings.

3.8 STANDARD NOTES AND SPECIFICATIONS FOR STEEL REPAIR

3.8.1 The specifications for steel repairs shall be as follows:

Table 3-01: Steel Repair Specifications

- DESIGN AND WORKMANSHIP:	AREMA (year) Chapter 15
- STRUCTURAL STEEL:	CAN/CSA G40.20-13/G40.21-13 (Latest Revision) or equal
- BOLTS:	(Unless noted otherwise) 7/8" dia. high strength ASTM A325-14 with heavy hex. nut and hardened round washer. Nuts to be tightened by using the turn-of-nut method.
- WELDING:	CSA W59 (Latest Revision).
- HOLES:	15/16" dia. (unless noted otherwise)
- PAINT:	See paint specifications

3.8.1 The standard notes for steel repairs shall be as follows:

REFERENCES:	EDRMS DWG's Nos. S-XXX Inspection report dated *year/month/date* Metrolinx General Guidelines for Design of Railway Bridges and Structures
NEAREST STATION:	xxxxxxx, mi. xxx.x xxxxxx subdivision
2	Indicates number of loose or missing bolts or rivets to be replaced by bolts
4 ¼”	Indicates length of field bolts

3.9 DESIGN OF FLANGE ANGLE FIELD SPLICES

3.9.1 Flange angle field splices are used when the existing flange angles are cracked or badly corroded in the vicinity of the bearing. A flange angle field splice is made by cutting the existing angle and removing the portion that is deteriorated. A new angle is then installed and spliced to the existing angle. Standard Drawing MS-09A shows the standard field angle flange splice.

3.9.2 The flange angle shall be cut far enough to remove the full length of the crack and the corrosion present at the end of the flange angle. This will ensure that a good friction connection is developed between the flange angle and the splice angle. The shape of the cut shall allow easy removal of the angle in the field.

3.9.3 Splices can be classified as single splice when only one of the flange angles is spliced or as double splice if both flange angles are spliced. If only one flange of the girder is cracked and the other flange does not show any signs of corrosion, a single splice is used. If either both flanges are cracked, or if one flange is cracked and the other flange is corroded, a double splice is used.

3.9.4 When using double splices, the location where the angles are cut shall be staggered on each side of the web in a manner that the splice angles do not overlap. If the geometry of the girder makes it impossible to do so, the location of the cuts shall be staggered by at least 0.305 m (one foot).

3.9.5 Flange angles shall be spliced with angles. However, this might not be possible for smaller size angles (6" x 6" and smaller) because the inside gauges line might be too close to the fillet of the splice angle. If the geometry will not allow a splice angle to be used, then two plates could be used instead as shown on Drawing MS-09B. In order to simplify the wording of this text, the term splice angle refers to splice plates as well.

3.9.6 Flange angle field splices shall be calculated according to the simplified method given below. This results in a design that is slightly conservative for the number of bolts to be used in the splice. If the geometry of the girder does not allow installation of all the bolts calculated according to this method, then the exact method can be used to refine the design.

3.9.7 SIMPLIFIED METHOD FOR THE DESIGN OF FLANGE ANGLE FIELD SPLICES

(a) Determine the size of the flange angle according to the original existing, as-built and shop drawings.

(b) Use a splice angle of the same size as the existing flange angle. Add a plate of the same width and thickness as the flange angle to compensate for drilling the extra holes and cutting the top of the vertical leg of the splice angle.

(c) Determine the number of bolts to be used for the splice according to the table on Standard Drawing MS-09C. This table is based on the gross area of the angle and an allowable stress of 18 ksi for the steel. This stress is usually adequate for structural steel rolled before 1930, although there are exceptions. The allowable stress can be confirmed by the Consultant if required. When determining the number of bolts, those in double shear are to be counted as two bolts.

3.9.8 EXACT METHOD FOR THE DESIGN OF FLANGE ANGLE FIELD SPLICES

(a) When designing flange angle field splices, the following requirements shall be considered:

(i) The net area of the splice angle must be equal to the net area of the flange angle in its original condition at the location where the flange is cut.

(ii) The number of bolts shall be sufficient to develop the capacity in the flange angle in its original condition based on the net section.

(b) The procedure to design the splice is as follows:

(i) Determine the size of the flange angle according to the original existing, as-built and shop drawings.

(ii) Find the gross area of the flange angle from a steel handbook.

(iii) Calculate the net area of the flange angle by removing the area of the existing and new holes (**DO NOT REMOVE THE AREA OF THE HOLES TO BE DRILLED IN THE FIELD [OUTSIDE OF CONNECTION AREA] TO ACCOMMODATE THE SPLICE ANGLE) according to the equation:**

$$A_n=A_g - \Sigma d \times t+\Sigma(s^2/4g) \times t$$

Where A_n = net area of flange angle
A_g = gross area of flange angle
d = diameter of hole
t = thickness of angle
s = spacing of staggered holes
g = gauge of staggered holes

Σ: refers the total number of holes along the critical path.

Note: The different failure paths across the angle shall be considered. The critical path corresponds to the one with the smaller net area. The net area of the critical path is to be used for designing the splice. The diameter of a 15/16" hole is usually taken as 1" when calculating net area.

(iv) Calculate the net area of the splice angle using the same equation. The area at the top of the vertical leg of the splice angle that is cut to avoid the accumulation of water shall also be deducted when calculating the net area. It shall be noted that the splice angle has holes in the horizontal leg which are usually not present when calculating the net area of the flange angle in its original condition.

(v) The net area of the splice angle calculated according to this method will be less than the area of the flange angle. In order to meet the first requirement for the design of splices, an additional plate must be used. Alternatively, a splice angle slightly thicker than the flange angle can be used if the geometry allows.

(vi) Multiply the area of the existing flange angle by the allowable stress to get the force to be transferred across the splice. The allowable stress can be taken as 60% of the yield stress. The yield stress for structural steel rolled before 1930 can usually be taken as 30 ksi, although there are exceptions. Yield stress can be confirmed by the Consultant if required.

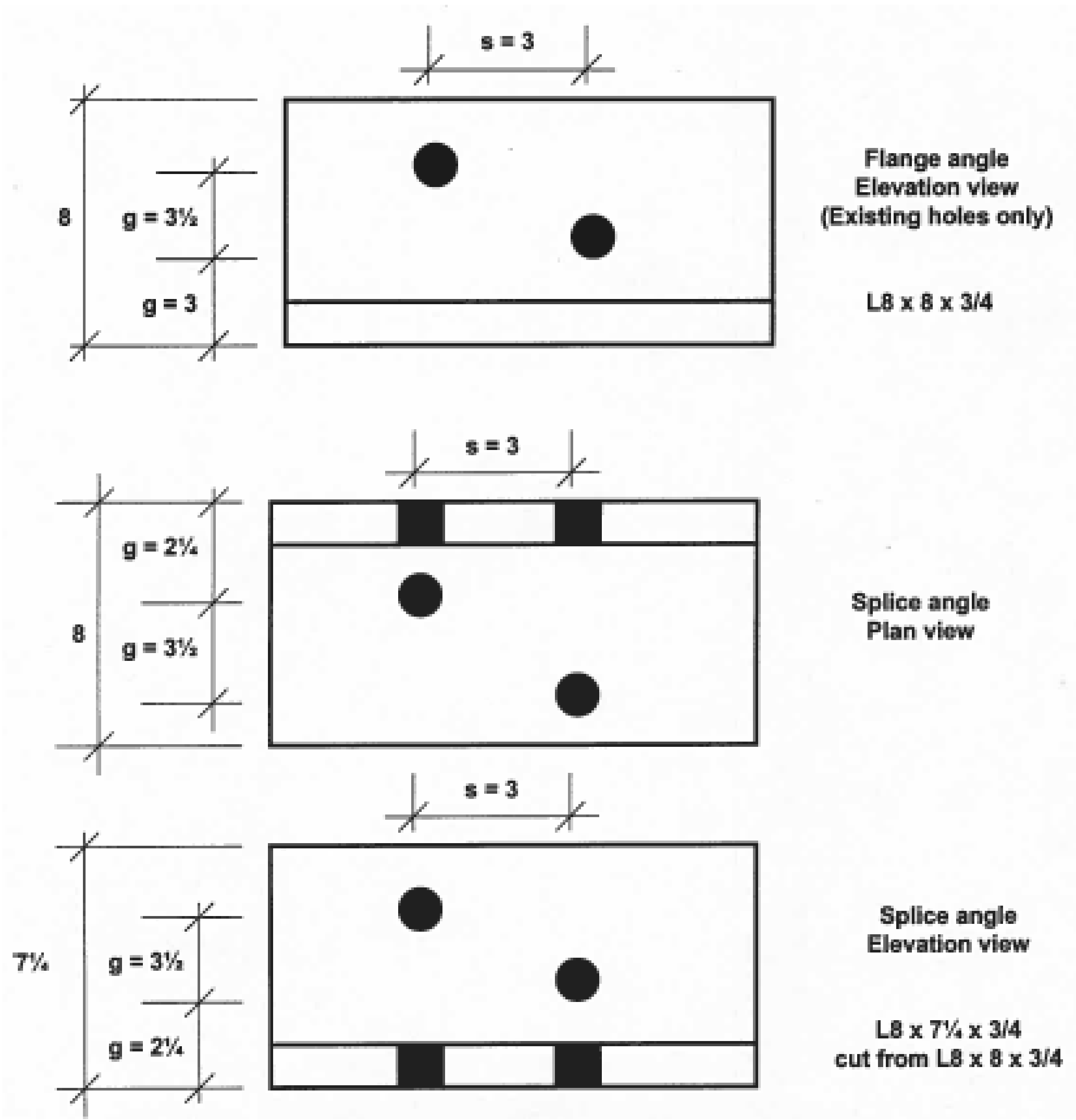
(vii) Determine the number of bolts to be used to transfer the force across the splice. The allowable shear in a 7/8 inch bolt for a friction connection is 10.22 kips. Bolts in double shear are to be counted as two bolts.

(c) Cracks in bottom flange angles shall not be repaired by welding because it is not possible to make a weld that is sound enough to properly fuse the steel back together. The welding process also creates stress concentrations which are favorable to the initiation of new cracks. Limited success has been achieved in the past with this practice.

3.10 SAMPLE CALCULATION OF FLANGE ANGLE FIELD SPLICE

3.10.1 The 8 x 8 x 3/4 angle of a bottom flange is cracked in the vicinity of the bearing. The splice needed for the repair must be designed. The hole pattern at a section located in the vicinity of the cut is as follows:

Figure 3-01: Flange and Splice View



3.10.2 SIMPLIFIED METHOD

(a) The sketch indicates that the angle size is 8" x 8" x 3/4"
(b) The splice angle should be L8 x 8 x 3/4 (cut to 8 x 7 1/4 x 3/4) with PL 8 x 3/4 on the underside of the horizontal leg
(c) From Drawing MS-09C, 20 bolts are required for the splice

3.10.3 EXACT METHOD

(a) The sketch indicates that the flange angle is 8x8x3/4, and the splice angle $8x7^1/_4x3/4$
(b) Using the Steel Handbook, we determine that Ag = 11.44 sq. in. for the flange angle
(c) Using the equation An = Ag - d x t + (s^2/4g) x t, we can determine the net area of the flange angle. Two paths must be considered:
  (i) path #1 (1 hole): An = 11.44 - 1 x .75 + 0 = 10.69 sq. in.
  (ii) **path #2 (2 holes)**: An = 11.44 - 2 x (1 x .75) + (32/(4 x 3.5) x .75) = **10.42 sq.in.**
(d) Using the same equation, the net area of the splice angle can be found. Try a splice angle the same size as the flange angle. Three paths must be considered:
  (i) path #1 (2 holes): An=11.44-2 x (1 x0.75)+0 = 9.94 sq. in.
  (ii) path #2 (3 holes): An=11.44-3 x (1 x0.75)+(32/(4 x 3.5) x .75)+(32/(4 x 4.5) x .75)= 10.05 sq. in.
  (iii) path **#3(4 holes)**: An=11.44-4 x (1 x0.75)+2 x (32/(4 x3.5) x.75)+(32/(4 x 3.75) x.75) = **9.85 sq. in.**

Note: To find the gauge between the hole in the horizontal leg and the vertical leg, add the gauge of the horizontal leg to the gauge of the vertical leg and subtract the thickness of the angle.

Remove the area that is cut at the top of the vertical leg to prevent water from accumulating.

An = 9.85 - (.75x.75) = 9.29 sq. in.

Obviously, the net area of the splice angle is less than the flange angle. A plate 8" x 3/4" will be installed under the horizontal leg of the flange angle to increase the area of the splice.

(e) Calculate the allowable force in the angle with the equation:

$T = An \times f_{all}$ = 10.42 sq. in x 18 ksi = 188 kips

(f) Determine the number of bolts

$n = T / V_{bolt}$ = 188 / 10.22 = 18.4 bolts use 19 bolts

3.11 CHECKLIST FOR FLANGE ANGLE FIELD SPLICES

3.11.1 Check if a single splice or double splice is required.

3.11.2 Determine the size of the flange angle. Verify if the gauge will accommodate a splice angle or if splice plates have to be used.

3.11.3 Check the design of the splice angle:

(a) the net area of the splice is greater than the original area of the flange
(b) enough bolts have been provided to develop the force in the flange angle

3.11.4 Check if the girder is Right or Left from the original existing, as-built and shop drawings and has been properly shown on the field detail. Pay attention to the location of stiffeners and the layout of rivets in the bottom flange.

3.11.5 Check the cut on the existing flange and verify that:

(a) the cut is far enough from the bearing to remove the crack and the corrosion
(b) the shape of the cut conforms to Standard Drawing MS-09B
(c) the cut allows easy removal of the flange angle in the field

3.11.6 Ensure that the location of the cut is correctly indicated on the field detail. This is done by taking the dimension given for the location of the cut on the field detail and identifying the corresponding location on the original drawing. The original drawing and the field detail are then checked to see if the cut is at the same location in relation to the adjacent rivets and stiffeners.

3.11.7 Check the holes in the vertical leg of the new flange angle.

3.11.8 Check the holes in the horizontal leg of the new flange angle at the bearing.

3.11.9 Check the holes in the horizontal leg of the new flange angle at the splice location:

(a) the holes are in the same pattern as the splice angle
(b) the holes are staggered between the vertical and horizontal legs
(c) the dimension between the holes at the bearing and at the splice is correct

3.11.10 Check the edge distance on the new flange angle.

3.11.11 Check the splice angle:

(a) the corner is ground to suit the fillet of the flange angle
(b) the top of the vertical leg is cut to prevent water from accumulating
(c) the first row of bolts fits with the reduced gauge on both legs

(d) the distance between the hole in the new flange and the hole in the existing flange
(e) the holes are staggered between the vertical and horizontal legs
(f) the splice angle does not interfere with existing intermediate or bearing stiffeners

3.11.12 Check the new holes in the existing angle:

(a) the distance from the cut edge is correct and the edge distance is adequate
(b) the holes match the splice angle

3.11.13 Check the field detail:

(a) the erection marks are correct on all views
(b) the length of countersunk bolts at the bearings is correct
(c) the protective sheet to prevent heating of the existing web has been specified

3.11.14 Check the erection diagram

(a) all marks are indicated
(b) the right or left identification is appropriate
(c) the field detail number has been indicated

3.11.15 Check the weight of the span to be jacked (including the ties and the track)

3.12 REPAIR OF GIRDER WEBS DAMAGED IN ACCIDENTS

3.12.1 Girder webs that are damaged in accidents are usually cut and a new plate is spliced on. The design of this repair shall conform to the requirements of the AREMA Manual for Railway Engineering, Chapter 15.

3.12.2 If the web is just slightly damaged, it may in some occasions be preferable to repair the web using heat straightening.

(a) This process consists in applying heat to the part of the steel that has yielded in a manner that the cooling process will return the steel to near its original position. Jacks and come-alongs are used just to initiate the movement. The cooling process itself does the straightening.
(b) The heat is applied on the side that would need to be jacked in order to straighten the web. The heat must be applied only to the section of steel that has yielded. The temperature shall not exceed 650 degrees Celsius in order to ensure that the microstructure of the steel is not modified. The heat must be applied in a manner that will minimize residual stresses. The cooling must be done by air only.
(c) This process shall not be attempted when it is raining.
(d) Specifications for heat straightening can be found in the document Development of Engineered Heat- Straightening Repair for the Twenty-First Century, R. Richard Avent, Department of Civil Engineering, Louisiana State University, January 1992.

3.13 SHIMMING OF STEEL SPANS

3.13.1 Before preparing drawings to shim a steel span, a survey must be requested to obtain the following information:

(a) Existing base of rail profile on bridge and 500 feet each side
(b) Top of ties at all four corners of the span
(c) Top of steel at all four corners of the span
(d) Top of bearing at all four corners
(e) Top of concrete at all bearings
(f) Size of rail, tie plate and tie pad on bridge
(g) Size of ties on each span and framed height
(h) Eccentricity of track on bridge
(i) Gap to the next span or backwall
(j) Cross-section of embankment behind abutment (if wingwalls must be raised)

3.13.2 The top of ties shall be taken as close as possible to the rail.

3.13.3 The concrete elevation shall be taken at the four corners of the bearings, and the highest point is given as the concrete elevation. Pictures shall be taken of the bearing surface.

3.13.4 The framed height of the tie is the actual depth of the wood over the girder.

3.13.5 An expansion bolt shall be installed in the abutment face or the backwall at each bearing. The nut shall be tightened so that the flat at the top is horizontal. The elevation of the top of the nut is then surveyed. These bolts will serve as a benchmark if the bridge seat elevations need to be adjusted.

3.13.6 For practical purposes, the bolts shall be installed at an elevation which would differ by 12 to 24 inches from the top of the bridge seat.

3.13.7 The size of the ties shall be measured and it shall be noted if they are dapped or sized on one side. The presence of track shims or additional tie pads under the tie plates, or shims under the ties, shall be noted.

3.13.8 If there are switches or level crossings in the vicinity of the bridge, they shall be indicated on the survey drawing.

3.13.9 The existing base of rail profile shall be plotted on a drawing. If the track is super-elevated, the lower rail shall be used to determine the profile. If any modification is required to the profile, it shall be plotted on the same view as the existing profile. Any modification to the track profile shall be approved by the Corridor Maintenance Senior Manager Track and Structures.

3.13.10 Shim height is determined as follows:

(a) From the proposed profile, determine the base-of-rail elevation at each end of the spans.

(b) Subtract the thickness of the tie plate, the tie pad and the framed height of the bridge tie to get the proposed top-of-steel elevation. This information shall not be taken from the survey but from Standard Drawings R9A-1.6 & R9A-1.7. If the survey indicates that the framed height differs by more than 1/8 inch from the standard, then a decision will have to be made as to whether to calculate the shims based on the actual or standard value. If the shims are calculated using the actual value, the track will not be at the proper elevation if the deck is replaced in the future according to the standard. If the shims are designed to the standard value, then plywood shims will have to be installed beneath the ties. The life of these shims rarely exceeds five years. If this problem occurs, it shall be brought to the Corridor Maintenance Manager Bridges and Structures attention.

(c) Subtract the height of the span according to the original shop drawings to get the proposed underside of the steel.

(d) Determine the proposed elevation of the bridge seat. Ideally, the proposed concrete elevation is 3/8" to 1/2" higher than the existing one.

(e) Determine the total height of the bearing by subtracting the proposed bridge seat elevation from the underside of the steel elevation.

(f) Determine the height of the shims by subtracting from the height of the bearing the height of the existing shoe plate, steel castings, bed plate, etc., that will be reused. The height of shims can be rounded to the nearest 1/8 inch.

3.13.11 The thickness of the shim plates shall be determined according to the following guidelines:

(a) The weight of the shims should be kept below 200 lbs. in order to facilitate their handling.
(b) Thin shims shall be avoided because they corrode quickly and have a tendency to warp. Shims of about 1 inch in thickness have proven to be successful over the years. The minimum thickness to be used is 3/8 inch. The thinner shims shall be installed in the middle of the stack. This helps in keeping the thin shims out of the pond of water that can form around the bearing and also prevents the shims from warping.
(c) A natural rubber pad 1/4 inch thick and having a hardness of 60 durometer shall be installed on the concrete in order to compensate for irregularities in the surface. Rubber pads thinner than 1/4 inch shall not be used because they have a tendency to tear easily.
(d) If shims thinner than 3/8 inch have to be used, it is preferable to use a rubber pad instead of a steel shim.
(e) No allowance shall be made for compression of the rubber pads when calculating shim thickness.
(f) The height of the shims shall not exceed 8 inches. Beyond this point, pedestals are preferable.

3.13.12 When substantial track lifts are done, backwalls and wingwalls may have to be modified in order to accommodate the additional ballast. The stability of the abutment shall also be investigated. No track lifts shall be allowed on bridges where the abutments show signs of instability without first consulting the Manager Bridges and Structures.

3.13.13 The existing anchor bolts represent an obstacle to the installation of shims. It is usually not practical to jack the span high enough to insert the shims from over the top of the anchor bolts (see Section 3.14). Also, the existing anchor bolts might not be long enough to accommodate the additional shims. Therefore, a decision must be made regarding what has to be done with the anchor bolts. The following guidelines are helpful in determining which solution shall be used (see also AREMA Chapter 8):

(a) If the anchor bolts are in good condition and are long enough to accommodate the new shims, then keep them and use split shims or slotted shims; if not then:
(b) If the existing bearing is made of rolled steel plates, cut the existing anchor bolts flush with the concrete surface and drill new anchor bolts beside them; if not, then:

(c) If the existing anchor bolts are straight and embedment is less than two feet, remove the anchor bolts by either pulling or coring and install new anchor bolts at the same location; if not, then:

(d) Use keeper plates as shown on Drawing MS-12A or MS-12B.

Refer to Drawings MS-11 or MS-12 for more information on shims and anchor bolts.

3.14 JACKING OF STEEL SPANS AND TOWERS

3.14.1 Steel spans must often be jacked when performing various works near the bearing area such as bridge seat repairs, shimming, bearing maintenance, bottom flange field splices, bottom gusset plate replacement, etc.

3.14.2 Before designing a jacking scheme, the following information must be gathered:

(a) Weight of span
(b) Required height of jacking
(c) Jack capacity, stroke, size and required clearances (see Figure 32)
(d) Gap between end of span and adjacent span or backwall
(e) Requirements for track loosening
(f) Soundness of substructure at jack location

3.14.3 The weight of the existing span is determined approximately as indicated in Section 3.15. This procedure is sufficiently accurate for jacking purposes. Alternatively, the weight of a span can be taken from the stress sheet drawing if this information is available. The weight given on the stress sheet may be less than the actual weight if the deck has been replaced with a heavier one. When using assumed dead load values given on a stress sheet, determine whether the values are for the whole span or just one girder.

3.14.4 For bearing maintenance, bottom flange field splices and replacement of bottom gusset plates, jacking is required due to the presence of countersunk rivets through the shoe plate and bottom flange angles. The top head of these rivets can be cut flush with the top of the bottom flange, but the rivet cannot be backed out because the bottom head is flush against the bridge seat (see Section 3.15). Therefore, the span will have to be jacked to allow removal of the rivet. Once the work is completed, a countersunk bolt will be installed to replace the former rivet. The span will then have to be jacked even higher since the bolt is longer than the cut rivet (the rivet is cut flush with the top of the bottom flange whereas the bolt projects on top of it). The required height of jacking in this case is usually equal to the length of the countersunk bolts plus an additional1/4 inch.

3.14.5 For bridge seat repairs and shimming, the required jacking height is dictated either by the projection of the existing anchor bolts above the bridge seat, the clearance required to chip out the loose concrete or the space required for the insertion of shims. A clearance of 4 inches is usually sufficient to break the concrete underneath the bearing. The minimum space required for the insertion of shims is equal to the shim thickness for slotted shims and twice the shim thickness for split shims. An additional clearance of ¼ inch shall be added to this minimum value.

3.14.6 Knowing the weight and the required stroke, the jack can then be chosen. Typical jack data is shown on Figure 32. Before planning work with low profile jacks, it shall be verified with the Corridor Maintenance Bridge Specialist that they are available for the project. It is common practice to use jacks with capacities well in excess of the theoretical jacking weight. An additional 30% - 50% shall be added to the jacking weight in order to determine required jack capacities. This will compensate for possible restraint forces that need to be overcome to jack the span and for reduced efficiency of the jack's hydraulic system.

3.14.7 The gap with the adjacent span or backwall is given on the inspection report.

3.14.8 To jack the span without any restraint from the rail, the track spikes need to be removed or loosened on the approaches. The rule of thumb is that the spikes shall be removed over a distance of 10 feet for every inch the span is jacked. The rail anchors usually have to be removed also. Even when the rail is jointed, it is easier to remove the spikes than trying to remove the rail joint. Removing the spikes will eliminate the lateral support that prevents the rail from buckling.

3.14.9 This is problematic during the summer months when the rail is in compression. Buckling can occur not only on continuous welded rail, but also on jointed rail. Jacking work is often done in the morning or even at night in order to minimize buckling problems during summer. Cutting the rail in order to install a rail joint to facilitate jacking is seldom a desired option because of the logistics involved in cutting and then rewelding the rail once the work is completed. It should only be considered if the spikes have to be removed beyond a switch or level crossing. The inspection report indicates the type of rail installed on the bridge (continuous welded rail or jointed rail) and any crossings or switches in the vicinity. If there is any doubt or concern about potential rail buckling problems, then the Corridor Maintenance Senior Manager Track and Structures shall be consulted.

3.14.10 The soundness of the substructure can be determined from the inspection report and photographs of the bridge. A site visit may be required if there are any concerns.

3.14.11 Once all the information is gathered, the jacking scheme can be designed. The following steps shall be followed:

(a) Examine drawings for any structural restraint that would increase jacking load
(b) Choose jacking bracket from the MS drawing series
(c) Determine bracket location from required jack clearances
(d) Determine eccentricity from required jack clearances
(e) Determine number of bolts once jacking load and eccentricity are known
(f) Check geometry to ensure bracket fits on bridge without any interference
(g) Check location of jack and base plate on bridge seat to ensure minimum edge distance of 6 inches
(h) Check soundness of substructure near the edge where the jack is located
(i) Check whether gap with adjacent span or backwall is sufficient
(j) Check if track loosening requirements will cause problems (buckling, crossings, switches, etc.)

3.14.12 It is important to ensure that all connections that would restrain the jacking operation are released. This is usually not a problem when the span is sitting on a concrete substructure. When the span is sitting on a steel substructure, drawings shall be checked for connections to the substructure, adjacent spans or other structural members. Additional restraint can also be caused by bearings which are frozen on the bridge seat or by friction on the anchor bolt. In order to avoid restraint due to torsional stiffness of the span, both bearings at the end of a span are jacked simultaneously. This will also prevent distress of the secondary members. The jacking procedure shall include steps to verify equal raise between jacks on a continual basis.

3.14.13 The jacking bracket shown on Drawing MS-01A&B is preferred when jacking on top of a concrete substructure. The design of this bracket is based on the assumption that the bolts are in bearing connection with threads in the shear plane.

3.14.14 If there is no room on the bridge seat for the jack or if the span is sitting on top of a steel substructure, it might be preferable to jack the spans from the top as shown on Drawings MS-04A&B and MS-05A&B. The condition of the top flange is critical when using the jacking bracket alone as shown on Drawing MS-04B. Corrosion could reduce the capacity of the top flange angles to resist the local bending caused by the jacking force. In this case, the force in the bolt is limited to 10 kips in order to control local bending in the flange. The jacking bracket shown on Drawing MS-05 can be installed with minimal track occupancy time and is often a preferred method. Jacking brackets shown on these drawings are designed so that they are no higher than the top of rail. They can therefore be left on the bridge without any clearance restrictions. If the concept shown on Drawing MS-05 is required for heavier spans, then a custom design can be made using larger rods and beams. Larger beams will most likely be in the clearance diagram and therefore will have to be installed during the work block.

3.14.15 If the jacking weight exceeds the capacity of the bracket shown on Drawing MS-01, then a jacking beam can be installed as shown on Drawing MS-06. The modifications to the end brace frames associated with the installation of the jacking beam are also included on this drawing. The concept shown on Drawing MS-06 is only applicable for square spans. Alternatively, the concept shown on Drawing MS-01 can be customized to suit the application.

3.14.16 The jacking bracket shown on Drawing MS-02 is used when both adjacent spans on a steel bent must be jacked. If the flange of the tower leg shows no sign of corrosion, experience has shown that the local bending in the flange will usually not be a problem if the tension in the bolt does not exceed 10 kips. This criterion controls the design of the connection between the bracket and the tower. The bracket is designed assuming 3 inch bolt spacing. Larger spacing will increase the capacity of the connection. The maximum eccentricity shown on the drawing shall not be exceeded.

3.14.17 The jacking bracket shown on Drawing MS-03 is used to jack steel bents. The criterion for local bending of the flange controls the design between the bracket and the tower leg. The bracket is designed assuming 3 inch bolt spacing. The maximum eccentricity shown on the drawing shall not be exceeded. Steel bents shall always be jacked at both legs simultaneously. The jacking procedure shall include steps to verify equal raise between jacks on a continual basis. Failing to do so could cause instability of the tower. A spacer block is installed at the bottom of each leg to prevent lateral movement of the bent. On viaduct bridges, steel bents are paired together to form a tower. It is possible to jack only one of the bents without affecting the adjacent one or the bracing in between the bents. The tower will just slightly rotate as a rigid body.

3.14.18 When selecting the jacking bracket, it is important to note that the actual force required to jack the span can be higher than the calculated weight. This is due to the restraints discussed above. For this reason, one should be conservative in the selection of the jacking bracket along with the number of bolts.

3.14.19 The criterion for minimum edge distance on the concrete can be relaxed if the bridge seat is made of reinforced concrete and the concrete is sound.

3.14.20 If the track loosening requirements are a problem, then a solution must be found to minimize the required jacking height. The existing anchor bolts are sometimes cut flush with the bridge seat and then replaced (see AREMA Chapter 8). The bearings can also be removed to provide more clearance for chipping the concrete. However, this may require removal of countersunk rivets which also requires a significant jacking height. If the concrete under the bearing is just encaved and not broken, it can be decided not to chip any concrete and to just blow the concrete dust away. In this case it is important to ensure that there will be sufficient space to meet the minimum thickness specified by the grout manufacturer (see Section 4.2). It should also be realized that the repair might not be as durable as if the concrete were chipped.

3.15 APPROXIMATE WEIGHT OF STEEL SPANS

3.15.1 The following equations apply to approximately determine the weight of steel railway spans made of riveted construction designed for Cooper E60 loading. The weights given by these equations are usually adequate to determine the jacking load of a span. They should not be used when determining the weight of a span that will be lifted with cranes.

3.15.2 Even though it is possible to adjust the weight of the span for design loads less than E60, this is usually not done when determining the jacking load of a span. There are too many uncertainties concerning the actual design of the span and the restraint forces required that must be overcome during jacking to justify such a reduction.

3.15.3 For other loads, increase or decrease the weight by 1% for each Cooper E1 above or below cooper E60, respectively.

Ex: for Cooper E50, decrease weight by 10%

3.15.4 The variables are defined as follows:

W = Weight of structural steel in kips
S = Length of span center to center of bearings in feet

3.15.5 See also graphs on Figures 16, 17 and 18 for weight vs. span length curves.

3.15.6 DECK PLATE GIRDER SPANS (DPG)

$$\mathrm{W} = \frac{\mathrm{S}^2}{42 + 0.25\,\mathrm{S}}$$

This formula is based on considerable experience and has proven to be reasonably accurate in practice.

3.15.7 THROUGH PLATE GIRDER SPANS (TPG)

$$\mathrm{W} = \frac{\mathrm{S}^2}{21 + 0.30\,\mathrm{S}}$$

This formula is based on limited experience and yields results similar to previous curves.

3.15.8 THROUGH TRUSS SPANS (TT)

$$W = \frac{S^2}{58 + 0.05\,S}$$

This formula is based on previous curves which have proven to be reasonably accurate for estimating purposes.

3.15.9 BRIDGE DECKS

The weight of timber bridge decks per foot of bridge can be taken as follows:

Table 3-02: Timber Bridge Deck Weights

size inch	length ft	WT/FT (lb/ft) Ties only	WT/FT (lb/ft) Ties and Track
8x8	12	320	520
8x8	13	345	545
8x12	12	485	685
8x12	13	525	725
10x10	12	430	630
10x10	13	465	665
10x12	12	515	715
10x12	13	555	755
10x14	12	600	800
10x14	13	650	850
12x12	12	540	740
12x12	13	585	785
12x14	12	635	835
12x14	13	685	885

Additional bridge deck data is provided in Section 7.1

3.16 CORROSION OF STRUCTURAL STEEL

3.16.1 CORROSION BASICS

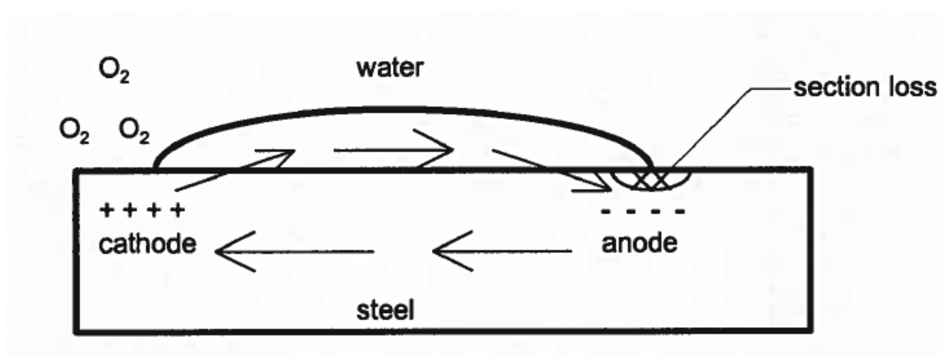
3.16.1.1 Corrosion occurs when steel reacts with the surrounding environment. The steel acts like a battery with a positive area (cathode) and a negative area (anode). Water joins the two sites and closes the electrical circuit.

3.16.1.2 By analogy, the cathodic site can be considered as the motor of the corrosion process. Oxygen is the fuel that makes the motor run. Cutting the supply of oxygen will stop the corrosion process.

3.16.1.3 The relative size of the cathode and the anode is one of the factors that can affect the rate of corrosion. A large cathode will have sufficient power to corrode a small anode very quickly, whereas the corrosion rate is slow when the cathode is small in comparison to the anode.

3.16.1.4 As long as oxygen is present at the cathode and moisture closes the electrical circuit, current is carried from the cathode to the anode through water and then back to the cathode through the steel. At the anode, the steel section will gradually be lost and replaced with a layer of rust. This transformation occurs with a large increase in volume which will destroy the protective coating, if any, on the area adjacent to where the rust has formed

Figure 3-02: Electrical Circuit



3.16.1.5 All of the following elements are required to initiate the corrosion process:

(a) bare steel
(b) moisture
(c) oxygen
(d) warm weather

3.16.1.6 Rain and condensation will provide the moist environment necessary for corrosion. Chemicals dissolved in water such as chlorides and sulfates will increase the speed of the corrosion current and therefore the rate of corrosion. Chlorides are present in marine environments and near roads where de-icing salts are used. Sulfur dioxides present in industrial environments will dissolve in the water as sulfates. Brine, a saline solution used in old refrigerated cars, also contains chemicals which accelerate corrosion rates.

3.16.1.7 Perfectly coated steel will not corrode since oxygen and moisture cannot reach the steel. Inert coatings such as epoxy paints are well suited for this application. Alternatively, galvanic coatings can be used. In addition to sealing the surface of the steel, they also serve as a sacrificial anode that will corrode in lieu of the steel. Zinc, either applied by painting, galvanizing or metallizing, is the most commonly used galvanic coating.

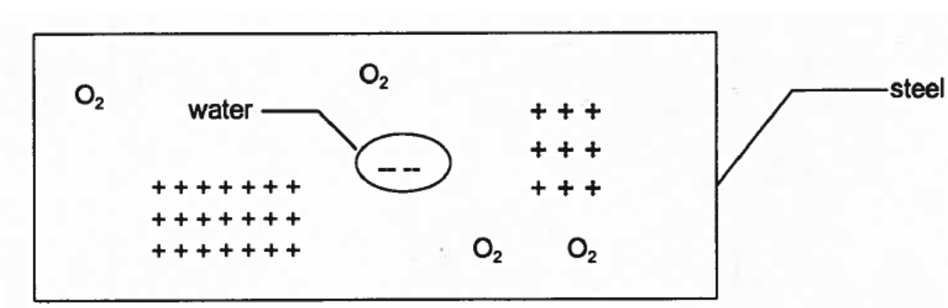
3.16.1.8 Corrosion rates are extremely slow at temperatures below the freezing point. Four different types of corrosion are identified in the SPC 4000 Commentary:

(a) Surface corrosion
(b) Pitting corrosion
(c) Dirt and debris corrosion
(d) Crevice corrosion

3.16.1.9 Surface corrosion will corrode the entire surface of the steel. Section loss can be up to 2 mils per year in severe corrosive environments.

3.16.1.10 Pitting corrosion will corrode small isolated areas of the steel when moisture is present. Steel covered with water has less oxygen than dry steel. An anode forms under the water, and the corrosion rate is much higher than for surface corrosion since the cathode is much larger than the anode.

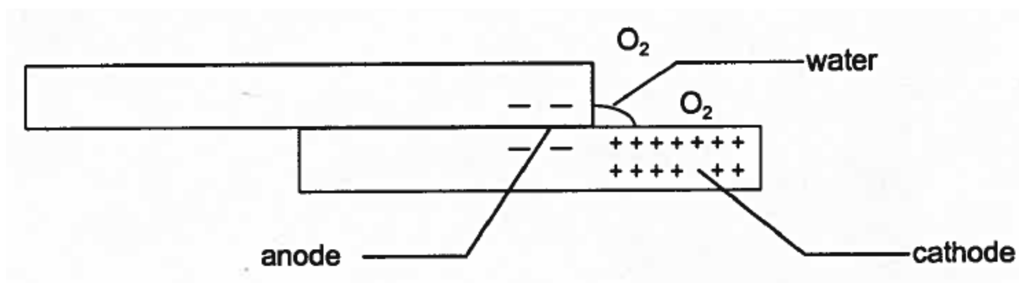
Figure 3-03: Pitting Corrosion



3.16.1.11 Dirt and debris corrosion will trap moisture and reduce the oxygen that reaches the steel. The corrosion process is similar to pitting corrosion. The corrosion rate is high due to the relative size of the cathode. This type of corrosion occurs more frequently over bearing areas and at other locations where the geometry allows retention of dirt and debris.

3.16.1.12 Crevice corrosion occurs when two plates are in contact but loosely connected. This is the case when rivets in a connection are loose. Water can penetrate into the joint, and the oxygen content is much lower in the interface between the plates. Corrosion will therefore occur. The corrosion stains around loose rivets are a sign of crevice corrosion.

Figure 3-04: Crevice Corrosion



3.16.1.13 If the plates are tightly in contact, only a thin layer of rust will form at the interface. This layer when expanding will seal tight the interface and prevent entrance of moisture and oxygen. No further corrosion will occur between the plates. This explains why old steel rarely exhibits section loss on the faying surface of the connection.

3.16.1.14 Protective coatings applied to steel often have defects. Pinholes (also called holidays) are always present in the coating even if the greatest care was taken during application. The coating can also be damaged during handling of the steel. These defects will allow water and oxygen to reach the bare steel. In the case of inert coating, a very small anode is created on the steel at this location and will corrode very quickly. If the coating is galvanic, the coating will still act as the anode and continue to protect the steel. Corrosion of the steel will only start after all the surrounding coating is gone.

3.16.1.15 Once steel starts corroding, the surface becomes irregular and promotes retention of water. Pitting corrosion is then promoted and the rate of corrosion becomes even larger. The rate of corrosion therefore increases over the years.

Additional information on corrosion can be found in the SPC 4000 Commentary.

3.16.2 SHOP COATINGS

(a) Steel to be used for the maintenance of railway bridges is coated in the fabrication shops using one of the following methods:
- (i) Painting
- (ii) Galvanizing
- (iii) Metallizing

3.16.3 PAINTING

(a) As a minimum standard, structural steel for repairs is painted according to the following guidelines:
- (i) Primer coat: inorganic zinc primer applied to a dry film thickness of 3 mils;
- (ii) Top coat: epoxy paint applied to a dry film thickness of 5 mils.
- (iii) Steel is to be sandblasted to a near-white finish according to SSPC-SP10 prior to application of the paint.
- (iv) A primer coat is to be applied to all steel surfaces. For the top coat only, the holes shall be masked on the faying surfaces two inches around each hole or group of holes.
- (v) Typically, gusset plates are painted with primer only on both sides because sometimes either side can be the faying surface. On angles, the top coat is omitted on the back of the angle in the vicinity of the holes and on both sides at the location of splices.
- (vi) Experience has shown that a better product is obtained if the requirements for top coat omissions are kept simple and touch-ups are done in the field. The alternative procedure consists in giving detailed instructions to the fabricator in order to minimize touch-ups. These instructions are easily misunderstood and the steel either gets installed with epoxy paint on the faying surface or has to be ground in the field. This latter operation will usually also damage the primer coat.

3.16.4 GALVANIZING

(a) The recommendations for galvanizing are according to the latest CSA-G164-18

(b) The following members are usually galvanized:
- (i) Grating
- (ii) Ladders
- (iii) Shim plates
- (iv) Bearing plates (except spherical bearings)

(c) Surface preparation consists in dipping the steel into an acid bath. After this has been done, the steel is then dipped into the molten zinc to obtain a minimum coverage of 610 g per square metre which is equivalent to 3 - 4 mils.

(d) Primary members and secondary members should preferably be painted rather than galvanized. The galvanizing process has some disadvantages from an erection point of view. The faying surfaces of galvanized members needs to be wire-brushed in order to obtain a friction connection. This operation is considered time consuming.

3.16.5 METALLIZING

(a) The recommendations for metallizing are according to the latest CSA G189-1966

(b) The following members are usually metallized:

  (i) Non sliding surfaces of spherical bearings

  (ii) Members that are too big to be galvanized

(c) The steel must be sandblasted to a white metal finish as per SSPC-SP5. The zinc shall be sprayed to obtain a minimum zinc thickness of 10 mils.

(d) The metallizing process produces a rough surface which is adequate to obtain a friction connection. No masking or wire-brushing is required.

(e) Zinc metallizing is not considered as durable as galvanizing. Metallizing should only be used where galvanizing is not practical.

3.16.6 NO COATING:

(a) Safety bars (see Drawing MF-01)

(b) Jacking brackets that will be removed from the bridge

(c) Other temporary steel

(d) Atmospheric corrosion resistant steel (except in marine environment)

(e) Steel to be encased in concrete

(f) Steel that will be welded in the field

(g) The standard specifications for the fabrication of structural steel indicate that the material shall be painted unless otherwise indicated on the drawings. If galvanizing or metallizing is required, it shall be indicated under the shipping mark or on the steel detail pointing to the location where the special coating is required. Any masking of holes which is not typical or any required wire-brushing shall be clearly indicated on the drawings or specifications.

3.16.7 FIELD TOUCH-UPS

(a) Paint touch-ups are required on newly installed structural steel where only the primer coat was applied in the shop or where the top coat has been damaged during handling. Bolt heads and nuts shall also be touched-up. Surface preparation for touch-ups is usually limited to ensuring surfaces are free of dirt. The touch-ups are done using the same epoxy paint as used for the top coat.

(b) Field touch-ups on galvanized or metallized surfaces are required when the coating has been destroyed by welding or during handling of the pieces. Touch-ups can be made using a cold zinc compound. Surface preparation for touch-ups is usually limited to ensuring the surfaces are free of dirt.

3.16.8 Field Painting

(a) Painting of existing structures is sometimes an economical way to prolong the life of an existing bridge. The decision whether or not to paint a bridge is based on the following factors:
   - (i) Available funds
   - (ii) Future of the bridge
   - (iii) Future of the line
   - (iv) Corrosion potential of environment (mild or corrosive)
   - (v) Environmental issues
   - (vi) Cost of painting versus replacing

(b) Prior to painting, an environmental risk assessment shall be conducted in order to identify all possible environmental hazards and mitigation measures that can be taken to minimize impact on the environment. Usually, enclosing the area to be painted in order to recover all debris from sandblasting, paint, and solvent products is an adequate mitigation measure.

(c) The surface shall be sandblasted to a near-white finish conforming to SSPC-SP10. An organic zinc primer and epoxy top coats are applied to the structural steel. The paint shall be cured according to the manufacturer's recommendations. Paint touch-ups shall be made only after curing has been completed. Particular attention should be given to the weather conditions to ensure compliance with the manufacturer's recommendations.

3.17 BASIC STEELWORK EQUIPMENT

3.17.1 RIVET REMOVAL

3.17.1.1 Rivets are removed using a rivet buster. The size of a typical rivet buster is shown on Figure 28. This equipment is pneumatically driven. A moil point chisel or a backout punch can be fitted to the end of the rivet buster.

3.17.1.2 The first step in removing a rivet consists of cutting the rivet head. The chisel is installed on the rivet buster. The chisel point is applied to the edge of the rivet head. The rivet buster is oriented at an angle of approximately 15 degrees from the plane of the connection. The impact of the chisel on the rivet head will eventually shear the head off. Once the head is removed, the backout punch is then installed on the rivet buster. The backout punch is applied to the rivet shank. The rivet buster is oriented along the axis of the rivet. The impact of the backout punch on the rivet shank will push the rivet out of the hole. Clearances required for rivet buster operation are indicated on Figure 28. These clearances should be taken into consideration when designing steel repairs.

3.17.1.3 When the plies of the connection are not perfectly in line, it can be difficult to back the rivet out of the hole. This problem becomes more frequent as the number of plies in the connection increases.

3.17.1.4 When attempts at backing out the rivet are unsuccessful, the first alternative is to drill a ½ inch hole in the middle of the shank. This can be done with a hand drill or even with a magnetic drill modified to suit this application. This hole will allow the rivet shank to be squeezed when pushed with the backout punch.

3.17.1.5 In most cases this will resolve the problem. If not, then the rivet can be drilled full size with a magnetic drill. In most cases, this will ovalize the hole since the magnetic drill will not be perfectly centered on the rivet. The hole shall therefore be reamed to either 1 or 1-1/16 inch after this operation. The use of torches for removing difficult rivets is seldom justified and this practice shall be abandoned.

3.17.1.6 When using the chisel, extreme care shall be exercised not to damage the steel. If damage occurs then nicks and burrs no deeper than 1/16 inch can be ground with a 10:1 slope. Pieces damaged more severely shall be replaced.

3.17.2 HOLE DRILLING

3.17.2.1 The most common tool used to drill holes is the magnetic drill. The size of a typical magnetic drill is shown on Figure 29. The magnetic drill is electrically driven. An electric magnet is used to hold the drill firmly onto the steel. A hollow bit is used to core out the steel and form the hole. A centering device extends through the middle of the core bit to properly align the hole.

3.17.2.2 Before drilling, the hole centres must be properly marked with a centre punch. This is best done using a steel template. After the holes are marked, the template is removed. The magnetic drill is then positioned so that the centering device is right on the mark. The magnet is activated to hold the drill on the steel. The drill can then be powered on. The penetration of the coring bit into the steel is controlled by a spin-handle on the side of the drill. During the coring operation, cutting fluid must be continually applied so that the bit does not overheat and eventually break. Oil-based fluids cannot be used for this purpose since they would make the faying surface of the connection slippery and therefore reduce its capacity.

3.17.2.3 Required clearances for the use of the magnetic drill are given on Figure 29. These clearances should be taken into consideration when designing steel repairs. Adjacent rivets must sometimes be removed to install the base of the drill on the steel. If the available clearance is not sufficient, then removing the restricting members should be considered. Drilling holes with a hand drill is a tedious operation that is not desired.

3.17.2.4 Burning holes with a cutting torch in structural members is not an acceptable method.

3.17.3 BOLT TIGHTENING

3.17.3.1 Impact wrenches are used for tightening bolts. The size of a typical impact wrench is shown on Figure 27. Impact wrenches used for bridge work are usually pneumatically driven. A one-inch drive shaft is preferable to develop the required torque for bolt tightening. Bolting sockets are installed on the drive shaft. For hard to reach locations, universal joints and extension bars can be fitted on the drive shaft.

The installation of high-strength bolts is discussed in Section 3.18.

3.17.3.2 The required clearances for typical impact wrenches are shown on Figure 27. These clearances should be taken into consideration when designing steel repairs. If the available clearance is not sufficient, then it is possible to use a smaller wrench. Some impact wrenches such as Model 6060 by Chicago Pneumatic have a pistol handle (like a carpenter's drill) and are more compact than typical impact wrenches. This equipment is barely powerful enough to torque a 7/8 inch bolt since it only has a 3/4 inch drive shaft. Frequent maintenance will be required if it is used extensively on a project.

3.17.3.3 Another alternative is to use a hydraulic wrench such as Model RSL2 by Sweeney Equipment. This equipment is very compact and requires minimal clearances. The hydraulic drive generates considerable torque. Production is much lower with this equipment than with standard impact wrenches. Railway forces and contractors also seldom have this equipment in stock.

3.17.3.4 Since alternatives to the typical impact wrench have some drawbacks, steel detailing should be done using the typical impact wrench clearances shown on Figure 27. Planning work with special impact wrenches shall be done as a last resort and only after confirming the availability of the equipment with the Bridge Specialist. If a smaller impact wrench is not available or still too big, then the bolts can be tightened by hand using either a small torque wrench, a socket with a flex- handle, or a spud wrench with a sledge hammer. Bolts that do not meet the clearances of Figure 27 should be clearly marked on the drawings as to be tightened by hand or using a specific model of impact wrench. This will facilitate job planning by field personnel.

3.17.4 HOLE REAMING

3.17.4.1 Holes must sometimes be reamed when the fit of the new steel onto the existing structure is not accurate. Before reaming holes to correct misfits, sufficient drift pins shall be used to line up the holes as best as possible. Reaming shall only be used as a last resort and would normally be limited to a small fraction of the holes. Severe misfits shall be handled by refabricating the steel (see Section 3.18).

3.17.4.2 Reaming can also be specified on the drawings for some tension members in order to improve the fatigue life at the bolt holes. The holes are then usually reamed to 1 inch diameter. This operation can be quite expensive since it usually involves a large amount of holes. Reaming requirements should be established by the Manager Bridges and Structures.

3.17.4.3 The pneumatic reamer is used to ream holes. The size of a typical reamer is shown on Figure 30. As its name implies, this equipment is pneumatically driven. Two steelworkers are required to use this piece of equipment, each of them holding one of the reamer handles. The pneumatic reamer is difficult to handle and should only be used by experienced steelworkers. Injuries can occur when the reamer bit jams and the torque is transferred to the handles. Alternative equipment for reaming holes is sometimes considered a better option. Required clearances for a typical pneumatic reamer are shown on Figure 30. These should be considered when planning to ream holes for fatigue purposes.

3.17.4.4 One alternative to the pneumatic reamer is the nut reamer. The nut reamer consists of a reamer bit with a nut or bolt head fitted to the end so that it can fit into the socket of an impact wrench. The bolt head/nut adapter can be fitted to the reamer bit either by forging or threading. Welded nut reamers are not acceptable since they usually are not fitted square and therefore ovalize the hole. One common problem with nut reamers is that the impact from the wrench will cause damage to the perimeter of the holes. This is especially true if the reamer bit is very sharp. A partly worn bit is preferable. However, when the bit is worn, it shall be replaced as it will generate too much heat when cutting the steel.

3.17.4.5 The key to successful nut-reaming is to pay careful attention, before reaming is started, to have the reamer bit straight in the hole. Reaming can then start and should not be interrupted until the bit has fully gone through the hole. If the reamer bit is not straight, the hole will be ovalized. If the operation is interrupted, the bit will make burrs inside the hole when reaming is resumed. Nut-reaming is a difficult operation that shall only be done by experienced steel workers.

3.17.4.6 Before using the nut reamer, a mock-up test must be done in the field with each operator and the operator’s equipment to determine if holes can be reamed to a circular shape without damaging the perimeter. If this test cannot be performed or fails, then the nut reamer shall not be used. The quality of the holes shall be checked continually as the work progresses.

3.17.4.7 Another alternative to the pneumatic reamer is to install the reamer bit on a magnetic drill equipped with a morse taper spindle. This equipment is safe and provides a good quality hole. However, production is much slower than with the nut reamer.

3.18 INSTALLATION AND INSPECTION OF BOLTS

3.18.1 High-strength bolts are used on steel railway bridges to connect the structural members. High- strength bolts shall conform to the ASTM A325 Standard. A490 bolts are not acceptable on railway bridges. Three different types of A325 bolts are used:

(a) TYPE 1 (also referred to as black bolts)
(b) TYPE 1 galvanized
(c) TYPE 3 (atmospheric corrosion resistant)

3.18.2 Bolt markings located on the head help in differentiating between TYPE 1 and TYPE 3 bolts (see Figure 15). TYPE 3 bolts are designed to be used with atmospheric corrosion resistant steel. Galvanized bolts shall be used on galvanized steel. Mismatching the bolt and type of steel could lead to corrosion in some cases due to contact between dissimilar metals.

3.18.3 Occasionally, other types of bolts will be used for structural steel connections. On viaduct bridges, anchoring the spans to the cap plates with A307 bolts is recommended. A307 bolts are more ductile and will minimize the possibility of brittle failure of the bolts due to longitudinal loads and thermal dilatation of the span. In order to ensure that vibration will not make the bolt fall out of the hole, techniques such as installing cotter pins, checking the thread, using thread locking fluid, etc., shall be employed. The bolt shall also be installed with the head on top of the bearing. The use of A307 bolts on similar applications with shorter span lengths, such as stringer/top-of-floor-beam connections or anchor bolts for steel trestles is not required. In this case, however, anchor bolts may have to be installed loose to allow rotation, expansion and contraction of the span without restraint.

3.18.4 Huck bolts are sometimes used on the top flanges of open deck bridges. Huck bolts have a rounded head which is similar to a rivet. This geometry will allow the head to crush into the tie during passage of the first trains. Therefore, no special notching of the ties is required. Huck bolts require special equipment for installation. The availability of this equipment should be checked with the Bridge Specialist before planning work with Huck bolts. If Huck bolts cannot be used, then the bridge ties will have to be notched to suit the bolt head pattern.

3.18.5 In order to proceed with bolt installation, all holes must be properly lined up by using sufficient drift pins. If the holes still do not fit after a reasonable amount of drifting has been done, then the holes should be reamed. Even though reaming should only be done if drifting cannot line-up the holes, overdrifting is not recommended since this will deform the steel around the holes. Therefore, proper judgment must be exercised. Reaming shall never exceed 10-20% of the holes in the connection, and holes shall not be off by more than 1/8 inch. When these limits are exceeded, the Manager Bridges and Structures should be informed. These misfits shall be handled by refabricating the steel.

3.18.6 Bolts used in connections for railway bridges shall be tightened using the turn-of-nut method. This method is described in the following paragraphs.

3.18.7 The first step is to install all the bolts in the holes. To facilitate inspection, the heads shall all be installed on the same side of the steel members. The washer is installed under the head or the nut depending on which will be turned when tightening the bolt. The nut is usually preferred since it is thicker and provides a better grip for the socket. However, due to clearance problems, the bolts must sometimes be tightened by turning the head.

3.18.8 All bolts shall be tightened to a condition called snug tight. This is defined as the normal effort of a man with one hand using a standard spud wrench or a few impacts with an impact wrench. However, this may not be sufficient to put the plies of the connection into contact. This is especially true when installing new steel on existing steel or when many plies of steel are involved. In this case, a greater effort must be applied to pull the plies together. The turn-of-nut method will not provide adequate tension in the bolts if the plies are not in contact before tightening.

3.18.9 Once the connection is snug tight, then a mark is made on the head or on the nut depending on which will be turned when tightening the bolt. A corresponding mark is also made on the steel to measure rotation. This marking works well as long as the other side is prevented from turning during bolt tightening. A more accurate method consists of marking the nut and the bolt shank. This method of marking more accurately indicates the rotation. The nut side is marked independently of which side is the turned element.

3.18.10 Bolt tightening can then proceed. The sequence shall be planned so that tightening starts in the centre of the connection and progresses towards the edges. This helps to ensure a tight fit between the connection plies without inducing any residual stresses. The impact wrench socket is installed on the nut or the head and turned for the appropriate rotation. The required nut rotation depends on the size of the bolt. Most bridge work is done with 7/8 bolts up to 3 ½ inches long. In this case the required rotation is 1/3 of a turn. For other cases, refer to the AREMA, Chapter 15.

3.18.11 When steel pieces must be dismantled during short work blocks (see Section 3.21), the rivets connecting them are usually removed ahead of time. A certain number of the rivets must be replaced with temporary high-strength bolts and the remaining holes can be left open until the work is finished.

3.18.12 The number of temporary bolts to be installed shall either be indicated in the construction procedure shown on the drawing, conform to the guidelines given in Section 3.20 or be confirmed by the Bridge Specialist. The temporary bolts must be tightened according to the turn-of-nut method before train passage can be allowed. During the work block, the temporary bolts are removed to allow dismantling of the steel pieces. The temporary bolts can then either:

(a) be specially marked and used as temporary bolts for the remainder of the work
(b) be re-used as the permanent bolts for the same connection if and only if:
  (i) the bolts are not galvanized
  (ii) the nut can be run for the full length of the thread
  (iii) the bolt does not show any sign of elongation
  (iv) the bolt has only been tightened once

3.18.13 When the work is performed by contractors, it is difficult to ensure that reuse of bolts is done properly. In this case, the first option where temporary bolts are marked and used as such for the entire job is preferable. Using Type 3 bolts as temporary bolts is advantageous because they have permanent markings on their heads. This reduces possibility of confusion between temporary and permanent bolts. Note that the temporary bolts will be reused many times; therefore, it is important to verify for signs of elongation and if the nut can run the full length of the thread. Galvanized bolts are not a good choice for temporary bolts since they cannot be reused.

3.18.14 Quality assurance for bolt tightening shall be done on 10% of the bolts but not less than two bolts per connection.

3.18.15 Quality assurance for bolt tightening can be performed using the Skidmore apparatus along with a torque wrench. The Skidmore apparatus can measure the tension in a bolt that is installed through it. The torque required to obtain the specified tension can then be measured using a torque wrench. The torque wrench is then used to verify the bolts installed on the bridge.

3.18.16 The relationship between the torque and tension in the bolts depends on various parameters that can vary from one connection to another. Among the most important factors affecting the torque-tension relationship are bolt length and bolt lubrication. For practical purposes, we will assume that all bolts in a specific heat are similarly lubricated. Bolts of different lengths all belong to different heats. Other factors do not influence the torque-tension relationship to the same extent and therefore will be neglected. Based on these assumptions, it is possible to take a representative sample of each heat of bolts used on a project and determine the required torque to obtain the specified tension in the bolts.

3.18.17 Five bolts are usually considered sufficient to obtain a representative sample of a heat of bolts. The specified tension for 7/8 bolts is 39 kips. For other bolt diameters, refer to the AREMA, Chapter 15. The bolts are installed in the Skidmore apparatus until they reach the specified tension. Then, the torque required to turn the bolt is measured. This procedure is repeated for all bolts in the sample. The torque to be used for bolt inspection in the field is the average torque measured for the sample plus an additional five percent. Erratic torque values, if any, should be discarded in calculating the average. The bolts used in the Skidmore apparatus can be reused if they meet the criteria mentioned above.

3.18.18 Alternatively, if a Skidmore apparatus is not available, quality control will have to be done using the torque wrench alone. This procedure is not accurate and has been deleted from most North American structural codes. It does not account for possible variations in bolt length, lubrication, etc. If this procedure has to be used, then the required torque for a 7/8 bolt should be between 500 to 600 ft. lb. Since this procedure is not accurate, quality control on the actual turn-of-nut method is essential.

3.19 ESTIMATING STEEL WORK

3.19.1 This section will assist in estimating steel repair work by providing some guidelines as far as cost of labour and production rates. These values are reasonable for some of the typical steel work done on railway bridges. THEY WILL NOT BE APPLICABLE IN ALL CASES. However, if used in combination with good experience and proper judgment, the values will lead to successful results.

3.19.2 The method presented in this section is based on a few assumptions which are stated below. The validity of these assumptions should be verified for each project. Some of the assumptions, such as the composition of the crew, are made for estimating purposes. The estimated project cost would not differ much if larger or multiple crews were used. Other assumptions may have a significant impact on the project cost. For example, the amount of traffic on the bridge would have a significant effect on the production rates for scaffolding; however, the effect on the production rates for rivet removal would not be significant. The assumptions are as follows:

(a) Six-man crew
(b) One flagman
(c) 10-hour work days
(d) Bridge easily accessible by road
(e) Average mainline traffic
(f) Sufficient work blocks are available to maintain progress of work
(g) Project of significant scope with several identical spans
(h) Single track bridge
(i) Bridge is fully scaffolded
(j) Boom truck is available and able to work on bridge
(k) Weather conditions are favorable to the progression of the work

3.19.3 For cost estimating purposes, we will assume that the work is done with a fictitious six-man crew composed as follows:

(a) one foreman
(b) four steelworkers
(c) one journeyman

3.19.4 It is assumed that the foreman and journeyman do not contribute directly to the production. The foreman looks after the steelworkers and the journeyman supports the steelworkers by doing miscellaneous tasks such as supplying bolts, for example. Flagging is performed by additional personnel and is included in a separate item of the estimate.

3.19.5 The bridge is easily accessible by road. This implies that no time is lost travelling back and forth to the bridge

3.19.6 Traffic on the bridge is assumed to be similar to the average mainline traffic across the system. Production rates could be different on heavily travelled main tracks or on secondary lines. There are sufficient work blocks available to maintain the production without any stoppage of work.

3.19.7 On typical steel erection projects, the production often starts slow and then increases as the job progresses. This is because the workers get better at the various procedures specific to the project. It is assumed that the project is of significant scope and consists of several identical spans. This allows the workers to achieve better production rates. On small projects, production rates could be somewhat lower.

3.19.8 It is assumed that the bridge is fully scaffolded ahead of the steel gang doing the work. This may-not be how the work is actually done, but this assumption is usually valid for estimating purposes. Partial scaffolding is more expensive to erect per unit area and work cannot progress as efficiently. Cost savings from partial scaffolding are often insignificant.

3.19.9 Cold or very hot weather and frequent rain can affect productivity. The expected weather conditions should be considered in the estimate.

3.19.10 The weekly rate of the crew can be calculated as indicated in the following tables. Some of the values may need to be adjusted for specific projects.

Table 3-03: Weekly Rate of Labour Force Steel Gang (Sample Only)

Number of men	6				
Days worked	4				
Hours worked - reg	40				
Hours worked – *o/t*	0 N.B.: Overtime is applicable after 40 hours				
Hourly pay	$48	Surcharge - reg	100%	Surcharge - o/t	50%
		Hourly rate - reg	$90	Hourly rate - o/t	$135
Daily expenses	$150				
Boom truck rate	$3,000 per week			Boom truck use	100%
Other vehicle rate	$1,200 per week				
Equipment rate	$2,000 per week				

Labour - reg	240 man-hours @	$90	=	$21,600
Labour - o/t	0 man-hours @	$135	=	$0
Expenses	24 man-days @	$150	=	$3,600
Boom truck	100% of	$3,000	=	$3,000
Other vehicles				$1,200
Equipment rate				$2,000
TOTAL GANG COST PER WEEK				$31,400

Table 3-04: Weekly Rate of Contractor Steel Gang (Sample Only)

Number of men	6				
Days worked	4				
Hours worked - reg	40				
Hours worked - o/t	0	N.B.: Overtime is applicable after 40 hours			
Hourly pay	$48	Surcharge - reg	150%	Surcharge – *o/t*	66%
		Hourly rate - reg	$90	Hourly rate – *o/t*	$150
Daily expenses	$150				
Boom truck rate	$7,000 per week			Boom truck use	50%
Other vehicle rate	$1,500 per week				
Rental rate	$3,000 per week				
Labour - reg	240	man-hours @	$90 =		$21,6000
Labour - o/t	0	man-hours @	$150 =		$0
Expenses	24	man-days @	$150 =		$3,600
Boom truck	50%	of	$7,000 =		$3,500
Other vehicles					$1,500
Rentals					$3,000
TOTAL GANG COST PER WEEK					$33,200

3.19.11 Mobilization and demobilization costs for contractors can be taken as 15% - 20% of the total steel gang cost for the project. This price includes the following

(a) Bonding and insurance fees
(b) Administrative fees
(c) Support staff
(d) Supervision fees
(e) Construction site installation and logistics
(f) Unloading and sorting of the steel pieces
(g) Miscellaneous supplies
(h) Site clean-up
(i) Etc.

3.19.12 Mobilization and demobilization costs for METROLINX forces include the labour for construction site installation, unloading and sorting of the steel pieces, and site clean-up. Depending on the site and scope of the job, one or two weeks should be added to the time required for steel erection to cover this work.

3.19.13 If the scaffolding is erected by a contractor, it should be estimated using a unit price per square foot. Suspended scaffolds can be installed for a price of $15 - $25 per square foot, depending on the amount of scaffolding installed and the difficulty of the job. This unit price also includes dismantling. When calculating the area to be scaffolded, add five feet on the outside of each girder or truss. Rental cost can be taken as $7 per month per square foot. When calculating rental, if the logistics of the project permit, it can be assumed that some of the scaffolding materials will be reused from one span to the other. Supported scaffold is somewhat cheaper than suspended scaffold. Unit prices for erection and dismantling can be taken as 65% of the ones for suspended scaffold. Obviously, for supported scaffolding, the width should be equal to the centre-to-centre distance between the girders.

3.19.14 If the scaffolding is to be erected by the Contractor forces, then the actual time required to install and dismantle the scaffolding should be added to the time required for erection of the steel. Production rates for scaffolding of TYPICAL SINGLE TRACK BRIDGES DONE BY THE CONTRACTOR FORCES can be taken as follows:

Table 3-05: Production Rates for Scaffolding of Typical Single Track Bridges Done by Contractor Forces

PRODUCTION RATES FOR SCAFFOLDING			FORCES
OPERATION	SCAFFOLD	SPAN	PRODUCTION
erection	suspended	DPG	50 - 60
erection	suspended	TPG	50 - 60
erection	suspended	TRUSS	50 - 60
erection	supported	DPG	80 - 90
dismantling	suspended	DPG	75 - 90
dismantling	suspended	TPG	75 - 90
dismantling	suspended	TRUSS	75 - 90
dismantling	supported	DPG	120 - 135

3.19.15 Production rates for replacement of secondary members in kind can be taken as follows:

Table 3-06: Production Rates for Replacement of Secondary Members

PRODUCTION RATES per day		**SECONDARY MEMBERS**
Member	Production	Example
Bracing gusset plate	8 - 10	TPG bottom gusset plate - 20 rivets
X - Bracing	4 - 5	TPG bottom bracing
Small gusset plate	12 - 16	Stringer gusset plate - 10 rivets
Small bracing	16 - 20	Stringer bracing - 6 rivets
Stiffener patch	7 - 9	Drawing MS-10A

3.19.16 The given production rates include handling of the pieces.

3.19.17 Other secondary members can be evaluated assuming 25 to 35 man-minutes per hole. The lower range applies to secondary members that can be man-handled. The upper range applies to larger secondary members. The production rates calculated on this basis should be double-checked by comparison to the ones given in the table to ensure that the value obtained is reasonable. Unusual members should not be estimated using this rule.

3.19.18 For more elaborate work such as replacing flange angles, the estimate should be prepared using the step-by-step erection procedure. Such work is usually done in three phases: preparations (before the work block see section 3.20), replacement of the member (during the work block see section 3.21) and completion (after the work block).

3.19.19 Preparatory and completion work can be estimated using the following production rates

Table 3-07: Preparation for Jacking Spans

PREPARATION FOR JACKING SPANS For one end of a span	
drawing number	days
MS-01	1
MS-02	2
MS-03	2
MS-04	1
MS-05	1
MS-06	2

Table 3-08: Production Rates for Miscellaneous Work (Per Day)

PRODUCTION RATES FOR MISCELLANEOUS WORK (per day)		
OPERATION	PRODUCTION	EXAMPLE
Rivet removal - horizontal surface	200 - 300	Bottom cover plate - 2 to 4 plies
Rivet removal - vertical surface	150 - 200	Bottom flange - vertical leg
Rivet removal - overhead	100 - 150	Top flange - vertical leg
Rivet removal - from deck	70 - 100	Top flange - open deck DPG
Drilling holes with template	300 - 350	Chord strengthening side plates
Reaming holes with nut reamer	400 - 500	Ream holes in bottom flange
Reaming holes with pneumatic reamer	200 - 300	Ream holes in bottom flange
Tightening bolts	800 - 1000	Bottom flange

3.19.20 The production rate per day for rivet removal also includes installation of high-strength bolts in 50% of the holes. Rivet removal from the deck includes moving the deck and installing high-strength bolts in all the holes. Since this work is done from the deck, production can be outside the specified range on secondary lines or on heavily travelled main line. The production rate for tightening bolts also includes paint touch-ups.

3.19.21 In calculating the number of rivets to be removed, consider that additional rivets in the members adjacent to the one that will be replaced may have to be removed.

3.19.22 These production rates were established for the members given in the table. Proper judgment must be used when applying the rates to other members.

3.19.23 Rivet removal will be much slower when the rivets pass through many plies of steel. Production rates should be adjusted accordingly.

3.19.24 Handling of the pieces should be added to the preparation time.

3.19.25 The estimate for the work to be done during the work block (removing old members, installing new members, pinning, installing bolts, etc.), must be based on judgment and experience (see also Section 3.21). Remember that according to the assumptions made in this section, only four workers are available to do the actual work. If additional labour is required, this should be considered in the estimate. Overtime rates may apply if work blocks are done during weekends or at night. The availability of work blocks should be considered in the estimate.

3.19.26 The information presented above allows the determination of the cost of mobilization and demobilization, scaffolding and steel gang. Other items that must be included for typical steel repair projects are:

(a) Structural steel
(b) Bolts
(c) Cranes (if required)
(d) Fall protection systems
(e) Flagging
(f) Railway forces (bridge, track and signal forces)
(g) Traffic or navigation protection (if required)
(h) Design
(i) Supervision
(j) Quality assurance
(k) Contingencies

3.19.27 Contractors may incur additional costs to allow a rubber-tired boom truck to work on a bridge. This may include modifications to the boom truck or planking of the bridge.

3.19.28 Other factors that must be considered when work is done by contractors are working in remote areas, complicated repairs or strengthening, lack of competition or of contractors with railway experience, frequent train traffic, etc. All these factors may encourage the contractor to put a higher contingency in the contractor's bid to compensate for unforeseen problems.

3.19.29 As a spot check, the estimate for the erection of structural steel (including mobilization and demobilization, scaffolding and labour) can be verified using unit prices per pound or per hole. General steel repairs can usually be made for a price varying between $25 and $45 per pound. Steel can be strengthened at a cost of $15 to $25 per pound of strengthening material. Steel strengthening can also be verified more accurately on a per hole basis. The price will vary between $60 and $80 per hole.

3.19.30 The procedure given in this section does not provide the actual duration of the entire project. The reason is that we have assumed that only one crew is working on the bridge. The contractor may choose to have more than one crew working. In this case, it may be decided to take the total number of weeks calculated according to the assumptions of this section and divide by the number of crews working. This procedure will be accurate if the job site is large enough to have all crews working in an efficient manner.

3.20 PERFORMING STEEL WORK UNDER TRAFFIC

When designing or planning steel work, it is important to determine which parts of a bridge can be dismantled, while trains are still running, without affecting structural integrity. It is also important to know how much of the work must absolutely be completed during a work block and what can be done once traffic has resumed.

The following guidelines provide conservative recommendations for typical steel work. These guidelines are usually sufficient for many situations and will assist in day-to-day planning of the work. Special cases where these guidelines are too restrictive or the work involved is more complicated should be looked at on a case-by-case basis. In these cases, the step-by-step construction procedure clearly indicating which work is to be performed as part of the preparations, during the work block and at completion should be indicated on the drawings.

3.20.1 DECK PLATE GIRDERS

The following guidelines apply to DPG spans as follows:

(a) Must be simple spans only
(b) The span must have at least three bays of bracing
(c) Brace frame spacing does not exceed 12 feet
(d) Cross lateral bracing is present on top and bottom of span
(e) The span is square (no skew)
(f) Bracing is sound

3.20.1.1 Rivets

(a) All rivets in flange angles and secondary members can be removed and replaced with bolts in 50% of the holes without any traffic restrictions. Before a train is allowed to cross the bridge, it shall be verified that all connections are 50% bolted and that the bolts have been tightened by the turn-of-nut method.

3.20.1.2 Bearing stiffeners

(a) Bearing stiffeners, which are not used for the connection of the end brace frame, can be removed without any traffic restrictions as long as the remaining effective bearing area is at least 50% of the original bearing area. When assessing the remaining effective bearing area, corrosion of the existing stiffeners shall be considered.

3.20.1.3 ONLY ONE OF THE FOLLOWING ITEMS CAN BE REMOVED AT ANY ONE TIME WHILE TRAINS ARE STILL RUNNING. (I.E., TOP AND BOTTOM BRACING CANNOT BE REMOVED AT THE SAME TIME).

Particular attention should be given when removing members such as gusset plates, stiffeners and other members which are used for connection purposes since this can affect more than one bracing system at the same time (i.e., removing the top gusset plate will disconnect the top bracing and also part of the brace frame).

(a) Top bracing:
  (i) Two diagonals forming one bay of the top bracing can be removed with a 20 MPH slow order. THE TOP STRUTS SHALL REMAIN IN PLACE. All brace frames and all bottom bracing shall be installed.

(b) Top bracing gusset plate:
  (i) One top bracing gusset plate can be removed with a 20 MPH slow order. The vertical leg of the top strut must remain connected.

(c) Bottom bracing:
  (i) Every second bay of bottom bracing including the bottom struts can be removed with a 30 MPH slow order. All top bracing and brace frames shall be installed.

(d) Bottom bracing gusset plate:
  (i) Bottom bracing gusset plates can be removed with a 30 MPH slow order as long as enough gusset plates remain to keep two of every four consecutive bays of bracing connected. A minimum of 50% of the total bays shall remain properly braced.

(e) End brace frames:
  (i) Train traffic SHALL NOT BE ALLOWED when the end brace frame is missing unless shown on the drawings or directed by the Corridor Maintenance - Manager Bridges and Structures.

(f) Intermediate brace frames:
  (i) Diagonal braces and the bottom strut of one intermediate brace frame can be removed with a 30 MPH slow order. THE TOP STRUT SHALL REMAIN IN PLACE. The top bracing and bottom bracing shall be installed.

3.20.1.4 Top struts:

(a) Train traffic SHALL NOT BE ALLOWED when a top strut is missing unless shown on the drawings or directed by the Corridor Maintenance - Manager Bridges and Structures.

3.20.2 THROUGH PLATE GIRDERS

The following guidelines apply to TPG spans as follows:

(a) Must be simple spans only
(b) Structural system consists of stringers, floor beams and girders
(c) Floor beam spacing is between 10 and 15 feet
(d) Cross lateral bracing is present at the bottom of the span
(e) Bracing and connections of stringers and floor beams are sound

3.20.2.1 Rivets:

(a) All rivets of flange angles and secondary members can be removed and replaced with bolts in 50% of the holes without any traffic restrictions. Stringer to floor beam connections and floor beam to girder connections shall be replaced with bolts in 80% of the holes. Before a train is allowed to cross the bridge, it shall be verified that all connections are either 50% or 80% bolted as indicated above and that the bolts have been tightened by the turn-of-nut method.

3.20.2.2 Bearing stiffeners:

(a) Bearing stiffeners, which do not serve for connection of the end floor beam, can be removed without any traffic restrictions as long as the remaining effective bearing area is at least 50% of the original bearing area. When assessing the remaining effective bearing area, corrosion of the existing stiffeners shall be considered.

3.20.2.3 Bottom bracing and gusset plates:

(a) Train traffic SHALL NOT BE ALLOWED when bottom bracing is missing unless shown on the drawings or directed by the Corridor Maintenance - Manager Bridges and Structures.

3.20.2.4 Stringer bracing and diaphragms:

(a) Train traffic SHALL NOT BE ALLOWED when stringer bracing or a diaphragm is missing unless shown on the drawings or directed by the Corridor Maintenance - Manager Bridges and Structures.

3.20.3 TRUSSES

(a) For trusses, recommendations should be given on the drawings or as directed by the Corridor Maintenance - Manager Bridges and Structures.

3.21 PERFORMING STEEL WORK UNDER WORK BLOCKS

3.21.1 Work that cannot be performed under traffic must be done during a work block. Required work blocks are given here as a guide only for general information. The required work blocks for particular projects should be looked at on a case-by-case basis.

(a)	Bottom flange field splice:	4 – 5 hours
(b)	Girder bottom flange replacement:	8 - 10 hours
(c)	Girder top flange replacement:	10 – 20 hours
(d)	Floor beam bottom flange replacement:	3 - 4 hours
(e)	Floor beam top flange replacement:	4 - 6 hours
(f)	Replace stringer from above deck:	6 - 8 hours
(g)	Replace stringer from under deck:	4 - 6 hours
(h)	Replace floor beam:	12 - 18 hours
(i)	Replace end brace frame:	4 - 6 hours
(j)	Replace intermediate brace frame:	3 - 4 hours
(k)	Replace one bay of top bracing:	3 - 4 hours
(l)	Replace one bay of bottom bracing:	2 - 3 hours
(m)	Repair concrete under bearings:	4 - 8 hours
(n)	Replace bearing stiffener:	2 - 4 hours

3.21.2 When dismantling part of a bridge during a work block, consideration shall always be given to maintaining structural integrity. Particular attention shall be given to compression and flexural members. Removal of bracing, lattice bars or other members providing lateral support could lead to failure. In some cases, stability shall be confirmed through calculation. Construction procedures shall be given on the drawings whenever restrictions apply during dismantling.

4 MAINTENANCE OF CONCRETE STRUCTURES

4.1 DESIGN CONSIDERATIONS

4.1.1 Concrete has been used extensively for construction of substructure elements. It has also been used for superstructures. Concrete structures are inspected as per Standard Practice Circular 4000. The condition of the concrete is noted in the inspection report. The most common problems found on concrete structures are:

(a) Scaling
(b) Delamination, Spalling
(c) Honeycombs
(d) Corrosion of reinforcing steel
(e) Cracking
(f) Deterioration of cement paste
(g) Abrasion

Other problems are also given in the SPC 4000 Commentary.

4.1.2 The extent of the repairs can be determined using the inspection report. A site visit should also be made for larger projects. If necessary, a more detailed condition survey can be conducted. There are many references on how to perform this survey, such as MTO's Structural Rehabilitation Manual and ACI Publication 201.1R-08.

4.1.3 Once all the data on the condition of the structure has been gathered, the structure must be evaluated. ACI Publication 364.1R-07 provides guidelines on performing this evaluation. The Corridor Maintenance - Manager Bridges and Structures shall be consulted if structural capacity is a concern.

4.1.4 One of the objectives of the evaluation is to determine what has caused the deterioration of the concrete. This must be done before any attempt is made to design a repair. There are many published cases of concrete repairs which have failed in a short period of time because the cause of the problem had not been addressed. The most common causes, which explain the deterioration of concrete railway structures, are freeze-thaw, corrosion of reinforcing steel and alkali- aggregate reactivity.

4.1.5 Deterioration of railway structures made of unreinforced concrete and not exposed to de-icing salts is often due to weathering caused by freeze-thaw cycles. The concrete mixes used to build the railway were typically of poor quality and did not have entrained air. A satisfactory repair method for these structures is to place a properly designed mix of air-entrained concrete over the original concrete. This repair will usually be successful if water is prevented from penetrating the interface between the new and existing concrete.

4.1.6 Structures exposed to marine environments or de-icing salts usually deteriorate due to the corrosion of the reinforcing steel in the presence of chloride ions. The corrosion process is similar to the one described in Section 3.16. An anode will form at the location where the concrete is contaminated with chlorides. The steel will be transformed into rust at this location. The expansion associated with the formation of rust will make the concrete burst and further expose the reinforcing steel. As part of the repair, the deteriorated concrete is removed and replaced with new chloride-free concrete. However, the concrete underneath or adjacent to the repair will still contain chlorides. Therefore, a new anode will be created in the concrete that was not repaired. The repaired area will act as the cathode. If the cathode is much larger than the anode, the corrosion rate will be high and the structure will deteriorate in a relatively short period of time. Reinforcing steel installed during repairs can be protected by painting it with an epoxy modified-cement corrosion coating such as SikaTop® Armatec-110 EpoCem® (or equivalent). This will help to prevent corrosion of the steel by the chlorides that will eventually enter the concrete. However, it will not prevent the galvanic reaction between the newly formed anode and cathode. If this is a concern, then using a single-component organic zinc coating, such as Fosroc® Galvafroid (or equivalent), shall be considered. This galvanic coating will help prevent the reaction between the anode and the cathode.

4.1.7 Some structures were built with concrete mixes containing reactive aggregates. These aggregates have reacted with the alkalis in the cement to form an expansive gel which in turn forces the concrete to burst. Structures suffering from alkali-aggregate reactivity usually are cracked in a random pattern. A white gel can often be seen leaching out of the cracks. If the structure is heavily reinforced, cracks will tend to be more parallel to the reinforcing rather than being randomly oriented. The presence of a white gel on the surface of the concrete is not sufficient to diagnose a structure with alkali-aggregate reactivity. The reactivity can only be confirmed positively by a microscopic examination of the concrete. The chemical reaction between the alkalis and the aggregate cannot be stopped. Typically, the deterioration process is slow. The reaction only occurs in the presence of water. It can therefore be retarded by covering the structure with an impervious layer of concrete or applying surface coatings and penetrating sealers, which have breathable properties to avoid retaining moisture in concrete. If the structure is in poor condition, the economics of replacing the structure should be investigated since it can be expected that deterioration will continue.

4.1.8 As part of the evaluation process, it may be decided to take core samples from the concrete structures in order to determine the quality of the concrete and the possible causes of deterioration. The following tests are often useful for the evaluation:

(a) Compressive strength
(b) Absorption test
(c) Determination of air-void parameters
(d) Half-cell potential
(e) Examination for alkali-aggregate reaction

4.1.9 Compressive strength will be useful to determine the quality of the concrete. The absorption test and air-void parameters will identify concrete which is susceptible to freeze-thaw. The half-cell potential will indicate if corrosion is occurring inside the structure. Examination of the concrete cores for reaction rims around the aggregate will positively confirm alkali-aggregate reaction. Other tests are given in ACI Publication 364.1R-07.

4.1.10 Different materials can be used for the repair of concrete structures. Portland cement concrete is the most often used. Proprietary products can also be used. The material to be used for the repair must be compatible with the existing concrete. Compatibility of modulus of elasticity, shrinkage, thermal dilatation and permeability should be considered. It is difficult to find a material that is fully compatible. A decision will have to be made as to which of these properties are the most important for the particular application.

4.1.11 Portland cement concrete can be easily designed to have an acceptable compatibility with all material properties of the existing concrete, except shrinkage. The existing concrete is usually old and does not shrink anymore. Concrete used for repairs is usually made with type 10 cement. Type 30 cement can also be used if high early strength is required. The possibility of increased shrinkage should be verified in this case. A typical specification for concrete mixtures is given in the table below and on drawing MC-09. The only admixtures allowed in the concrete mix are an air-entraining admixture and a non-retarding water-reducing admixture. Superplasticizers (also called high-range water reducing admixtures) and other admixtures can be used as per General Guidelines.

Table 4-01; Typical Specifications for Concrete Mixtures

Environment	Corrosive Environment		Mild Environment	
Concrete Thickness	t<200 mm	t>200 mm	t<200 mm	t>200 mm
Exposure Class	Class C-1	Class C-1		
Concrete Strength	35 MPa	35 MPa	30 MPa	30 MPa
Min. Cement Content	365 kg	365 kg	365 kg	365 kg
Max W/C Ratio	0.40	0.40	0.45	0.45
Coarse Aggregate	10 mm	20 mm	10 mm	20 mm
Entrained Air	7-10%	5-8%	5-8%	4-7%

4.1.12 Proprietary cement grouts and mortars can be used in numerous concrete repair applications. A variety of products exist on the market and have different characteristics such as non-shrink, low temperature, high early strength, resistant to freeze-thaw and de-icing chemicals, abrasive resistant, etc. These products generally have good compatibility with the existing concrete. Many manufacturers supply these products. Their suitability should be investigated for each individual application.

4.1.13 Phosphate magnesium grouts such as SET® 45 by Master Builders (or equivalent) are often used when high strengths are required within hours to resume traffic. This material has proven to be very successful in repairing concrete under bearings. This is a one component grout to which water is added to initiate the chemical reaction. This product is one of the easiest to use among the prepackaged grouts. It is not as sensitive as other products to workmanship errors that can occur in the field. Minimum grout thickness shall be the greater of; 12mm or per the Manufacturer’s instructions. The concrete surface shall be roughened to 6 mm. The minimum ambient temperature when using the grout shall be the greater of; 2 degrees Celsius or per the Manufacturer’s instructions.

4.1.14 Alternatively, a modified methacrylate mortar such as Sika® Pronto-11 or equivalent can be used instead of the phosphate magnesium grouts. This is a two-component grout with an optional third sub-zero component which can be used in cold weather. One of its main advantages is that it can be used at a minimum thickness of 6 mm. The required minimum ambient temperature is -10 degrees Celsius for the two component grout and -25 degrees Celsius with the sub-zero component. The drying of the concrete substrate and the mixing of the components are some of the most sensitive operations which can jeopardize the success of the repair. This material shall always be used carefully according to the manufacturer's instructions.

4.1.15 Epoxy grouts and mortars have a thermal dilatation coefficient quite different than the existing concrete so they shall not be used on railway structures.

4.1.16 In order to have a better understanding of material incompatibility problems and durability of concrete repairs, it is important to differentiate between thin and thick repairs. The thickness of thin repairs does not exceed 75 to 100 mm. Above these values, the repair is qualified as thick. Repairs often fail due to debonding at the interface between the new and existing concrete or due to cracking of the repair material. Thick repairs can be well anchored to the existing concrete with dowels and are sufficiently thick to resist some tensile forces. On the other hand, thin repairs cannot be anchored and only rely on the strength of the bond with the existing concrete. Since they are so thin, they cannot resist much tensile force and will easily crack. Portland cement concrete is an adequate repair material for most thick repairs. The material selected for thin repairs must have superior bonding properties and close compatibility with the existing concrete. Proprietary products are suggested for thin repairs. Selection should be made with the assistance of the manufacturer.

4.1.17 Proper concrete cover may increase the durability of a concrete repair. The minimum concrete cover shall be 75 mm in a corrosive environment and 50 mm in a mild environment.

4.2 CONCRETE REPAIRS

4.2.1 Proper workmanship is one of the keys to making a durable concrete repair. A standard procedure for the repair of concrete structures is given on Drawings MC-07A and MC-07B. This procedure is applicable to the repair details shown on Drawings MC-03 and MC-04. However, the general principles would apply to most thick repairs.

4.2.2 Concrete is first inspected by sounding with a hammer to determine the area to be repaired. This procedure is appropriate unless the cause of deterioration is found to be corrosion of the reinforcing steel (see Section 4.1). In this case, sounding will not identify the areas already contaminated with chlorides where corrosion has started but not yet affected the integrity of the concrete.

4.2.3 If it has been established that the deterioration is due to corrosion, then one of the following options shall be considered:

(a) If most of the surface has already deteriorated, then repair the concrete over the full surface
(b) If only a small part of the surface has deteriorated, then determining the area to be repaired by sounding may be acceptable
(c) On larger scale projects, consider conducting half-cell corrosion potential tests on the concrete surface. These tests are commonly done by road authorities for this application

4.2.4 The shape of the area to be repaired shall be as indicated on Drawing MC-07B, Figure 1. Re-entrant corners shall be avoided since stress concentrations occur when the concrete shrinks. This often leads to cracking of the repair.

4.2.5 Once the area has been delimited, a 20 mm deep saw cut shall be made around the repair. The saw cut will ensure that the side of the repair is not feather-edged. It will also prevent cracks caused by chipping to extend beyond the repair into the sound concrete. If the concrete cover is very small, then the depth of the saw cut shall be reduced so that it does not cut the existing reinforcing. If additional concrete must be removed to provide access for the pour as shown on Drawing MC-04, this should be considered when locating the saw cut.

4.2.6 The concrete can then be chipped with a 15 lb. jackhammer. A larger tool could damage the sound concrete underneath or adjacent to the repaired area. Such equipment shall not be used without the prior approval of the Bridge Specialist. Concrete shall be demolished until the sound concrete is reached. A sound concrete substrate will increase the probability of success of the repair. However, concrete shall not be removed to a point where the structural integrity would be affected. The maximum concrete removal shall be indicated on the drawings. The profile of the concrete substrate before the repair should be as indicated on Drawing MC-07B, Figure 2. A constant repair thickness will prevent the concrete from cracking at a weak section due to shrinkage stresses.

4.2.7 If, after chipping, more than half the circumference of reinforcing bars is exposed, then additional concrete should be chipped in order to provide a minimum of 25 mm clearance behind the reinforcing bars as shown on Drawing MC-07B, Figure 3. This will ensure that the coarse aggregate will get behind the bars. In this case, the concrete mixes shown on Drawings MC-09A and MC-09C should be used regardless of the total thickness of the repair. The smaller coarse aggregate will get more easily behind the bars.

4.2.8 All bars corroded by more than 30% of their diameter, or deemed a concern by the Consultant, shall be cut and lap spliced with a new reinforcing bar. The length of the splice is shown on drawing MC-07B, Table 1. Additional concrete might have to be removed to lap the bars. Attention should be given to the shape of the demolition to avoid sharp corners that will eventually cause cracking. Alternatively, a mechanical splice can be used.

4.2.9 All reinforcing steel shall be sandblasted in order to remove corrosion products that will prevent proper bond of the concrete. Loose concrete shall then be removed using oil-free compressed air or a light sandblast.

4.2.10 The reinforcing steel shall be protected from corrosion using an epoxy modified-cement corrosion coating such as SikaTop® Armatec-110 EpoCem® (or equivalent) or a single component organic zinc compound such as Fosroc® Galvafroid (or equivalent).

4.2.11 The SikaTop® Armatec-110 EpoCem® shall also be applied as a bonding agent to the perimeter walls of the repair. This will improve the bond to the walls and reduce the possibility of debonding due to shrinkage. Wet curing is the best process to cure the concrete. This shall be done for seven days. Alternatively, a chemical curing agent such as Sikagard®CureHard (or equivalent) can be used.

4.2.12 Whenever practicable, a breathable concrete sealer should be applied on the repaired area. Silane based sealers with 40% solids such as MasterProtect H400 (or equivalent) are the most durable. The concrete must be at least three weeks old before this product is applied. Greater penetration is obtained on dry surfaces. A light sandblast should be done on the concrete surface prior to application of the sealer.

4.2.13 Concrete repairs can also be made using shotcrete. This method may be more practical in some cases since formwork is not required; however, the durability is not as good as for cast-in-place concrete. A standard detail for shotcrete repairs is given on Drawing MC-05. The general principles given above for the preparation, curing and sealing are usually applicable. More information on shotcrete is given in ACI Publication 506R-16.

4.2.14 The dry-mix process is the more practical for railway applications. The shotcrete mix should contain steel fibers in order to reduce plastic shrinkage. Silica fume, added to the mix in proportions varying from 7 to 10 percent of the weight of cement, will improve the adhesion of the wet shotcrete on the concrete substrate. The use of an air-entraining powder shall also be considered.

4.2.15 Concrete under bearings is repaired as shown on Drawing MC-01. A minimum clearance of 100 mm is usually required under the bearing to perform the repair. This is obtained by jacking the span and/or removing the bearing. The surface shall be roughened to a 6 mm profile in order to improve the bond of the repair material. These can be made grout-tight with a urethane compound supplied in a spray can. The repair material is then installed and allowed to cure before the span is lowered and traffic resumes.

4.2.16 The following practices shall be avoided when repairing concrete under bearings:

(a) Improper drying of the concrete surface (when drying is required by manufacturer)
(b) Using a dirty pale for batching
(c) Eyeballing batching quantities
(d) Batching with water from the river

(e) Improper mixing of the repair material
(f) Trowelling the mix in place
(g) Heating the repair material with a torch
(h) Retempering the mix

4.2.17 In cold weather, the extended curing times for the repair material may be problematic when the repair is made during short work blocks. In this case, the water used for batching can be heated. A small enclosure, just large enough to go around the bearing, can also be made around the bearings. Hot air from a hand-held dryer can be blown into the enclosure.

4.2.18 Concrete repairs can also be made underwater using divers. The procedure is similar as for the above water repairs. Pneumatic or hydraulic equipment can generally be used. The latter should be considered for depths greater than 6 metres (20 ft). The surface of the piers must be thoroughly cleaned using a high-pressure water jet in order to determine the extent of the repairs. The saw cut can be made with a hydraulic saw mounted on rails. The deteriorated concrete can be removed using either a pneumatic or hydraulic jackhammer. Reinforcement can be cut with arc-air cutting equipment. A high-pressure water jet should be applied to the concrete and reinforcing surface in order to remove loose concrete and clean reinforcing. Abrasives can be added to the water jet in order to increase its effectiveness. The environmental impacts associated with the use of abrasives must however be investigated. Dowels can be drilled using a pneumatic or hydraulic drill.

4.2.19 Repairs can be made using a trowel-applied mortar or by pumping an underwater concrete mix into the formwork. In this latter case, the key to success is in preventing the water from coming into contact with the fresh concrete. Otherwise, the cement will be washed out of the mix, therefore reducing the strength and durability of the repair. In order to prevent washout, the hose should be sealed in order to prevent water from entering. The hose is held at the bottom of the formwork and concrete is pumped. The seal will then be expelled from the hose and float to the surface. A mound of fresh concrete will form around the hose therefore reforming the seal. It is of prime importance to always keep the hose inside this fresh concrete seal. Once the formwork has been filled, additional concrete is pumped in order to expel out of the formwork the fresh concrete that was part of the original seal. Superplasticizers are usually added to the mix in order to bring it to a self-leveling consistency. This waives the need for vibrating the concrete. An anti-washout admixture should also be used to reduce the risk of washout of the concrete mix.

4.2.20 Alternatively, concrete can be placed using the preplaced aggregate method. Injection ports are first installed into the formwork. The coarse aggregate is then placed into the formwork. Grout is injected starting at the bottom of the formwork and progressing towards the top.

4.2.21 Other methods for surface preparation and placement are given in the book "Underwater Concreting and Repair," Andrew McLeish, Edward Arnold Editors.

4.3 CRACK REPAIRS

4.3.1 Concrete often cracks because of its inability to resist tensile stresses. Cracks are usually a problem in concrete structures because they allow water (and dissolved aggressive chemicals) to get into the concrete. This eventually leads to corrosion of the reinforcement or deterioration of the concrete due to freezing action. Although most cracks are not of any structural concern, some of them might affect the integrity of the structure. ACI Publication 224.1R-07 contains relevant information on concrete cracking.

4.3.2 In typical railway applications, only relatively wide cracks (larger than 0.3mm) are required to be repaired. The discussion in this Section applies to wide cracks only. The repair of hairline cracks is not covered.

4.3.3 Some of the most common causes of cracking are as follows:

(a) Plastic shrinkage
(b) Restrained drying shrinkage or thermal shrinkage
(c) Construction overloads
(d) Restrained temperature movements
(e) Differential settlements
(f) Structural overloads

4.3.4 Before an attempt is made at designing a crack repair procedure, it is important to determine the cause of cracking. Failure to do so could cause the crack to reappear shortly after the repair.

4.3.5 Crack repair techniques can be classified in two groups: structural and non-structural crack repairs. The following are some common repair techniques that can be used for railway applications:

4.3.5.1 STRUCTURAL REPAIRS

(a) Stitching
(b) Pinning
(c) Injection with epoxy

4.3.5.2 NONSTRUCTURAL REPAIRS

(a) Routing and sealing
(b) Injection with polyurethane

4.3.6 Structural repairs shall not be made when the crack is suspected to be moving. Structural repairs are stiff and will not accommodate the movement. Cracking may then reoccur. Cracks caused by ongoing differential settlement, regular structural overload or large temperature variations are expected to move. Non-structural repairs are more flexible and may be better suited for these applications.

4.3.7 Stitching and pinning techniques are shown on Drawing MC-11. These repairs will restore some of the structural integrity but do not prevent water from entering the crack. The standard details shown on Drawing MC-11 restore across the crack an apparent shear and tensile strength of 1 MPa for pinning and an apparent tensile strength of only 1.5 MPa for stitching.

4.3.8 The routing and sealing technique is also shown on Drawing MC-11. This repair will not restore structural integrity but will prevent further deterioration that would be caused by water entering the crack. Since the bond between the sealant and the concrete has limited strength and can be affected by less-than-perfect workmanship, it is debatable whether this repair can accommodate large crack movements.

4.3.9 Injection with epoxy will both seal the crack and bond it back. The crack should preferably be dry before injection. However, damp cracks have been injected successfully with moisture-tolerant epoxies. Actively leaking cracks cannot be injected with epoxies. Epoxies are available as low- viscosity material called resins and also as thicker material called gels. Resins are most often used for crack repairs. Gels that are appropriate for injection have a consistency similar to toothpaste. Gels are useful when injecting retaining walls or other structures where the backside of the crack cannot be capped. Gels usually work best with cracks wider than 1.5 mm and at ambient temperatures higher than 15 degrees Celsius. However, it is possible to use them in more difficult conditions.

4.3.10 Injection with polyurethane resin will only seal the crack and not restore structural integrity. Polymerized resin is flexible and can accommodate crack movement. Resins usually come as a one component system that reacts with the water at the nozzle and/or in the crack to initiate polymerization. The crack must therefore be wet in order to polymerize the resin. The resin can also be injected in some actively leaking cracks. Concrete structures may be injected with cement grout, but this practice is not common. This method is more suitable for injection of masonry structures (see AREMA Chapter 8) and rock foundations since the cracks are usually larger than those in concrete structures.

4.3.11 Injection procedures shall be designed for each particular application. The design shall specify the type of material to be injected, recommendations for port spacing, maximum injection pressures, procedure for progressing from port to port, required slow orders or work blocks, and any other particularities of that specific project. General steps for injection of epoxy and urethane are given here as a guide:

4.3.12 EPOXY INJECTION PROCEDURE

4.3.12.1 Procedure

(a) Clean concrete surface 25 mm on each side of the crack
(b) Install injection ports on crack surface
(c) Install capping material on crack
(d) Inject crack from the first port in the sequence
(e) Move to next port in the injection sequence, inject and so on until completion of injection
(f) If aesthetics is a concern, remove capping material once the resin has cured

4.3.12.2 Concrete surfaces are best cleaned with a hand wire brush. Mechanical wire brushes and other powered equipment have a tendency to fill the crack with dust, which makes injection difficult. Proper cleaning will ensure good adhesion of the cap to the concrete surface.

4.3.12.3 There is no strict rule for port spacing when injecting with resins. For typical railway applications, ports need not be any closer than the thickness of the concrete. However, for application on thin or thick members, port spacing shall not be less than 200 mm or more than 900 mm. Ports shall be installed at wider crack openings if possible and at locations where cracks intersect.

4.3.12.4 When injecting with gels, port spacing shall not be less than the thickness of the concrete. This is based on the fact that gels will usually penetrate as deep into the crack as they do along the crack. This ensures that the epoxy has fully filled the crack when the gel appears at the adjacent port.

4.3.12.5 Concrete dust created during drilling can get into the crack and make injection difficult. The holes shall be drilled using a vacuum-bit drill. Alternatively, the dust can be removed from the crack using a shop vacuum cleaner.

4.3.12.6 Installation of the capping material is the most critical stage of the job. Leaking caps will jeopardize the success of an injection project. Manufacturer's recommendations for the batching and mixing of the gel shall be followed closely. Preparation shall be done in small batches that can be installed during the recommended working life. If the material becomes too stiff, then it shall be discarded. The working life has been exceeded. The cap should be 50 mm wide by 3 to 5 mm thick.

4.3.12.7 If a cap leaks during injection, the following methods have been recommended for repair (from book by Trout; see reference at the end of this section):

(a) Knead a 25 mm plug of epoxy putty until it is warm. This putty is the type used by plumbers for sealing leaks. Just before placing the putty against the leak, wipe the surface dry with a cloth. Hold the putty for a minute or two, then resume injection at reduced pressure.
(b) Rub a paraffin block over the leaking area until the paraffin plugs the leak. This will only be strong enough to prevent the leak from draining by gravity. Injection must be stopped until the resin has set.
(c) Put a rag over the tip of a 75 mm nail and pound it in at the leak, then resume injection.

4.3.12.8 One of these methods shall be applied to stop the leak and allow injection to proceed. If not, then they may be able to prevent the leak from draining by gravity. In this case, wait for the resin in the crack to set before resuming injection. If this is not successful, let the resin drain out. Repair the faulty cap and start over. In any of these cases, it might be a good idea to inspect the cap before resuming injection to find other locations where leaking might be a problem.

4.3.12.9 On small injection projects for short-term loads only, the capping material shall be a fast set epoxy gel such as Sikadur Injection Gel - Fast Set (or equivalent). This will allow workers to perform the capping in the morning and inject in the afternoon. Cracking of the capping material due to the overnight temperature variation will therefore be prevented. If the capping has to be done over several days, then use an epoxy gel such as Sikadur Injection Gel.

4.3.12.10 Flushing of the crack is often specified in injection procedures; however, this is difficult to properly execute, and mitigated success is usually achieved. In lieu of flushing the crack, it is recommended during injection to let the resin drool from the injection ports until appearance of clean material.

4.3.12.11 When using gels, the injection sequence shall progress from the widest part of the crack to the narrowest. Start injection at the first port in the sequence until clean material drips out of the adjacent port. The injection can then be stopped and moved to the third port. The port that has dripped must be skipped. Inject every second port until the end of the crack is reached. Then return to the starting point and inject the remaining ports.

4.3.12.12 When using resins, the injection sequence shall progress from bottom to top for vertical cracks and from the widest part of the crack to the narrowest for horizontal cracks. Start injection at the first port in the sequence until clean material drools out of the adjacent port. Block the ports that drool and proceed with injection until the specified maximum injection pressure is reached or the crack will not accept any more resin (refusal). Then, hold the pressure for about one minute in order to ensure that the resin has penetrated the narrower parts of the crack. Only then should the injection port be blocked and injection proceed to the next port in the sequence. Ports that have drooled do not have to be injected. Ports located at wide crack openings should preferably be chosen for injection. Obviously, these ports should be marked before the cap is installed over the crack.

4.3.12.13 The capping material need not be removed after the epoxy has set. Removing the cap can be a tedious job. If the cap has to be removed, it can be done using a rotary grinder.

4.3.13 POLYURETHANE INJECTION PROCEDURE

4.3.13.1 Procedure:

(a) Clean concrete surface to expose crack
(b) Seal crack if required
(c) Install injection ports by drilling at 45 degrees to intercept crack at mid-depth of the member
(d) Flush crack thoroughly with water
(e) Inject crack from the first port in the sequence
(f) Move to next port in the injection sequence, inject and so on until completion of injection
(g) Remove resin extrusion or seal at the surface

4.3.13.2 The concrete surface is best cleaned with a hand or mechanical wire brush. The use of a mechanical wire brush or other powered equipment is not a major concern for polyurethane injection since the crack is not injected from the surface and can also be flushed with water.

4.3.13.3 Sealing of the crack surface is usually not required when injecting polyurethane resins. This is because the resin polymerizes almost instantly. In some cases, when the crack is very wide or leaking large flows, a surface seal might be required. In this case, the use of a foam backer rod soaked in polyurethane resin is often sufficient.

4.3.13.4 There is no strict rule for port spacing when injecting polyurethane resins. For typical railway applications, ports need not be any closer than the thickness of the concrete. Ports shall be installed at wider crack openings if possible and at locations where cracks intersect. Port spacing can be verified during flushing of the crack. When injecting water through one port, the spacing will be adequate if flow occurs at the adjacent port.

4.3.13.5 Concrete dust created during drilling can weaken the anchorage of the injection port onto the concrete. The holes shall be drilled using a vacuum bit drill. Alternatively, the dust can be removed from the hole using a shop vacuum cleaner.

4.3.13.6 The injection sequence shall progress from bottom to top for vertical cracks and from the widest part of the crack to the narrowest for horizontal cracks. Start injection at the first port in the sequence until the resin drools out of the adjacent port. Injection can then be stopped and moved to the next port. It is not necessary to inject until refusal with polyurethane resins.

4.3.13.7 It is important to note the differences between the procedures for injecting epoxy and polyurethane. One should be familiar with the particularities of the material to be injected before starting the job.

4.3.13.8 All that is needed for injecting epoxy or polyurethane is a dual-cartridge caulking gun. If the cracks are large or numerous, then the use of a pump is recommended. Piston pumps are recommended for this application. Gear pumps have also been used successfully. Backup equipment shall be on standby.

4.3.13.9 Epoxy resin and gels of different viscosity are available for different applications. ASTM Standard C-881-15 is a good tool to properly select injection materials. However, materials should not be selected without consulting the manufacturer. The latter can then advise on materials, equipment and a specific procedure for the particular application. Gels can often be pre-conditioned in order to reduce their viscosity. This is best accomplished by putting the gel in a pail of water and heating it with a propane torch. The manufacturer can provide recommendations on pre-conditioning gels.

4.3.13.10 Epoxy can also be injected underwater using divers. Some epoxy formulations are specifically designed for this application. The procedure is similar to the above-water application. Usually, the concrete will have to be thoroughly cleaned in order to determine the extent of the cracking. This is best done using a high-pressure water jet. The crack can be capped using an underwater epoxy gel such as Sikadur® UW Gel (or equivalent).

4.3.13.11 Polyurethane resins can be classified as hydrophilic (likes water) or hydrophobic (dislikes water). Hydrophilic resin will require a lot more water than hydrophobic resin for polymerization. It will also retain more water after polymerization. Each type has its advantages and disadvantages. Hydrophilic resins may shrink if the cracks eventually get dry. Hydrophobic resins are not as flexible and may break if larger movements are anticipated. Hydrophilic resins may be better suited for injection of cracks which flow continuously. The manufacturer should be consulted when selecting the resin for each particular application.

4.3.13.12 Maximum injection pressures should be specified for each application. For epoxy injection, pressures of 200 psi for unreinforced concrete and 300 psi for reinforced concrete are usually adequate to inject the cracks successfully without causing any distress to the structure. If the structure is severely deteriorated, then reduced maximum injection pressures should be considered. For polyurethane injection, the required injection pressures can be much higher than for epoxy. This is not a problem as long as the resin is flowing. Pressure should be quickly reduced if refusal is encountered. The polymerization of the resin is accompanied by swelling which also induces pressure in the surrounding concrete. When planning polyurethane injection in unreinforced concrete, these issues should be discussed with the manufacturer.

4.3.13.13 To increase the possibility of success, injection work on concrete structures shall only be done by Companies experienced in this type of application. If this is not possible, then assistance from the manufacturer of the resin or a specialized contractor shall be sought.

4.3.13.14 Relevant information concerning epoxy injection can be found in the book Epoxy Injection in Construction, John Trout, The Aberdeen Group, 1998. For polyurethane injection, refer to the articles Stopping Leaks with Polyurethane Grouts, Joseph Salomon and Mike Jaques, Concrete Repair Digest, June 1994, and Choosing the Right Polyurethane Grout, Brent D. Anderson, Concrete Repair Digest, December 1996.

4.4 PREPARATION OF CONCRETE REPAIR DRAWINGS

4.4.1 Before any drafting is attempted, it is important to verify that the original drawings of the structure actually represent what is on site. Check all sources of information (bridge list, photos, plan files, bridge folder and correspondence file if necessary) to make sure that no work has been carried out which would affect the repair. If any records are misleading or incorrect, make a note of correction or explanation wherever the misinformation occurs and date and initial your comment.

4.4.2 Standard size drawings (559mmx864mm (22''x34'')) should be used for all work which would influence future maintenance of the bridge. Full size drawings shall be numbered and dated unless otherwise instructed. Drawing numbers are to be given by the Bridge Specialist at some time during the preparation of the drawings. All drawings issued prior to signing by the Engineer shall be stamped "PRELIMINARY." Minor work which would not affect future maintenance of the structure can be done according to sketches or standard drawings.

4.4.3 The original issue of the drawings is not identified with a revision letter. A comment can still be indicated in the revision block such as "PRELIMINARY" or "ISSUED FOR TENDER." Revisions of the drawings are then identified by the letters A, B, C, etc., in the bottom right corner of the drawing. Revised items shall be identified with a revision mark located as close as possible to the revision. A description of the revision such as "BAR 25099F MODIFIED" is given in the revision block. Alternatively, a comment such as "ISSUED FOR FABRICATION" can be indicated. If too many items are revised and the description does not fit into the revision block, then the comment "GENERAL" can be indicated instead.

4.4.4 On the drawings, the extent of the repairs shall be shown on a general view of the structure which is drawn to scale. The typical repair details showing the expected depth of the repair, dowels, reinforcing details, joints, etc., shall be drawn to a large scale. Reference to standard drawings can also be made. If the dimensions of the repairs are known accurately, a bar list shall be made. Bar marks shall be given as per Drawing C-7 of METROLINX's General Guidelines for Design of Railway Bridges and Structures. Other detailing standards are found in the RSIC manual. Concrete covers shall conform to CSA Standard A23.1-14. A repair procedure giving all the steps to be followed and specifying all products to be used shall also appear on the drawing.

4.4.5 When preparing repair drawings, attention shall always be given to maintaining the structural integrity of the bridge. The maximum depth of concrete that can be removed without affecting the structural integrity shall be indicated on the drawings. All work to be done under work block or slow orders should be clearly indicated as such. Any shoring required during the repairs shall be properly designed or specified.

4.4.6 Once the job is finished in the field, place the appropriate stamp on the drawing indicating that the job has been completed in whole or in part and note the date completed. Indicate any work that has been done differently than as indicated on the drawing. A revision shall be made to the drawings with the comment "AS CONSTRUCTED." Work completion comments and revision notes can be printed by hand.

4.5 STANDARD NOTES AND SPECIFICATIONS FOR CONCRETE REPAIRS

4.5.1 SPECIFICATIONS

(a) DESIGN AND WORKMANSHIP AREMA Chapter 8
(b) CONCRETE CSA CAN3-A23.1 & A23.2 M (2014)
(c) REINFORCING STEEL CSA G30.18-09 (R2014)

4.5.2 NOTES:

(a) CONCRETE SHALL BE XX MPa AT 28 DAYS
(b) REINFORCING STEEL SHALL BE 400 MPA (400W or 400R) BILLET STEEL DEFORMED BARS
(c) CONCRETE COVER SHALL BE A MINIMUM OF XX mm UNLESS NOTED
(d) ALL EXPOSED EDGES SHALL BE GIVEN 20 x 20 CHAMFERS

4.5.3 REFERENCES:

(a) EDRMS
(b) Metrolinx Drawing Number S-XXX
(c) Inspection report dated year/month/date
(d) METROLINX's General Guidelines for Design of Railway Bridges and Structures

4.5.4 NEAREST STATION:

(a) xxxxxxx, mi. xxx.x xxxxxx subdivision

5 MAINTENANCE OF TIMBER STRUCTURES

5.1 MAINTENANCE OF TIMBER STRUCTURES

5.1.1 Timber structures are inspected as per Standard Practice Circular 4000. The condition of the timber is noted in the inspection report. Timber is inspected visually, by sounding and by drilling.

5.1.2 The most common types of problems found on timber bridges are the following:

(a) Rot of piles, caps and stringers
(b) Structural failure of members
(c) Crushing of the timber at points of bearing
(d) Excessive sway of the bridge
(e) Enlarged bolt holes

Other problems are also given in the SPC 4000 Commentary and SPC 4300.

5.1.3 The structural integrity of the bridge is evaluated as per Standard Practice Circular 4300. Guidelines are given on the rejection of stringers, caps and bracing, the allowable rot in piles, the number of piles that can be posted, etc. The sway of the bridge is evaluated by inspection under traffic. Loose bolts and cracked lateral bracing may also be signs of excessive sway.

5.1.4 Standards for maintaining timber bridges are given in Standard Practice Circular 4300. The typical solutions used to maintain timber bridges are as follows:

(a) Replacement of structural members in kind
(b) Posting piles
(c) Framing bents
(d) Double-capping
(e) Shimming at points of bearing
(f) Tightening of existing bolts or installation of new bolts in new holes
(g) Installation of additional bracing
(h) Other maintenance solutions given in SPC 4300

5.1.5 Rotten piles can be repaired either by posting individual piles or framing the entire bent. Bents are framed when the number of reject piles exceeds the limits for posting given in SPC 4300. The bent is dug down to sound timber and all the piles are cut. A sill is placed on top of the piles and the new framed bent is installed on top of the sill. A typical framing detail is given on Drawing R3A-3.9. If the number of rotten piles does not exceed the posting limits, the piles can be posted as shown on Drawing MT-01A. In this case, only the rotten piles are cut. A sill and a post are placed on top of these piles. Posted piles shall be tied to the adjacent sound piles as shown on drawing MT-01B. If the rot is at the level of the cap, the pile can be cut short and a second cap installed underneath the original one. Double-capping is also done to obtain better load distribution within the pile bent, to correct the elevation of pile bents which have settled, or to replace thin shims under stringers. Details for double-capping are given on Standard Drawing MT-02.

5.1.6 If a bridge sways excessively, all bearing points shall be inspected to ensure that the sway is not caused by structural members not bearing. Bolts shall be inspected to make sure that they are tight and the bolt holes are sound. If shimming and tightening the bolts does not resolve the problem, then additional bracing can be installed.

5.1.7 More information on maintenance of timber structures can be found in METROLINX Lesson 33 entitled "Timber Trestle Maintenance - Recommended Methods".

6 MAINTENANCE OF MASONRY STRUCTURES

6.1 MAINTENANCE OF MASONRY STRUCTURES

6.1.1 Masonry has been used extensively for the construction of substructure elements and culverts. The stones most often used were granite, sandstone and limestone. Masonry structures are inspected as per Standard Practice Circular 4000. The condition of the masonry is noted in the inspection report. The most common problems found on masonry structures are:

(a) Deterioration of the mortar in joints
(b) Broken, spalled or weathered stones
(c) Cracked stones

Other problems are also given in the SPC 4000 Commentary.

6.1.2 Deteriorated masonry joints can be pointed in order to secure the masonry blocks or just to prevent future deterioration of the element. The latter is most important on horizontal surfaces such as bridge seats. The water tends to accumulate in the open joints and seeps down into them. The water then expands when freezing and deteriorates the joints even further down. If pointing is done as preventive maintenance on substructure elements, the bridge seat shall be done first. A delay of one or two years is then given to allow the water to seep out of the masonry by the joints in the vertical faces. The joints in the vertical faces are then pointed.

6.1.3 If the joints are not deteriorated to depths greater than 125 mm, pointing can be done according to Standard Drawing MM-01. If the deterioration is deeper, then the joints shall be injected with a cement grout. In this case, the pointing is done exactly as per MM-01 except that a stiff cementitious mortar such as SikaRepair® 223 is used to point the joints. The pointing does not need to go any deeper than 75 mm from the surface since grout will be injected underneath. Obviously, injection ports must be installed before the joints are pointed. If the deterioration is deeper than 125 mm, and injection is not considered practical, the joints can be pointed in multiple passes not exceeding 125 mm.

6.1.4 Granite stones are usually very durable and do not often require maintenance. Stones made of limestone and sandstone are more porous and have a greater tendency to break, spall or weather. In these cases, the stones shall be replaced according to Standard Drawing MM-02A, MM-02B or MM-02C. Stones which are not located under the bearings are replaced with cast-in-place concrete as shown on drawing MM-02A. Vent holes must be installed through the stone above in order to prevent entrapment of air. Stones under bearings can be replaced with a precast concrete block during a work block as shown on MM-02B. Injection of non-shrink grout is specified to fill the voids that are not filled by the precast block. Alternatively, stones under the bearings can be removed and replaced with cast-in-place concrete after the span has been supported on a steel pedestal. This concept is shown on drawing MM-02C. Stones with several cracks are defined as broken stones and shall be replaced as discussed.

6.1.5 Cracked stones can either be replaced as discussed above or injected. Epoxy, cement grout or polyurethane can be used to inject the crack. Before attempting any of these repairs, it is important to determine the cause of cracking. Cracks caused by ongoing differential settlement, regular structural overload or large temperature variations are expected to move in the future. This shall be considered when selecting the repair material. Cracks in masonry structures can either be limited to individual stones or can go through a series of stones. The latter can be a sign of serious structural distress and shall be investigated. As discussed in Section 4.3, epoxy will not only seal the crack but also bond it back. The same applies to cement grout. These materials shall not be used if the crack is expected to move. If a structural repair is not required, then a polyurethane resin can be used. The polymerized resin is flexible and can accommodate potential crack movements.

6.1.6 If injection is considered for the repair of joints or cracks, the following factors shall be taken into account:

(a) Size of cracks and joints to be injected
(b) Linear feet of cracks and joints to be injected
(c) Presence of a rubble core inside the masonry
(d) Expected volume of material to be injected

6.1.7 If the scope of the job is considered small, then injection with proprietary epoxies or urethanes is a good idea. If the scope of the job is considered large, then cement grout shall be injected. The savings in material costs will more than compensate for the cost of the additional logistics related to grout injection. Injection with cement grout is a good option for masonry structures since the size of the cracks and joints to be injected is usually quite large. With the proper grout formulation, cracks as narrow as 1 mm can be injected.

6.1.8 The procedure for grout injection is as follows:

(a) Prepare joints as indicated on Drawing MM-01
(b) Clean crack surface
(c) Install injection ports
(d) Install pointing material in joints
(e) Install capping material on cracks
(f) Flush the crack with water
(g) Inject crack from the first port in the sequence
(h) Move to next port in the injection sequence, inject and so on until completion of injection
(i) If aesthetics is a concern, remove capping material on cracks once the grout has cured

6.1.9 Cleaning of the crack surface is best done using a medium-to high-pressure water jet. The joints can be prepared with the water jet or a 15 lb. jackhammer. Care shall be taken when using high water pressures as softer stones such as limestone and sandstone can be damaged. A hooked chisel shall be used in combination with the jackhammer to prevent additional damage to the structure due to the impact of the jackhammer.

6.1.10 There is no strict rule for port spacing when injecting cement grouts. A spacing of 300 to 600 mm is usually adequate. Ports shall be installed at locations where cracks and joints intersect and at wider crack openings if possible. Injection ports in joints can be inserted before the joint is pointed. Ports in cracks usually have to be drilled in.

6.1.11 As discussed in Section 4.3, proper cap installation is of prime importance. This obviously also applies to the pointing material. Leaking caps are a problem. The method to repair leaking caps suggested in Section 4.3 may help in resolving it.

6.1.12 The pointing material for joints shall be a stiff cementitious mortar such as SikaRepair® 223 (or equivalent). For cracks, the capping material shall be Sikadur® 31, Hi-Mod Gel (or equivalent).

6.1.13 The crack is flushed with water. This not only serves to clean the crack, but also as an injection test to verify continuity between the ports and determine expected quantities to be injected.

6.1.14 The injection sequence shall progress from bottom to top. Start injection at the first port in the sequence until grout drools out of the adjacent ports at the same elevation. Block the ports that drool and proceed with injection until the material drools out from a port at the next highest elevation. The injection port shall then be blocked. Injection can then proceed from a port at the next highest elevation. Ports that have drooled do not have to be injected.

6.1.15 The capping material on cracks need not be removed after the grout has set. Removing the cap can be a tedious job. If the cap has to be removed, a rotary grinder may be used.

6.1.16 The equipment required for cement grout injection is either a Moyno type screw pump or a Hany type piston pump with a pressure capacity of 500 psi. Mixers can either be high speed colloidal or centrifugal. Backup equipment shall be on stand-by.

6.1.17 Many different grout formulations will work for this application to different degrees of success. Specifications for grout mix should be as follows:

(a)	Compressive strength at 28 days	
(b)	(ASTM C942-15):	25 MPa
(c)	Washout test:	< 10%
(d)	Mini-slump:	130 - 190 square cm
(e)	Marsh cone:	30 - 60 seconds
(f)	Grout temperature:	15 - 25 degrees
	Celsius	
(g)	Plastic viscosity:	60 to 80 cp
(h)	Apparent viscosity at 3 rpm: < 1000 cp	
(i)	Final set:	10 - 15 hours

6.1.18 The following grout formulation has been extensively tested in the laboratory and in the field to confirm its suitability for injection in masonry structures. This formulation is given here as a guide. Trial batches of this formulation must be made with the local materials in order to confirm specification requirements. Batching weights are arbitrarily given per 100 kg of cement.

(a)	Cement type 30:	100 kg
(b)	Silica fume :	6 - 8 kg
(c)	Water:	40 - 55 kg
(d)	Whelan gum anti-washout admixture:	.015 - .030 kg
(e)	Naphthalene based superplasticizer:	1.2 - 1.8 kg

6.1.19 The anti-washout admixture is used to prevent the grout from segregating when injected under pressure. The superplasticizer makes the grout fluid enough to ease injection. Type 10 cement and other admixtures are not recommended.

6.1.20 Quality control testing during injection shall consist of:

(a) Grout temperature
(b) Washout test
(c) Marsh cone
(d) Mini slump

6.1.21 When starting injection, these tests shall be conducted on the first three batches. Afterward, they shall be conducted every hour or 500 litres, whichever comes first. Grout cubes shall also be taken every 2,000 litres for compression tests. A minimum of three cubes shall be tested on each project.

6.1.22 Cement grout can also be injected underwater using divers. Since an anti-washout admixture is already incorporated, the same grout mix can be used underwater. The surface of the pier must be thoroughly cleaned with a high-pressure water jet in order to determine the extent of deterioration. Underwater capping and pointing is possible with an underwater epoxy gel such as Sikadur® UW Gel (or equivalent).

6.1.23 Typical injection pressures vary from 50 to 75 psi. Structural integrity must be assessed. The maximum injection pressure for the application shall be specified.

6.1.24 If a rubble core is present inside the masonry or severe core deterioration is expected, the amount of grout injected into the masonry may be substantial. In this case, reducing the water in the mix to obtain a thicker grout mix shall be considered. Unless the substructure shows sign of distress, the core of masonry substructure elements usually does not need to be injected.

7 MAINTENANCE OF BRIDGE DECKS AND WALKWAYS

(a) The role of the deck is to transfer the load from the rail to the main members, maintain the gauge and alignment of the track, and prevent the rails from buckling during hot weather. The most common decks for railway structures are open decks and ballasted decks. A few bridges have direct fixation decks.

7.1 OPEN DECKS

7.1.1 An open deck consists of timber ties laid perpendicular to the rail and closely spaced. Open decks are the most common type of deck found on existing steel bridges.

7.1.2 Bridge decks are inspected according to SPC 4000. The condition of the deck is noted on the inspection report. The ties are inspected visually, by sounding and by drilling. The most common problems found on open decks are:

(a) Tie rot around spikes and under tie plates
(b) Rail fastening problems on lines with high tonnage and high speeds
(c) Checks and splits on the surface of the ties
(d) Cracking at the corners of daps
(e) Splitting of ties due to horizontal shear
(f) Ties bearing on gusset plates

Other problems are also given in the SPC 4000 Commentary and SPC 4001.

7.1.3 The structural integrity of an existing deck is evaluated according to SPC 4001. The criteria for identifying reject ties are given in this document. The allowable percentage of reject ties on the entire deck is given for different possible traffic conditions. If the number of reject ties on a bridge is greater than allowable, the deck shall be replaced or spotted. Particular attention shall be given to the presence of consecutive reject ties. Two consecutive reject ties shall have at least one sound tie on each side. The maximum distance between ties with firmly holding spikes is also given in SPC 4001 for continuous welded rail and jointed rail.

7.1.4 Maintenance of bridge decks is covered in detail in SPC 4001. Some common solutions for maintaining timber decks are:

(a) Plugging spike holes and re-spiking or double-spiking
(b) Replacement of tie plates with bridge plates
(c) Spot renewing of deteriorated ties
(d) Deck renewal with ballast deck or open deck
(e) Installing shims to lower gusset plates
(f) Other maintenance indicated in SPC 4001

7.1.5 If track problems are present on a bridge but the tie is still structurally sound, the problem may possibly be solved by chemically plugging the existing spike holes and re-spiking. If this is believed to be insufficient, double-spiking can be done by adding two extra spikes in the open holes of the tie plates. On lines which carry heavy traffic with high speeds, the conventional tie plate system has been found inadequate to properly maintain the gauge. Also, tie plates are often found to be crushing into the tie. These problems are even more frequent on curved track. On these bridges, the tie plates are replaced with bridge plates. Bridge plates are discussed in Section 7.6.

7.1.6 If there are reject ties on a bridge, but not in a quantity sufficient to justify the replacement of the entire deck, spot renewing of the deteriorated ties is done in order to ensure that two consecutive reject ties have at least one sound tie on each side. For this purpose, it is preferable to use partly worn ties of similar age as the existing ones. The use of new ties, which are much stiffer than the existing ones, will introduce stiffness transitions in the riding surface which will cause higher impacts on the bridge. ·

7.1.7 If a deck needs to be replaced, it shall be verified whether the bridge has enough capacity to carry a ballasted deck and if the track profile can be raised to accommodate it. The capacity of the foundations and the stability of the abutments shall also be investigated. These verifications must be done with the assistance of the Consulting Engineer and Geotechnical Engineer respectively. If an open deck has to be used, the design shall conform to Standard Drawings R9A-1.6 and 1.7 for steel bridges and R3A-3.7 for timber bridges. These drawings indicate the required tie sizes and the standard tie details. When replacing decks on through truss spans, the vertical clearance shall be verified if the deck must be upgraded to a larger tie size. Standard Drawings MD-01 and MD-02 can be useful to complement Drawings R9A-1.6 and 1.7.

7.1.8 According to the standard drawings, ties must either be dapped 1/2 inch or sized on one side. This has to be done since their exact dimensions will vary slightly from one tie to another. The depth of wood over the girder must be exactly the same throughout the bridge in order to have a smooth rail profile. The biggest disadvantage of dapping is the presence of a notch over the girder which tends to initiate cracks in the tie. When more than two beams support the ties, such as in timber bridges and steel beam spans, ties that are sized on one side are preferred because it is easier to achieve proper bearing on all of them. Standard Drawings R9A-1.6 and 1.7 specifies that dapped ties shall be used when two beams or girders support the ties. However, this practice is progressively being abandoned in favour of sized ties.

7.1.9 For girder spacing greater than 10'-0", the size of the tie shall be determined by analysis, either by considering the rail being a beam on an elastic foundation or by conservatively assuming that the axle weight is distributed over three ties. No impact is considered for the design of bridge ties.

7.1.10 On curved track, there shall be means to resist the lateral loads due to the centrifugal force by supplying enough hook bolts to resist the force. Dapping the ties only provides an additional safety factor against lateral loads.

7.1.11 Each tie shall have a standard tie plate and tie pad. Alternatively, a bridge plate can be used instead of the tie plate. Ties of all sizes are installed with a clear distance of 4 inches between them. The weight of rails and fastenings can be taken as 200 lbs. per linear foot per track. The weight of treated timber can be taken as 60 lbs. per cubic foot.

Table 7-01: Tie Properties

Size	Length	Area	Vol	Wt/tie	Wt/ft deck	Ties/ft
in.	ft.	sq.ft.	cu.ft.	lbs.	lbs.	
10 x 10	13	.69	8.97	538	461	.857
10 x 12	13	.83	10.79	647	554	.857
12 x 12	13	1.00	13.00	780	585	.750
12 x 14	13	1.17	15.21	913	685	.750
12 x 16	13	1.33	17.33	1040	780	.750

7.1.12 Timber ties are made of Douglas Fir as indicated on drawings R9A-1.6 and 1.7. Grading shall be done as per METROLINX Standard L-57b. The timber must be treated in order to increase the service life of the deck. Creosote is the most common treatment for timber ties. Ties are treated as per METROLINX specification 12-30B

7.1.13 Dapped ties shall not be allowed to bear on the top gusset plates because this usually causes cracking of the gusset plate. This situation is usually corrected by installing a steel shim to lower the gusset plate. Parts of the brace frames must be modified in order to accommodate the shim. Standard Drawing MS-13 shows the required modifications. Alternatively, the dap of the ties can be enlarged or the ties sized in order to clear the gusset plate. However, these operations have many disadvantages such as:

(a) The original treatment of the ties is removed
(b) The problem will still exist when the deck is replaced
(c) The cross-section of the tie is reduced
(d) Cracking can be initiated if the dap is poor

7.1.14 Steel repairs often require removing the track spikes. This operation has the tendency to shorten the life of the bridge ties. If possible, steel repairs shall be scheduled to coincide with or be made just before replacement of decks.

7.2 BALLASTED DECKS

7.2.1 Ballasted decks are the preferred option for deck renewal mainly because they are more durable. They also ease and reduce the maintenance required on bridge approaches (see AREMA Chapter 8).

7.2.2 Deck slabs to hold the ballast are made of precast prestressed concrete as shown on Standard Drawings R9A-3 series. A minimum of 400 mm of ballast must be installed to ensure that the deck slab will not be damaged by the track maintenance equipment. In order to allow for future track lifts, deck slabs are designed geometrically to hold an additional 150 mm of ballast and structurally for an additional 305 mm. On superelevated tracks, the height of the curbs has to be adjusted in order to account for the restricted clearance on the low side of the track and the additional ballast on the high side. PVC pipes (100 mm) should be installed through the curbs to allow proper drainage of the deck.

7.2.3 Standard timber ballast decks for timber bridges are shown on Standard Drawing R3A-3.8.

7.3 WALKWAYS

7.3.1 Trainmen's walkways must be installed on bridge decks to allow train crews to inspect trains and to ensure the safety of railway maintenance employees. Many standard designs are available.

7.3.2 The standard steel walkway as shown on Drawings R1A-9.1 to 9.9 represents the most versatile design. The main advantage is that this walkway has been standardized to fit on any bridge independently of the location of the walkway supports. This walkway is often more expensive than alternate designs, but scale economies can be made by ordering the walkway for several bridges at the same time. The main disadvantage of this design is the impossibility of removing one handrail section without cutting it. This operation is often required when performing maintenance on bridges.

7.3.3 Alternatively, a steel walkway can be custom designed for each particular bridge. The length of the grating panels has to be designed so that it does not cantilever past the support ties. The steel details are simpler with this walkway because no grating panel splices are required. Drawing R1A- 9.11 can be used as a guide to detail custom designed steel walkways.

7.3.4 The standard timber walkways are shown on Drawings R3A-3.10 for timber bridges and R9A-1.3 for steel bridges. Standard Drawings MD-01 and MD-02 can be useful in complementing Drawing R9A- 1.3.

7.3.5 It shall be noted that cable or chain hand-railings are not considered an acceptable substitute for the rigid hand-railings shown on the standard drawings.

7.3.6 On bridges exceeding 150 feet in length, refuge bays shall be installed at intervals not exceeding 150 feet. The refuge bays are usually staggered on each side of the track except on curved track where they are located on the inside of the curve and on double track bridges where they shall be located on both sides. Drawing R9A-1.3 provides the typical arrangement and details of refuge bays.

7.3.7 In order to prevent accidents, additional handrail sections shall be added on the wingwalls where the height exceeds 8 feet. If the walls are parallel to the track, the handrail shall extend to the point where the height will be less than 8 feet. In other cases, adding one section on the wingwalls is usually sufficient.

7.4 GUARD RAILS

7.4.1 Guard-rails shall be installed on bridges in accordance with the METROLINX General Guidelines for Design of Railway Bridges & Structures, GO Track Standards and, according to Standard Drawing GTS-1108. Guard-rails on ballast deck bridges are sometimes installed further from the running rail in order to allow the operation of track maintenance equipment. The guard-rails are installed to a distance of 60 feet beyond the backwall. They are fastened to every second tie with two spikes on the bridge deck and every tie on the approaches. The ends of the guard-rails are either cut or bent and double-spiked to the ties.

7.4.2 According to Standard Practice Circular 3700, temporary slow orders of 30 miles per hour on core network and 10 miles per hour on secondary lines are required when guard-rails are temporarily removed to perform maintenance work.

7.5 RAIL ANCHORS

7.5.1 Rail anchors are used on continuous welded rail to transfer the forces induced in the rails by temperature changes. In some cases, rail anchors are installed on each side of the track ties. This installation is referred to as a box anchor.

7.5.2 On ballasted deck bridges, the number of rail anchors to be installed is the same as for normal track. Details are given in the GO Track Standard.

7.5.3 On open deck bridges, rail anchors are not installed on bridges with continuous welded rail in order to allow thermal movement of the span without affecting the rail. The rail is box anchored on the approaches as indicated in Standard Practice Circular 3205. On bridges longer than 300 feet, a rail expansion joint is required at one end of the bridge.

7.5.4 Jointed rail is often used on bridges instead of continuous welded rail since rail expansion joints are considered a costly maintenance item. The continuous welded rail must end 90 feet before the backwall and be box-anchored for 200 feet at the end. Rail anchors may be installed on the bridge at ties that are connected to the structure with hook bolts, except at rail joints, on the backwall tie or where they would interfere with track or signal appliances. Details are given in the GO Track Standard.

7.6 BRIDGE PLATE SYSTEMS

7.6.1 As mentioned in Section 7.1, bridge plates are used instead of tie plates on lines which carry heavy traffic with high speeds. These plates resolve the rail fastening and gauge widening problems which are common on open deck bridges located on such lines. The plates are connected to the ties with four 7/8 inch x 8 inch long lag screws. The plate is 30 inches wide instead of 14 inches for a conventional tie plate. The load is therefore distributed over a larger area, which reduces the tendency of the plate to crush into the tie. On curved tracks, gauge plates can be installed in order to maintain better gauge. These problems are even more frequent on curved track. Zero-restraint Pandrol or Portec rail fasteners are used to hold the rail. These fasteners allow the rail and the structure to respond to temperature changes without transferring force from one to the other.

7.7 DIRECT FIXATION CONCRETE DECKS

7.7.1 On direct fixation concrete decks, the rail is fastened to a rail chair which sits directly onto the concrete. The METROLINX acceptable direct fixation system to be used for concrete bridge decks is LOADMASTER (coated/insulated) rail plate and fastener assembly (or approved equivalent). The more common problems found on these decks are deterioration of concrete under rail chairs and shearing of the anchor bolts holding the rail chairs. The concrete deterioration is due to excessive impact on the concrete surface. This problem can be worsened by the presence of rail joints, which generate even higher impacts. Anchor bolts usually shear just below the concrete surface due to the repeated stress cycles they have to withstand.

7.7.2 The concrete surface can be repaired according to the guidelines given in AREMA Chapter 4 of this document and the MC drawing series. Particular attention shall be given to the spacing of control joints in the concrete repair, which should be determined with the assistance of the manufacturer of the grout used to repair the slab. To avoid reoccurrence of the problem in the near future, the design of the rail chair should be improved. A neoprene pad can be installed between the concrete and the rail chair to absorb the impact energy of the train going over the deck. Neoprene is preferred to natural rubber because it is more resistant to oils and other lubricants which can leak from the engines. The rail chair can be made wider in order to distribute the load over a larger bearing area. HILTI HRC rail anchors (or equivalent) can be used instead of conventional anchor bolts to fasten the rail chairs. These bolts are made of very high strength steel. Their surface is machined in order to remove the defects which initiate fatigue cracks. The bolts are set in the concrete using Hilti HIT-RE 500 adhesive. Availability should be checked with the supplier before planning work with this material. Pandrol plates and clips can be used to provide better fastening of the rails. The requirement for zero-restraint clips should be investigated.

7.8 DIRECT FIXATION STEEL DECKS

7.8.1 On direct fixation steel decks, the rail is fastened to a rail chair which sits directly onto a steel plate. The METROLINX acceptable direct fixation system to be used for steel bridge decks is LOADMASTER (coated/insulated) rail plate and fastener assembly (or approved equivalent). Steel decks provide electrical continuity between the two rails of the track. Proper insulating hardware consisting of bushings, washers and pads must therefore be used in order to avoid short-circuiting the track signals. When the insulating material deteriorates, frequent problems occur with the signal system.

7.8.2 The best solution for resolving problems related to insulation of steel decks is to separate that part of the track which is on a steel deck from the signal system. This is done by installing insulated rail joints at each end of the deck and bypassing the current to the other side of the bridge. However, the cost of this alternative can be prohibitive.

7.8.3 Alternatively, the insulation hardware can be replaced. The materials used for new insulation hardware are rubber fabrics, such as Fabreeka, or other fabrics, such as Nylatron and ABS.

7.8.4 Other problems found on bridges with steel decks are corrosion, cracking of the deck plate and shearing of the anchor bolts holding the rail chairs. Deck plates and bolts are usually replaced in kind. Refer to AREMA Chapter 3 and the MS drawing series for other useful information.

8 MAINTENANCE OF BEARINGS

8.1 MAINTENANCE OF BEARINGS

8.1.1 The function of bridge bearings is to spread the load over an adequate area. They must also be able to respond to movements of the span due to live load, thermal movements or differential settlements without developing harmful restraining forces. They also prevent the span from moving or lifting due to live loads or flooding conditions. Bearings are inspected as per Standard Practice Circular 4000. The condition of the bearings is noted in the inspection report. The most common problems found on bearings are:

(a) Pumping under traffic
(b) Bearing encaved in concrete
(c) Seizing
(d) Flat rollers
(e) Deteriorated keeper plates for rollers
(f) Corroded anchor bolts

Other problems are also given in the SPC 4000 Commentary.

8.1.2 Bearings pumping under traffic shall be repaired because this movement can initiate cracks in the bottom flange over the bearings and also cause further deterioration of the bridge seat. This situation is usually corrected by repairing the concrete under the bearing as shown on Standard Drawing MC-01. Alternatively, a rubber pad or shim can be installed as shown on Standard Drawing MS-11A.

8.1.3 Bearings encaved in the bridge seat are a sign that the concrete underneath the bearing is soft. It can be expected that the deterioration will progress in the future and will create pumping of the bearings. This situation shall be corrected by repairing the concrete under the bearing as shown on Standard Drawing MC-01.

8.1.4 Roller bearings are difficult to maintain because they usually fall apart when dismantled. When this is the case, maintenance of these bearings is not recommended as long as the safety of the structure is assured. Beyond this point, the bearings shall be replaced.

8.1.5 Corroded anchor bolts are always difficult to maintain because they cannot be removed easily. The preferred way to replace an existing anchor bolt is by jacking the span and cutting the bolt flush with the concrete surface. A new one is then installed at a minimum distance of 75 mm from the centre of the existing bolt. New holes must be drilled in the existing bearing or else the bearing must be replaced with a new one having relocated holes. This solution is well suited for steel bearings consisting of rolled steel plates.

8.1.6 For cast iron bearings, the previous solution is not recommended because this material is too brittle. Cracking often occurs when drilling the new holes. Installing a new bearing with relocated holes is usually expensive. The best solution is to jack the span and remove the bearing temporarily. This will allow the anchor bolt to be pulled or cored out. This works well if the anchor bolt is straight and embedded less than two feet. This solution might also be attractive for other manufactured bearings.

8.1.7 If the anchor bolts cannot be removed or the span cannot be jacked, then the anchor bolts can be repaired using keeper plates as shown on Drawings MS-12A and MS-12B. Note that cast iron and cast steel require welding procedures which are difficult for field welding conditions. Bolting to the existing casting is the preferred option in this case.

8.1.8 Anchor bolts shall not be repaired or lengthened by welding. Limited success has been achieved using this practice in the past.

8.1.9 Additional information on bearings is given in the SPC 4000 Commentary.

9 MAINTENANCE OF BRIDGE APPROACHES

9.1 MAINTENANCE OF BRIDGE APPROACHES

9.1.1 The condition of the bridge approach is periodically evaluated by the track supervisor. It is also noted in the bridge inspection report. The most common problems associated with bridge approaches are the following:

(g) Settlement of the approach
(h) Softening of the approach causing track pumping

9.1.2 These problems increase lateral and vertical impact when the train goes over a bridge. This will create other problems such as deterioration of the concrete under the bearing, cracks in the bottom flange at the bearings and loss of track alignment. Moreover, slow orders are required if the condition of the approach does not allow safe railway operation.

9.1.3 Settlement of approaches is mainly due to the difference in the stiffness of the riding surface between the bridge deck and the approach. An additional impact is created both on the bridge and on the approach every time the train goes through such a transition. This impact causes settlement of the approach. The bridge, however, does not settle since it is usually built on a stiff foundation. The differential settlement will cause a local depression in the track profile. A snowball effect is associated with this problem since the depression will generate even higher impacts. This will lead to the track and structural problems mentioned above. This problem is more severe on open deck bridges than on ballasted deck bridges because the bridge ties provide a very stiff riding surface compared to the approach.

9.1.4 The approaches of newly constructed bridges can also settle because of improper compaction of the embankment fill or consolidation of the foundation material below the fill. This settlement will usually occur in the first years after construction. However, clayey foundations can settle for periods of up to one hundred years.

9.1.5 Softening of approaches and formation of ballast pockets usually occurs due to poor drainage behind the abutment and also as a result of accumulated settlement of the approach. When the approach settles, the ballast will penetrate into the top of the subgrade material forming a concave surface which can retain water. This surface is usually referred to as a ballast pocket. The material forming the base of the ballast pocket softens and becomes muddy. The fines may then rise to the surface by pumping action and contaminate the ballast. Again, a snowball effect is associated with this problem since the contaminated ballast will reduce the draining capacity of the approach. This problem is also more common on open deck bridges.

9.1.6 Low approaches are usually raised to the proper profile. A smoother transition can be achieved on a ballasted deck bridge since the track profile can be adjusted both on the bridge and on the approach.

9.1.7 Maintenance of soft approaches is usually limited to replacing the ballast section and improving lateral drainage. However, if a ballast pocket has developed, corrective measures shall extend down to the undisturbed foundation. New geogrid or geoweb reinforced sub-ballast shall be installed to increase the stiffness of the approach and therefore reduce the impact. If the ballast pocket has reached the subgrade, lateral drainage trenches shall also be installed. It has been found convenient to take advantage of the work blocks for deck replacement in order to perform work on the bridge approach.

9.1.8 To rectify the issues at the bridge approaches, approach slabs shall be constructed in accordance with the METROLINX General Guidelines for the Design of Railway Bridges and Structures; for all new bridges, all existing bridges with superstructure replacement work and, when there is ballast replacement work on the bridge & approaches.

10 MAINTENANCE OF CULVERTS

10.1 MAINTENANCE OF CULVERTS

10.1.1 Culverts are inspected as per Standard Practice Circular 4400. The condition of the culvert is noted on the inspection report. The inspection report also includes information on the condition of the water channel and the embankment.

10.1.2 Common problems associated with culverts are the following:

(a) Accumulation of debris constricting the openings
(b) Damming by beavers
(c) Erosion or siltation of channel at the openings
(d) Accumulation of embankment fill at the openings
(e) Deterioration of culvert material
(f) Deformation or sagging of the structure
(g) Open joints between culvert sections

Other problems are also given in SPC 4401.

10.1.3 The structural integrity of a culvert is evaluated by inspecting it under traffic. A hydraulic analysis shall be carried out to assess the hydraulic capacity and performance of culverts, especially at locations with a known history of hydraulic issues. In addition, the hydraulic performance will be evaluated by inspecting it during high runoff periods. Bank stability will also be evaluated by periodic inspections. The track and its draiunage will be inspected during high runoff by the Track inspectors.

10.1.4 Standards for maintaining culverts are given in Standard Practice Circular 4401. The typical solutions used to maintain culverts are as follows:

(a) Cleaning the openings of culverts
(b) Dismantling beaver dams
(c) Installing rip-rap at the openings
(d) Extending culverts or raising headwalls
(e) Concrete repairs (see AREMA Chapter 4 and MC drawing series)
(f) Masonry repairs (see AREMA Chapter 6 and MM drawing series)
(g) Installing urethane at opened joints
(h) Installing joint liners
(i) Installing culvert liners
(j) Replacing culverts
(k) Raise or lower stream bed or barrels of perched culverts to improve flow

Other maintenance solutions given in SPC 4401

10.1.5 The openings of the culvert shall be kept clean in order to maintain its hydraulic capacity. If the speed of the flow is fast, scour problems could occur at the openings. This can be corrected by installing rip-rap at the opening. If the embankment fill is accumulating at the opening of the culvert, then consideration shall be given to extending the culvert or raising headwalls as per Drawing MC- 02.

10.1.6 If the joints are open between culvert sections and embankment fill is leaking through the culvert, then the preferred solutions are to install an interior joint liner on corrugated steel pipes (CSP) and inject urethane in the joint for concrete pipes. For the latter, the commercial injection process similar to the one used for insulating buildings must be used. Urethane compounds supplied in spray cans have not been successful in this situation.

10.1.7 For culvert installations under emergency situations, CSPs are the preferred option for small- and medium-size culverts. For large culverts, multi-plate pipes are preferred. These can easily be installed with an open cut during a work block. Under emergency situations, the size of the opening is usually determined by experience with the existing culvert or by measuring the size of the culverts downstream and providing an adequate safety factor. However, in general, the size of the opening shall be determined by hydraulic analysis. The diameter of the pipe shall not be less than 760 mm. The wall thickness and corrugation are determined according to METROLINX Standard Drawing R7A-80.1i/m-2. The pipe shall be long enough so that headwalls are not required. The pipe shall be installed according to METROLINX Standard Drawing R7A-80.1i/m-1. Proper compaction of the base and sides of the pipe is essential in order to prevent settlement and deformation of the pipe.

10.1.8 If it is not possible to replace the culvert by open cut, then lining it with a CSP or multi-plate pipe is an alternative viable solution. The pipe is assembled at one end of the existing culvert and then jacked or pulled into place. When the pipe is in place, the space between the pipe and walls of the old culvert must be completely filled in. This can be done by placing crushed stone or other granular fill, bulkheading the ends and pressure grouting. Lining of culverts will only be performed once a hydraulic study has determined that even with the reduction the culvert meets all the peak flow requirements. This would only be performed on culverts that have overcapacity or on culverts locations that will have additional culverts installed in order to meet the peak flows.

10.1.9 For new or replacement culverts, a hydraulic analysis/study shall be carried out to determine the required cross-section area (i.e. diameter, dimensions, etc.) at the culvert site that satisfies the projected peak flow. The peak flow design shall be based on the worst-case of the 100-year storm event + 25% or the Regional Storm event.

11 FALL PROTECTION AND SCAFFOLDING

11.1 FALL PROTECTION AND SCAFFOLDING

11.1.1 Fall protection must be installed in accordance with Canada Labour Code requirements. These requirements are given in Metrolinx Safety Guidelines for Contractors, Consultants, and Project Coordinators Section 5.7 Working at Heights.

11.1.2 In order to facilitate maintenance work and increase productivity, scaffolding is usually installed. Scaffolding must be designed by a professional engineer to the applicable standards. The most common scaffolds used for maintenance work are scaffold systems suspended from the deck and scaffold systems supported on the bottom flange.

11.1.3 According to METROLINX policy, fall protection may not be required in some cases where scaffolding is installed when meeting the requirements of OHSA, O. Reg. 213/91.

11.1.4 In order to comply with the requirements of the OHSA and METROLINX Safety Guidelines for Contractors, Consultants and, Project Coordinators, everyone shall be equipped with personal fall protection equipment consisting of a full body harness, a lanyard and a shock absorber conforming to the applicable CSA standards.

11.1.5 When maintenance work is performed, additional personal fall protection equipment is installed in order to provide tie-off points for the workers. The equipment most often used consists of horizontal lifelines, vertical lifelines and retractable lifelines. Detailed information on this equipment is provided in the OHSA and METROLINX Safety Guidelines for Contractors, Consultants and, Project Coordinators.

11.1.6 Fixed fall protection equipment is also installed on bridges. This equipment is shown on the MF drawings series. This equipment consists of:

(a)	Safety bars on DPG	(drawing MF-01 and MF-02)
(b)	Safety cables	(drawing MF-03)
(c)	Safety ladders	(drawing MF-04)
(d)	Ladder rungs	(drawing MF-05)

11.1.7 Safety bars are installed on deck plate girder spans through the stiffeners, as shown on Drawing MF- 01, when the span is deeper than 5 feet. For shallower spans with bottom bracing, it has been found more practical to install the bars on the brace frames as indicated on Drawing MF-02. If the brace frame spacing exceeds 11 feet, safety bars shall not be installed on brace frames since this configuration has not been tested. Safety bars for DPG spans are neither painted nor galvanized. Galvanized safety bars have cracked on occasion when they were hot-bent in the field around the stiffener. Painting of safety bars is relatively expensive and not justified since these members are not overly susceptible to corrosion.

11.1.8 Safety cables are installed on truss spans as shown on Drawing MF-03 to serve as a horizontal lifeline. The minimum sag requirement must be adhered to in order to minimize pretension in the cable and reduce anchorage loads generated by a fall. The cable shall be detailed to the exact length with a flemish eye at each end. The use of cable clamps and cable clips is not acceptable.

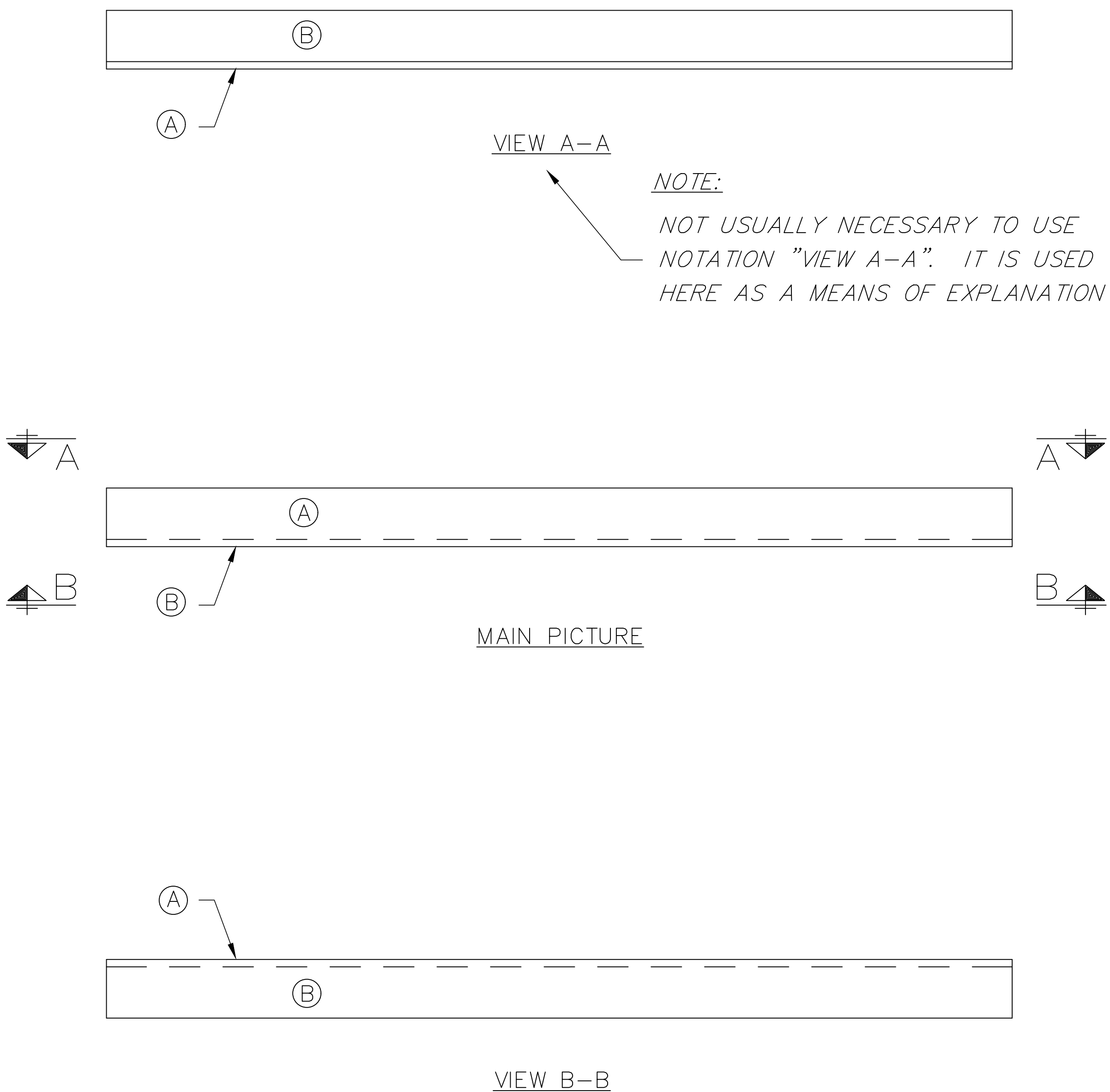
11.1.9 Safety ladders as shown on Drawings MF-04A, MF-04B and MF-04C are installed to access the bridge seat or the bottom chord of a deck truss. A security cage conforming to ANSI Standard A14.3 is required when the length of the climb is greater than 20 feet or when the top of the ladder is higher than 20 feet from the ground.

11.1.10 Ladder rungs are installed on abutment and pier faces to access the inside of the span and facilitate inspection of the bearings.

11.1.11 Vertical anchorage points of fall protection systems shall be designed for a factored load of 18 KN. Horizontal points of fall protection systems shall be designed for a factored load of 90 KN. For horizontal anchorage points only, this force can be reduced if an energy absorber is installed on the horizontal lifeline.

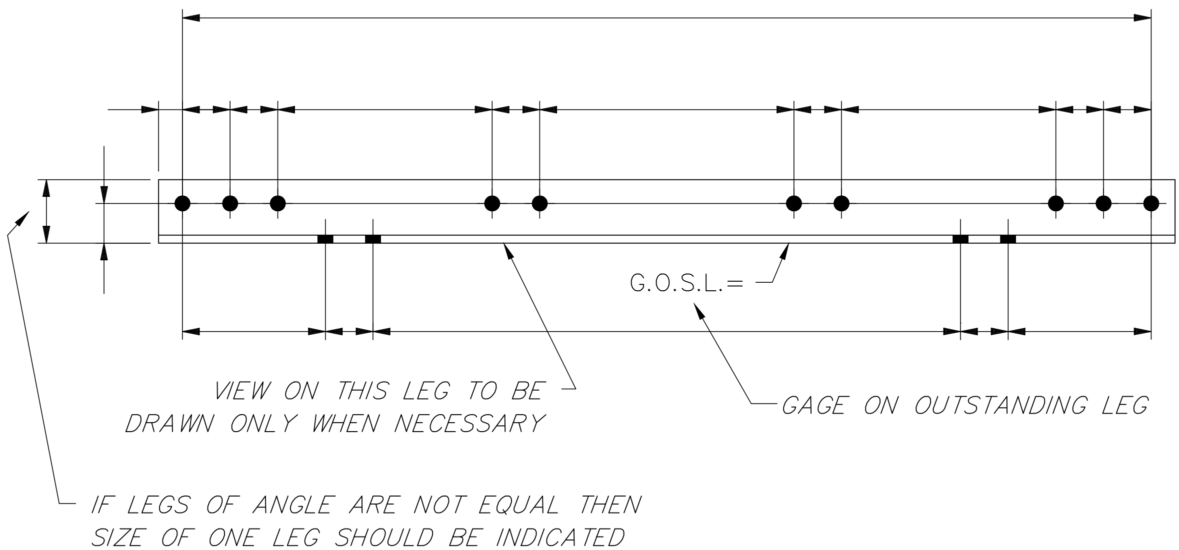
11.1.12 The design of scaffolding shall comply with the requirements of the Canada Labour Code OHSA, O. Reg. 213/91 and CSA S269.2. Components of the scaffold shall be designed with a minimum safety factor of 4. The cables which hold the suspended systems shall be designed with a minimum safety factor of 10.

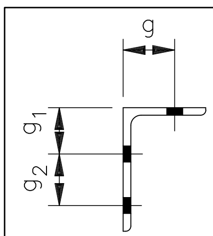
11.1.13 Live load requirements shall be established for the specific needs of each project but not be less than what is specified in the applicable standards.



COMMENTS SHOWN IN ITALIC

Drawn Dessin	JE	Checked Verification	S.Kh.	STANDARD WAY TO SHOW ANGLE VIEWS
Scale Echelle	NONE	Date	March 2019	
METROLINX			Drawing Number Dessin Numero FIGURE 1	Rev.





USUAL GAUGE FOR ANGLES							
LEG	8	7	6	5	4	3½	3
g	4½	4	3½	3	2½	2	1¾
g_1	3	2½	2¼	2			
g_2	3	3	2½	1¾			

COMMENTS SHOWN IN ITALIC

Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

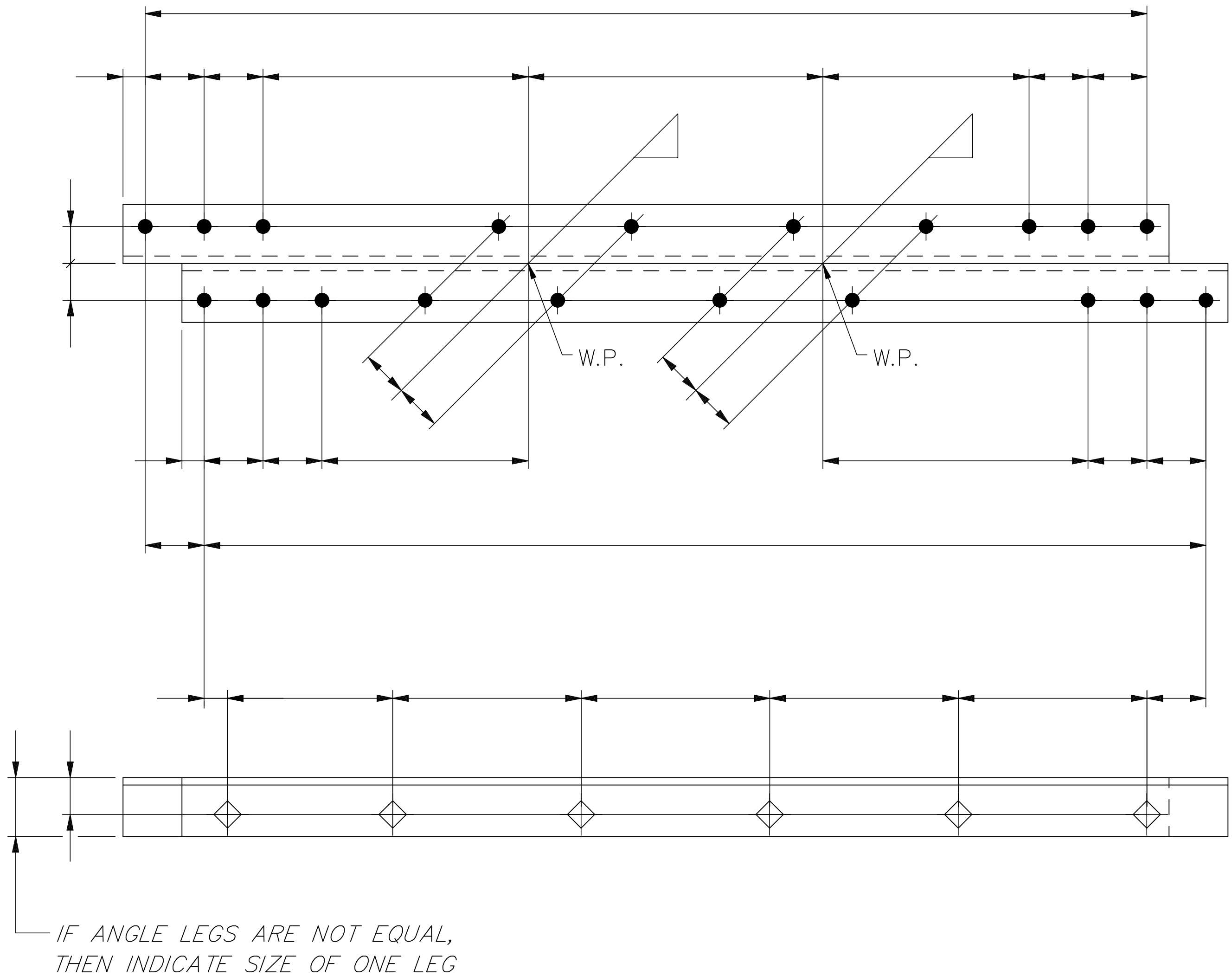
STANDARD WAY TO DETAIL ANGLES

METROLINX

Drawing Number Dessin Numero

FIGURE 2

Rev.



FIELD BOLTS ARE INDICATED ON DRAWING BY SYMBOL ●

SHOP BOLTS ARE INDICATED ON DRAWING BY SYMBOL ◇

ANGLES WHICH ARE CONNECTED TOGETHER AS SHOWN ABOVE ARE FABRICATED SEPARATELY AND LATER JOINED TOGETHER IN SHOP. EACH ANGLE SHOULD BE DETAILED WITH ENOUGH DIMENSIONS SO THEY CAN BE FABRICATED AS SEPARATE PIECES.

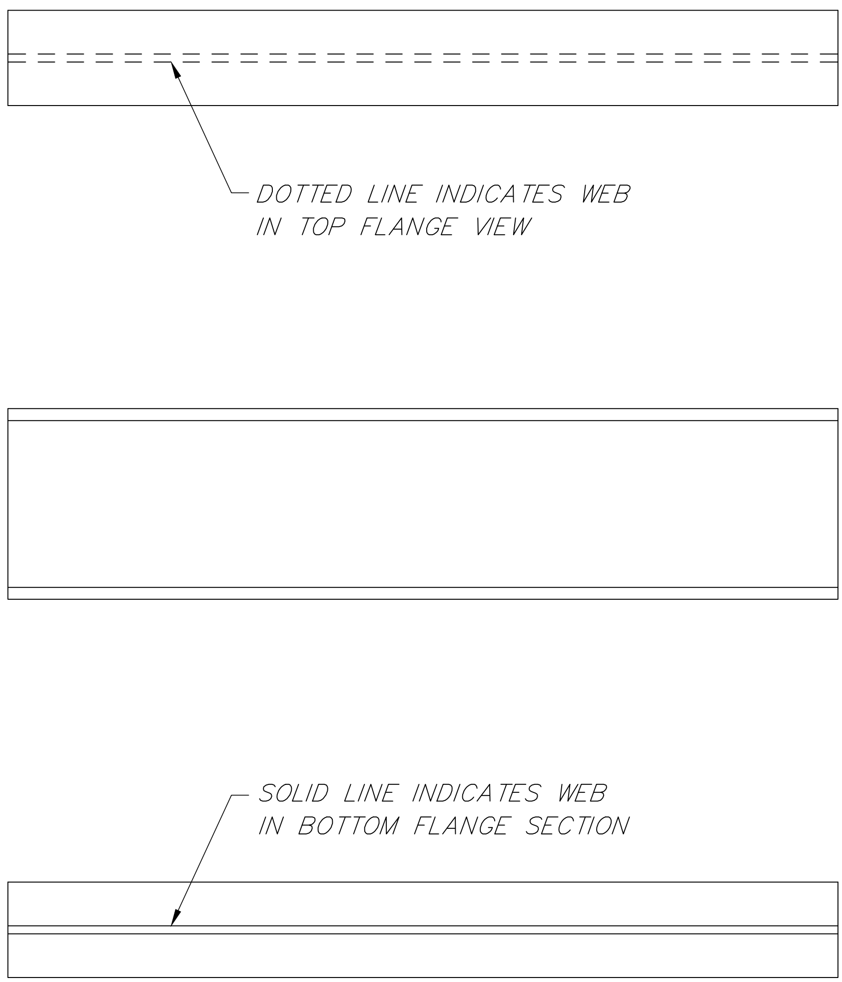
COMMENTS SHOWN IN ITALIC

Drawn / Dessin	FM	Checked / Verification	S.Kh.	STANDARD WAY TO DETAIL ANGLES WHICH ARE CONNECTED BACK TO BACK
Scale / Echelle	NONE	Date	March 2019	

METROLINX

Drawing Number / Dessin Numero	Rev.
FIGURE 3	

VIEW ON TOP FLANGE AND SECTION ON BOTTOM FLANGE ALWAYS DRAWN LOOKING DOWN ON BEAM



NOTE:

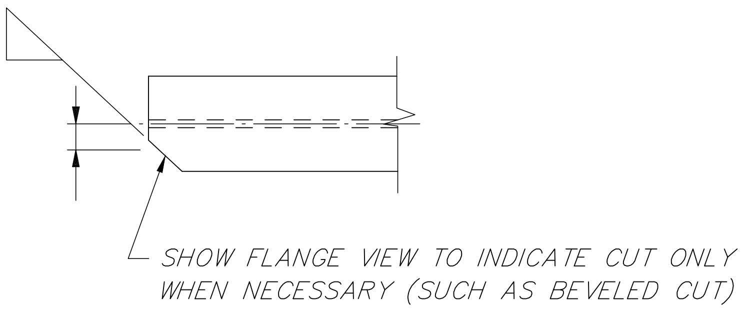
SAME RULES APPLY TO CHANNELS

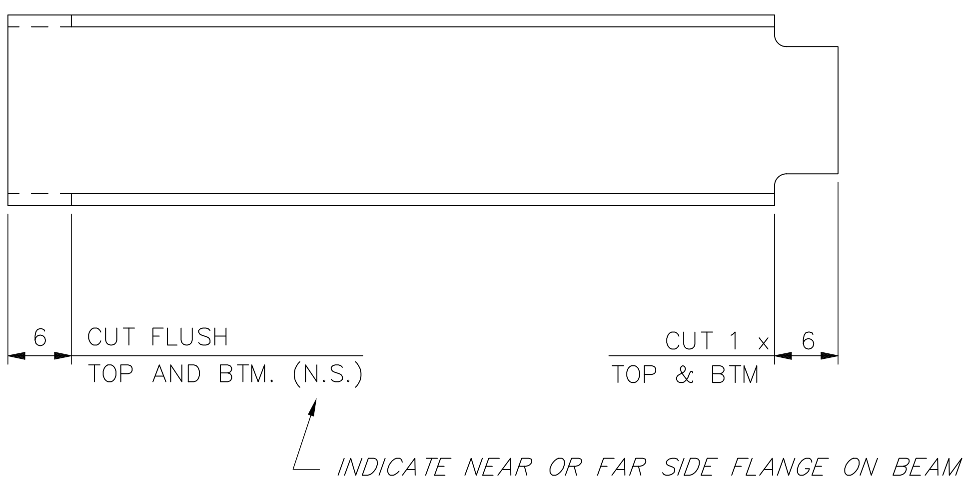
COMMENTS SHOWN IN ITALIC

Drawn Dessin	FM	Checked Verification	S.Kh.	STANDARD WAY TO SHOW TOP AND BOTTOM FLANGES OF BEAMS
Scale Echelle	NONE	Date	March 2019	

METROLINX

Drawing Number Dessin Numero	Rev.
FIGURE 4	





NOTE:

SEE ALSO METROLINX STD DWG No. S6 FOR
STANDARD COPING DETAILS APPLICABLE ON BRIDGES

COMMENTS SHOWN IN ITALIC

Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

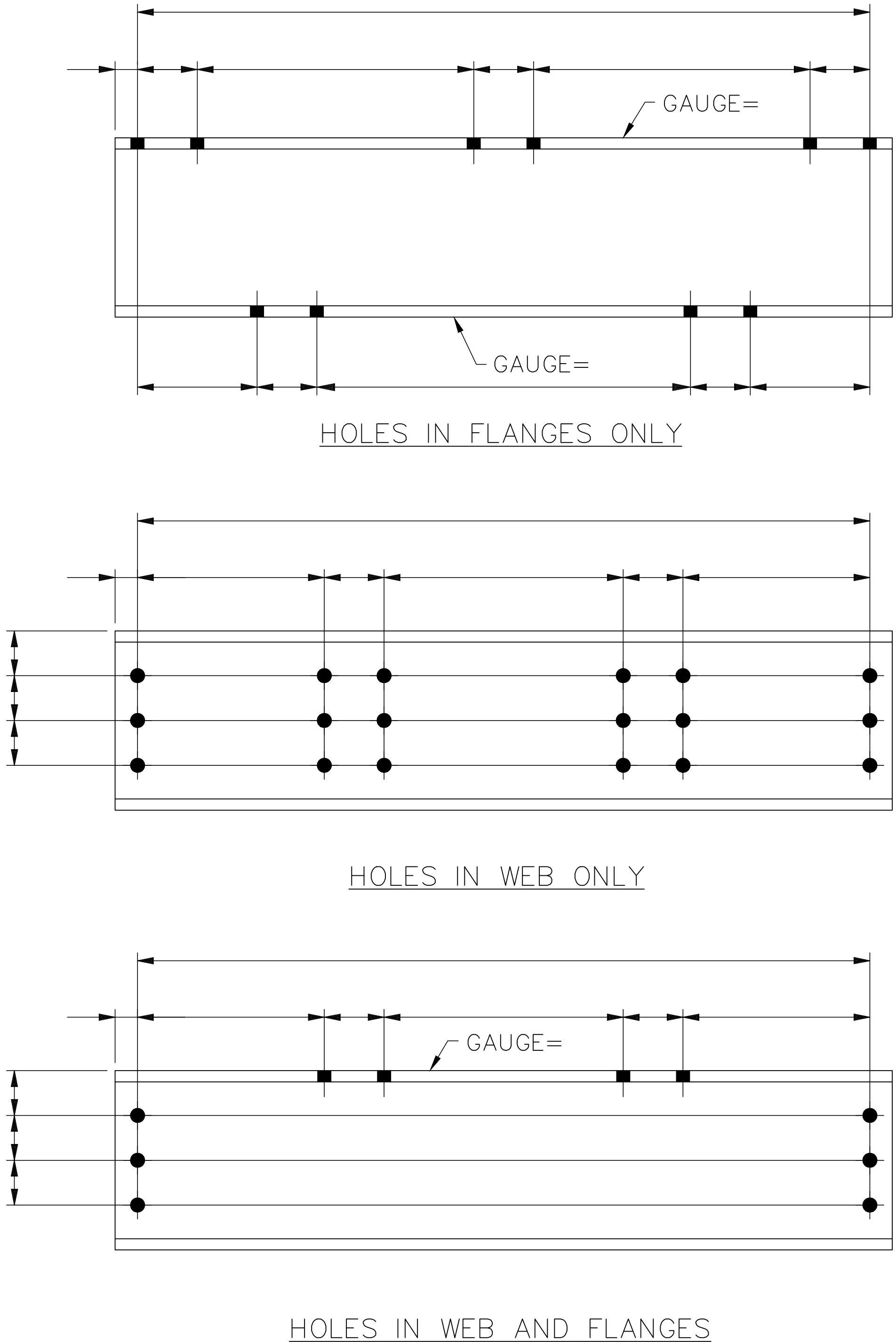
STANDARD WAY TO SHOW CUTS OR
COPES IN BEAMS AND CHANNELS

METROLINX

Drawing Number
Dessin Numero

FIGURE 5

Rev.



Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

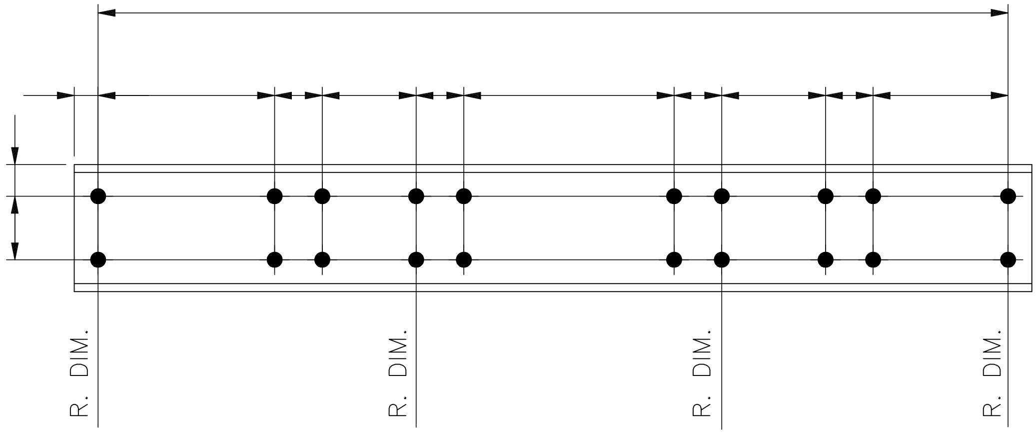
STANDARD WAY TO DIMENSION
HOLES IN BEAMS AND CHANNELS

METROLINX

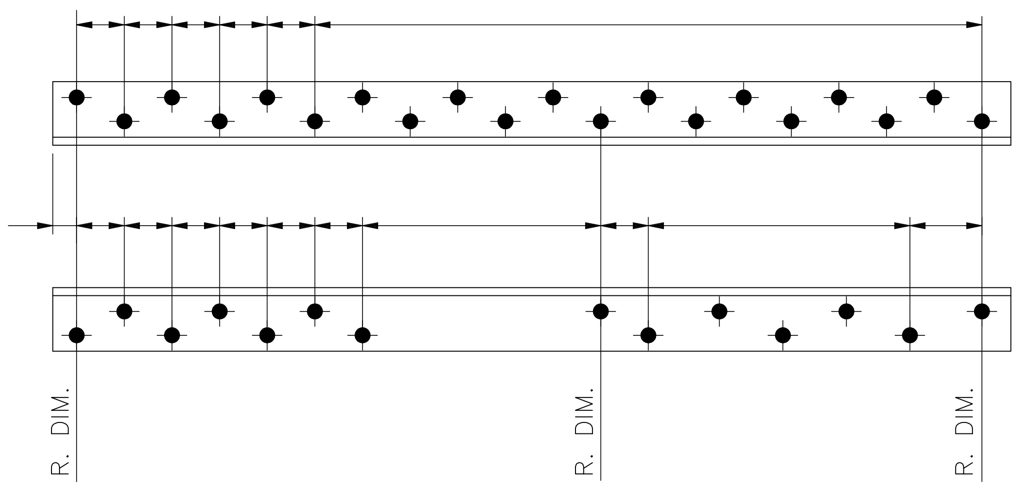
Drawing Number
Dessin Numero

FIGURE 6

Rev.



WHEN DIMENSIONING MEMBER LONGER THAN 12 FEET WHICH HAVE MANY HOLES, RUNNING DIMENSIONS ARE TO BE GIVEN AT APPROXIMATELY 6 FEET AS A CHECK FOR THE SHOP.

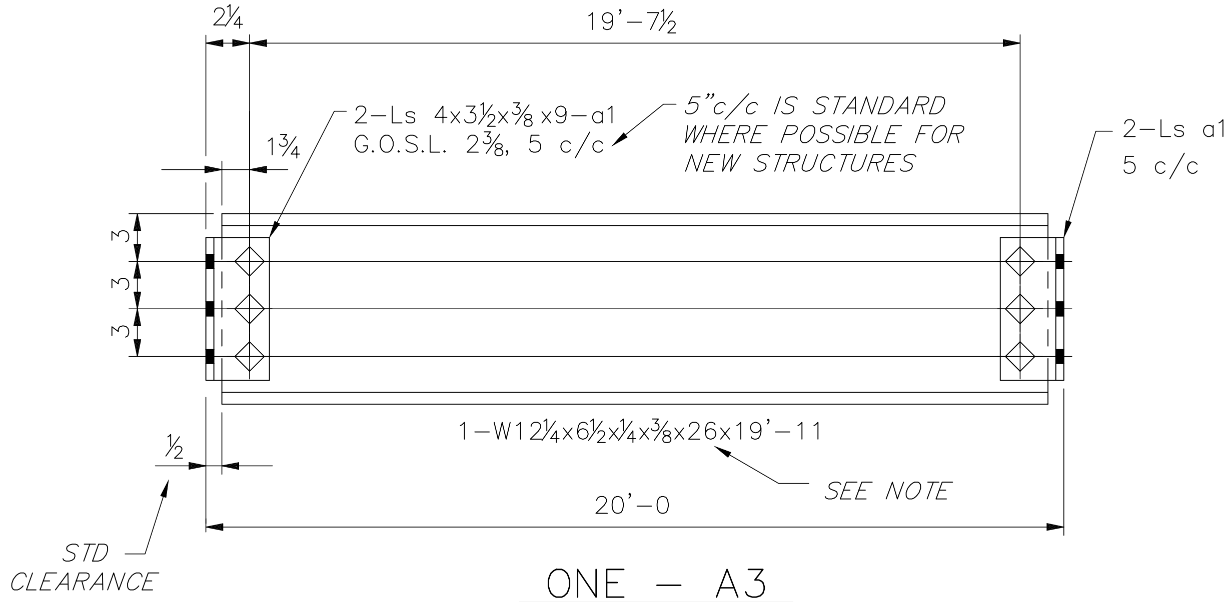


COMMENTS SHOWN IN ITALIC

Drawn Dessin	FM	Checked Verification	S.Kh.	STANDARD WAY TO DIMENSION MEMBERS WITH MANY HOLES
Scale Echelle	NONE	Date	March 2019	

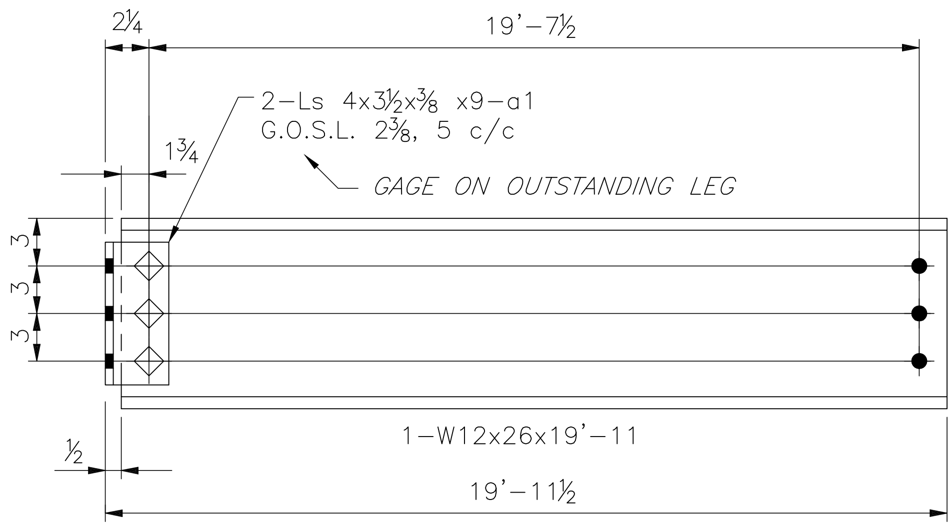
METROLINX

Drawing Number Dessin Numero	Rev.
FIGURE 7	



ONE – A3

WEIGHT: 175 LBS



4 – A3

UNIT WT: 152 LBS

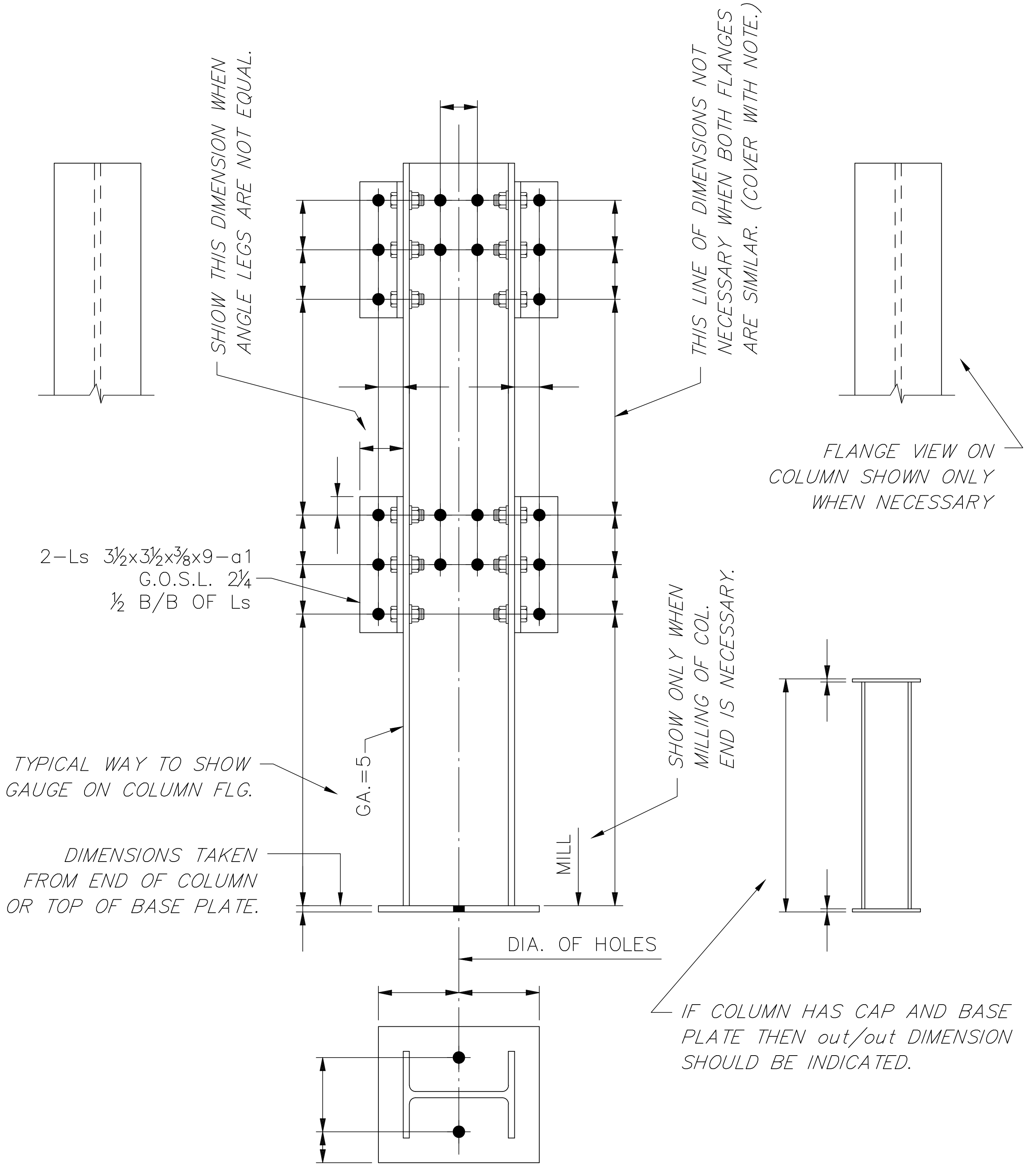
NOTE:

WHEN BILLING OUT A BEAM OR A CHANNEL FOR THE FIRST TIME ON A DRAWING OR A SET OF DRAWINGS, IT IS A GOOD PRACTICE TO GIVE A FULL DESCRIPTION OF THAT MEMBER AS SHOWN ABOVE. THE DEPTH OF MEMBER, WIDTH OF FLANGE, THICKNESS OF WEB, THICKNESS OF FLANGE, WEIGHT AND LENGTH ARE THEN SHOWN IN THIS ORDER.

COMMENTS SHOWN IN ITALIC

Drawn Dessin	FM	Checked Verification	S.Kh.	STANDARD WAY TO DETAIL BEAMS AND CHANNELS WITH CONNECTION ANGLES
Scale Echelle	NONE	Date	March 2019	

METROLINX	Drawing Number Dessin Numero FIGURE 8	Rev.



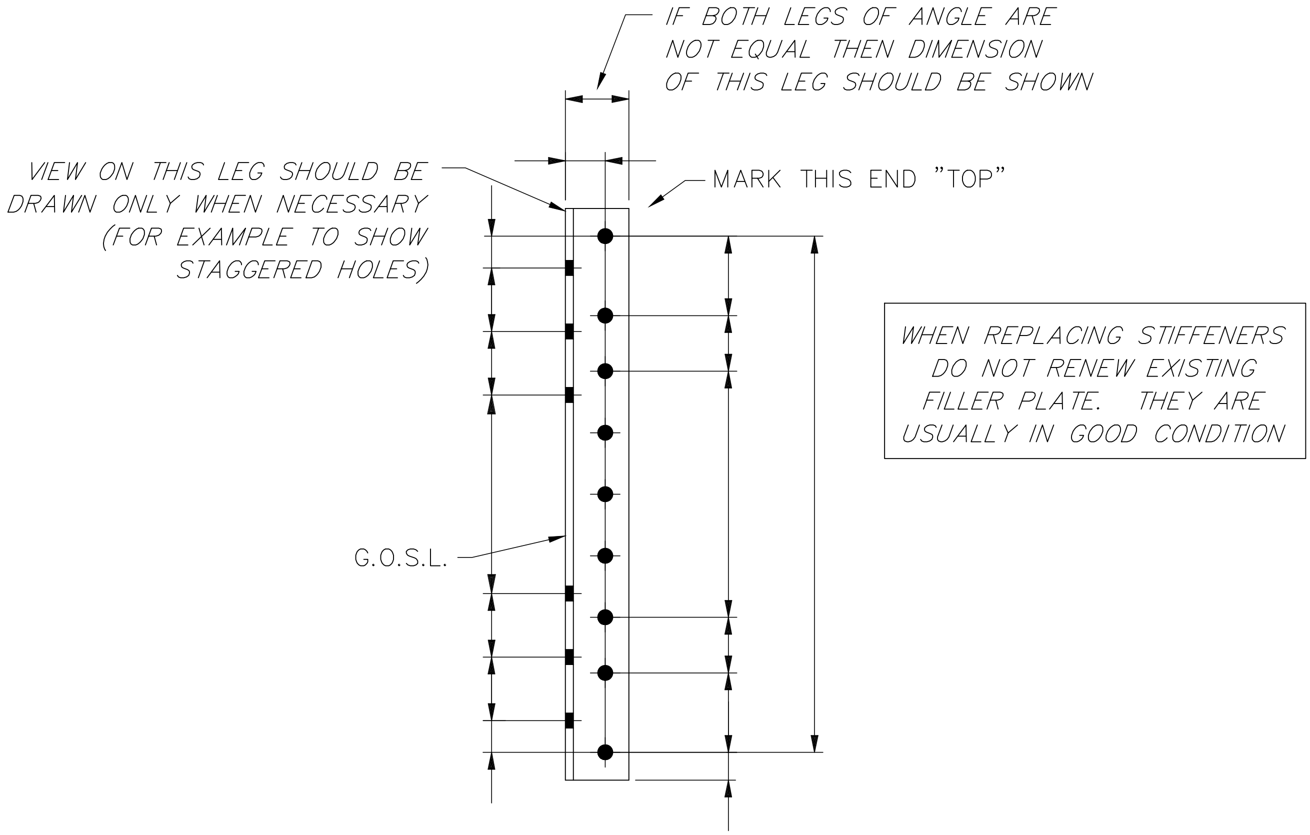
COMMENTS SHOWN IN ITALIC

Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

STANDARD WAY TO DETAIL COLUMNS

METROLINX

Drawing Number Dessin Numero	Rev.
FIGURE 9	



NOTE:

GRIND ENDS OF BEARING STIFFENERS IN FIELD TO BEAR.

— BECAUSE FLANGES OF EXISTING GIRDERS MAY BE LESSENED BY CORROSION, ¼" SHOULD BE ADDED TO LENGTH OF BEARING STIFFENERS AT BOTTOM END AND ⅛" AT TOP END FOR GRINDING IN FIELD.

COMMENTS SHOWN IN ITALIC

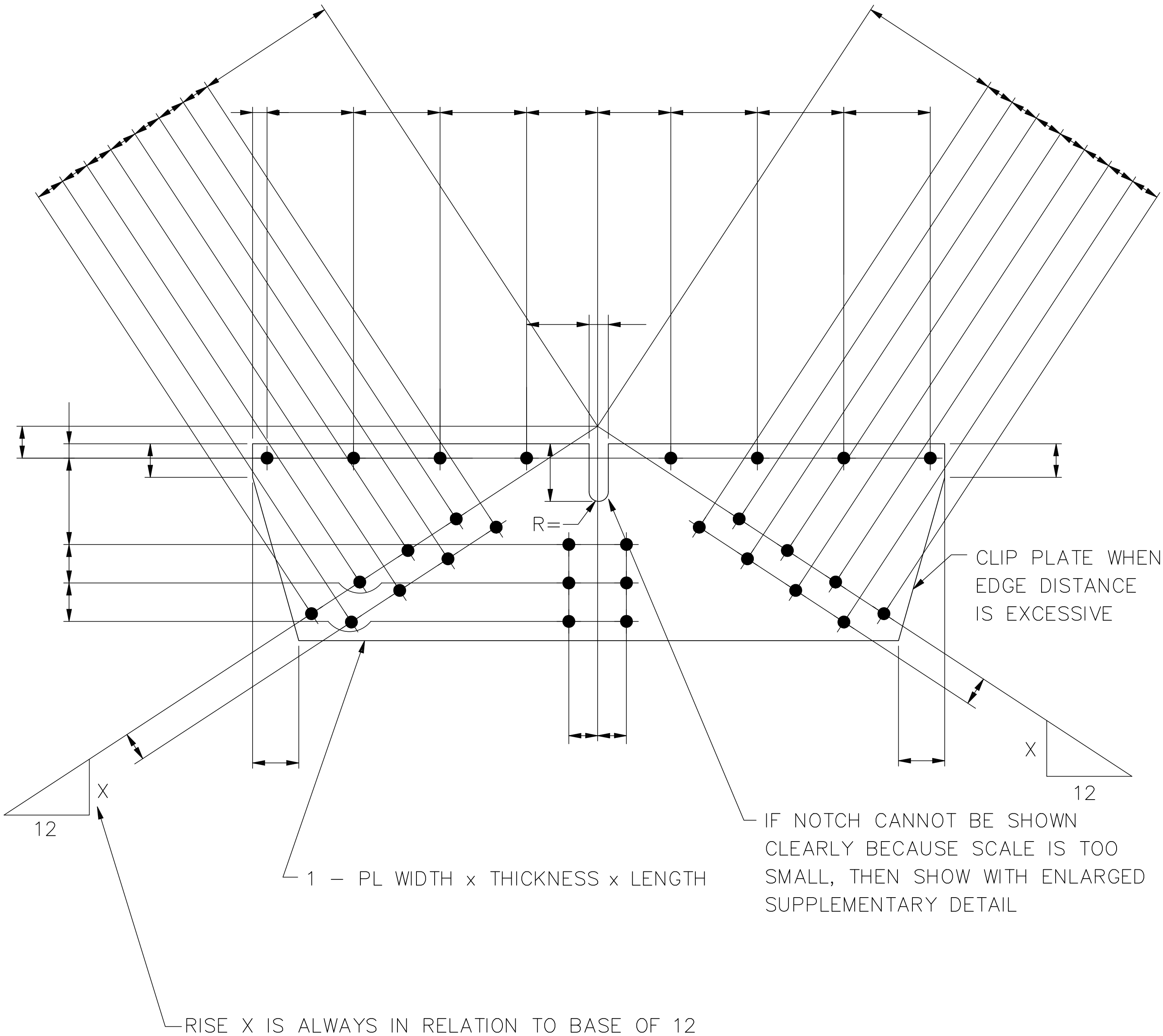
Drawn / Dessin	FM	Checked / Verification	S.Kh.
Scale / Echelle	NONE	Date	March 2019

STANDARD WAY TO DETAIL STIFFENERS

METROLINX

Drawing Number / Dessin Numero: FIGURE 10

Rev.



COMMENTS SHOWN IN ITALIC

Drawn Dessin	FM	Checked Vérification	S.Kh.
Scale Échelle	N.T.S.	Date	March 2019

STANDARD WAY TO DETAIL

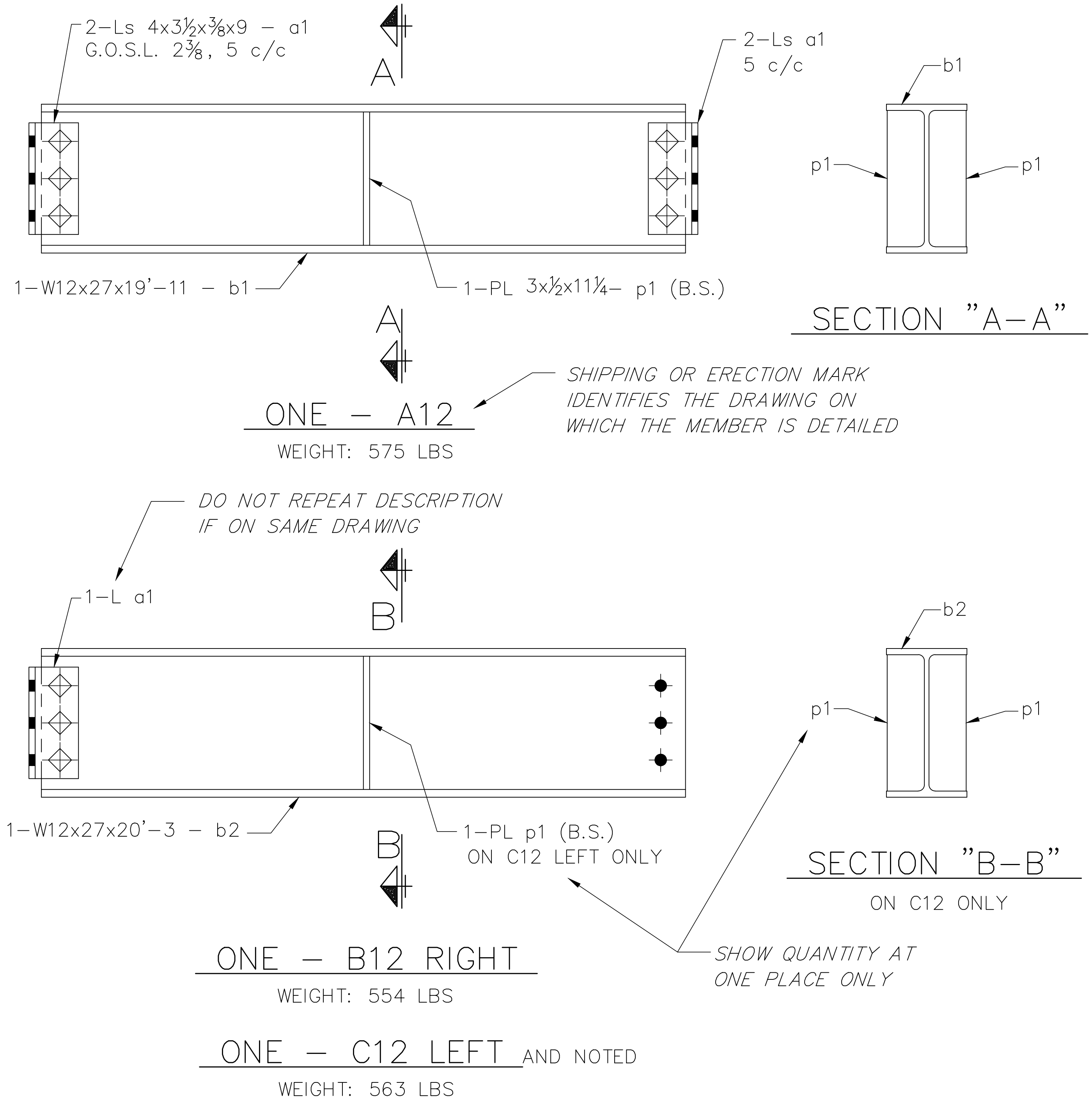
GUSSET PLATES

METROLINX

Drawing Number
Dessin Numero

FIGURE 11

Rev.



COMMENTS SHOWN IN ITALIC

Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

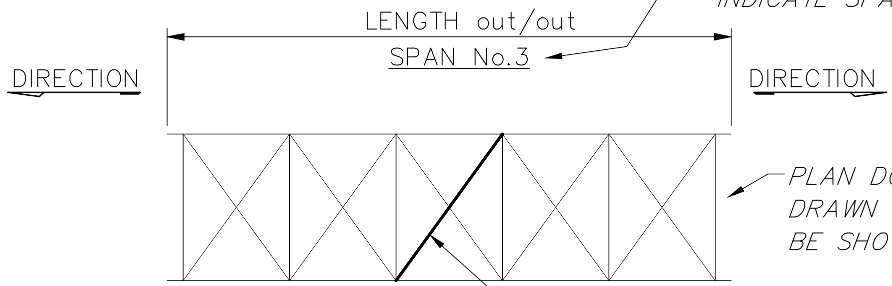
STANDARD WAY TO SHOW ASSEMBLY,
SHIPPING AND ERECTION MARKS

METROLINX

Drawing Number Dessin Numero	Rev.
FIGURE 12	

ERECTION DIAGRAM SHOULD BE DRAWN SO THAT INCREASING MILEAGE IS SHOWN TOWARDS LEFT SIDE OF DRAWING.

WHEN MORE THAN ONE SPAN INDICATE SPAN No. FOR EACH SPAN



PLAN DOES NOT HAVE TO BE DRAWN TO SCALE BUT SHOULD BE SHOWN IN PROPER PROPORTION

TO LOCATE NEW STEEL ON ERECTION DIAGRAM OF EXIST. BRIDGE, DRAW HEAVY LINE AS SHOWN.

ERECTION DIAGRAM

SHOWING BOTTOM FLANGE UNLESS NOTED

WHEN REPAIRS ARE REQUIRED ON TOP AND BOTTOM FLANGES, SHOW THIS NOTE

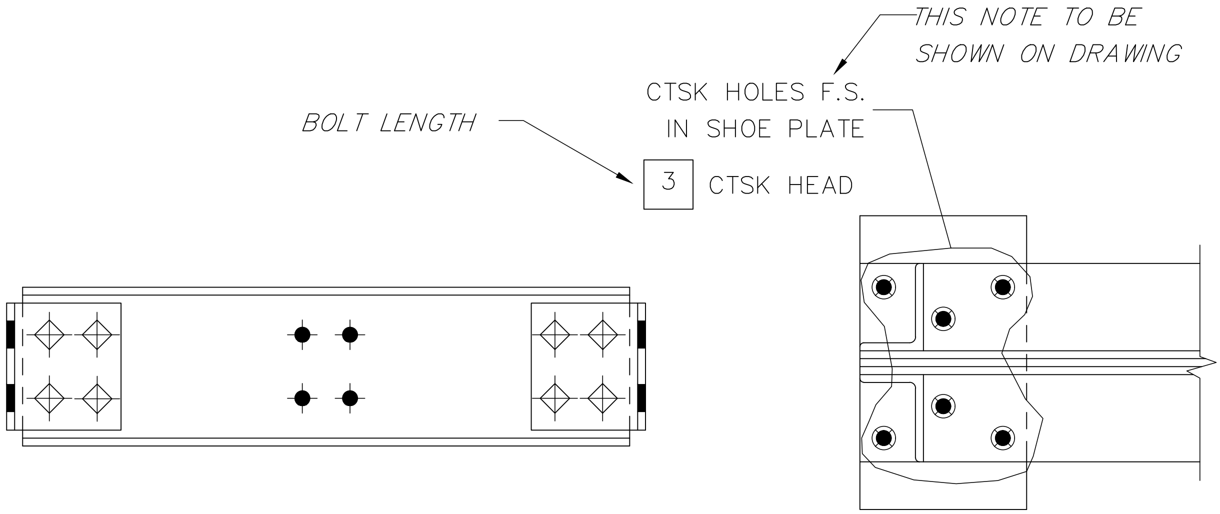
- *IF IT IS NOT POSSIBLE TO CLEARLY INDICATE NEW STEEL ON PLAN, THEN SECTION THROUGH BRIDGE SHOULD BE SHOWN AT POINT WHERE NEW STEEL OCCURS.*
- *IF SHOP AND FIELD DETAILS ARE ON SAME DRAWING THEN FIELD DETAILS SHOULD BE KEPT ON UPPER PART OF DRAWING WITH ERECTION DIAGRAM. THEY SHOULD BE KEPT SEPARATE FROM SHOP DETAILS.*

STANDARD NOTES SHOULD APPEAR RIGHT HAND SIDE OF DRAWING ABOVE TITLE BLOCK.

COMMENTS SHOWN IN ITALIC

Drawn Dessin	FM	Checked Verification	S.Kh.	STANDARD WAY TO SHOW ERECTION DIAGRAM FOR STEEL REPAIRS
Scale Echelle	NONE	Date	March 2019	

METROLINX	Drawing Number Dessin Numero: FIGURE 13	Rev.



- HIGH STRENGTH BOLTS ARE USED FOR FIELD AND SHOP CONNECTIONS.
- FIELD BOLTS ARE INDICATED ON DRAWING BY SYMBOL
- SHOP BOLTS ARE INDICATED ON DRAWING BY SYMBOL
- COUNTERSUNK BOLTS ARE INDICATED BY SYMBOL FOR NEAR SIDE AND FOR FAR SIDE. TO AVOID POSSIBLE FABRICATION ERRORS, A NOTE DESCRIBING THESE SPECIAL HOLES SHOULD ALSO BE ADDED ON THE DRAWING.
- BOLT LENGTHS ARE NOT LISTED EXCEPT FOR COUNTERSUNK BOLTS, ⅞"dia. BOLTS OF 5 INCHES OR LONGER, BOLTS OF DIAMETER OTHER THAN ⅞"dia. AND OTHER UNUSUAL BOLTS.
- EACH HIGH STRENGTH BOLT IS TO HAVE ONE HEAVY HEX. NUT AND ONE HARDENED ROUND WASHER. BEVEL WASHERS SHOULD BE USED WITH I BEAMS OR CHANNELS. THE WASHER SHOULD ALWAYS BE PLACED UNDER THE TURNED ELEMENT (HEAD OR NUT). THIS INFORMATION IS ASSUMED TO BE KNOWN BY THE FIELD PERSONNEL AND DOES NOT HAVE TO BE INDICATED ON THE DRAWING.
- CLIPPED HARDENED ROUND WASHERS AND CLIPPED HARDENED BEVEL WASHERS CAN BE USED FOR INSTALLATION WHERE CLEARANCES ARE LESS THAN STANDARD (SEE FIG. 15). LOCATIONS WHERE CLIPPED WASHERS ARE REQUIRED SHOULD BE INDICATED ON THE DRAWINGS.
- ⅞"dia. BOLT LENGTHS SHOULD BE CALCULATED ACCORDING TO TABLE ON FIGURE 15
- LENGTH OF ⅞"dia. BOLTS CAN ALTERNATIVELY BE CALCULATED USING THICKNESS OF MATERIAL CONNECTED AND ADDING 1¼".

COMMENTS SHOWN IN ITALIC

Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

METHOD OF INDICATING BOLTS ON DRAWING

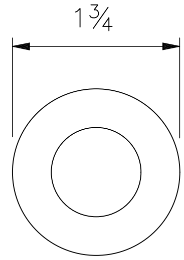
METROLINX

Drawing Number Dessin Numero: FIGURE 14

Rev.

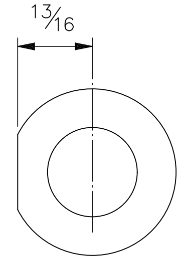
GRIP LENGTHS FOR 7/8dia. A325 BOLTS			
RANGE OF GRIP LENGTHS IN.	BOLT LENGTH IN.	RANGE OF GRIP LENGTHS IN.	BOLT LENGTH IN.
9/16–3/4	2	4 9/16–4 3/4	6
13/16–1	2 1/4	4 13/16–5	6 1/4
1 1/16–1 1/4	2 1/2	5 1/16–5 1/4	6 1/2
1 5/16–1 1/2	2 3/4	5 5/16–5 1/2	6 3/4
1 9/16–1 3/4	3	5 9/16–5 3/4	7
1 13/16–2	3 1/4	5 13/16–6	7 1/4
2 1/16–2 1/4	3 1/2	6 1/16–6 1/4	7 1/2
2 5/16–2 1/2	3 3/4	6 5/16–6 1/2	7 3/4
2 9/16–2 3/4	4	6 9/16–6 3/4	8
2 13/16–3	4 1/4	6 13/16–7	8 1/4
3 1/16–3 1/4	4 1/2	7 1/16–7 1/4	8 1/2
3 5/16–3 1/2	4 3/4	7 5/16–7 1/2	8 3/4
3 9/16–3 3/4	5	7 9/16–7 3/4	9
3 13/16–4	5 1/4		
4 1/16–4 1/4	5 1/2		
4 5/16–4 1/2	5 3/4		

THICKNESS OF A SINGLE FLAT WASHER IS INCLUDED IN COMPUTATION OF BOLT LENGTH. WHEN A BEVEL WASHER IS USED WITH A FLAT WASHER, ADD 5/16" TO GRIP LENGTH.



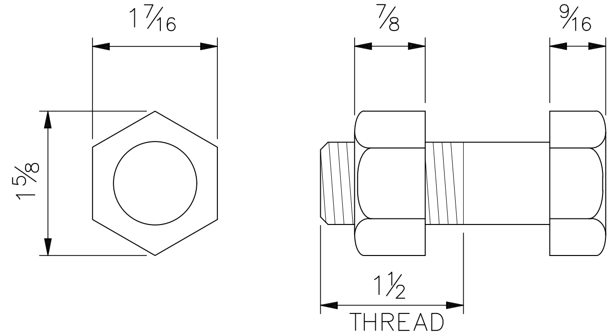
STANDARD WASHER

THICKNESS: 3/16

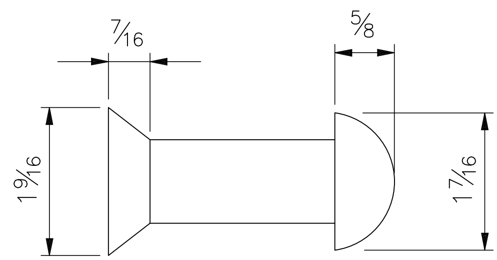


CLIPPED WASHER

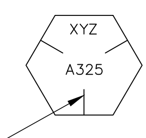
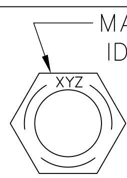
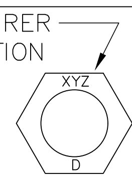
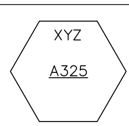
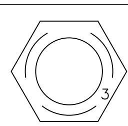
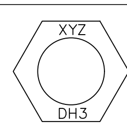
THICKNESS: 3/16



DIMENSIONS OF 7/8dia. BOLT AND NUT



DIMENSIONS OF 7/8dia. RIVET

TYPE	A325		
	BOLT	NUT	
		MANUFACTURER IDENTIFICATION (TYP)	
1	 OPTIONAL	 ARCS INDICATE GRADE C	 GRADE MARK (2) D, DH, 2 OR 2H
3			

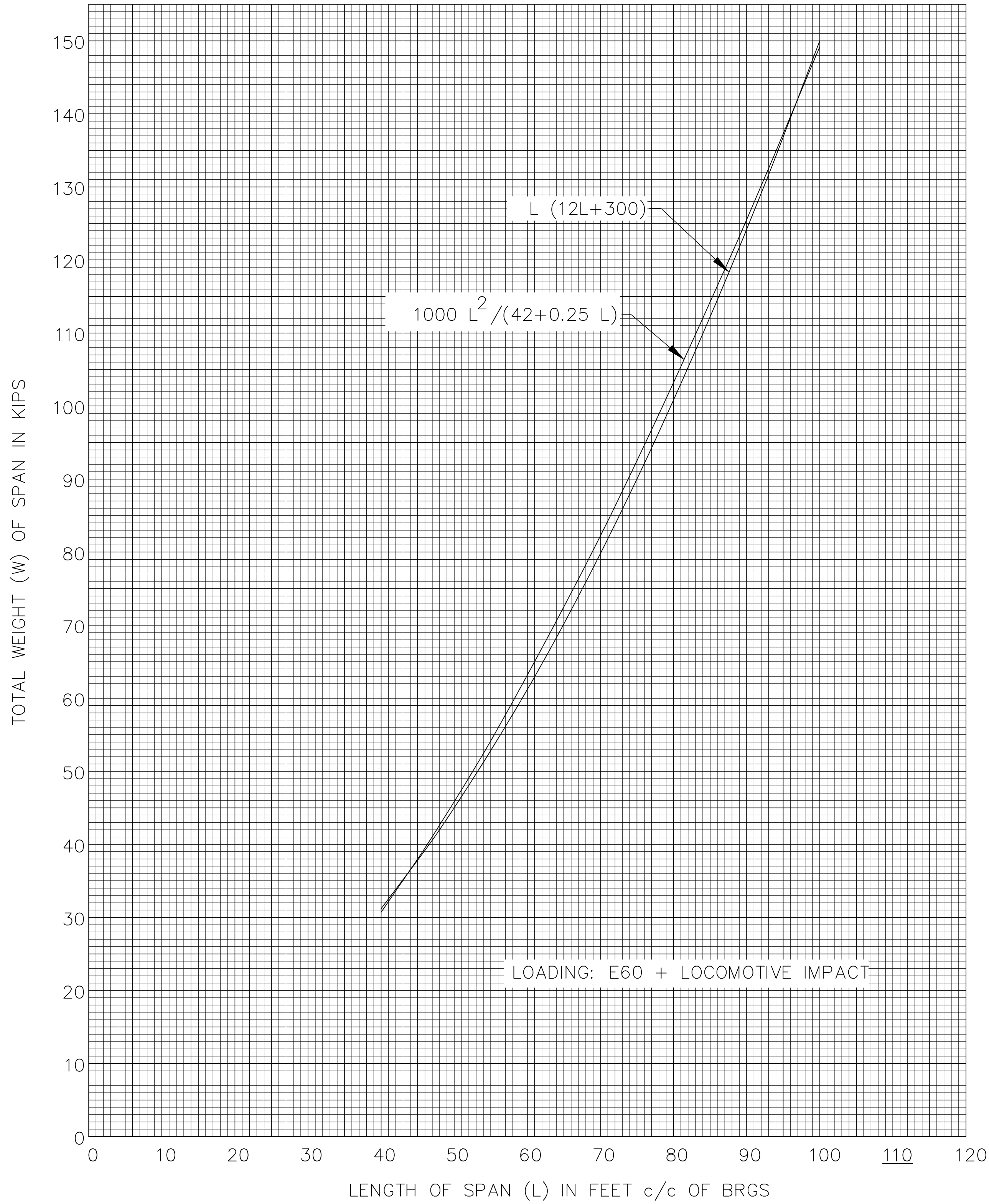
Drawn / Dessin	FM	Checked / Verification	S.Kh.
Scale / Echelle	NONE	Date	March 2019

BOLT INFORMATION

METROLINX

Drawing Number / Dessin Numero: FIGURE 15

Rev.



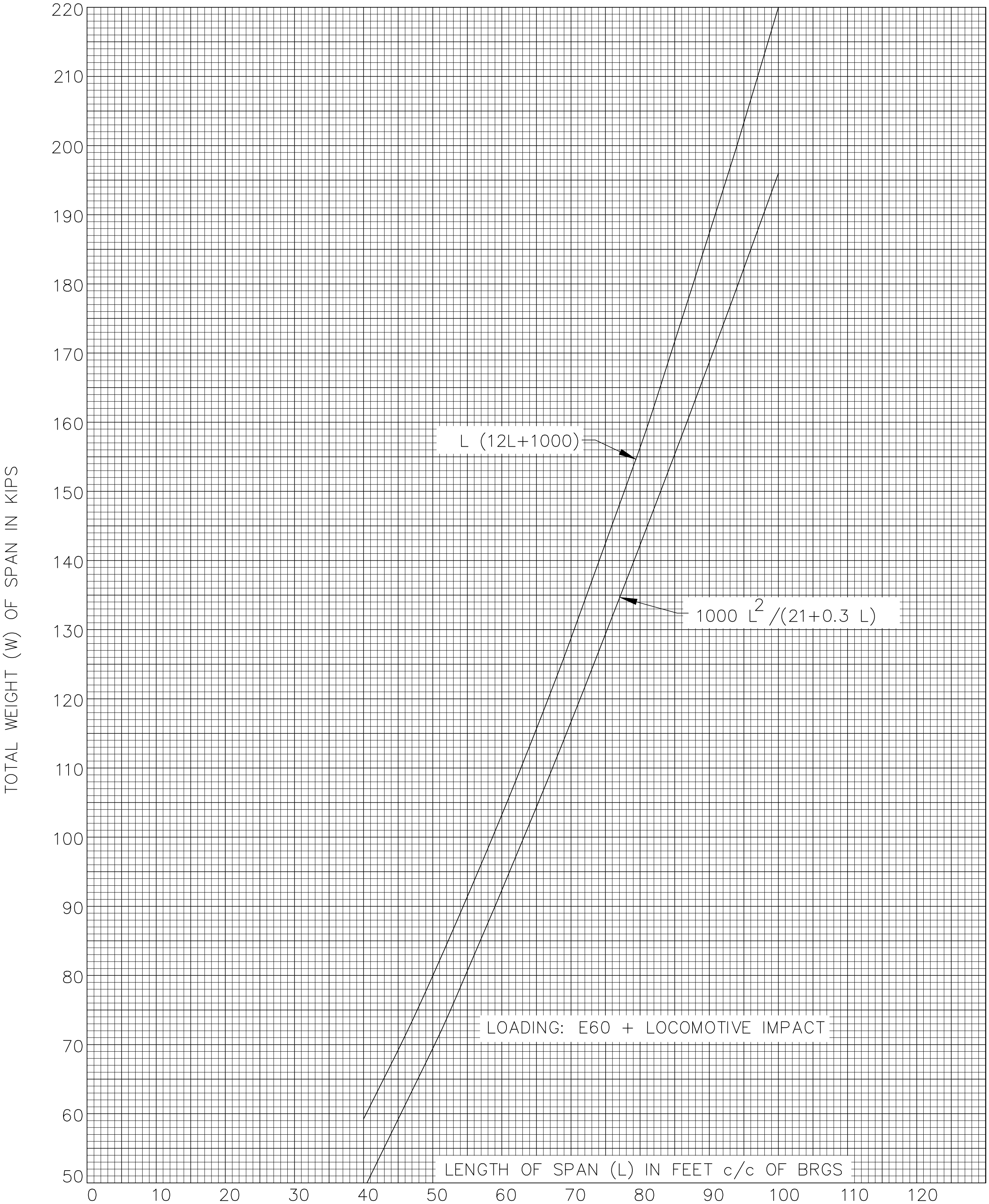
Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

WEIGHT OF D.P.G. OPEN DECK
SINGLE TRACK SPAN
RIVETED CONSTRUCTION

METROLINX

Drawing Number Dessin Numero: FIGURE 16

Rev.

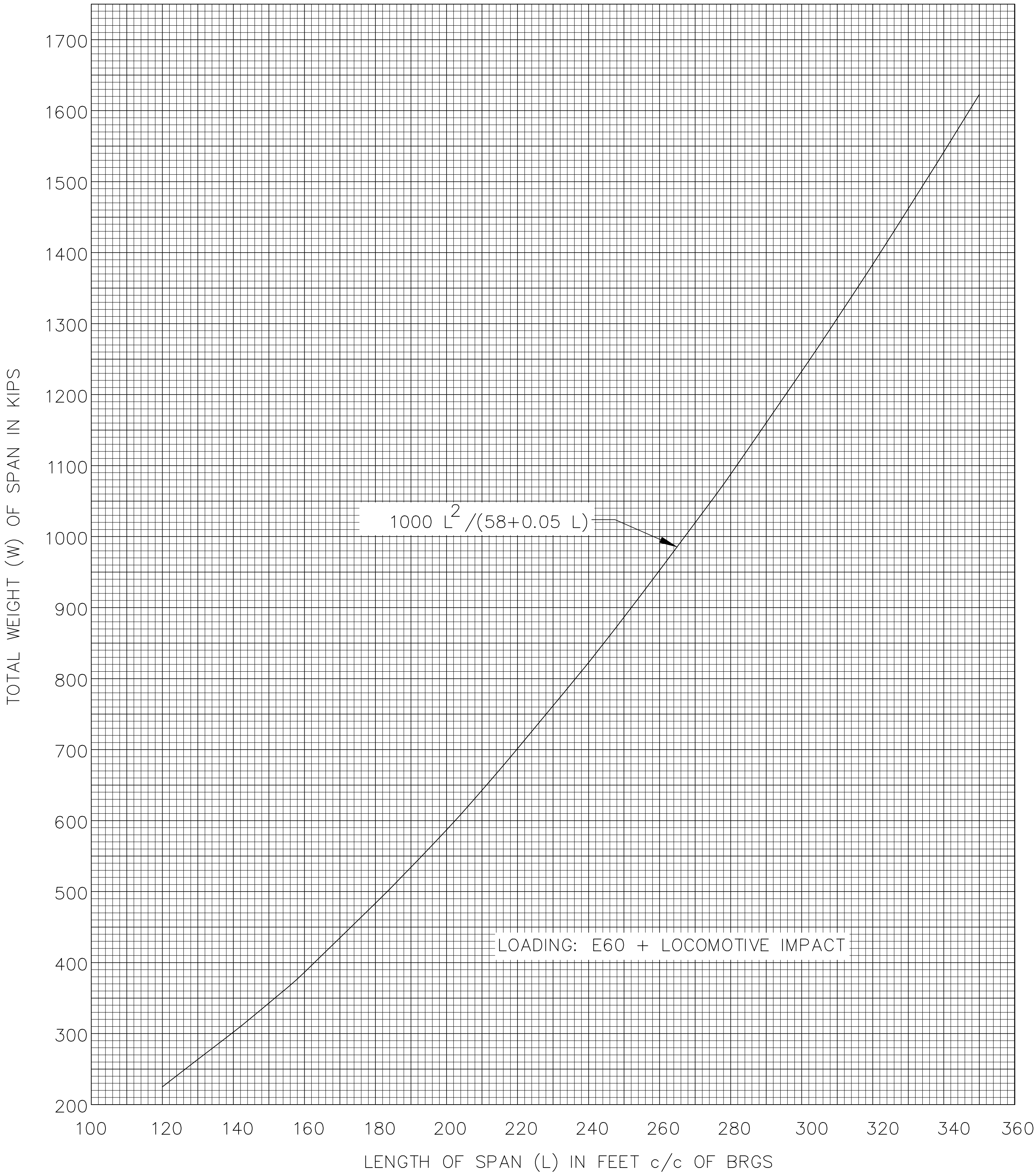


Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

WEIGHT OF T.P.G. OPEN DECK, SINGLE TRACK SPAN RIVETED CONSTRUCTION

METROLINX

Drawing Number Dessin Numero	Rev.
FIGURE 17	



Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

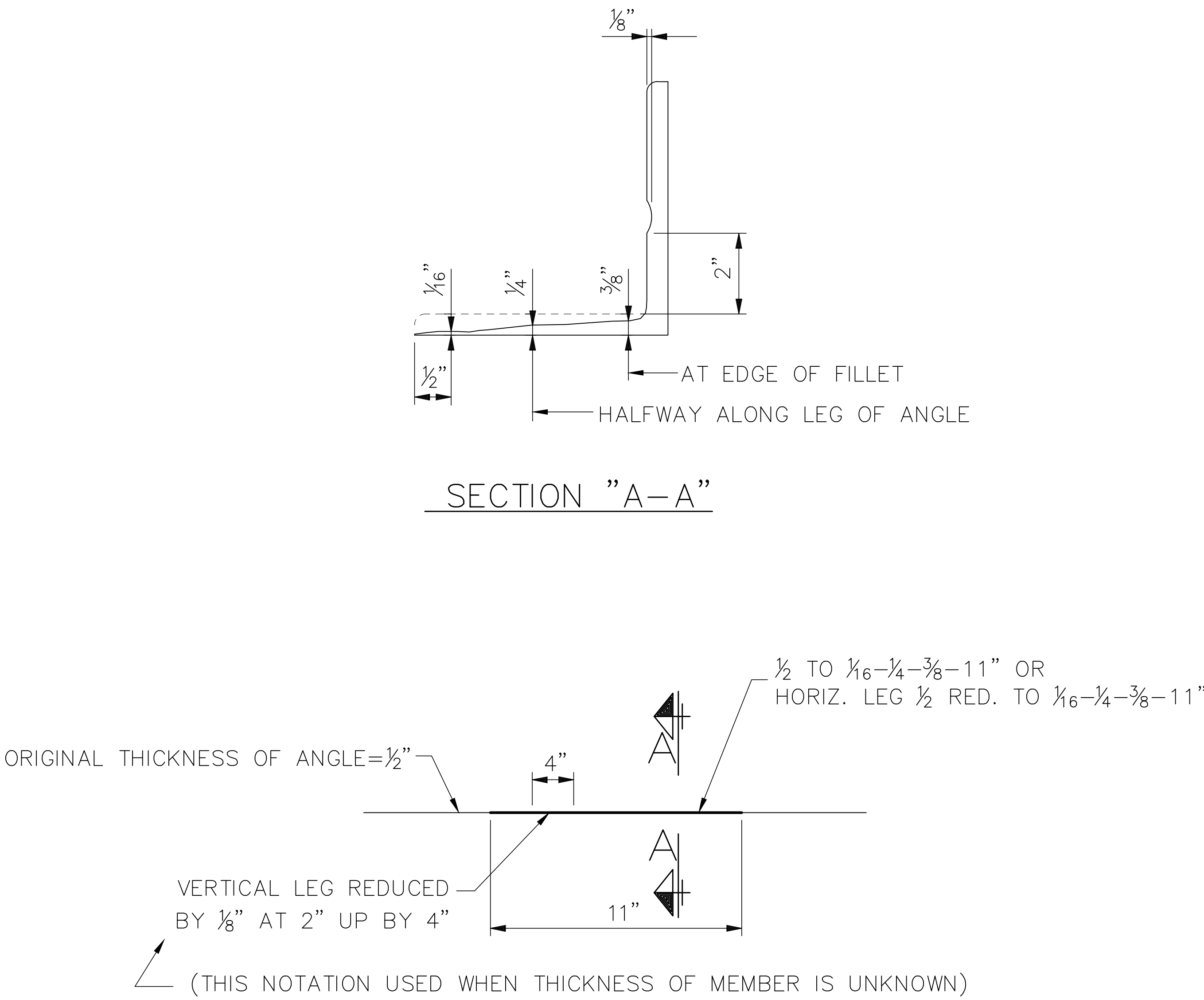
WEIGHT OF T.T. OPEN DECK,
SINGLE TRACK SPAN
RIVETED CONSTRUCTION

METROLINX

Drawing Number
Dessin Numero

FIGURE 18

Rev.



Drawn / Dessin	FM	Checked / Verification	S.Kh.	INTERPRETATION OF STEEL INSPECTION REPORT
Scale / Echelle	NONE	Date	March 2019	

METROLINX

Drawing Number / Dessin Numero	Rev.
FIGURE 19	

ABBREVIATIONS	DESCRIPTIONS	ABBREVIATIONS	DESCRIPTIONS
A.B.	Anchor bolt	Ingr	In groove
&	and	Ins	Inside
@	at	Insp bar	Inspection bar
B/W	Back wall	K.B.	Knee brace
BP	Batten plate	KE	Knife edge
Brg	Bearing	Lat br	Lateral bracing
Bed pl	Bed plate	Lat bar	Lattice bar
Bet	Between	LT	Left
Btm	Bottom	Lg	Long
Btm chord	Bottom chord	N	North
Btm Flg	Bottom flange	Off SQ	Off square
Btm lat br	Bottom lateral bracing	OSL	Out Standing Leg
B\f	Brace frame	Outs	Outside
CL	Center line	Pl	Plate
C.A.	Compound angle	Pump	Pumping
Conn L	Connecting angle	Red	Reduced
CWR	Continuous welded rail	Reinf	Reinforcing
CTSK	Countersunk	Rt	Right
CSRH	Counter sunk river head	Riv	Rivet
Cover pl	Cover plate	Roller Brg	Roller bearing
CR	Crack	Run rail	Running rail
Diag	Diagonal	Scour	Scouring
Diap	Diaphragm	Shelf L	Shelf angle
DN	Down	Shim pl	Shim plate
DSL	Down standing leg	Shoe pl	Shoe plate
E	East	S	South
End brg stiff	End bearing stiffener	Spc	Space
Exp joint	Expansion joint	Stiff	Stiffener
Filler pl	Filler plate	Stri	Stringer
Flg L	Flange angle	Strut br	Strut bracing
F\B	Floor beam	Sw br	Sway bracing
Fract	Fracture	Tie pl	Tie plate
FL	Full length	Top flg	Top flange
FS	Full size	Tranv br	Transverse bracing
FW	Full width	USL	Up Standing Leg
GFA	Girder flange angle	VGP	Vertical gusset plate
GP	Gusset plate	V leg	Vertical leg
H	Hole	W	West
H GP	Horizontal gusset plate	W/W	Wing wall
H leg	Horizontal leg	W/...	With ...

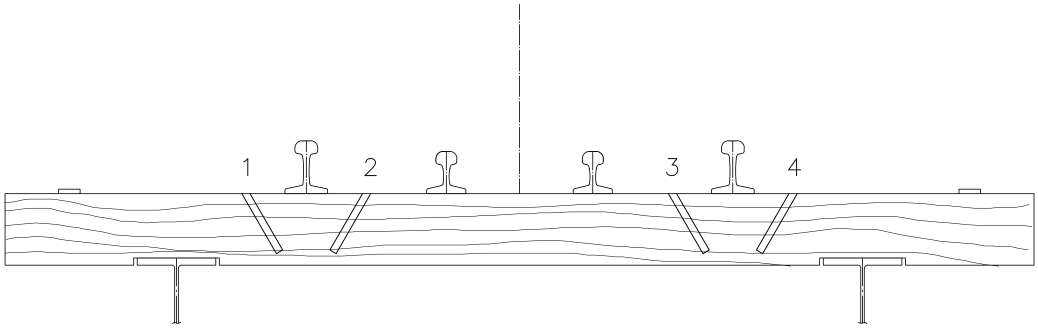
Drawn / Dessin	FM	Checked / Verification	S.Kh.
Scale / Echelle	NONE	Date	March 2019

ABBREVIATIONS USED IN STEEL INSPECTION REPORT

METROLINX

Drawing Number / Dessin Numero: FIGURE 20

Rev.



VIEW LOOKING IN INCREASED MILEAGE DIRECTION

TIE No.	INSPECTION BORE HOLE	#1	#2	#3	#4	REMARKS
37	SPIKES	L	OK	L 1"H	OK	CRACK ," x 3" DEEP, 3' LONG STARTING FROM LEFT END. OTHER SMALL CRACKS. THE TIE SURFACE IS ROTTEN TO A DEPTH OF 1" ON A LARGE AREA. SPIKES TRANSPOSED.
		2"R 4" 3"V	1"R 7" 1"V	4"R	1"R	

LEGEND:

1"R – MEANS 1" OF ROT AT SURFACE.
7" – MEANS 7" OF GOOD TIMBER.
1"V – MEANS 1" VOID OR ROT.

SPIKES:

OK – MEANS SPIKE IS NOT LOOSE & IS SITTING TIGHT.
L – MEANS SPIKE IS LOOSE.
1"H – MEANS SPIKE EXTENDS BY 1" IN HEIGHT.
X – MEANS SPIKE IS MISSING

Drawn Dessin	FM	Checked Vérification	S.Kh.
Scale Échelle	N.T.S.	Date	March 2019

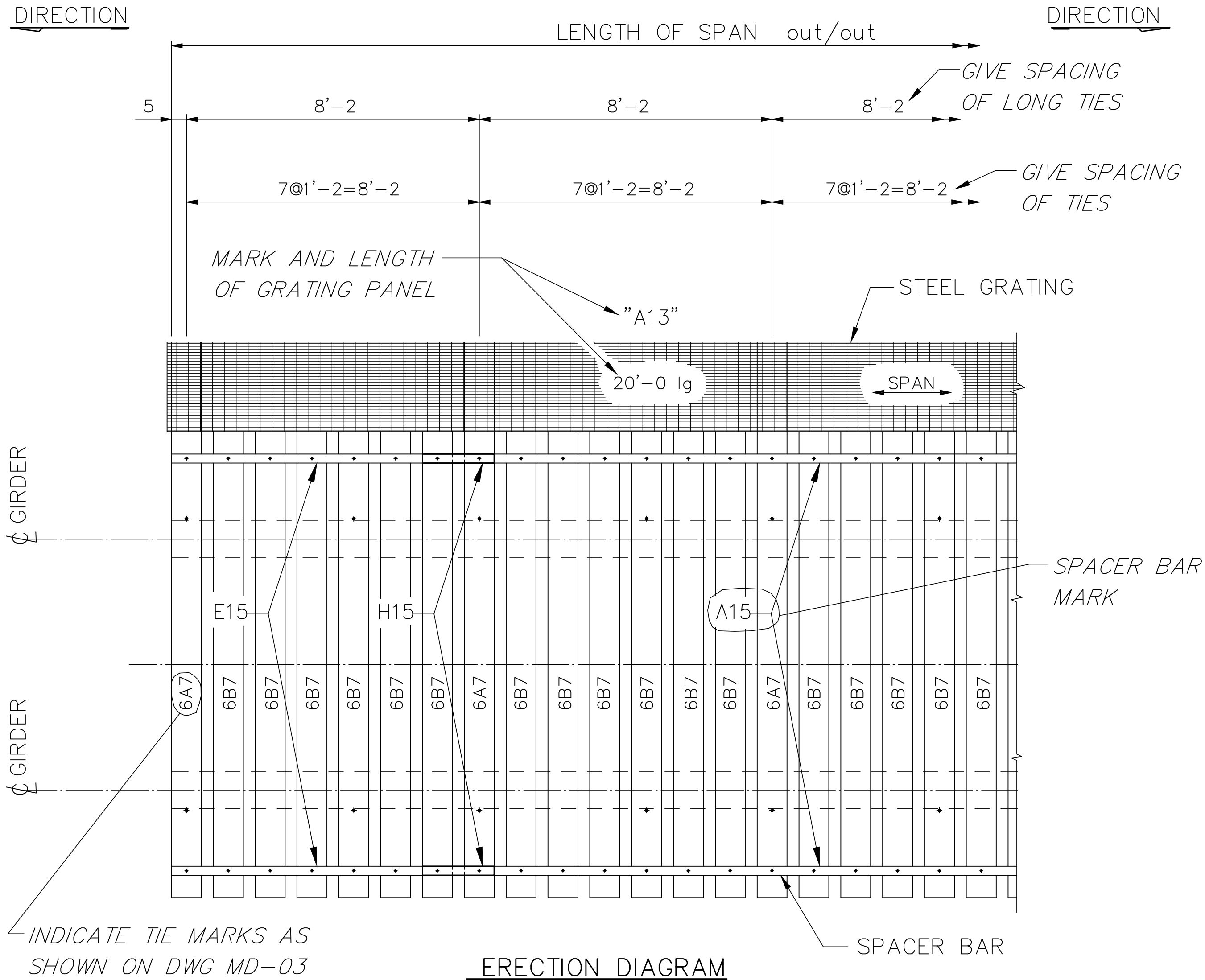
INTERPRETATION OF TIMBER DECK INSPECTION REPORT

METROLINX

Drawing Number Dessin Numero: FIGURE 21

Rev.

ERECTION DIAGRAM SHOULD BE DRAWN SO THAT INCREASING MILEAGE IS SHOWN TOWARDS LEFT SIDE OF DRAWING.



ERECTION DIAGRAM

SPAN No. 3

WHEN MORE THAN ONE SPAN INDICATE SPAN No. FOR EACH SPAN

–ERECTION DIAGRAM SHALL BE DRAWN TO SCALE RANGING BETWEEN (¼"=1'–0") TO (½"=1'–0")

STANDARD NOTES SHOULD APPEAR ON RIGHT HAND SIDE OF DRAWING ABOVE TITLE BLOCK.

COMMENTS SHOWN IN ITALIC

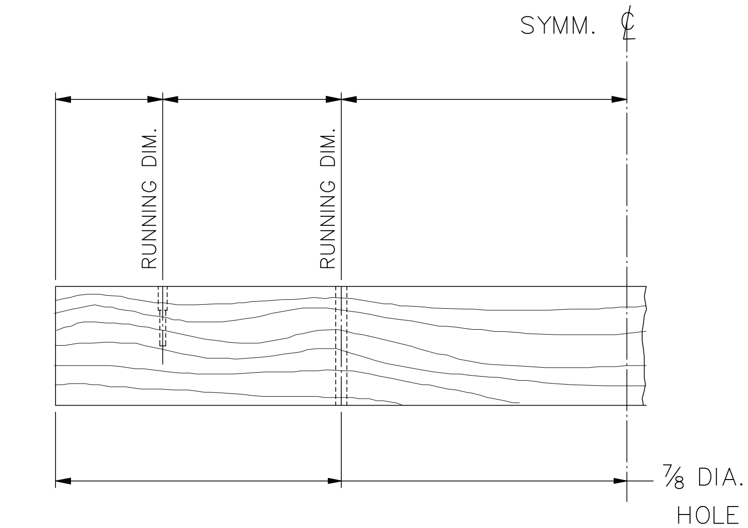
Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

STANDARD WAY TO SHOW ERECTION DIAGRAM FOR TIMBER DECKS

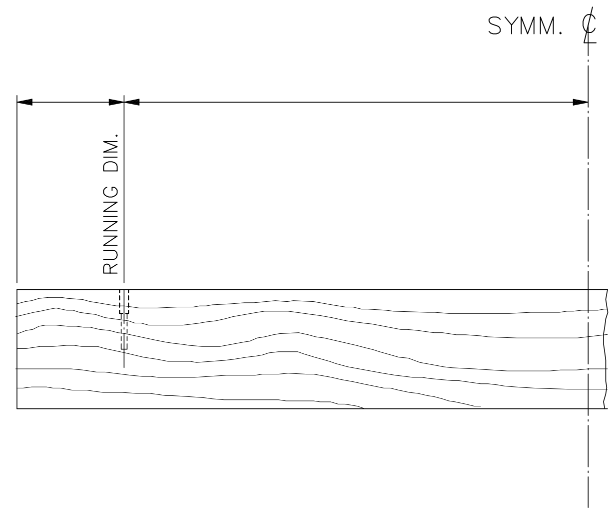
METROLINX

Drawing Number Dessin Numero: FIGURE 22

Rev.

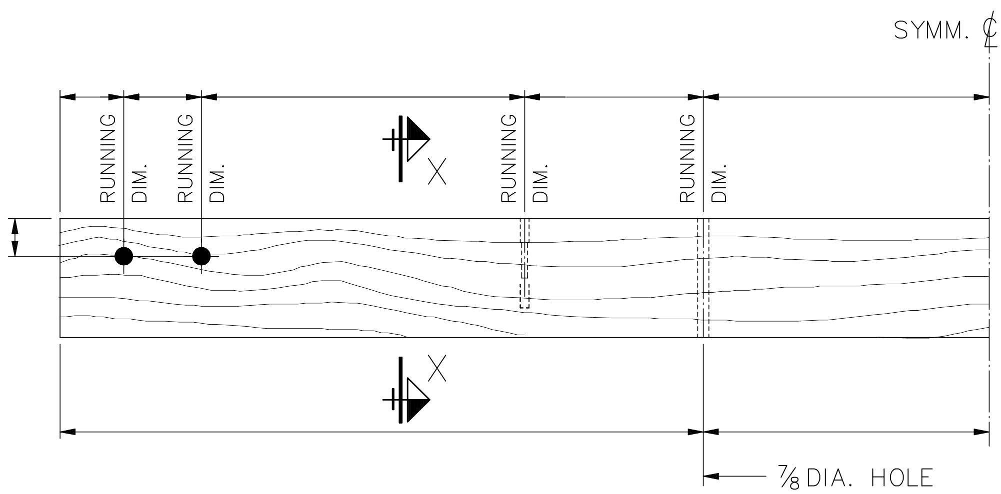


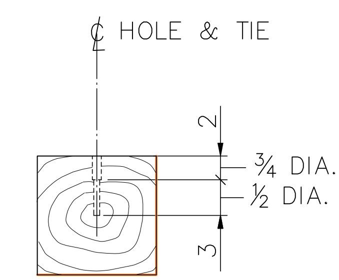
WIDTH"x DEPTH"x LENGTH'–"(SIZE FRAMED HEIGHT)
QTY –REQUIRED – MARK



WIDTH"x DEPTH"x LENGTH'–"(SIZE FRAMED HEIGHT)
QTY –REQUIRED – MARK

WITHOUT WALKWAYS





SECT."X–X"

WITH WALKWAYS

WIDTH"x DEPTH"x LENGTH'–"(SIZE FRAMED HEIGHT)
QTY –REQUIRED – MARK

NOTE:
- BRIDGE TIES TO BE INSTALLED IN ACCORDANCE WITH METROLINX STD. DWG'S.: R9A–1.3(B) & R1A–1.4(A).

COMMENTS SHOWN IN ITALIC

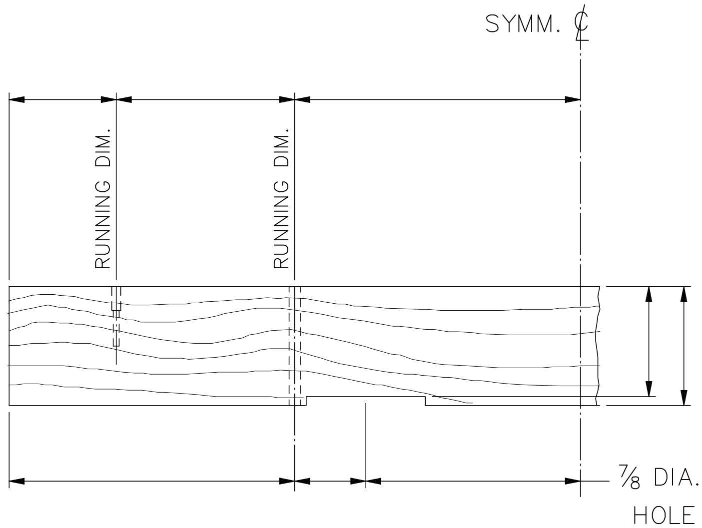
Drawn / Dessin	FM	Checked / Vérification	S.Kh.
Scale / Échelle	N.T.S.	Date	March 2019

STANDARD WAY TO DETAIL SIZED TIES

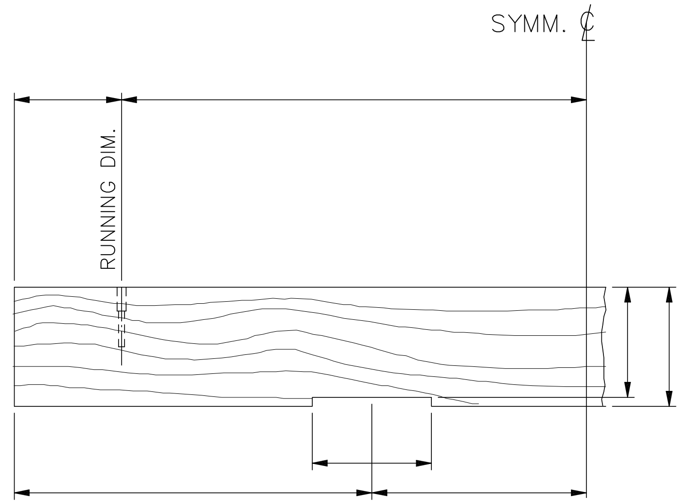
METROLINX

Drawing Number / Dessin Numero: FIGURE 23

Rev.

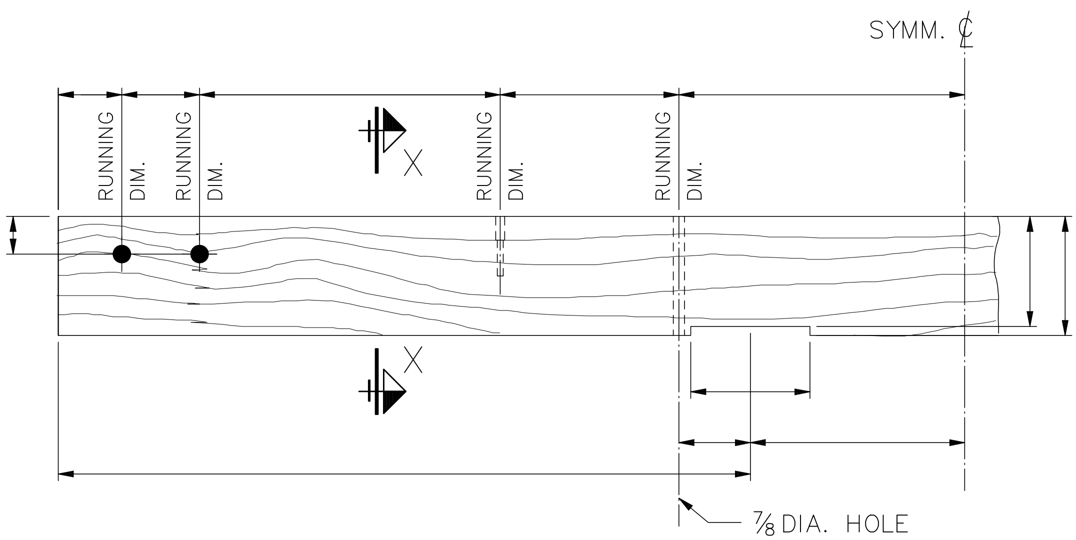


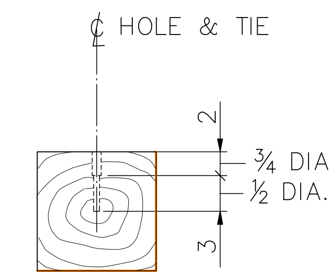
WIDTH"x DEPTH"x LENGTH'–"(SIZE FRAMED HEIGHT)
QTY –REQUIRED – MARK



WIDTH"x DEPTH"x LENGTH'–"(SIZE FRAMED HEIGHT)
QTY –REQUIRED – MARK

WITHOUT WALKWAYS





SECT."X–X"

WITH WALKWAYS

WIDTH"x DEPTH"x LENGTH'–"(SIZE FRAMED HEIGHT)
QTY –REQUIRED – MARK

NOTE:
- BRIDGE TIES TO BE INSTALLED IN ACCORDANCE WITH METROLINX STD. DWG'S.: R9A–1.3(B) & R1A–1.4(A).

COMMENTS SHOWN IN ITALIC

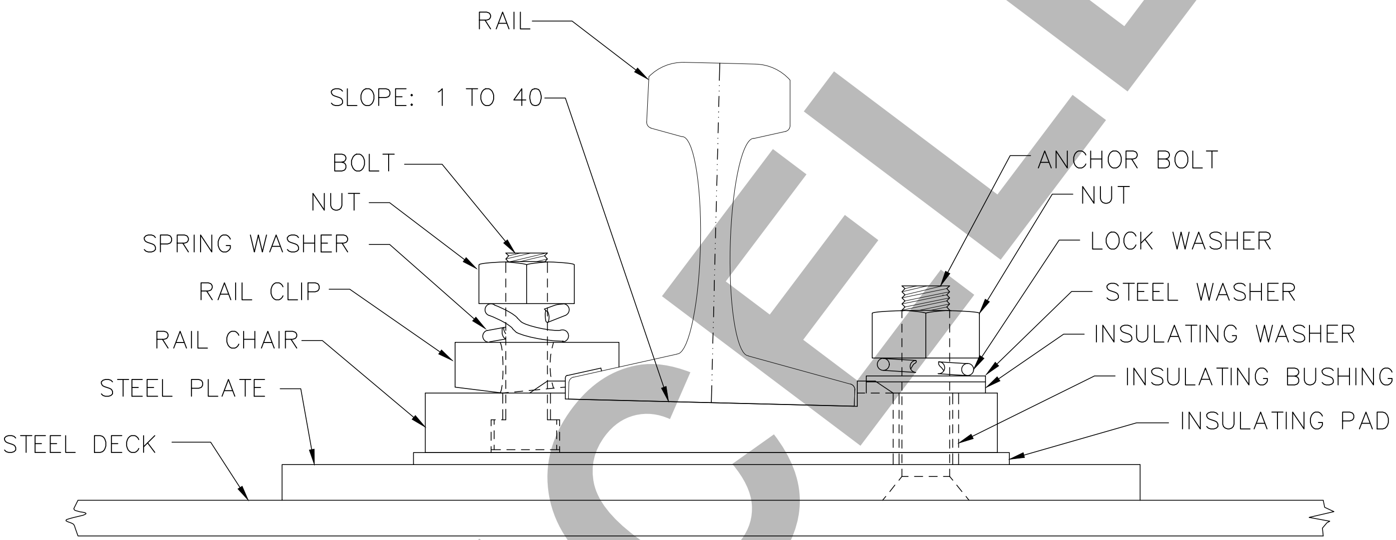
Drawn Dessin	FM	Checked Vérification	S.Kh.
Scale Échelle	N.T.S.	Date	March 2019

STANDARD WAY TO DETAIL
DAPPED TIES

METROLINX

Drawing Number
Dessin Numero
FIGURE 24

Rev.



NOTE: PLAN VIEW SIMILAR TO FIGURE 26

CANCELED

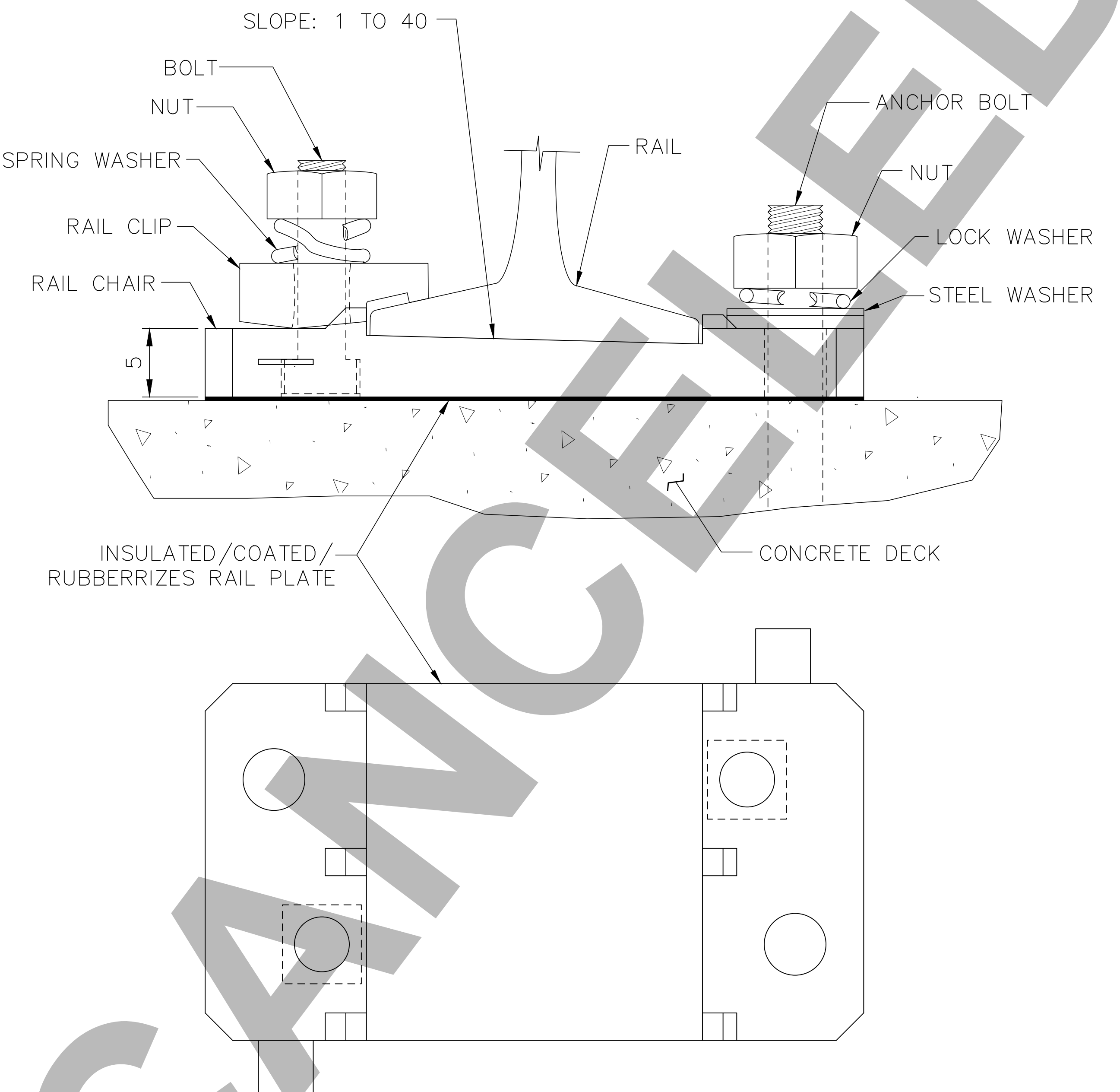
Drawn Dessin	FM	Checked Vérification	S.Kh.
Scale Échelle	NONE	Date	March 2019

TYPICAL RAIL CHAIRS ON STEEL DECK

METROLINX

Drawing Number Dessin Numero FIGURE 25

Rev.



RAIL CHAIR

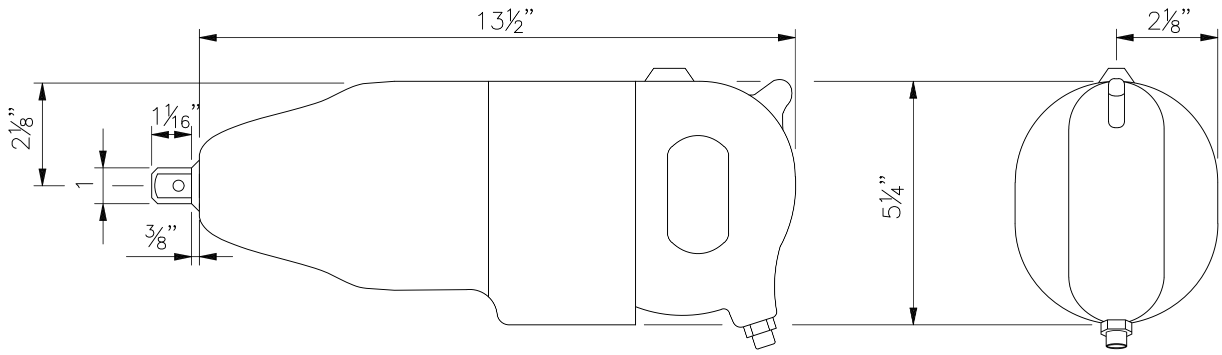
Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

TYPICAL RAIL CHAIR
ON CONCRETE DECK

METROLINX

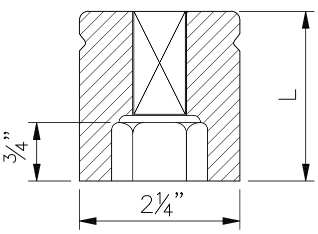
Drawing Number Dessin Numero: FIGURE 26

Rev.



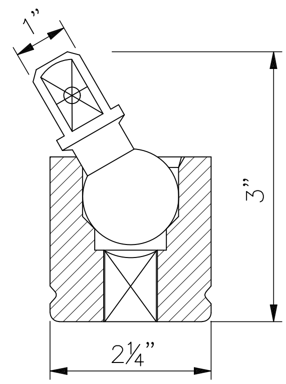
IMPACT WRENCHES

– APPROXIMATE WEIGHT : 20 Lbs



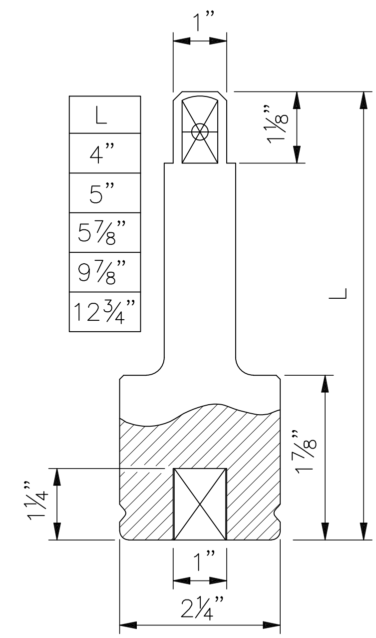
NOMINAL OPENINGS		LENGTH L	
ACROSS CORNERS	ACROSS FLATS	REG	LONG
1⅝"	1⁷⁄₁₆"	2⅜"	4"

SOCKETS

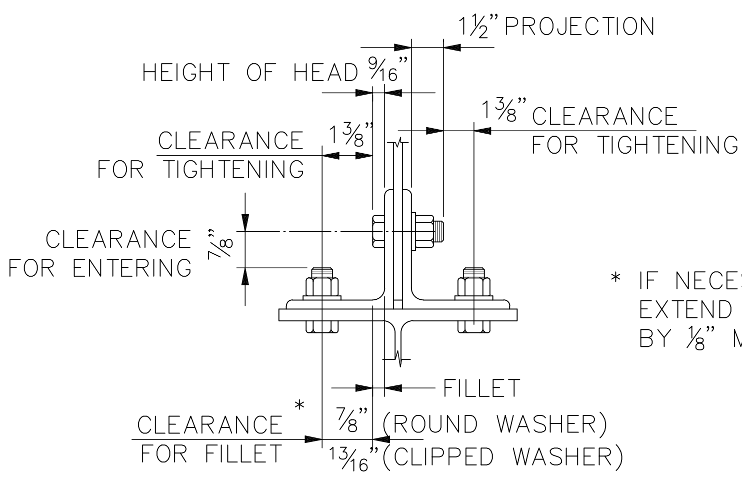


UNIVERSAL JOINTS

L
4"
5"
5⅞"
9⅞"
12¾"



EXTENSIONS



ERECTION CLEARANCES FOR IMPACT WRENCH

* IF NECESSARY, WASHER MAY EXTEND PAST EDGE OF FILLET BY ⅛" MAX.

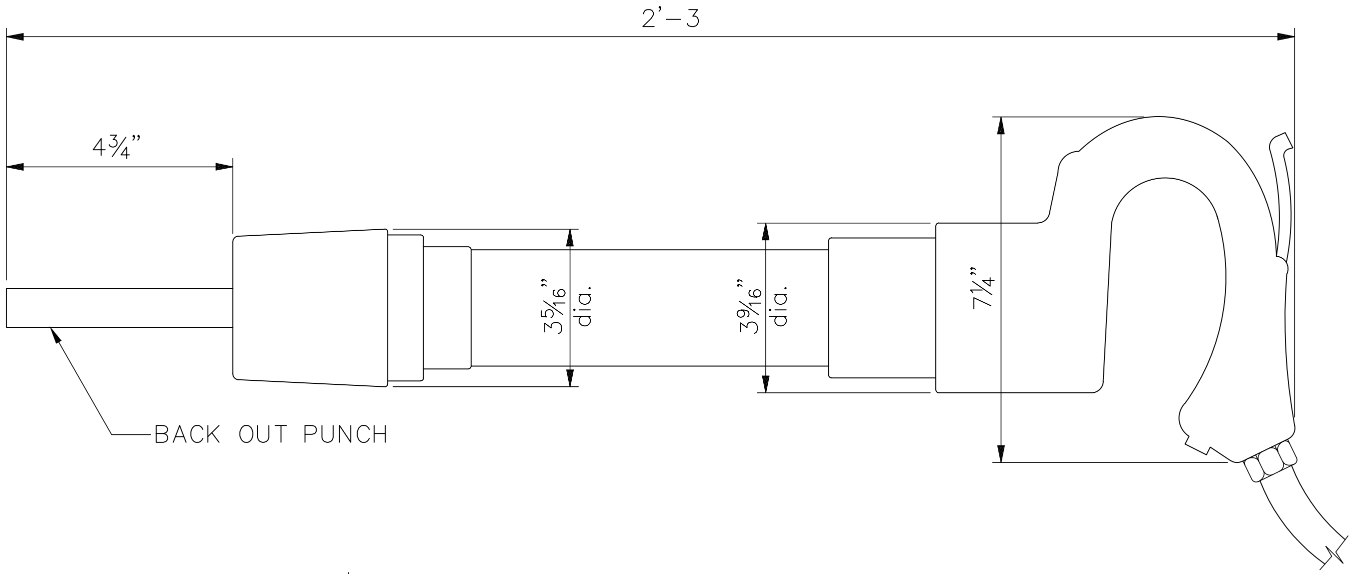
N.B.: * CLEARANCES FOR OPERATOR ARE TO BE ADDED TO REQUIRED CLEARANCES GIVEN FOR THE EQUIPMENT.

ALL UNITS ON THIS DRAWING ARE IN IMPERIAL.

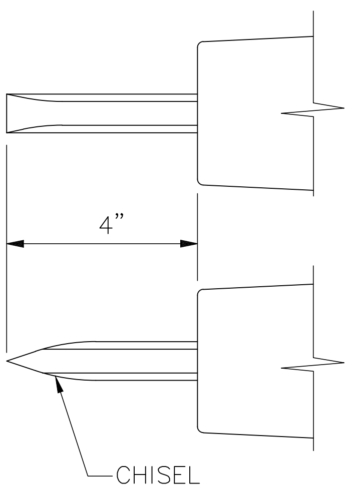
Drawn Dessin	FM	Checked Verification	S.Kh.	CLEARANCES FOR IMPACT WRENCHES
Scale Echelle	NONE	Date	March 2019	

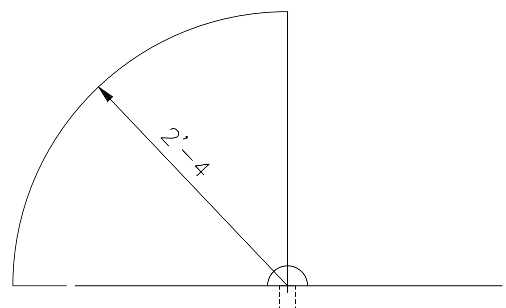
METROLINX

Drawing Number Dessin Numero	Rev.
FIGURE 27	



– APPROXIMATE WEIGHT : 30 Lbs





REQUIRED CLEARANCES FOR RIVET BUSTER

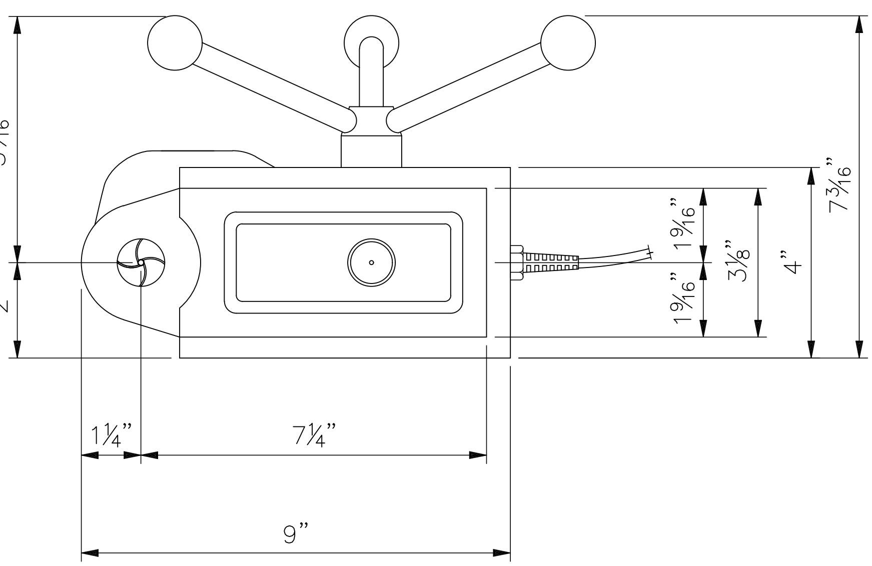
N.B.: CLEARANCES FOR OPERATOR ARE TO BE ADDED TO REQUIRED CLEARANCES GIVEN FOR THE EQUIPMENT.

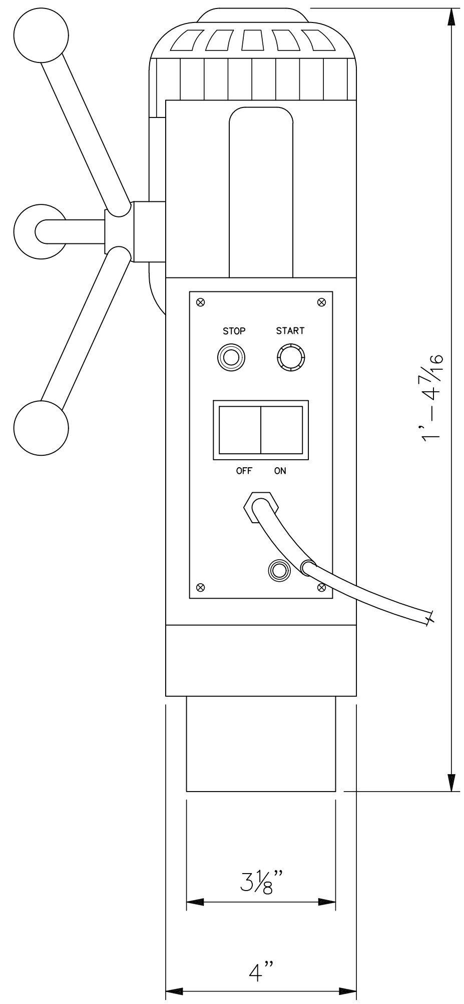
ALL UNITS ON THIS DRAWING ARE IN IMPERIAL.

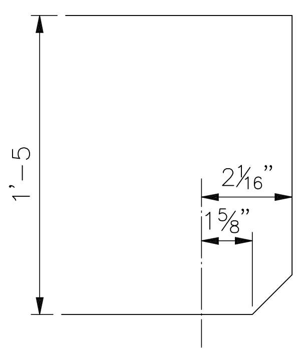
Drawn Dessin	FM	Checked Verification	S.Kh.	CLEARANCES FOR RIVET BUSTER
Scale Echelle	NONE	Date	March 2019	

METROLINX

Drawing Number Dessin Numero	Rev.
FIGURE 28	







– APPROXIMATE WEIGHT : 30 Lbs

N.B.: CLEARANCES FOR OPERATOR ARE TO BE ADDED TO REQUIRED CLEARANCES GIVEN FOR THE EQUIPMENT.

REQUIRED CLEARANCES FOR MAGNETIC DRILL

ALL UNITS ON THIS DRAWING ARE IN IMPERIAL.

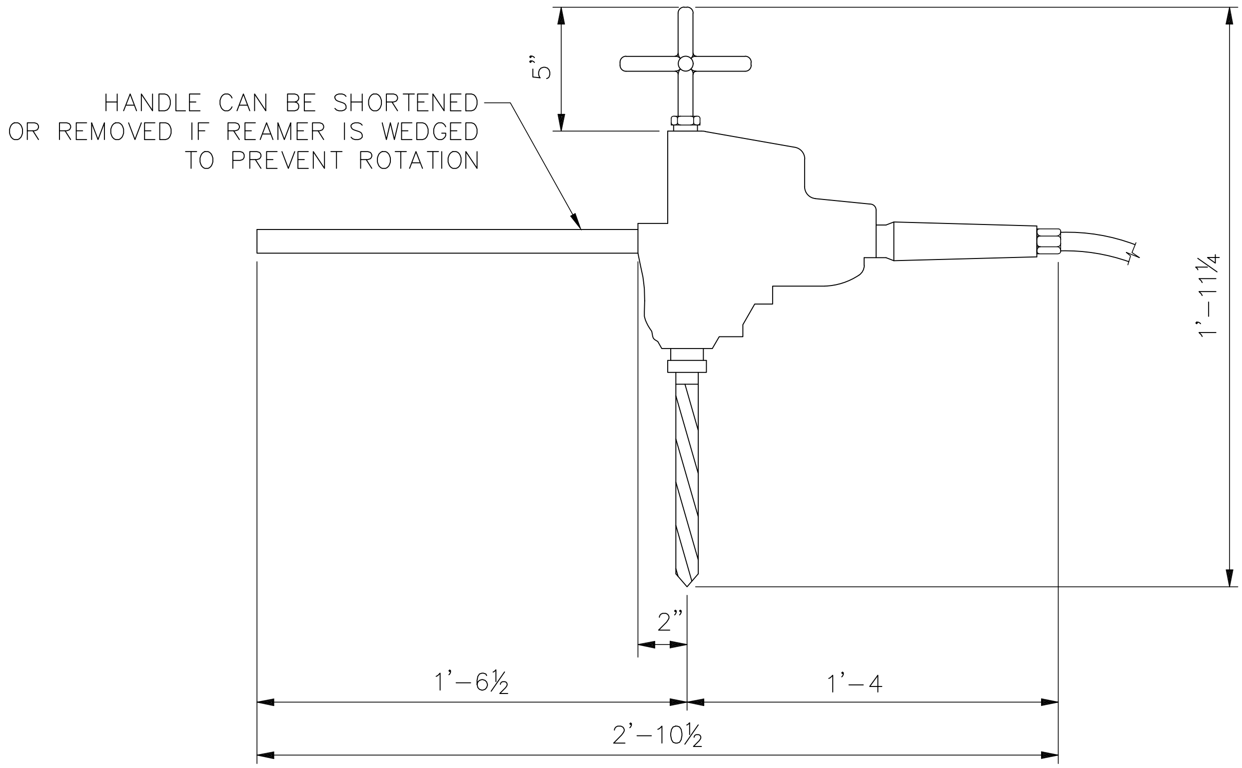
Drawn / Dessin	FM	Checked / Verification	S.Kh.
Scale / Echelle	NONE	Date	March 2019

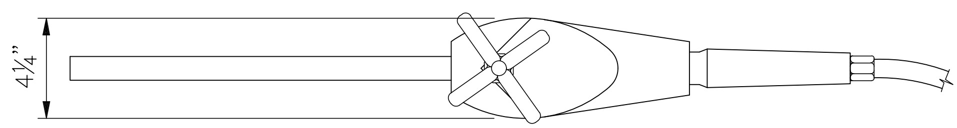
CLEARANCES FOR MAGNETIC DRILL

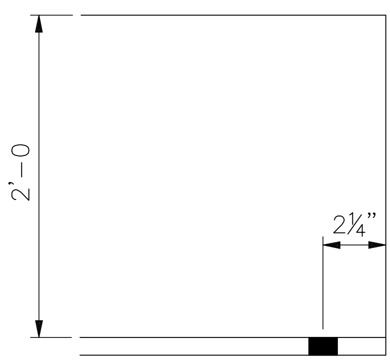
METROLINX

Drawing Number / Dessin Numero: FIGURE 29

Rev.







REQUIRED CLEARANCES FOR
PNEUMATIC REAMER
(WITH HANDLE REMOVED)

N.B.: CLEARANCES FOR OPERATOR ARE TO BE ADDED TO REQUIRED CLEARANCES GIVEN FOR THE EQUIPMENT.

ALL UNITS ON THIS DRAWING ARE IN IMPERIAL.

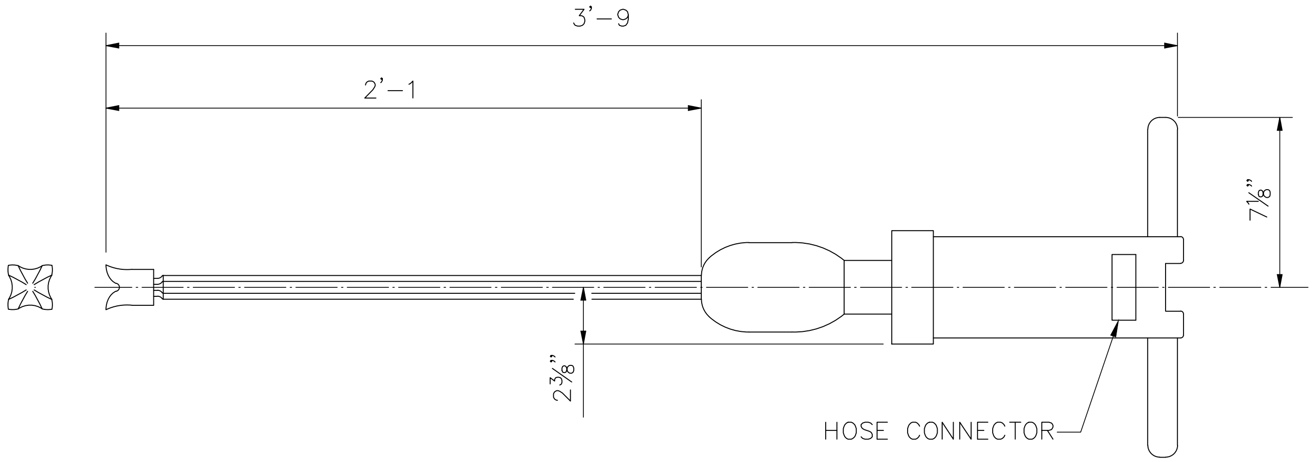
Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

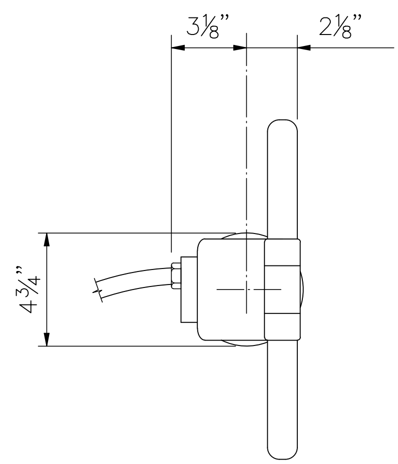
CLEARANCES FOR PNEUMATIC REAMER

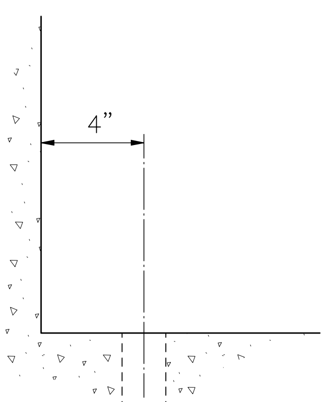
METROLINX

Drawing Number Dessin Numero: FIGURE 30

Rev.







N.B.: CLEARANCES FOR OPERATOR ARE TO BE ADDED TO REQUIRED CLEARANCES GIVEN FOR THE EQUIPMENT.

REQUIRED CLEARANCES FOR CONCRETE DRILL

ALL UNITS ON THIS DRAWING ARE IN IMPERIAL.

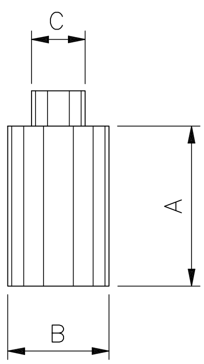
Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

CLEARANCES FOR CONCRETE DRILL

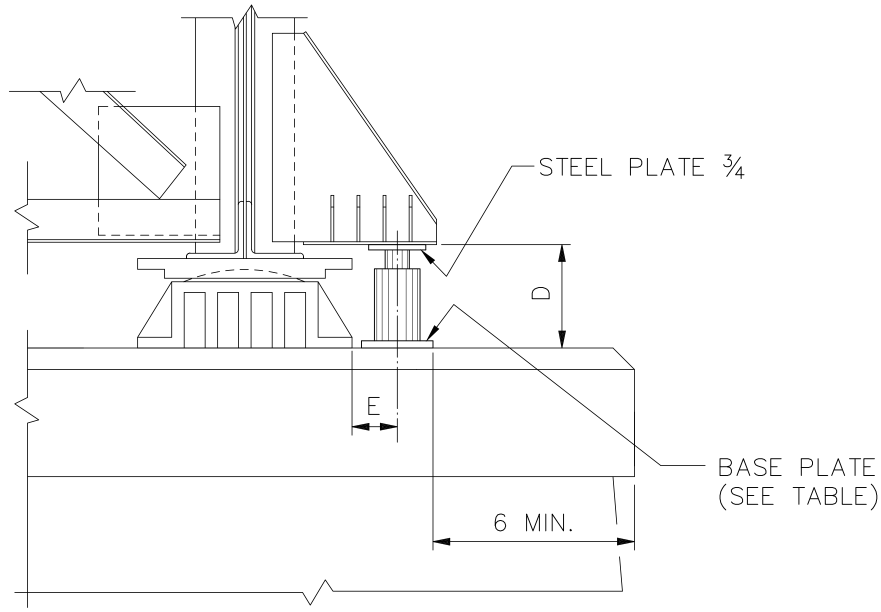
METROLINX

Drawing Number
Dessin Numero: FIGURE 31

Rev.



JACK



ELEVATION

CAPACITY (TON)	STROKE	A	B	C	D (1) MIN.	E MIN.	BASE PLATE
30 STANDARD	6	8⅞	4	2	13¾	3½	6x6x¾
30 LOW PROFILE	½	2 5/16	4½	2	5¼	3½	6x6x¾
50 STANDARD	6	9	5½	3	14	4½	8x8x¾
50 LOW PROFILE	⅝	2⅝	5½	2¼	5½	4½	8x8x¾
100 STANDARD	6	10 1/16	6⅜	3⅜	15⅝	5½	10x10x1
100 LOW PROFILE	⅝	3⅜	7	3⅜	7	5½	10x10x1

(1) HEIGHT OF JACK WAS INCREASED BY 35% (FOR STANDARD JACK) AND 50% (FOR LOW PROFILE JACK) TO ACCOUNT FOR POSSIBLE VARIATIONS IN HEIGHT BETWEEN VARIOUS MANUFACTURERS.

ALL UNITS ON THIS DRAWING ARE IN IMPERIAL.

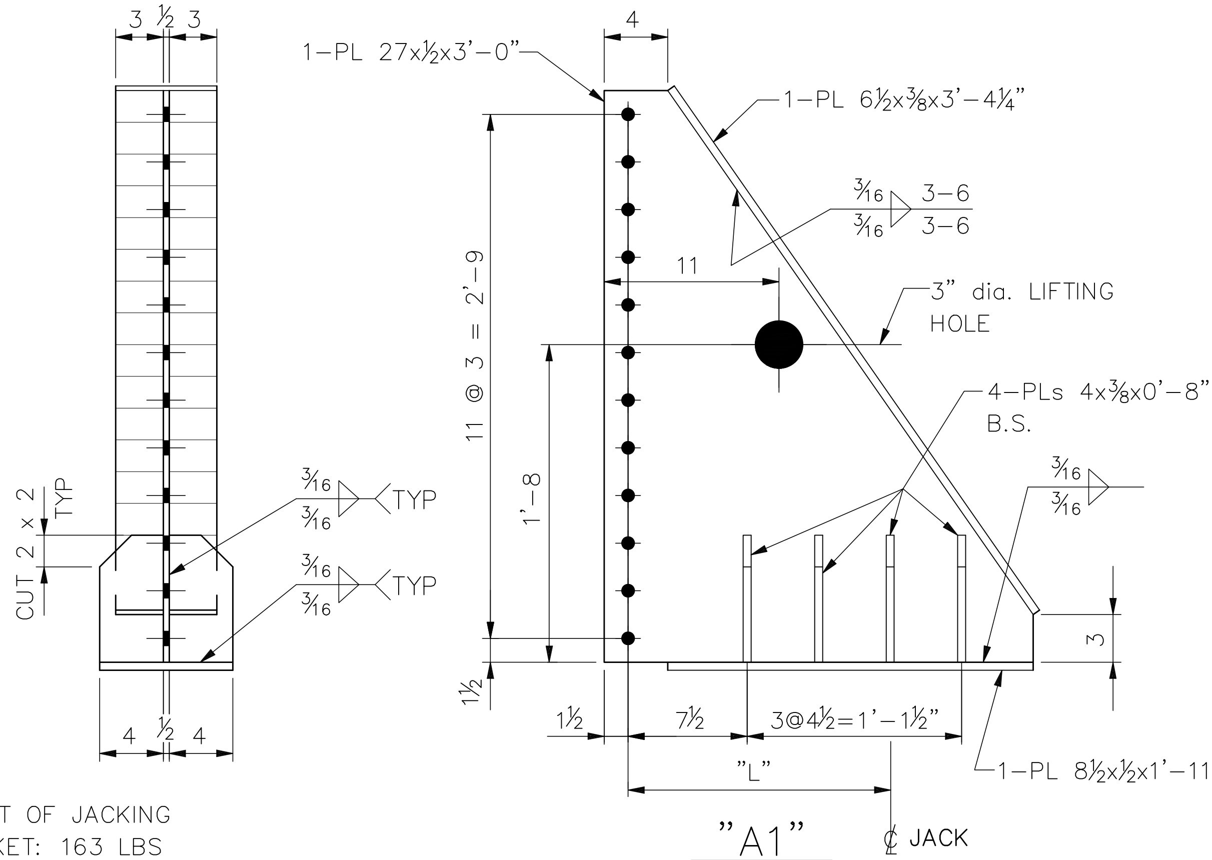
Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

CLEARANCES FOR HYDRAULIC JACKS

METROLINX

Drawing Number Dessin Numero: FIGURE 32

Rev.



WEIGHT OF JACKING BRACKET: 163 LBS

FABRICATION DETAILS

QUANTITY OF BOLTS REQUIRED														
DISTANCE "L"	LOAD IN KIPS													
	10	15	20	25	30	35	40	45	50	55	60	65	70	75
12"	6	6	6	6	6	6	6	8	8	8	8	10	10	12
15"	6	6	6	6	6	6	8	8	8	10	12			
18"	6	6	6	6	6	8	8	10	12					
21"	6	6	6	6	8	8	10	12						

NOTES:

–SEE STD DRAWING MS–01B FOR INSTALLATION OF BOLTS

SPECIFICATIONS:

– STRUCTURAL STEEL: CAN/CSA–G40.21–M92 GRADE 300W
– BOLTS: ⅞"dia. HIGH STRENGTH ASTM A325 WITH HEAVY HEX NUT AND HARDENED ROUND WASHER
– HOLES: UNLESS NOTED OTHERWISE, 15⁄16"dia.
– WELDING: CSA W59–M1989
– MAXIMUM SHEAR IN BOLT = 13 KIPS (BEARING TYPE CONNECTION)

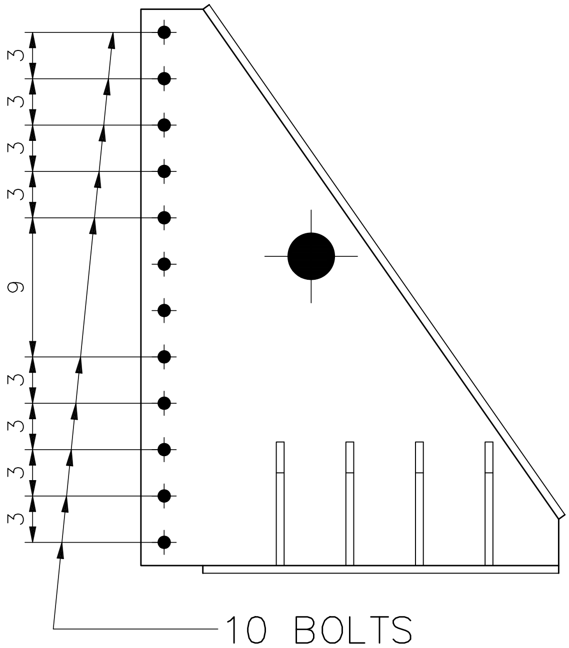
Drawn / Dessin	JE	Checked / Verification	S.Kh.
Scale / Echelle	1"=1'–0	Date	March 2019

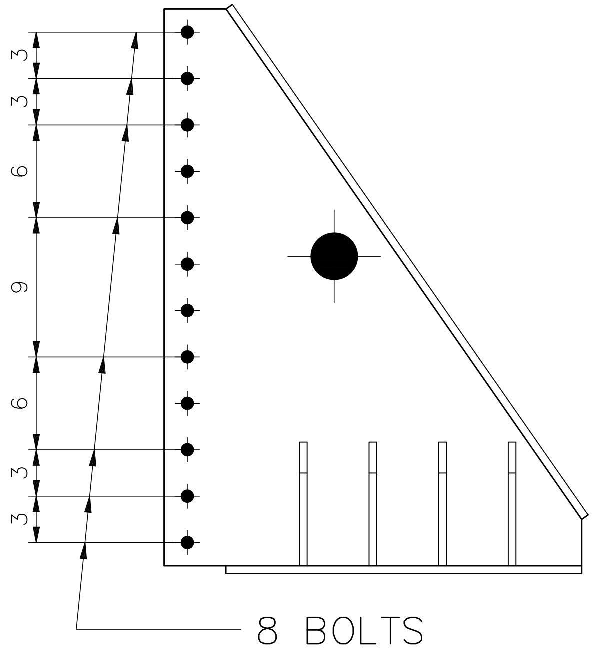
FABRICATION OF STANDARD JACKING BRACKET
(INSTALLED ON STIFFENERS)

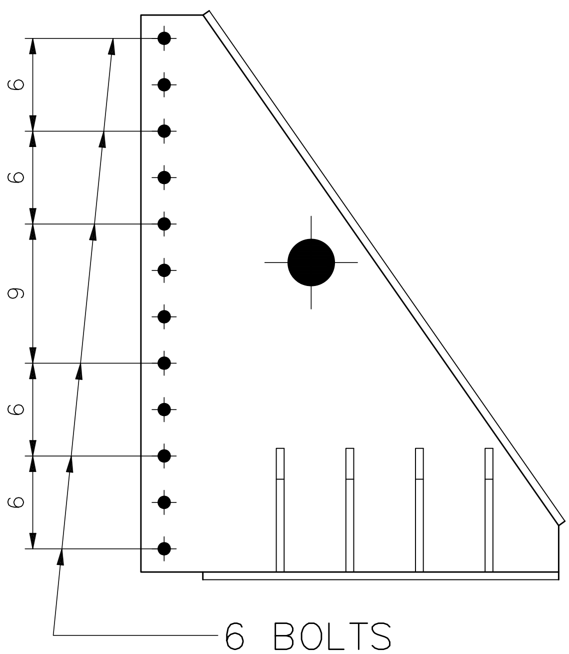
METROLINX

Drawing Number / Dessin Numero: MS–01A

Rev.







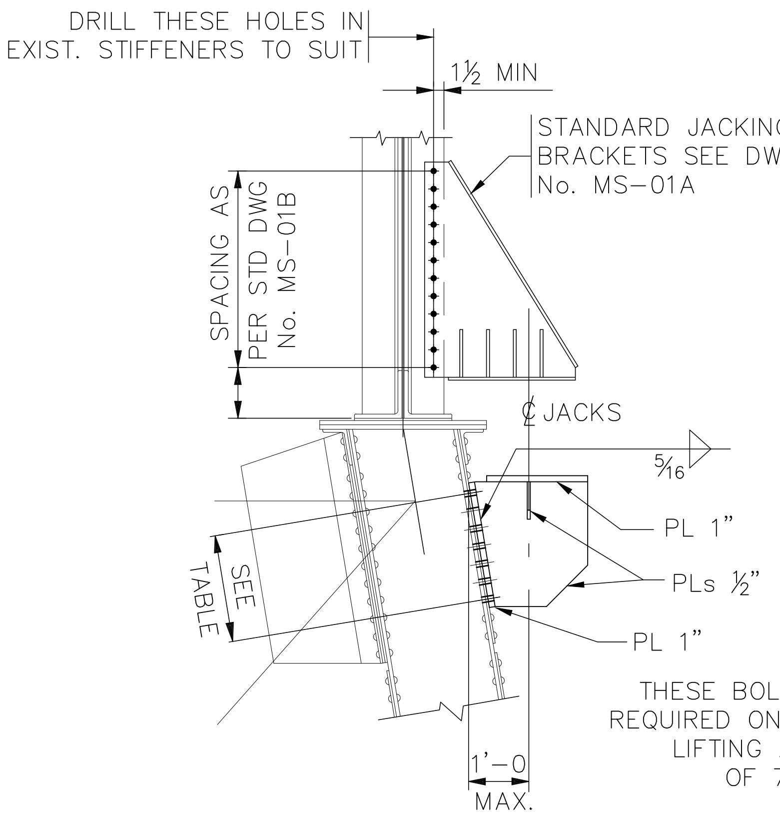
NOTE:

– SEE STANDARD DRAWING MS–01A FOR FABRICATION DETAILS OF STANDARD JACKING BRACKET

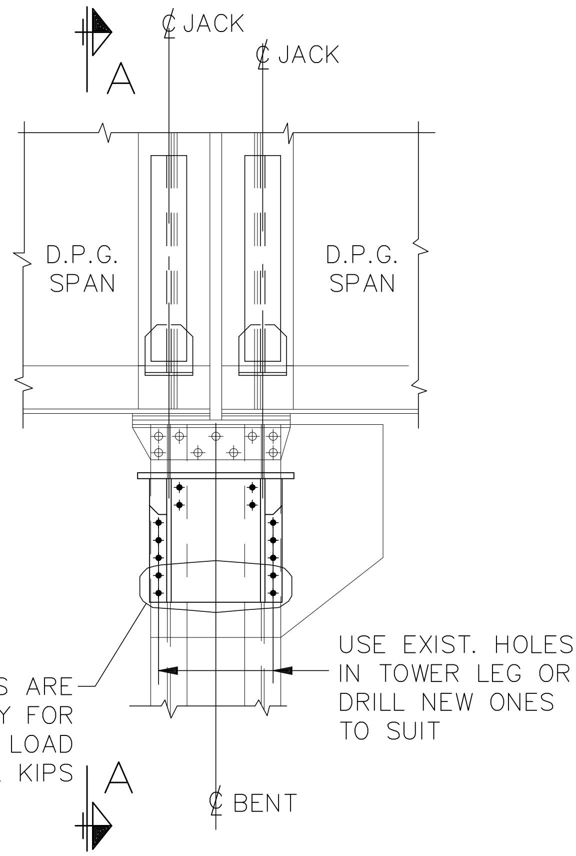
Drawn Dessin	JE	Checked Verification	S.Kh.	INSTALLATION OF STANDARD JACKING BRACKET (ON STIFFENERS)
Scale Echelle	1"=1'–0	Date	March 2019	

METROLINX

Drawing Number Dessin Numero	Rev.
MS–01B	



VIEW "A-A"



ELEVATION

NUMBER OF BOLTS REQUIRED	
NUMBER OF BOLTS *	LOAD ON EACH BRACKET (KIPS)
14	45
18	72

* BASED ON A SPACING OF 3" BETWEEN BOLTS

— MAX TENSION IN BOLT: 10 KIPS

— MAX SHEAR IN BOLT: 4 KIPS FRICTION CONNECTION

NOTE:

— BRACKETS SHOULD BE PLACED AS HIGH AS POSSIBLE ON TOWER LEG BUT LOW ENOUGH TO PERMIT THE REPLACEMENT OF ALL NECESSARY PARTS ON TOWER CAP.

— TENSION IN BOLT IS LIMITED TO 10 KIPS (WITHOUT PRYING ACTION) TO AVOID BENDING THE FLANGE OF THE TOWER LEG.

— ALL WELDS TO BE ¼" FILLET UNLESS NOTED OTHERWISE.

Drawn Dessin	JE	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

STANDARD JACKING BRACKET

FOR JACKING SPANS FROM TOWER

METROLINX

Drawing Number Dessin Numero: MS-02

Rev.

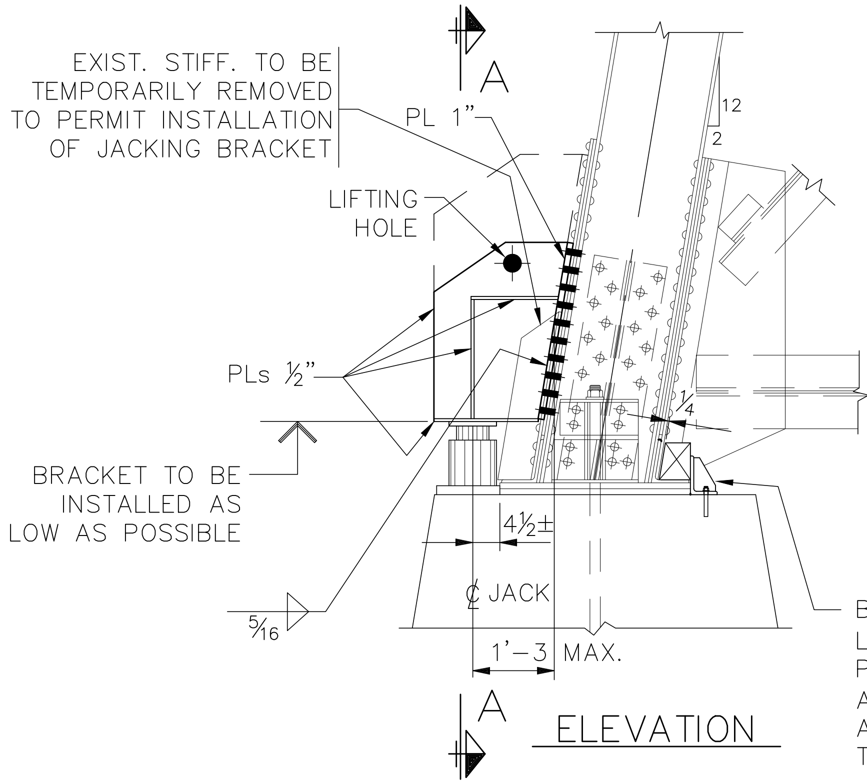
NUMBER OF BOLTS REQUIRED	
NUMBER OF BOLTS *	LOAD ON EACH BRACKET
26	70
32	125

* BASED ON A SPACING OF 3" BETWEEN BOLTS AND ON AN ECCENTRICITY "L" OF 15 IN.

— MAX TENSION IN BOLT: 10 kips

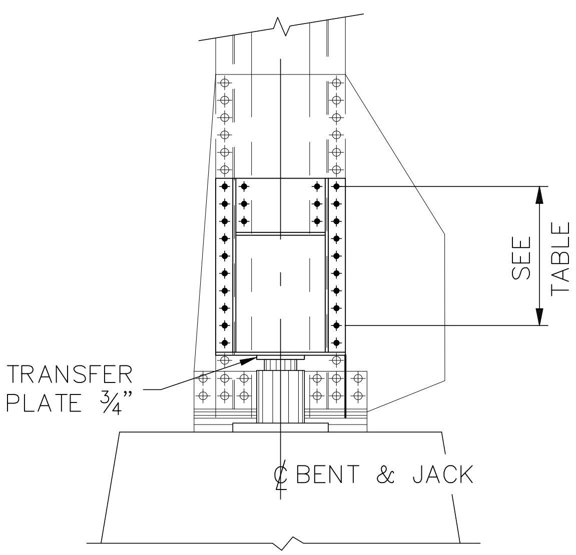
— MAX SHEAR IN BOLT: 4 kips FRICTION CONNECTION

— TENSION IN BOLT IS LIMITED TO 10 KIPS (WITHOUT PRYING ACTION) TO AVOID BENDING THE FLANGE OF THE TOWER LEG.



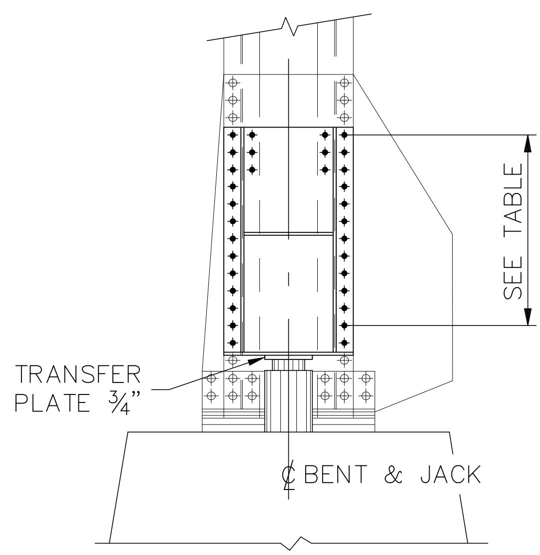
BEFORE LIFTING THE BENT, INSTALL L8x6x½x4'–0"± AGAINST THE BASE PLATES USING 4–HILTI STD STUD ANCHOR KB–TZ ¾"dia.x7" ANCHORS AND A 6x6 HARD WOOD BLOCKING CUT TO THE TOWER LEG BEVEL AS SHOWN.

ELEVATION



VIEW "A–A"

24–BOLTS



VIEW "A–A"

30–BOLTS

NOTES:

— JACK BOTH LEGS OF BENT SIMULTANEOUSLY

— TRAFFIC CAN BE RESTORED ON BRIDGE ONLY AFTER CONCRETE OF REPAIRED AREAS HAS REACHED 10 MPa

— ALL WELDS TO BE ¼" FILLET UNLESS NOTED OTHERWISE.

Drawn Dessin	JE	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

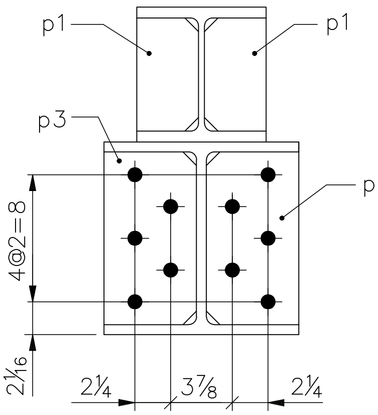
STANDARD JACKING BRACKET

FOR JACKING STEEL BENTS

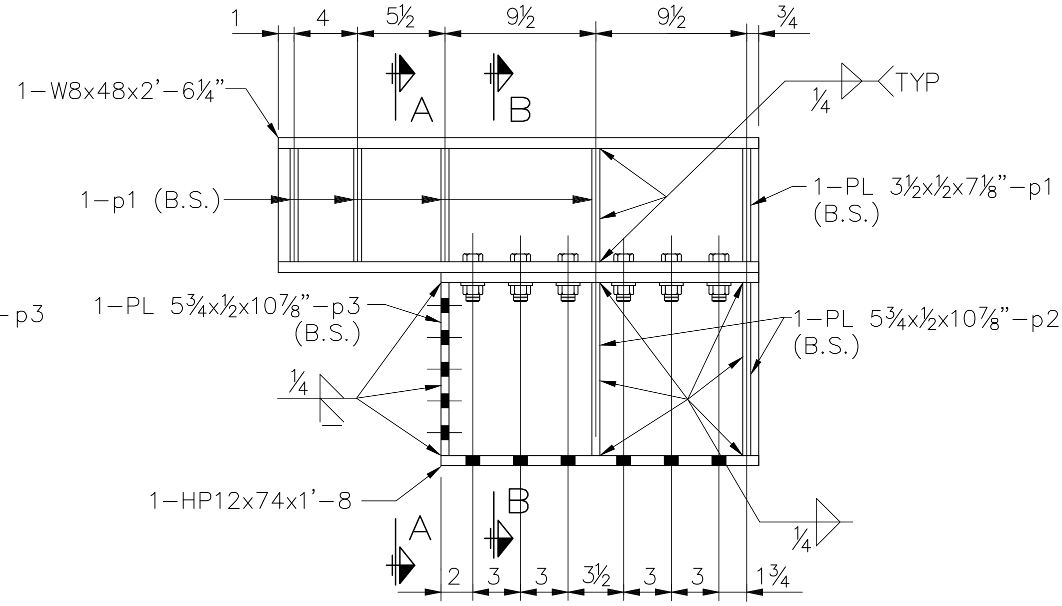
METROLINX

Drawing Number Dessin Numero: MS–03

Rev.

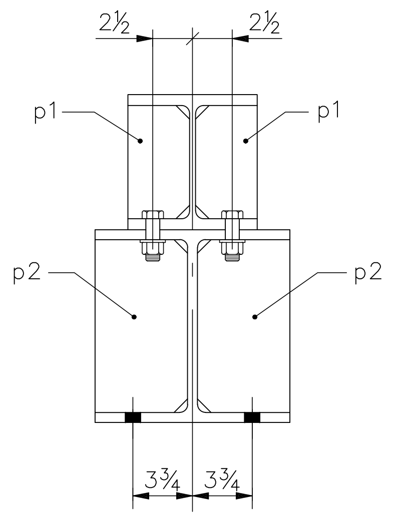


SECTION "A–A"

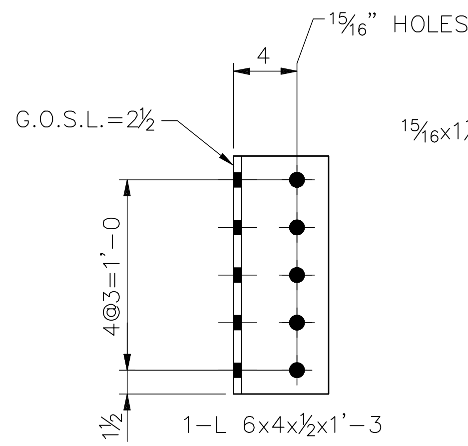


"A4"

UNIT WT: 305 LBS

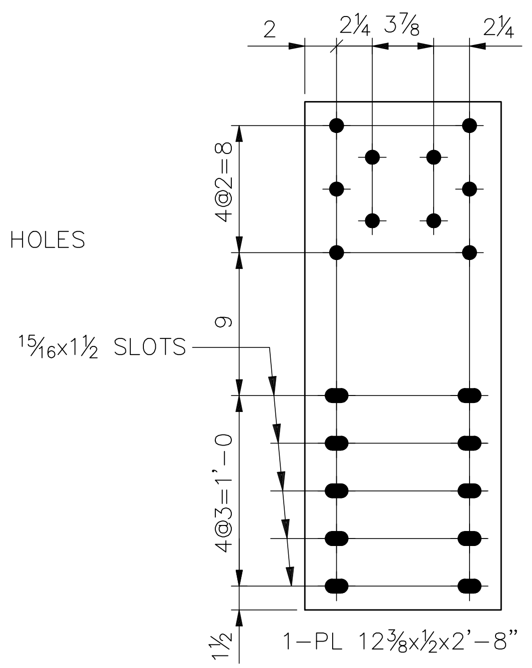


SECTION "B–B"



"C4"

UNIT WT: 20 LBS



"B4"

UNIT WT: 56 LBS

– READ THIS DRAWING IN CONJUNCTION WITH MS–04B

Drawn Dessin	JE	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

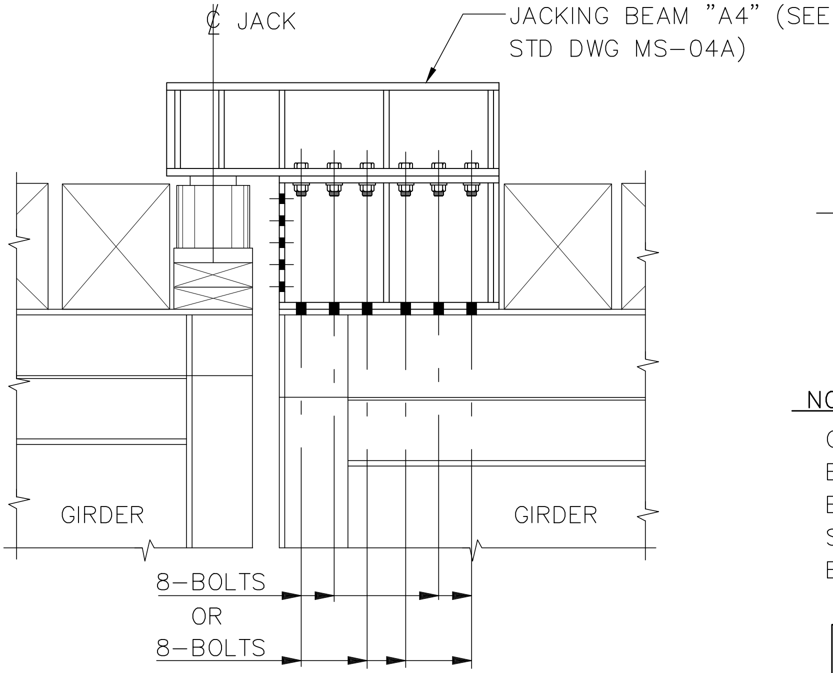
FABRICATION OF STANDARD JACKING BEAM

(INSTALLED ON TOP FLANGE)

METROLINX

Drawing Number Dessin Numero: MS–04A

Rev.



BOLT PATTERN

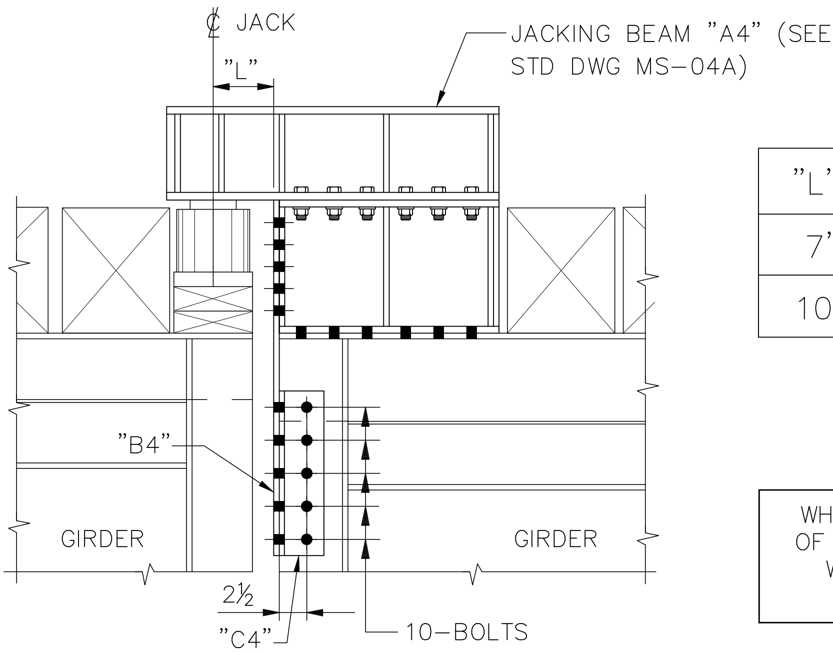
MAXIMUM CAPACITY:
20 kips PER JACKING BEAM

— TENSION IN BOLT IS LIMITED TO 10 kips (WITHOUT PRYING ACTION) TO AVOID BENDING THE FLANGE OF THE GIRDER

NOTE:

CONDITION OF EXIST. TOP FLANGE SHALL BE VERIFIED PRIOR TO INSTALLING JACKING BEAMS. IF CORRODED, INSTALLATION SHALL BE APPROVED BY THE STRUCTURAL ENGINEER — MAINTENANCE

JACKING BEAM ALONE ON SMOOTH TOP GIRDERS



BOLT PATTERN

"L"	BEAM CAPACITY
7"	80 kips
10"	60 kips

— MAX SHEAR IN C4 CONNECTION: 10 kips (FRICTION CONNECTION)

WHERE RIVET HEADS EXIST ON TOP OF GIRDER, USE A ⅝ FILLER PLATE, WITH HOLES 1¾ dia, TO MATCH RIVETS LOCATIONS.

JACKING BEAM WITH END PLATE "B4"

— READ THIS DRAWING IN CONJUNCTION WITH MS-04A

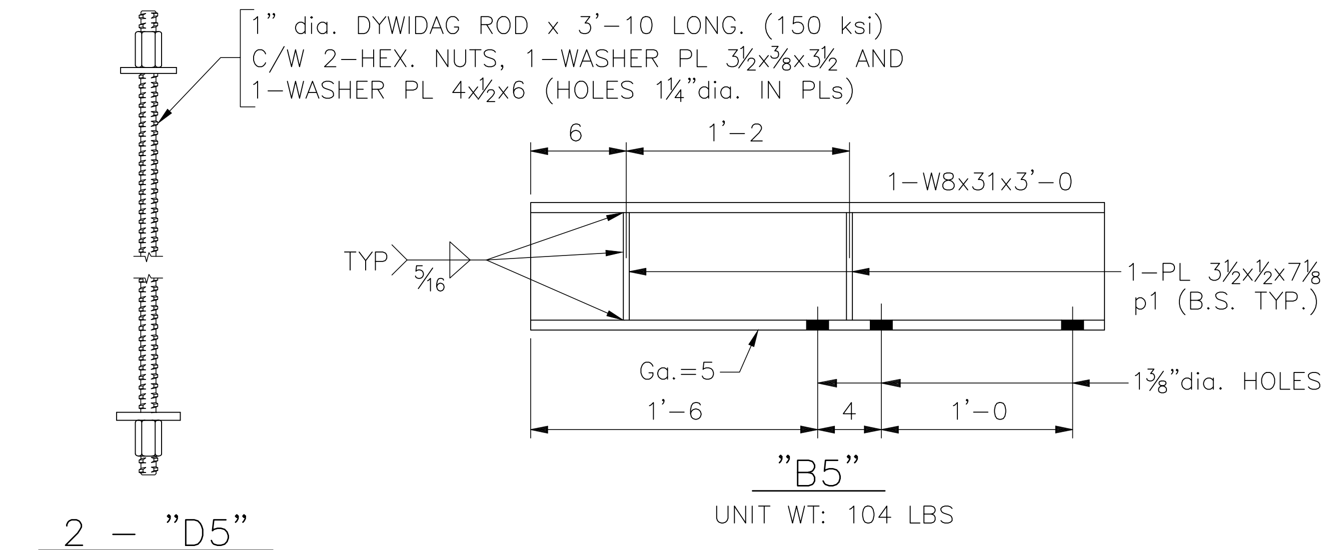
Drawn Dessin	JE	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

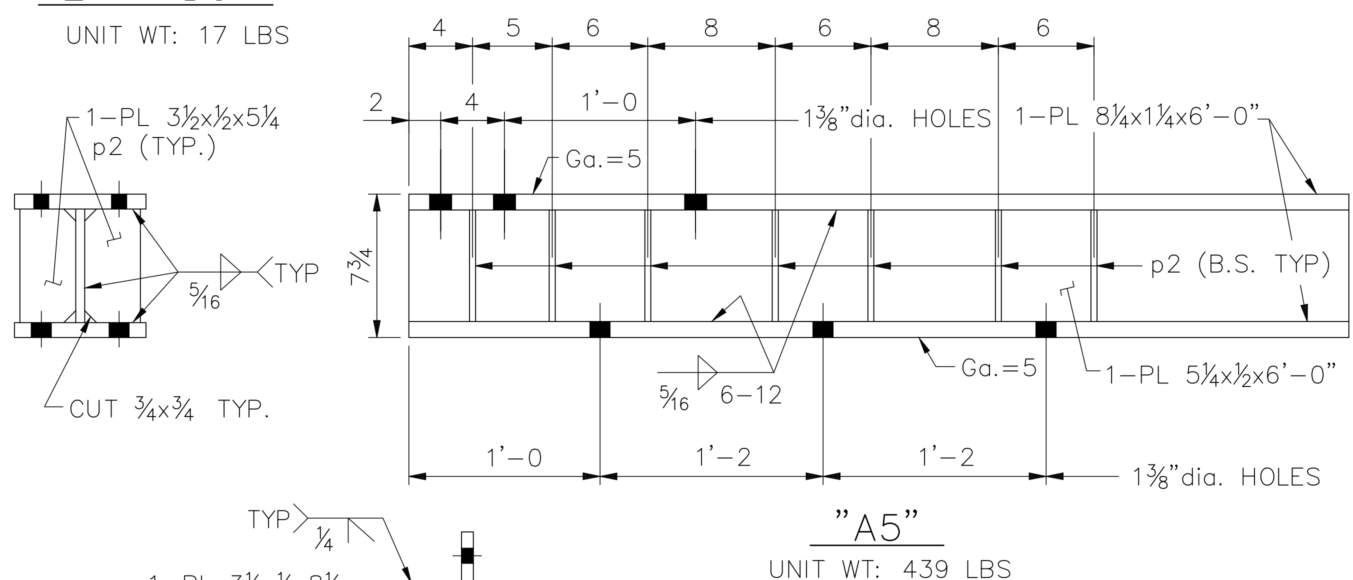
INSTALLATION OF STANDARD JACKING BEAM
(ON TOP FLANGE)

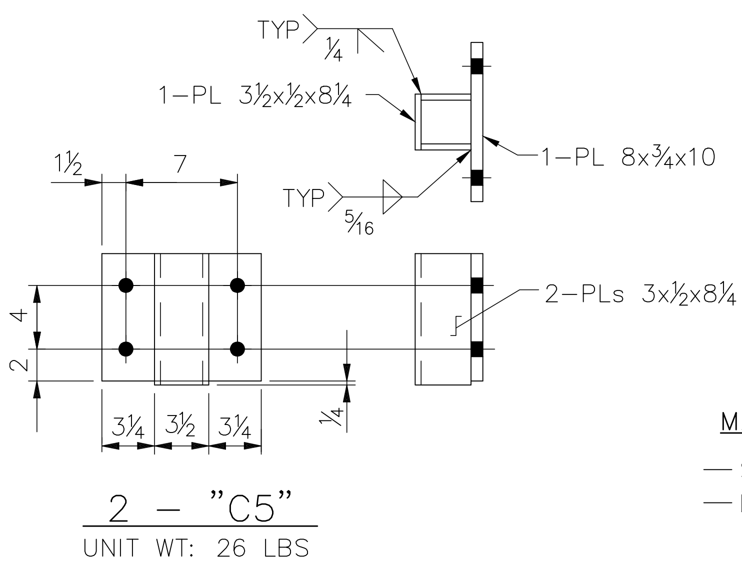
METROLINX

Drawing Number Dessin Numero: MS-04B

Rev.







NOTES:

— FOR ERECTION DETAILS SEE DWG Nos. MS–05B AND MS–05C

MATERIAL:

— STEEL : CAN3–G40.21, 300W
— DYWIDAG RODS: CSA–G279, 150 ksi

Drawn / Dessin	JE	Checked / Verification	S.Kh.
Scale / Echelle	1"=1'–0	Date	March 2019

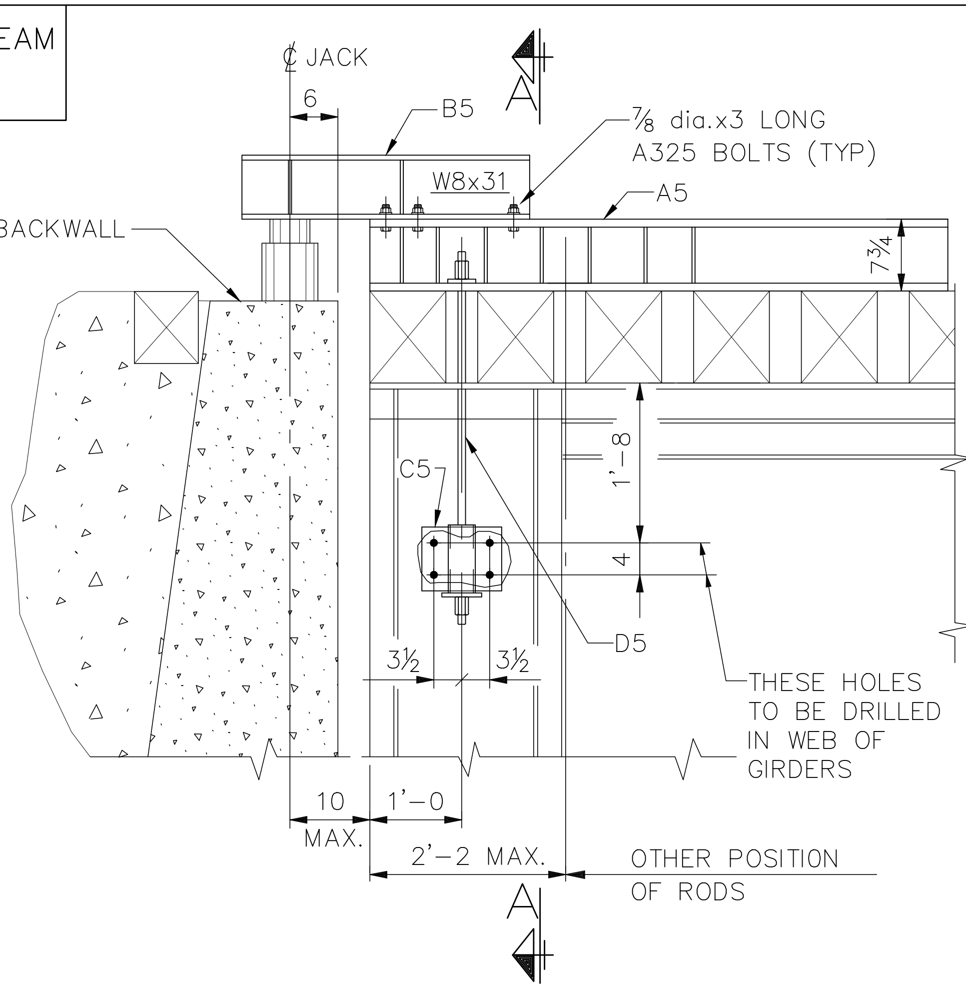
FABRICATION OF
STANDARD JACKING BEAM
(INSTALLED ON TIES)

METROLINX

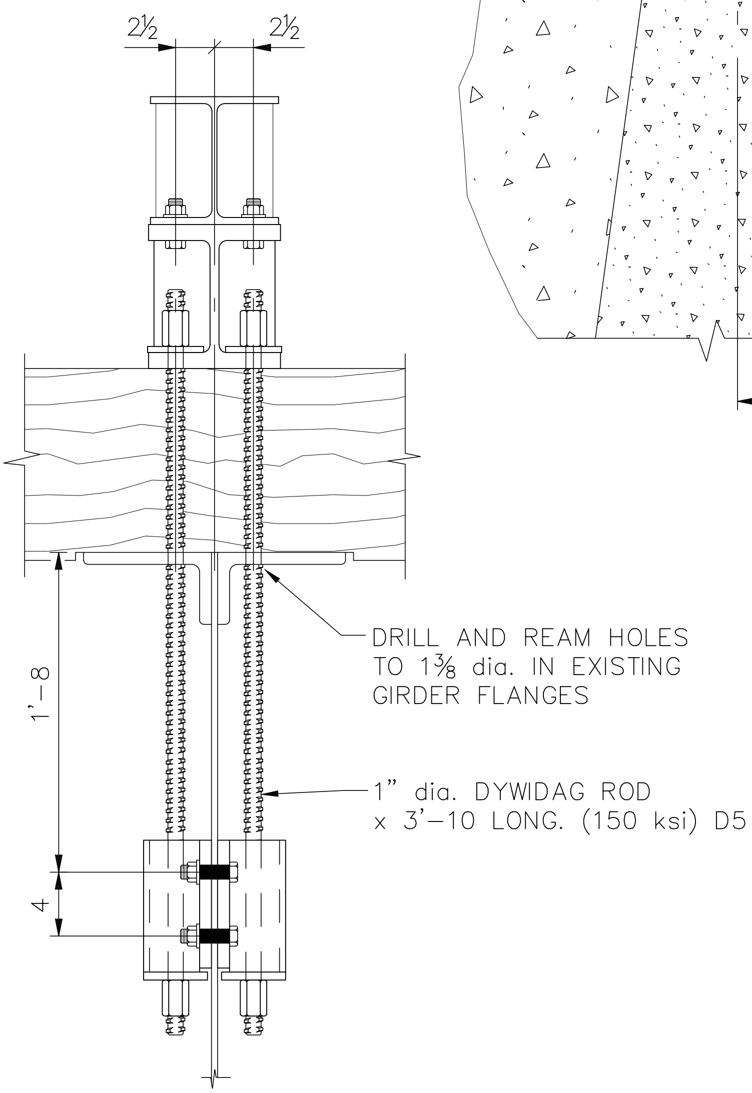
Drawing Number / Dessin Numero: MS–05A

Rev.

CAPACITY OF JACKING BEAM
12 TONS



ELEVATION
INSTALLATION OF JACK
ON BACKWALL
SCALE: ½"=1'–0



SECTION "A–A"
SCALE: 1"=1'–0

NOTES:

- FOR FABRICATION DETAILS SEE DWG No. MS–05A
- MAXIMUM JACKING LOAD: 24000 LBS
- RODS TO BE TORQUED TO A COUPLE OF 650 LBS.FT OR TO A TENSION OF 20,000 LBS
- ALL HOLES TO BE FILLED WITH HIGH STRENGTH BOLTS AFTER REMOVAL OF BRACKET

Drawn / Dessin	JE	Checked / Verification	S.Kh.
Scale / Echelle	AS NOTED	Date	March 2019

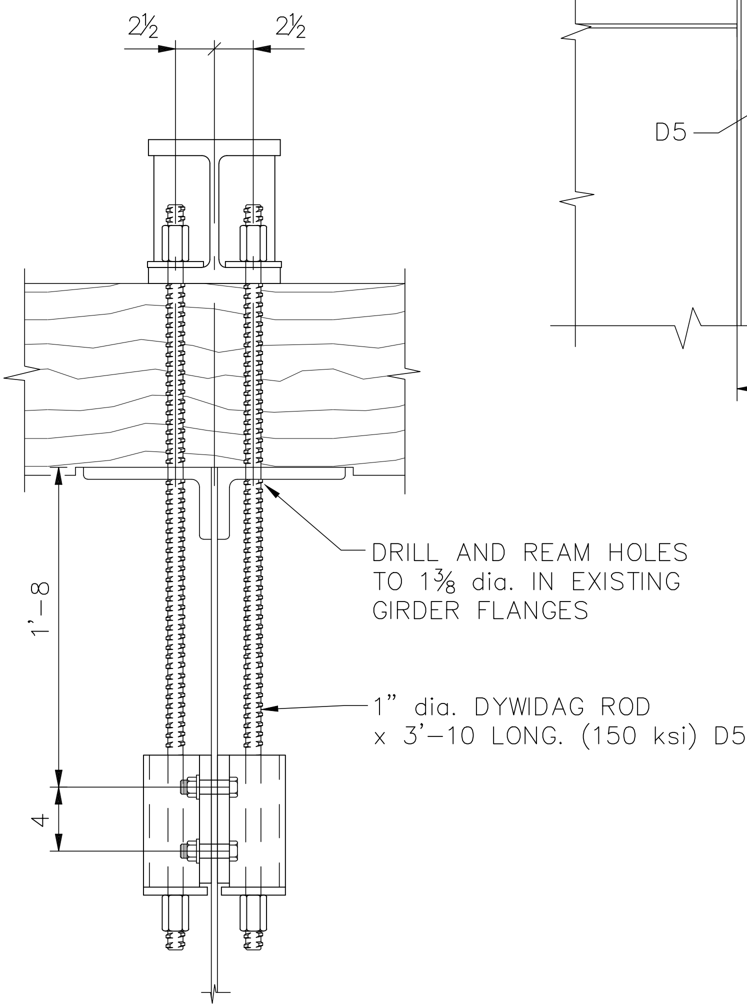
INSTALLATION OF
STANDARD JACKING BEAM
(ON TIES) SHEET 1 OF 2

METROLINX

Drawing Number / Dessin Numero: MS–05B

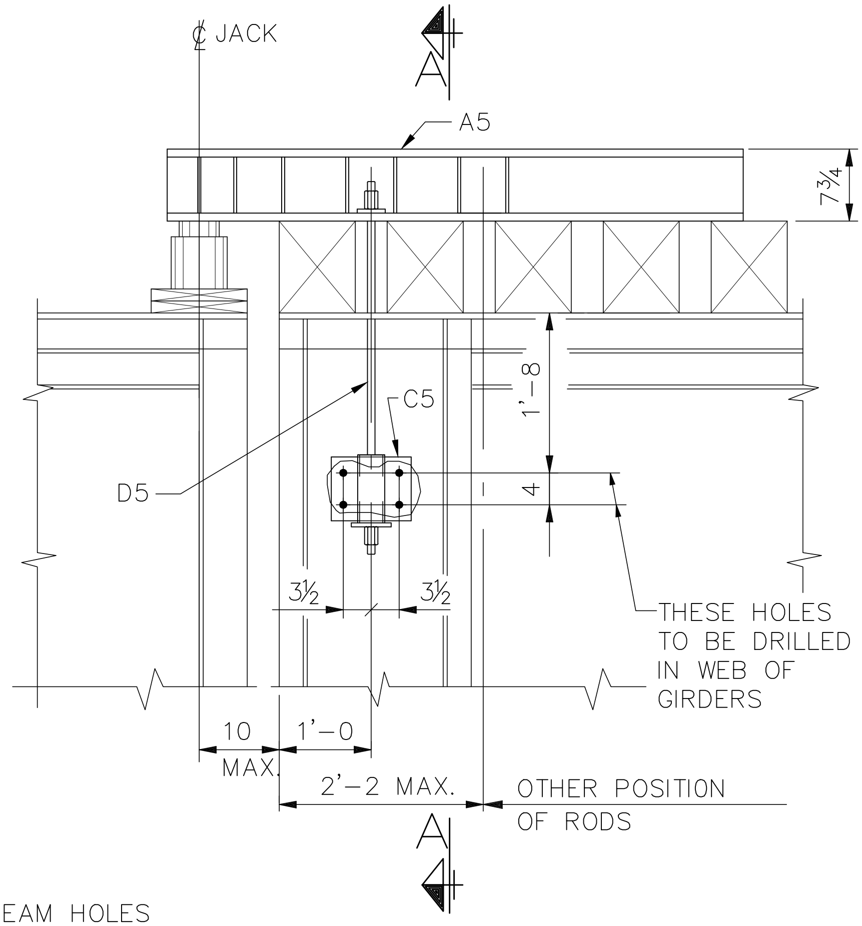
Rev.

CAPACITY OF JACKING BEAM
12 TONS



SECTION "A–A"

SCALE: 1"=1'–0



ELEVATION

INSTALLATION OF JACK
ON ADJACENT SPAN

SCALE: ½"=1'–0

NOTES:

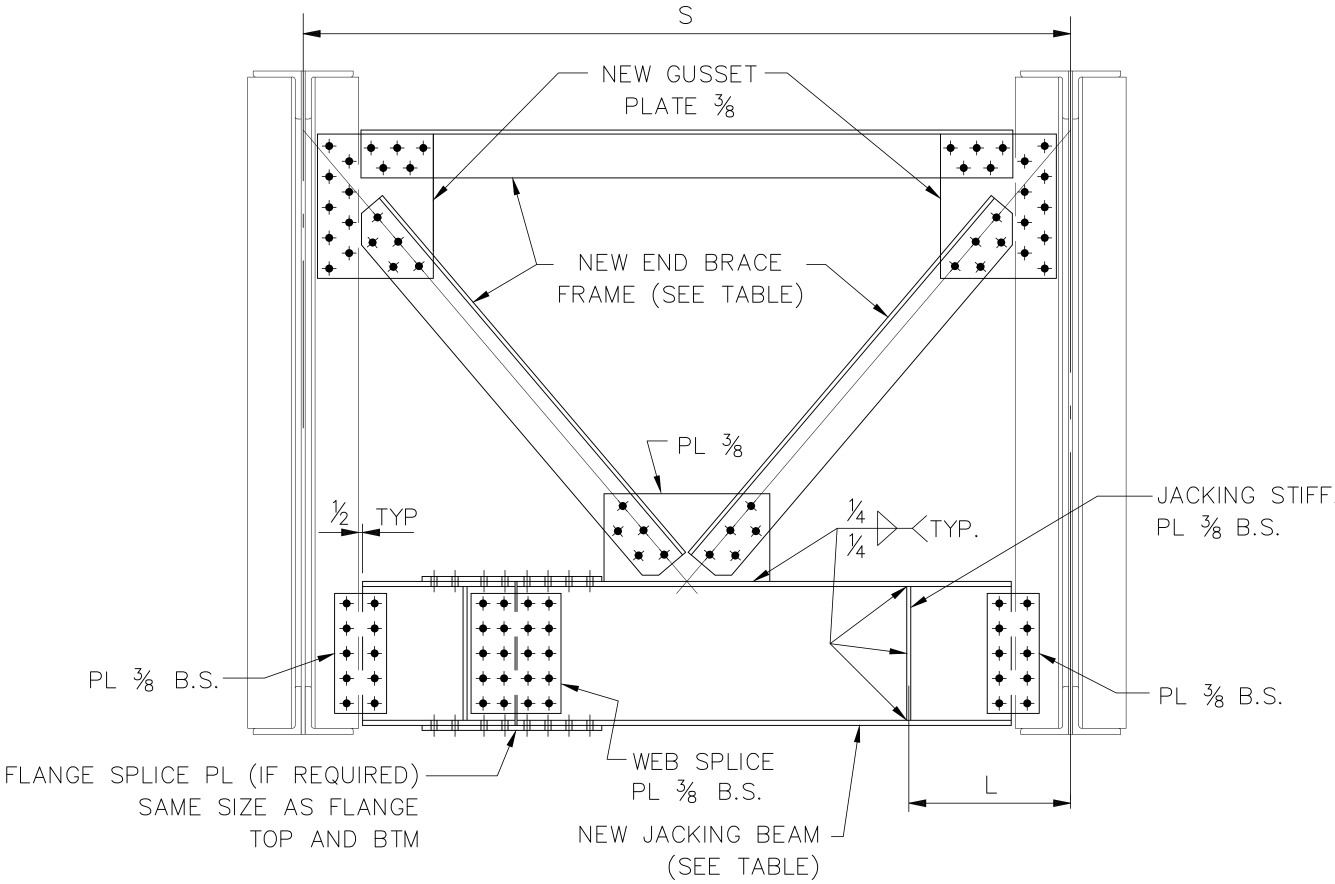
- FOR FABRICATION DETAILS SEE DWG No. MS–05A
- MAXIMUM JACKING LOAD: 24000 LBS
- RODS TO BE TORQUED TO A COUPLE OF 650 LBS.FT OR TO A TENSION OF 20,000 LBS
- ALL HOLES TO BE FILLED WITH HIGH STRENGTH BOLTS AFTER REMOVAL OF BRACKET

Drawn / Dessin	JE	Checked / Verification	S.Kh.
Scale / Echelle	AS NOTED	Date	March 2019

INSTALLATION OF
STANDARD JACKING BEAM
(ON TIES) SHEET 2 OF 2

METROLINX

Drawing Number / Dessin Numero	Rev.
MS–05C	



JACKING BEAM	NUMBER OF BOLTS			S max.	L max.	JACKING LOAD (KIPS)
	END	WEB SPLICE *	FLANGE SPLICE *			
W12x45	3	6	6	8'–0	2'–6	40
W18x50	5	10	8	9'–0	2'–6	60
W24x68	7	14	10	10'–0	2'–6	100

* NUMBER OF BOLTS TO BE USED ON EACH SIDE OF SPLICE

FOR SQUARE SPANS ONLY

SPAN LENGTH	TOP STRUT		DIAGONALS		GUSSET PLs	
	SIZE	No. OF BOLTS	SIZE	No. OF BOLTS	THICK.	No. OF BOLTS
60'–0	L6x4x⅜	5	2–Ls 6x4x⅜	5	⅜	9
80'–0	L6x4x⅜	6	2–Ls 6x4x⅜	6	⅜	10
100'–0	L6x4x½	7	2–Ls 6x4x½	7	⅜	11

NOTE: NUMBER OF BOLTS FOR WEB SPLICE, CONNECTIONS AT ENDS OF BEAM AND DIAGONALS ARE CALCULATED AS DOUBLE SHEAR.

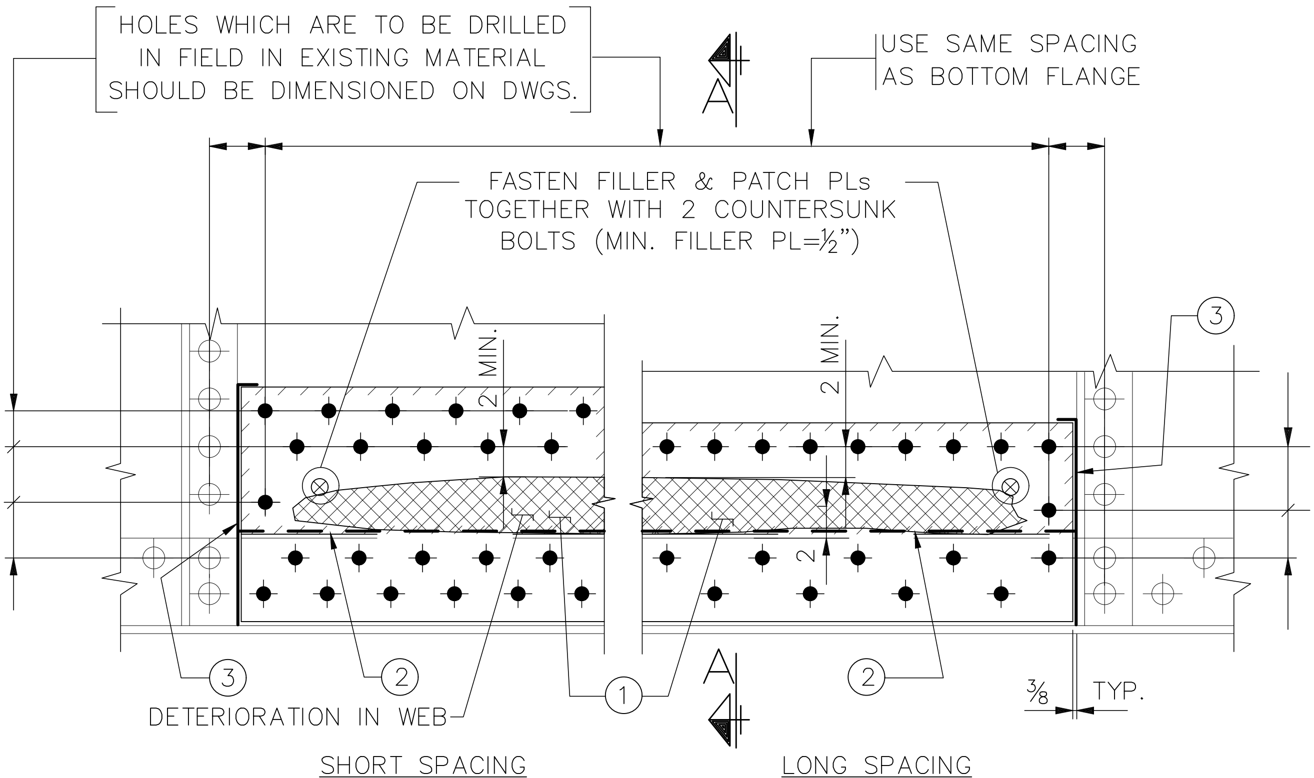
Drawn Dessin	JE	Checked Verification	S.Kh.
Scale Echelle	½"=1'–0	Date	March 2019

STANDARD JACKING BEAM AT END BRACE FRAME OF DPG SPANS

METROLINX

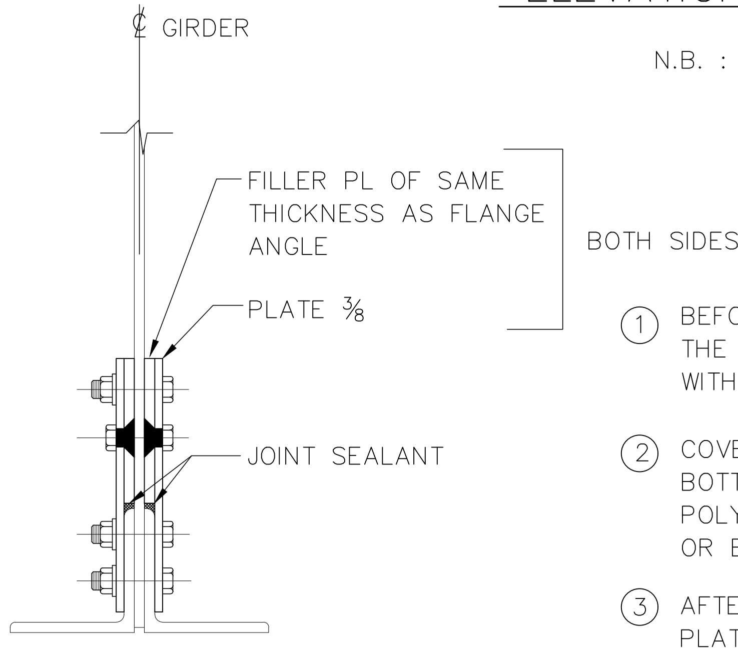
Drawing Number Dessin Numero: MS–06

Rev.



ELEVATION

N.B. : IF FILLER PLATE IS THINNER THAN ½"
USE WIRE TO TIE FILLER AND PATCH PLATES.

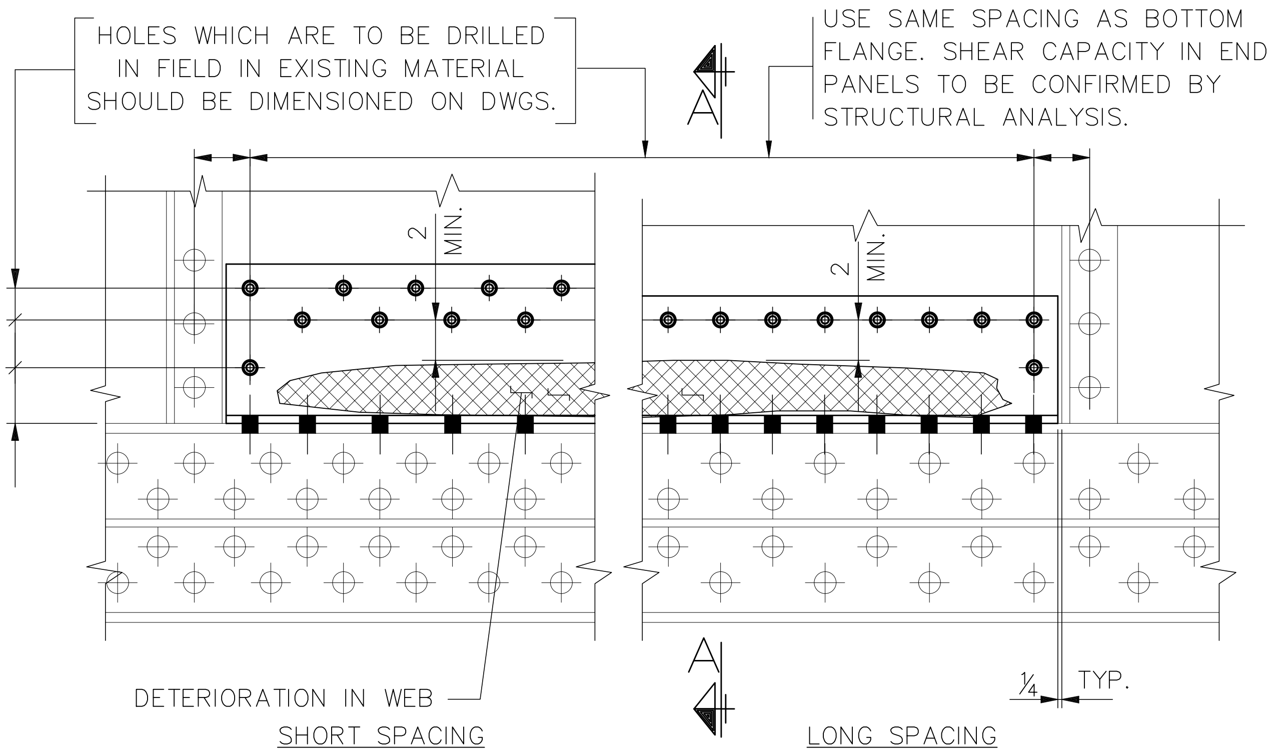


SECTION "A-A"

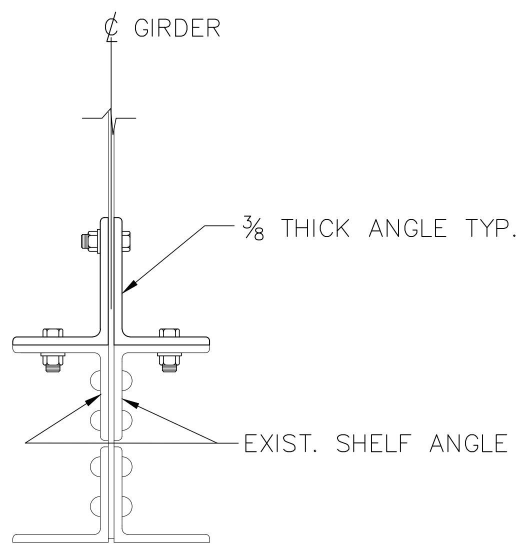
1. BEFORE INSTALLING NEW PLATES, CLEAN THE CORRODED PORTION OF THE WEB WITH POWER TOOLS AND PAINT.
2. COVER WEB AND FILLET OF VERTICAL LEG OF BOTTOM FLANGE WITH A HEAVY COAT OF POLYURETHANE JOINT SEALANT (SIKAFLEX -1A) OR EQUIVALENT.
3. AFTER INSTALLATION, FILL JOINT BETWEEN NEW PLATES AND EXISTING STIFFENER WITH A POLYURETHANE JOINT SEALANT (SIKAFLEX -1A) OR EQUIVALENT.

Drawn / Dessin	JE	Checked / Verification	S.Kh.	**WEB SPLICE ON FULL PANEL**
Scale / Echelle	1"=1'-0	Date	March 2019	FOR WEB CORRODED MAX. 3" IN LENGTH AND UP (FLANGE ANGLE ONLY)

METROLINX	Drawing Number / Dessin Numero: MS-07A	Rev.



ELEVATION



SECTION "A–A"

NOTE

IF CONNECTION IS MADE ON SOUND STEEL, NO PAINTING OR SEALANT IS REQUIRED WHEN USING THIS DETAIL

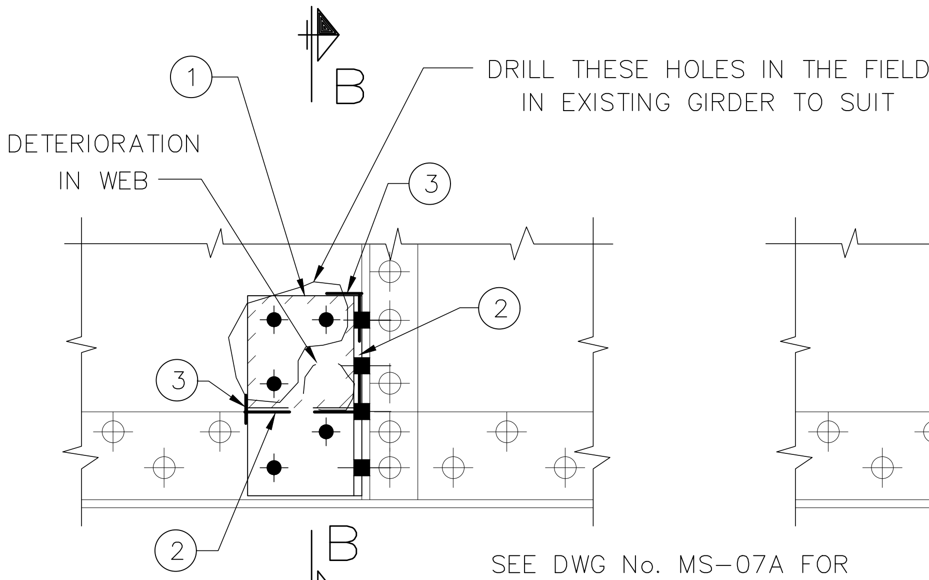
Drawn Dessin	JE	Checked Verification	S.Kh.
Scale Echelle	1"=1'–0	Date	March 2019

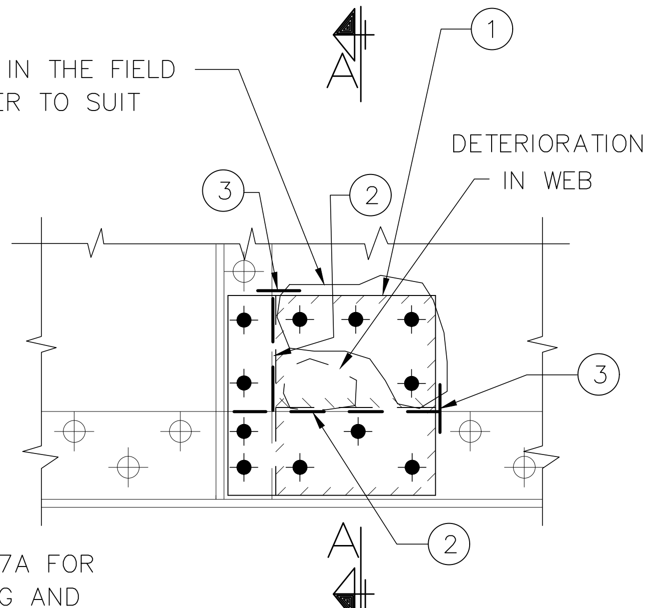
WEB SPLICE ON FULL PANEL

FOR WEB CORRODED MAX. 3" IN LENGTH AND UP (WITH SHELF ANGLES)

METROLINX

Drawing Number Dessin Numero	Rev.
MS–07B	





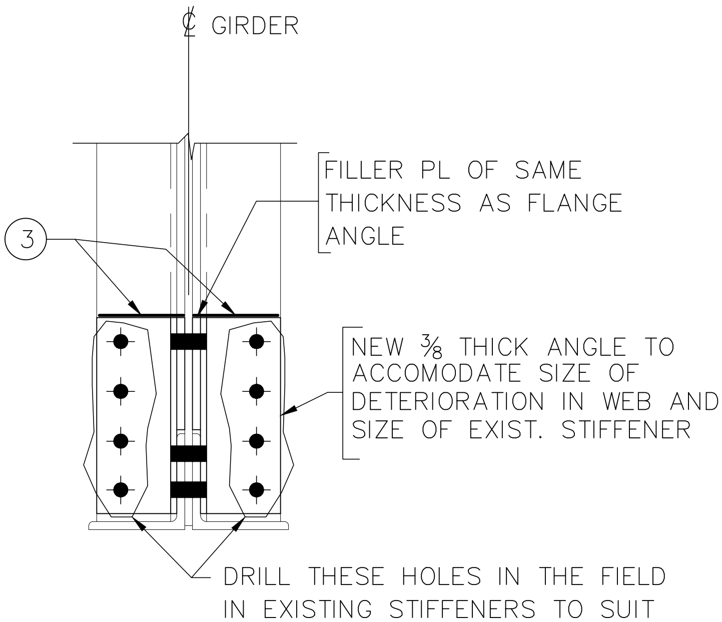
SEE DWG No. MS-07A FOR
NOTES ON PAINTING AND
JOINT SEALING

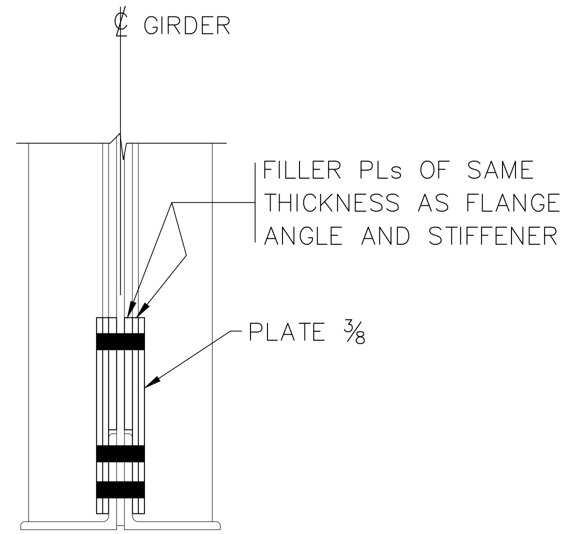
ELEVATION 2

HOLE BEHIND STIFF.

ELEVATION 1

HOLE IN FRONT OF STIFF.





SECTION "A-A"

SECTION "B-B"

Drawn Dessin	JE	Checked Verification	S.Kh.
Scale Echelle	1"=1'-0	Date	March 2019

WEB PATCH

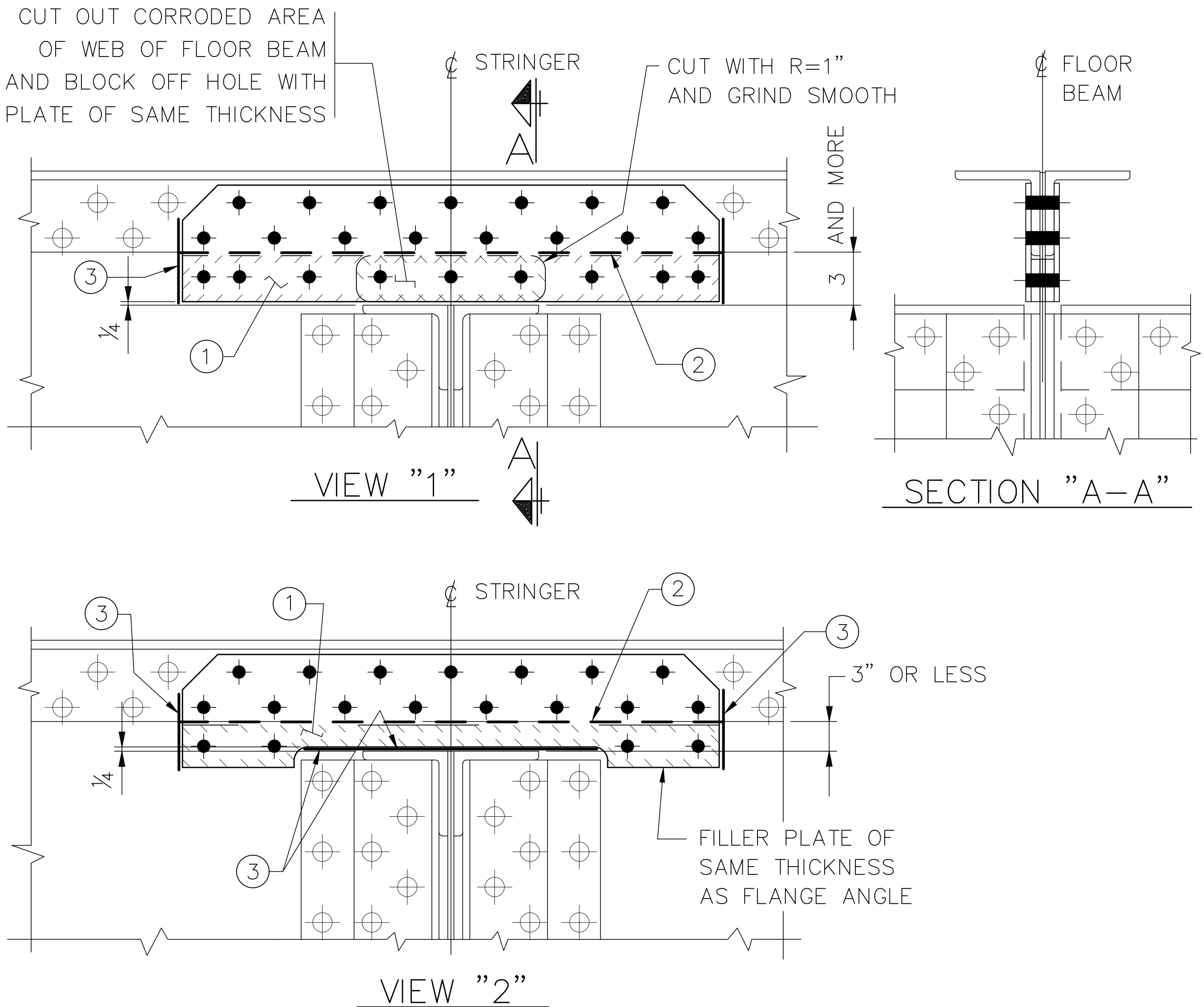
(FOR HOLES SMALLER THAN 4" dia)

METROLINX

Drawing Number
Dessin Numero

MS-07C

Rev.



1. BEFORE INSTALLING NEW PLATES, CLEAN WEB UNDER SPLICE PLATE WITH POWER TOOLS AND PAINT;

2. COVER WEB AND FILLET AT BOTTOM OF VERTICAL LEG OF TOP FLANGE WITH A HEAVY COAT OF POLYURETHANE JOINT SEALANT (SIKAFLEX 1A) OR EQUIVALENT.

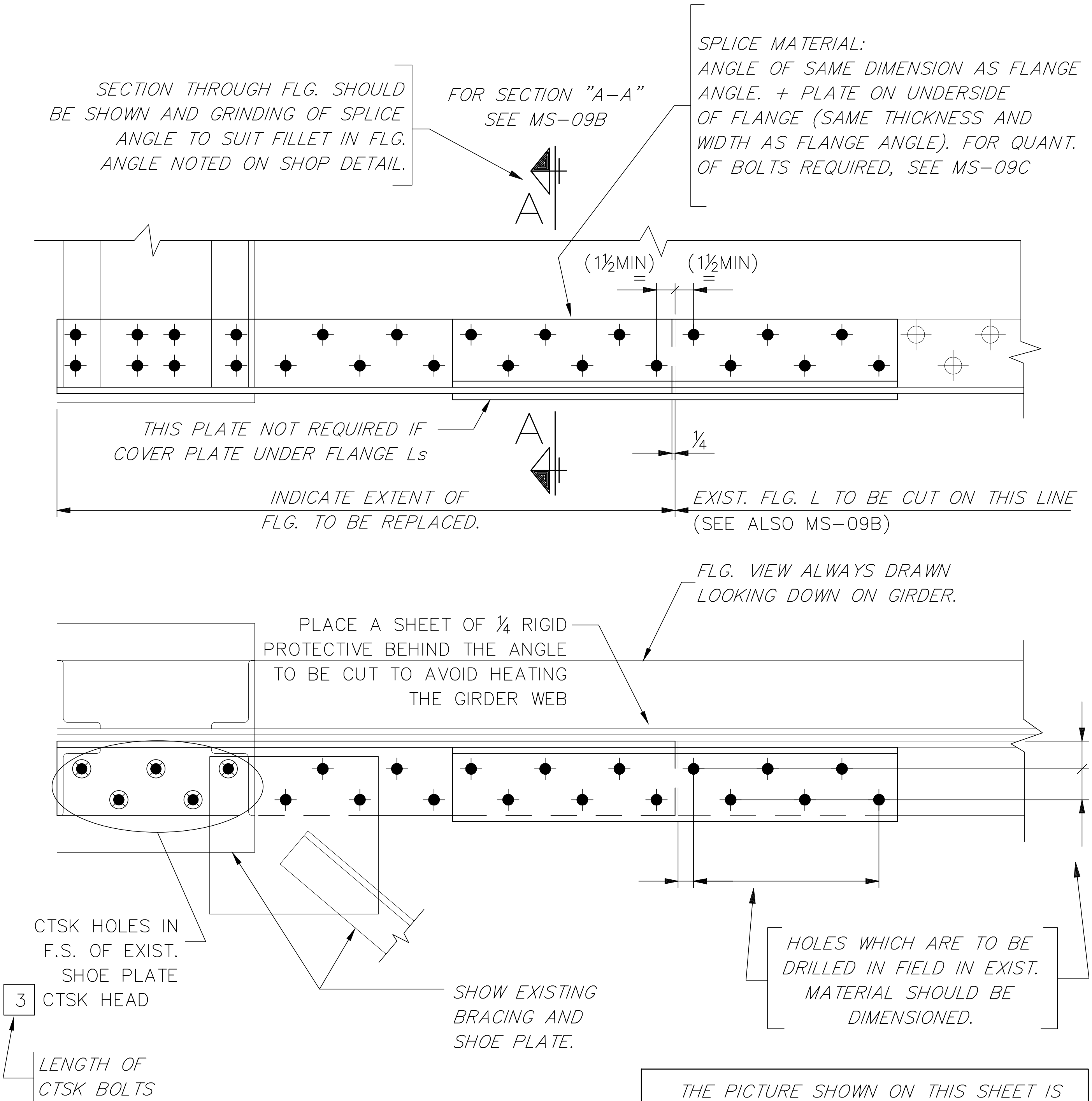
3. AFTER INSTALLATION, APPLY ON THE SIDES OF THE PLATES AND ON THE BOTTOM (VIEW "2" ONLY) A POLYURETHANE JOINT SEALANT (SIKAFLEX 1A) OR EQUIVALENT.

Drawn / Dessin	JE	Checked / Verification	S.Kh.
Scale / Echelle	1"=1'-0	Date	March 2019

FLOOR BEAM WEB PATCH

METROLINX

Drawing Number / Dessin Numero	Rev.
MS-08	



NEW ANGLE USED TO REPLACE EXIST. FLG. ANGLE SHOULD BE THE SAME SIZE AS EXISTING ANGLE SECTION.

THE PICTURE SHOWN ON THIS SHEET IS A FIELD DETAIL. NEW MATERIAL FOR SPLICE WHICH IS FABRICATED IN SHOP SHOULD BE DETAILED SEPARATELY FOR SHOP ONLY.

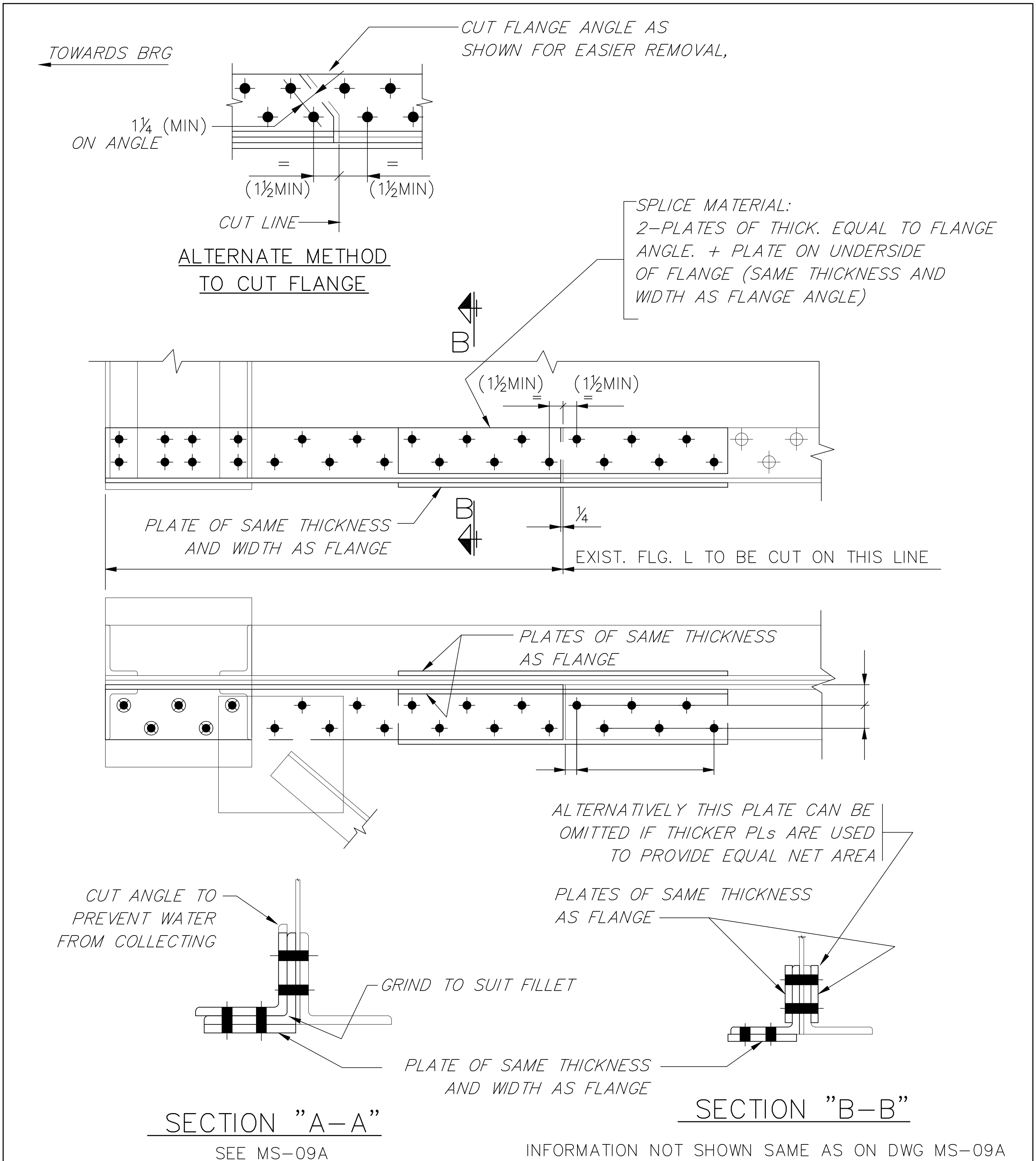
COMMENTS SHOWN IN ITALIC

Drawn Dessin	JE	Checked Verification	S.Kh.
Scale Echelle	1"=1'–0	Date	March 2019

STANDARD GIRDER BOTTOM FLANGE FIELD SPLICE USING ANGLES

METROLINX

Drawing Number Dessin Numero	Rev.
MS–09A	



Drawn Dessin	JE	Checked Verification	S.Kh.
Scale Echelle	1"=1'-0	Date	March 2019

STANDARD GIRDER BOTTOM FLANGE FIELD SPLICE USING PLATES

METROLINX

Drawing Number Dessin Numero: MS-09B

Rev.

SIZE OF FLANGE ANGLE	NUMBER OF BOLTS REQUIRED
L6x3½x⅜	6
L6x3½x½	8
L6x4x⅜	7
L6x4x½	9
L6x6x½	10
L6x6x⅝	13
L6x6x¾	15
L8x6x½	12
L8x6x⅝	15
L8x6x¾	18
L8x8x⅝	17
L8x8x¾	20

— GROSS AREA OF ANGLES WAS USED FOR CALCULATIONS

— BOLTS INDICATED ARE REQUIRED ON EACH SIDE OF THE CUT

— NUMBER OF BOLTS IS BASED ON ⅞"dia. BOLTS IN FRICTION (CAPACITY OF 10.22 kips).

— THIS TABLE IS APPLICABLE FOR STEEL WITH A YIELD POINT OF 30 KSI

NOTE:

SEE DWGS MS-09A AND MS-09B FOR FIELD DETAIL OF SPLICES

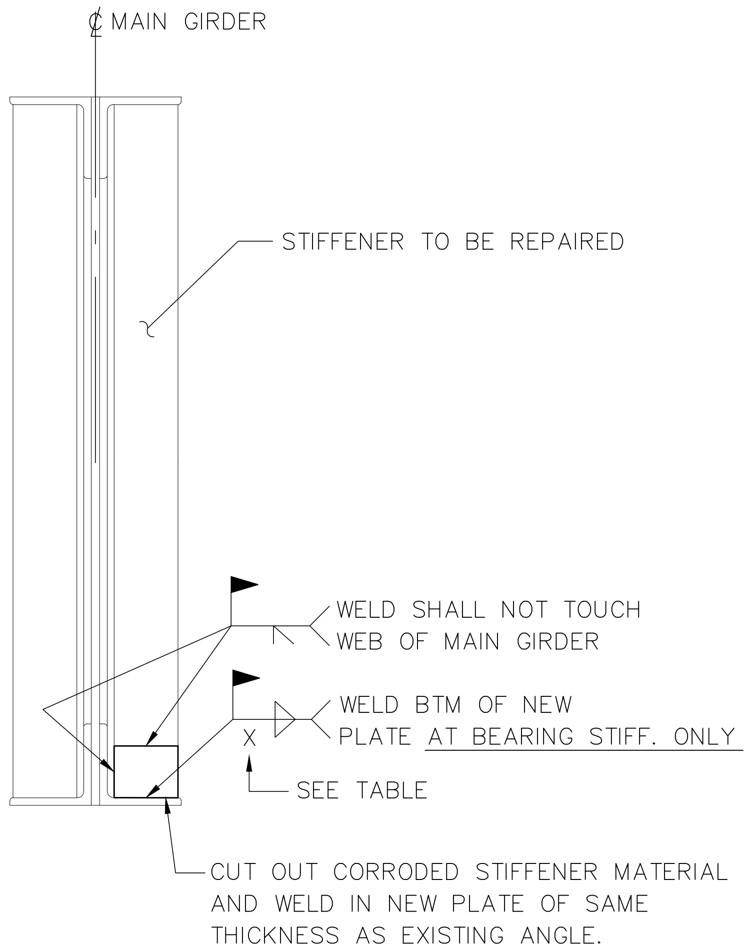
Drawn / Dessin	JE	Checked / Verification	S.Kh.
Scale / Echelle		Date	March 2019

NUMBER OF BOLTS FOR GIRDER BOTTOM FLANGE FIELD SPLICE

METROLINX

Drawing Number / Dessin Numero: MS-09C

Rev.



THICKNESS OF THICKER PART JOINED	MINIMUM FILLET SIZE
UP TO AND INCL. ½"	3/16"
FROM ½" TO ¾" INCL.	¼"
OVER ¾"	5/16"

— CORRODED STIFFENERS SHALL BE REPAIRED WHEN CORROSION OCCURS ON OUTSTANDING LEG ONLY. WHEN CORROSION OCCURS ON BOTH LEGS, THE STIFFENER SHOULD BE REPLACED

— WHEN CORRODED STIFFENER IS 4'–0 OR SHORTER, IT IS USUALLY PREFERABLE TO REPLACE THE STIFFENER THAN TO REPAIR IT.

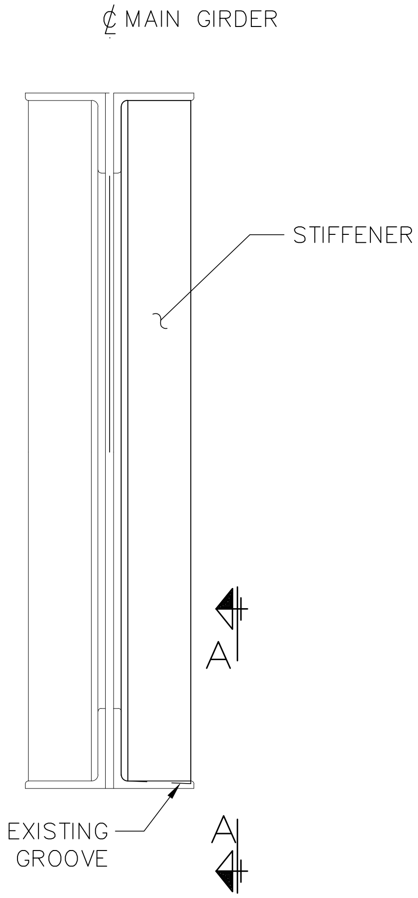
Drawn / Dessin	JE	Checked / Verification	S.Kh.
Scale / Echelle	NONE	Date	March 2019

FIELD REPAIR OF CORRODED BEARING STIFFENERS

METROLINX

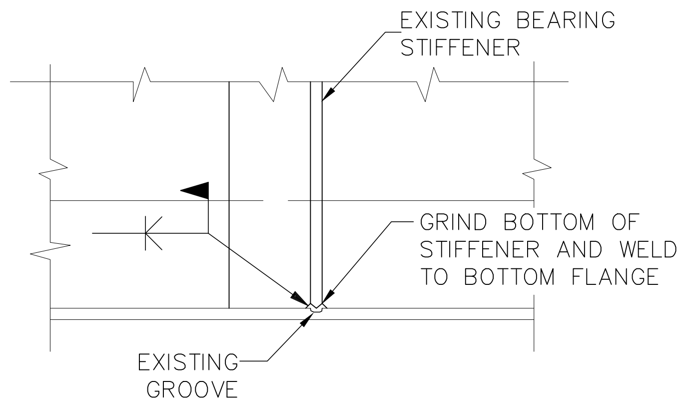
Drawing Number / Dessin Numero: MS–10A

Rev.



TYPICAL SECTION

SCALE: 1"=1'-0



VIEW "A-A"

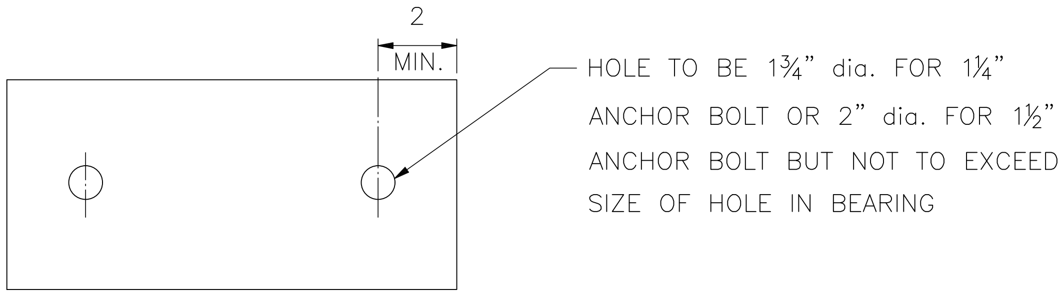
SCALE: 1½"=1'-0

Drawn Dessin	JE	Checked Verification	S.Kh.
Scale Echelle	AS NOTED	Date	March 2019

FIELD REPAIR OF
ENCAVED BEARING STIFFENERS

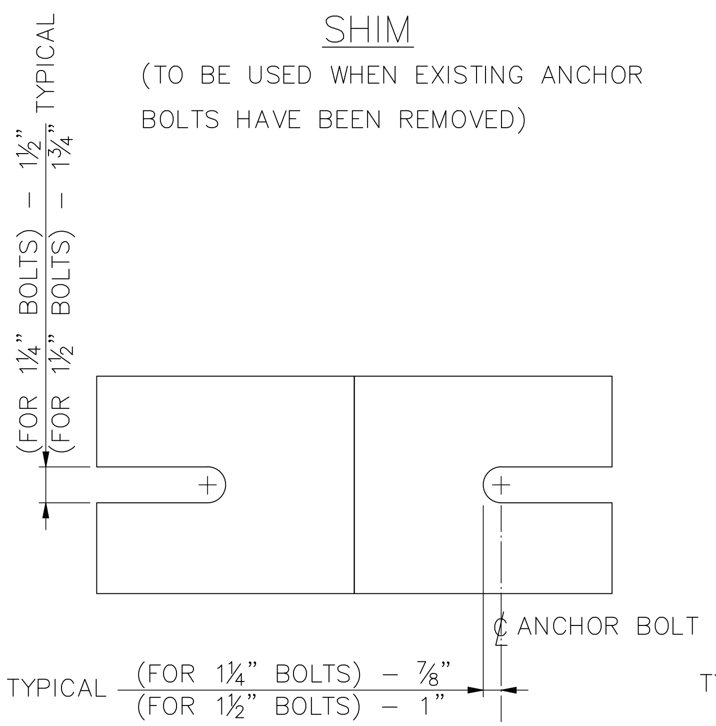
METROLINX

Drawing Number Dessin Numero	Rev.
MS-10B	

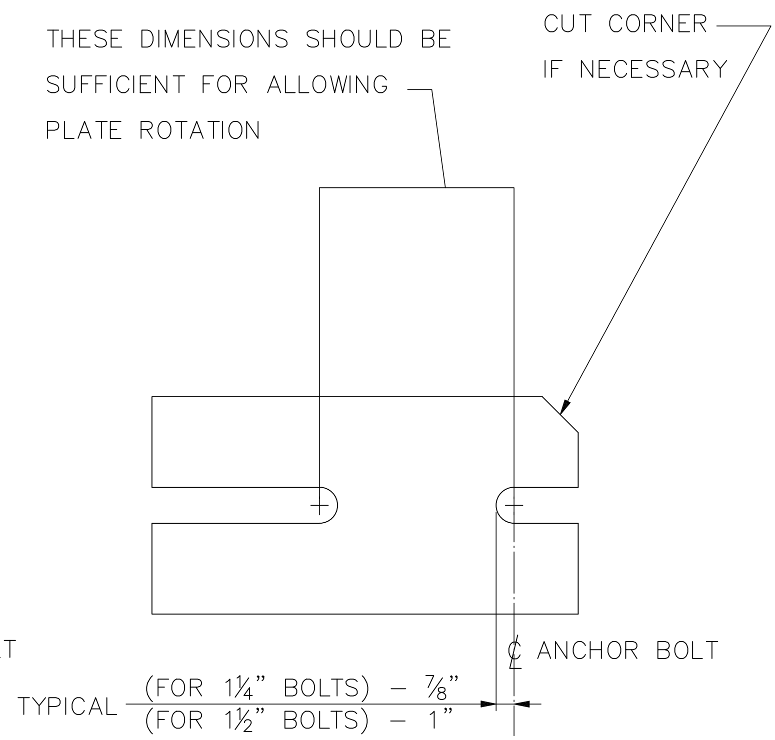


SHIM

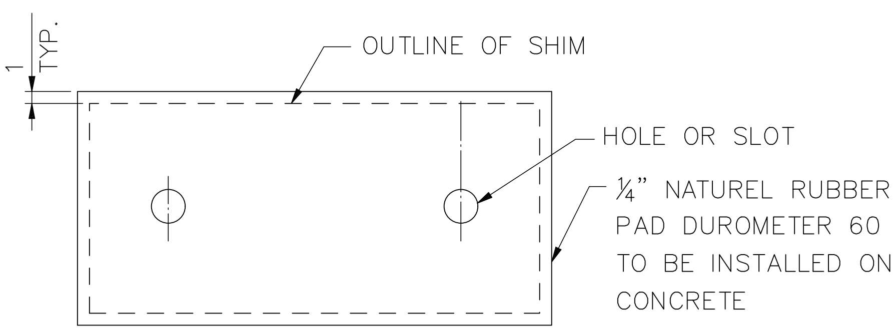
(TO BE USED WHEN EXISTING ANCHOR BOLTS HAVE BEEN REMOVED)



SPLIT SHIM



SLOTTED SHIM



RUBBER PAD

NOTE:

- PREFERABLE WEIGHT <= 200 lbs.
- PREFERABLE SIZE = 1"
- MINIMUM SIZE = ⅜"
- USE A RUBBER PAD FOR SHIMS THINNER THAN ⅜"
- VERIFY THE REQUIRED CLEARANCE FOR THE CONCRETE DRILL WHEN LOCATING HOLES FOR NEW ANCHOR BOLTS.
- ALL UNITS ON THIS DRAWING ARE IN IMPERIAL

SPECIFICATIONS:

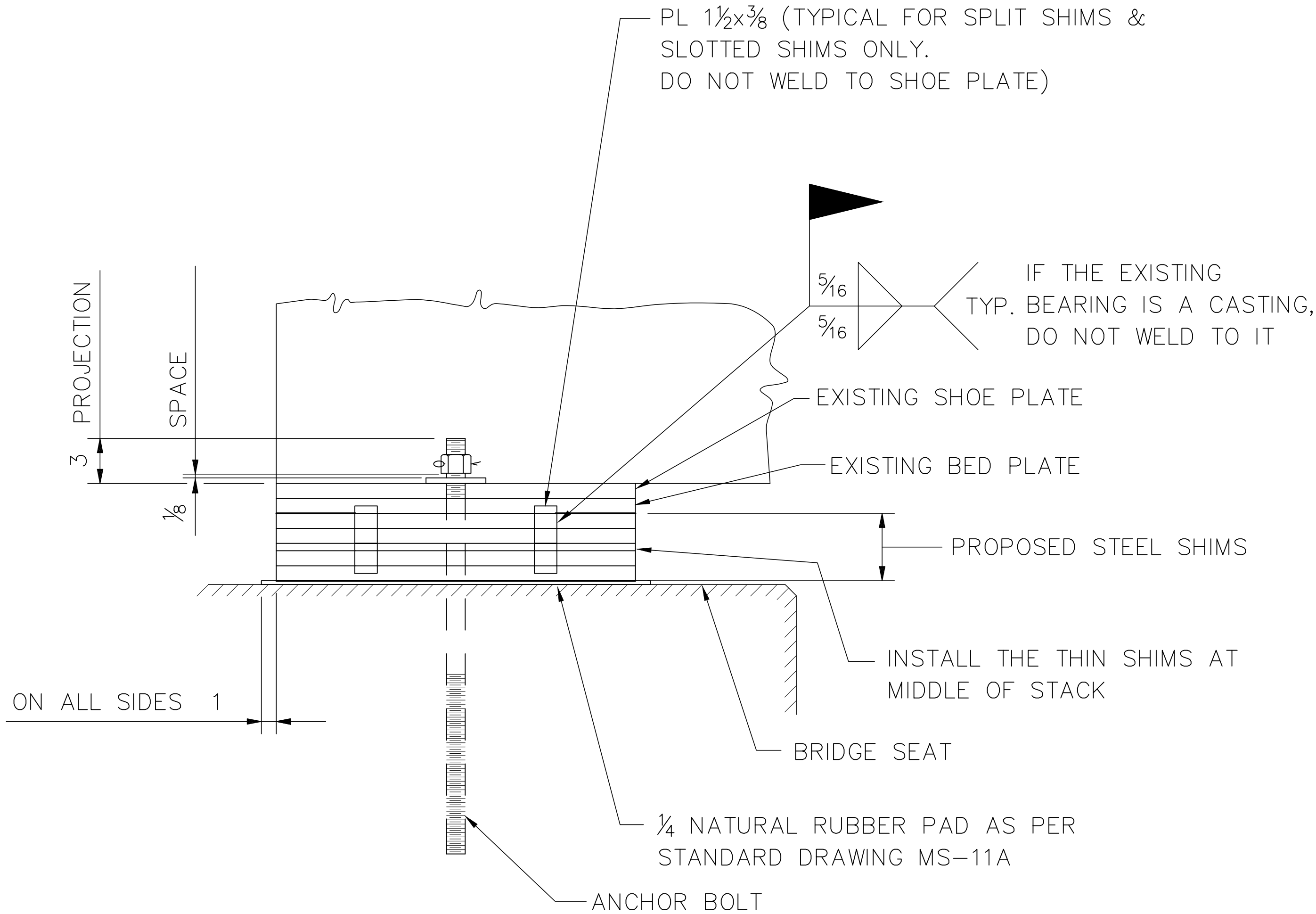
- STEEL: C.S.A. G40.21–M92 GRADE 300W

Drawn Dessin	FM	Checked Vérification	S.Kh.
Scale Échelle	N.T.S.	Date	March 2019

STANDARD SHIM

METROLINX

Drawing Number Dessin Numero	Rev.
MS–11A	



NOTES:

1.– IF REQUIRED, CONCRETE UNDER BEARINGS TO BE REPAIRED AS PER STANDARD DRAWING MC–01.

2.– SHIMS TO BE INSTALLED IN ONE PIECE IF ANCHOR BOLTS ARE TO BE REPLACED. IF THE ANCHOR BOLTS REMAIN, SLOTTED SHIMS OR SPLIT SHIMS CAN BE USED.

3.– SEE STANDARD DRAWING MS–11A FOR DETAILS OF STANDARD SHIMS.

4.– IF EXISTING BED PLATE IS THICKER THAN INDIVIDUAL SHIMS, INSTALL SHIMS BETWEEN SHOE PLATE AND BED PLATE.

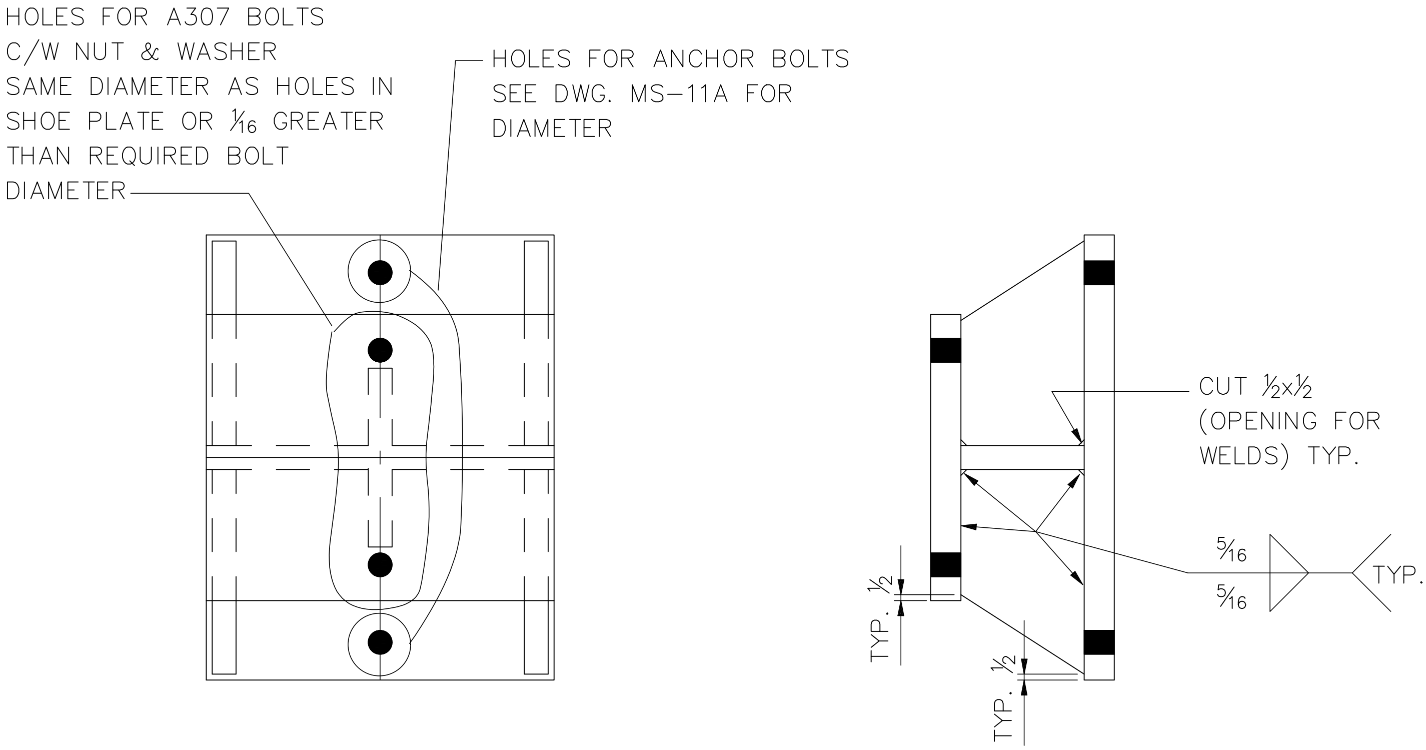
Drawn / Dessin	JE	Checked / Verification	S.Kh.
Scale / Echelle	NONE	Date	March 2019

STANDARD FIELD DETAILS FOR INSTALLATION OF SHIMS

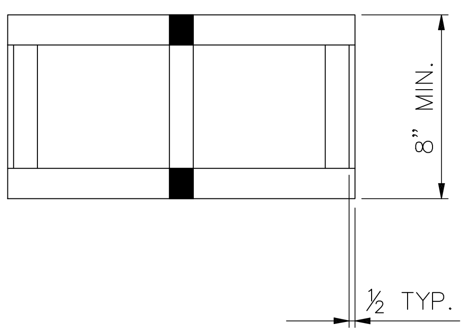
METROLINX

Drawing Number / Dessin Numero: MS–11B

Rev.



P L A N



THICKNESS OF PLATES:
TOP & BTM PL'S. : 1¼"
WEB & STIFFENER PL's. : 1"

NOTE:
- INSTALL NATURAL RUBBER PAD UNDER PEDESTAL AS SHOWN ON DWG. MS–11A AND MS–11B.
- VERIFY THE REQUIRED CLEARANCE FOR THE CONCRETE DRILL WHEN LOCATING ANCHOR BOLT HOLES.
- NUMBER OF ANCHOR BOLTS SHOULD BE SAME AS EXISTING OR DETERMINED BY STRUCTURAL ANALYSIS.
- ANCHOR BOLTS TO BE AS PER METROLINX STD. S3A

SPECIFICATIONS:
- STEEL: C.S.A. G40.21 GRADE 300W
- WELDING: C.S.A. W59

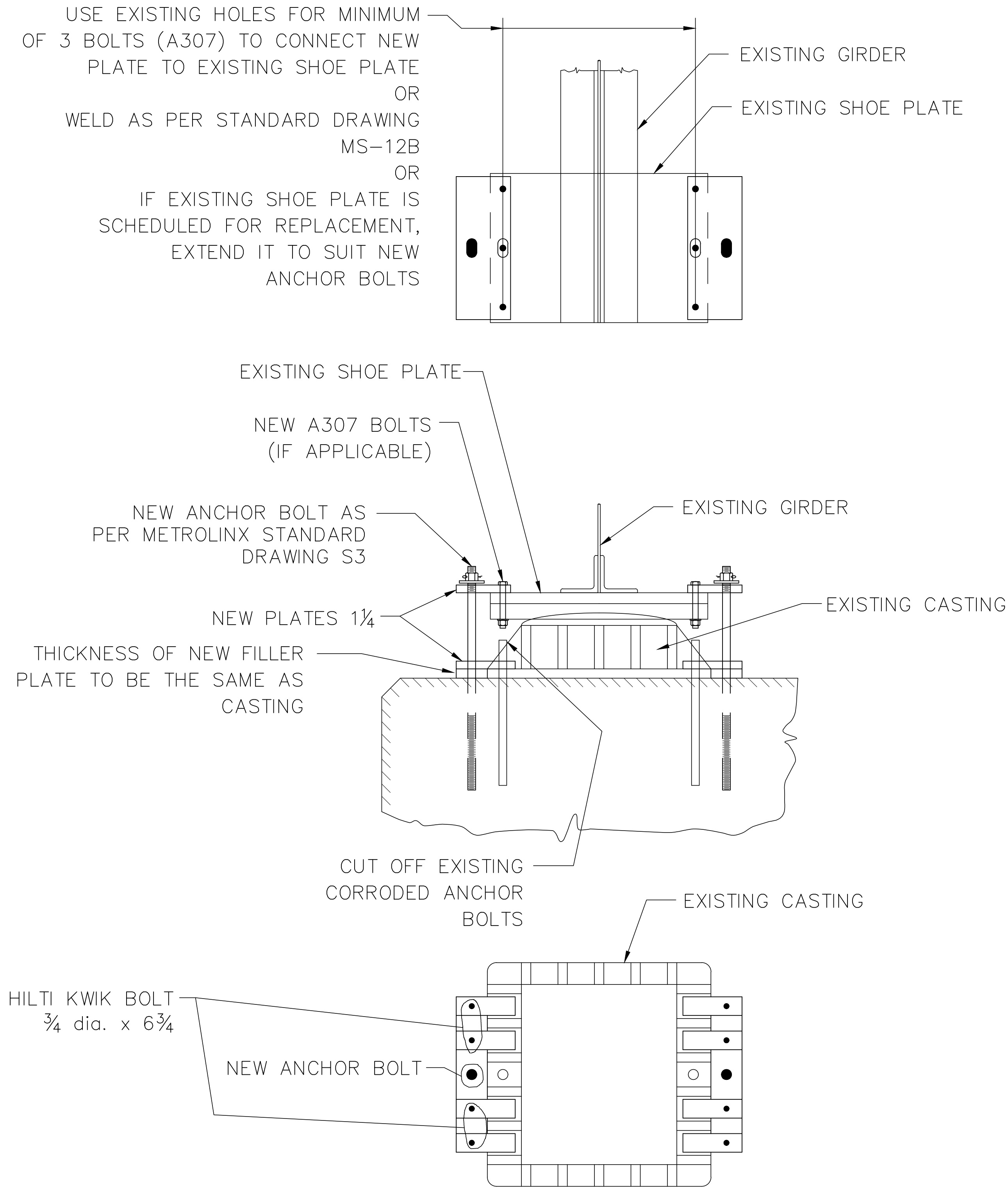
Drawn Dessin	JE	Checked Vérification	S.Kh.
Scale Échelle	N.T.S.	Date	March 2019

STANDARD PEDESTAL

METROLINX

Drawing Number
Dessin Numero
MS–11C

Rev.



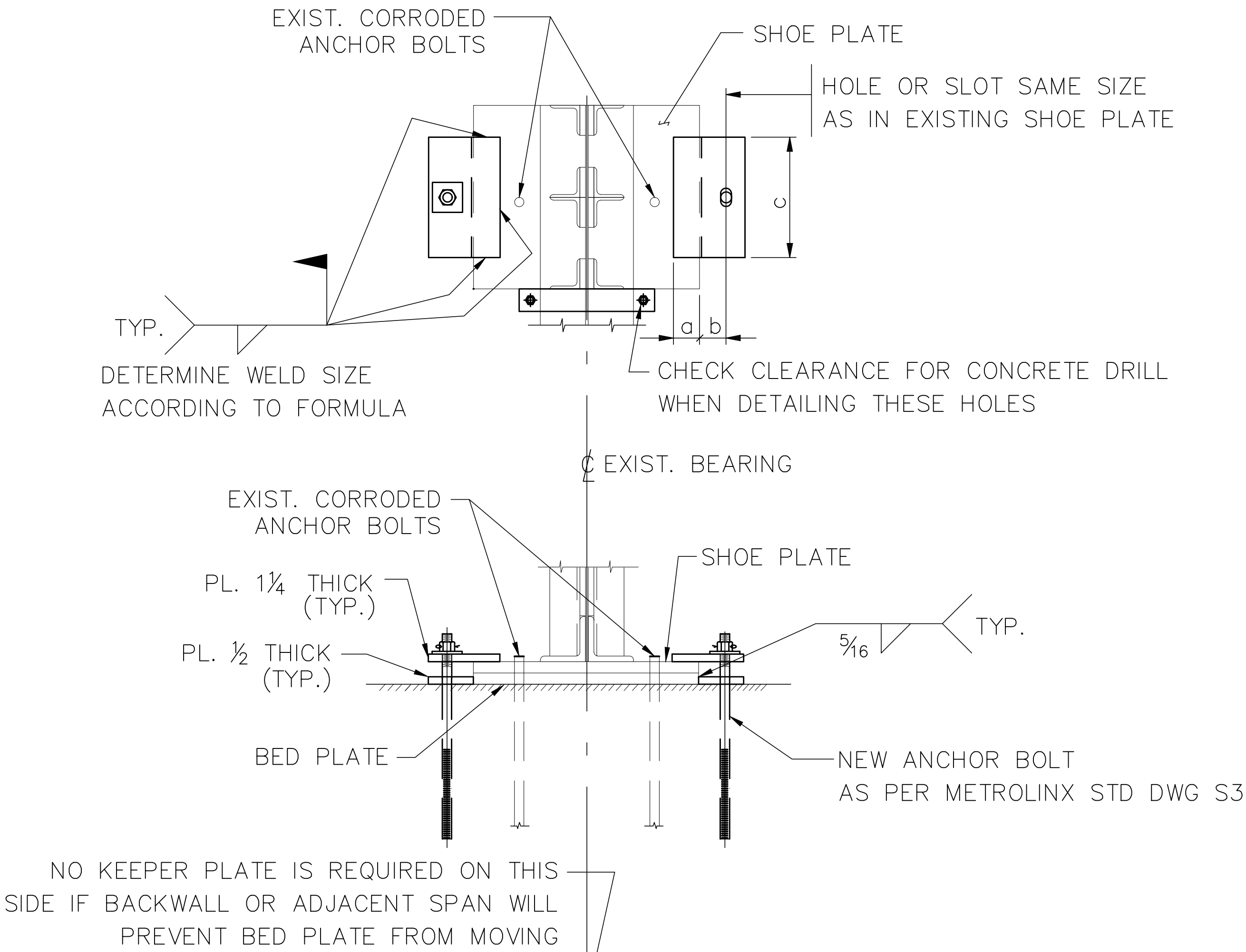
Drawn / Dessin	JE	Checked / Vérification	S.Kh.
Scale / Échelle	NONE	Date	March 2019

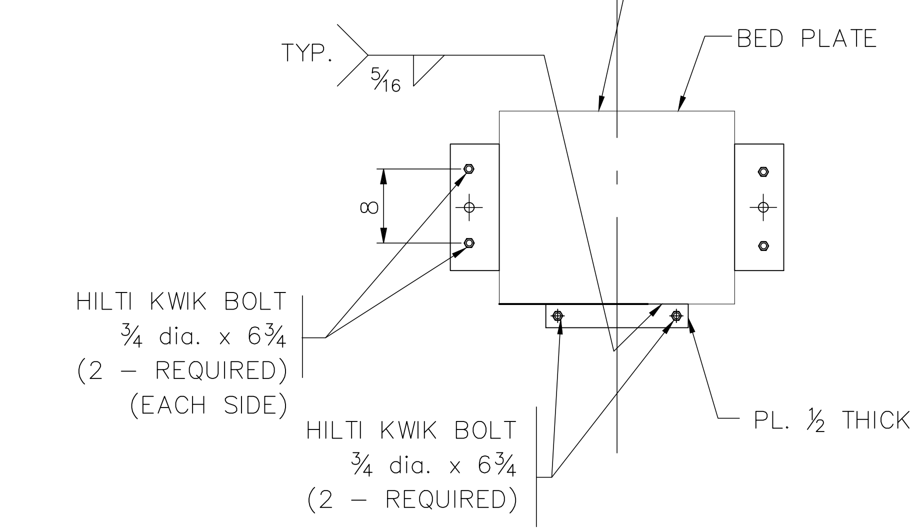
STANDARD FOR REPLACEMENT OF ANCHOR BOLTS ON CAST SPHERICAL BEARINGS

METROLINX

Drawing Number / Dessin Numero: MS-12A

Rev.





ANCHOR BOLT	ALLOWABLE LOAD	
DIAMETER	TENSION	SHEAR
1¼	19.4 Kips	12.3 Kips
1½	28.1 Kips	17.7 Kips

FORMULA TO DETERMINE
SIZE OF FILLET:

– $q > \frac{T \times b}{c \times a}$ $\quad q > \left(\frac{6V \times (a + b)}{c^2} + \frac{V}{c}\right)$

– T = TENSION – V = SHEAR
– q = 4000 lbs/in. FILLET 5/16
– q = 4800 lbs/in. FILLET 3/8
– q = 5600 lbs/in. FILLET 7/16
– q = 6400 lbs/in. FILLET ½
– c = LENGTH OF FILLET WELD

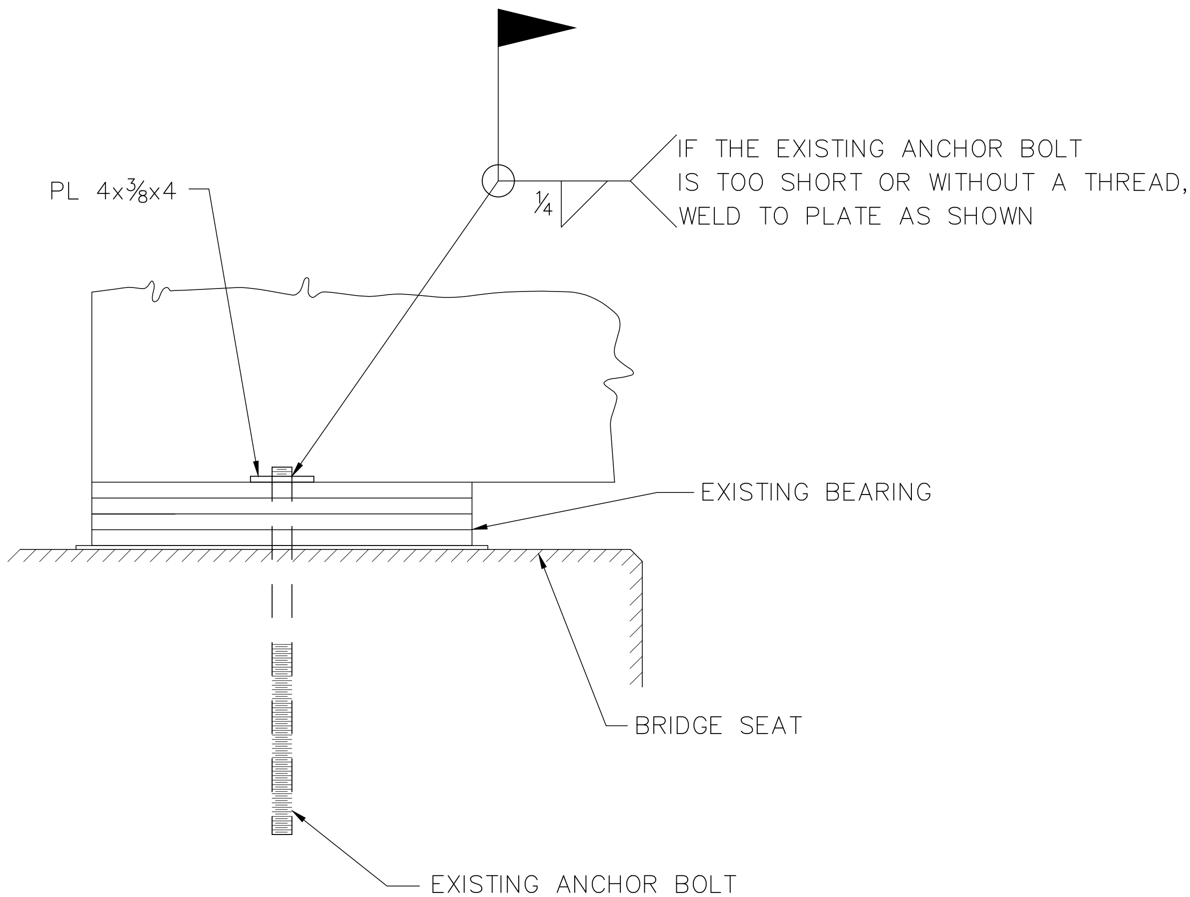
Drawn Dessin	JE	Checked Vérification	S.Kh.
Scale Échelle	N.T.S.	Date	March 2019

STANDARD FOR REPLACEMENT OF ANCHOR BOLTS FOR STEEL PLATE BEARINGS

METROLINX

Drawing Number
Dessin Numero
MS–12B

Rev.



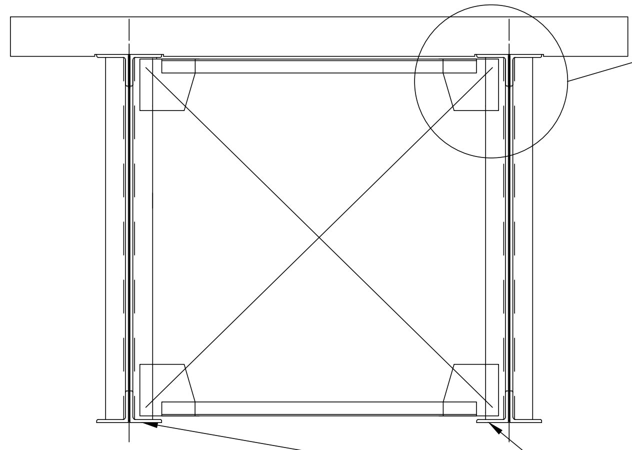
Drawn Dessin	JE	Checked Vérification	S.Kh.
Scale Échelle	NONE	Date	March 2019

DETAIL FOR ANCHOR BOLT WHERE NUT CAN NOT BE INSTALLED

METROLINX

Drawing Number
Dessin Numero
MS–12C

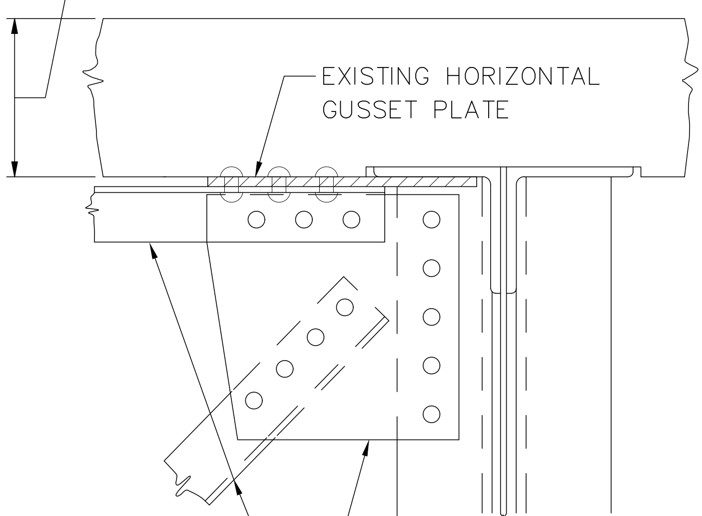
Rev.



WHERE TIES SIT ON GUSSET PLATES:
LOWER HORIZONTAL GUSSET PLATES & HORIZONTAL BRACING BY INSTALLING 1" (MIN.) SHIM PLATES BETWEEN TOP FLANGE & GUSSET PLATES TO ENSURE TIES ARE NOT N CONTACT WITH BOLT HEADS. RETAIN ALL EXISTING BRACING WHERE POSSIBLE & REPLACE TOP VERTICAL GUSSET PLATES TO SUIT LOWERED HORIZONTAL BRACING.

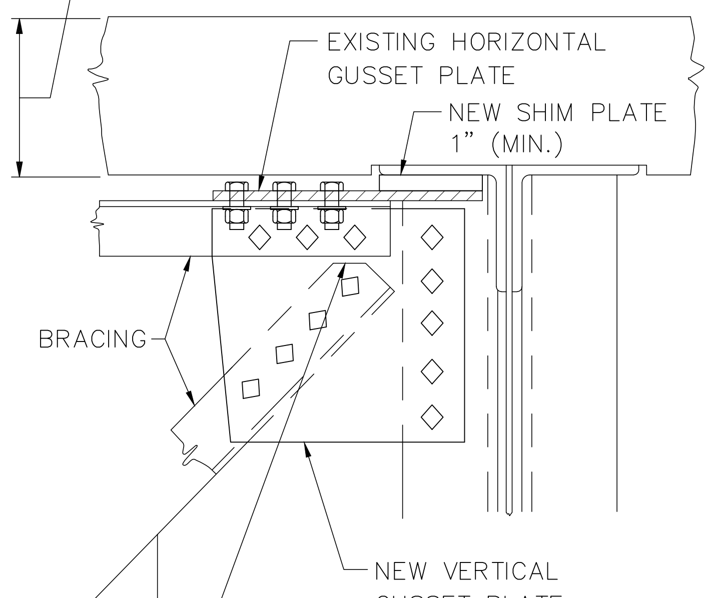
TYPICAL SECTION

NOT TO SCALE



TYPICAL DETAIL (A) (EXISTING)

SHOWING TIE SITTING ON GUSSET PLATE



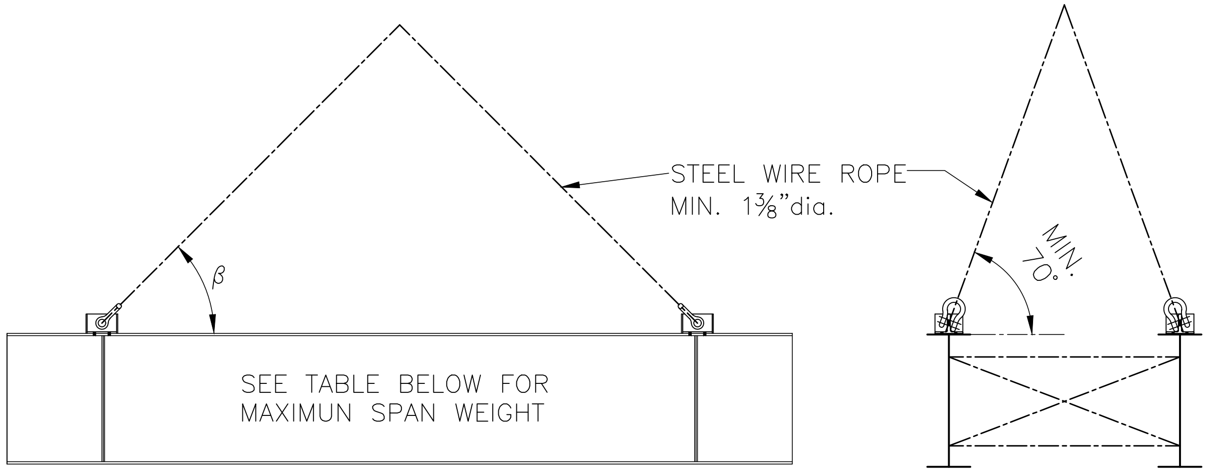
TYPICAL DETAIL (A) (NEW)

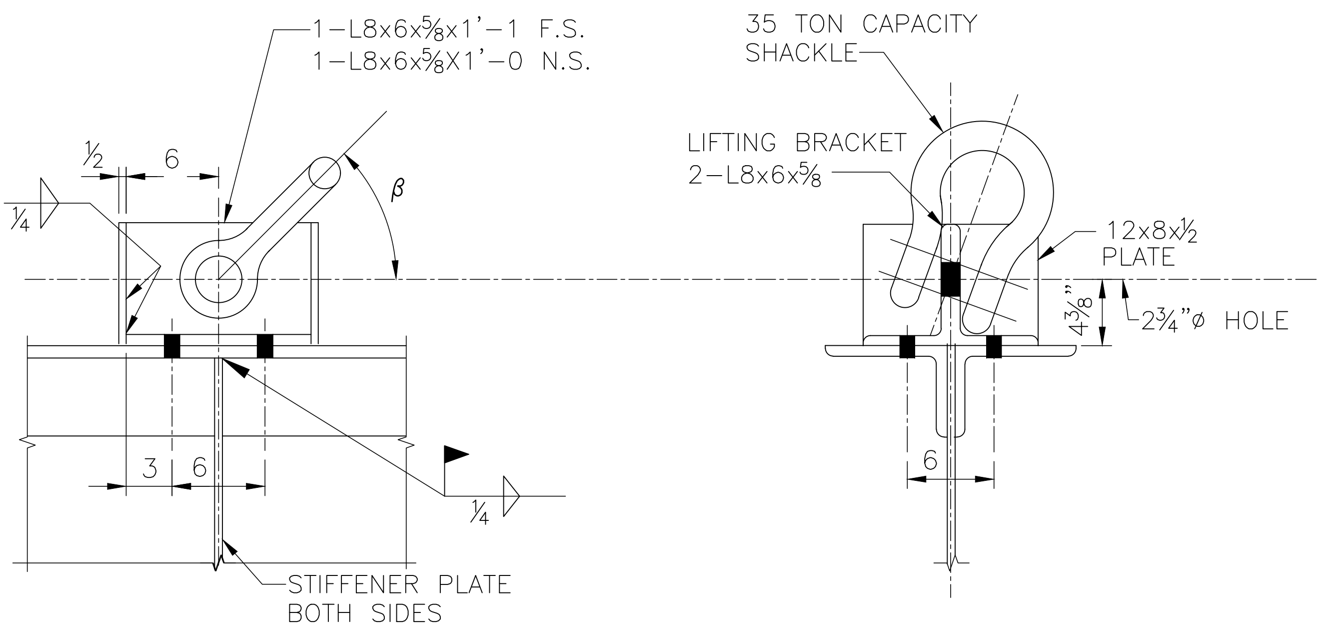
SHOWING NEW POSITION OF GUSSET PLATE

Drawn / Dessin	JE	Checked / Vérification	S.Kh.	STANDARD FOR INSTALLATION OF SHIMS TO LOWER GUSSET PLATES
Scale / Échelle	NONE	Date	March 2019	

METROLINX

Drawing Number / Dessin Numero	Rev.
MS–13	





MAX. WEIGHT OF STEEL SPAN (TON)				
β	MIN. FLANGE THICKNESS			
	⅝	¾	⅞	1
30	7	9	11	13
37.5	8	11	13	16
45	9	12	15	18
52.5	10	13	17	21
60	12	15	19	23

1 TON = 2000 LB

NOTE: FOR SPECIFICATIONS SEE DRAWING No: MS–14B

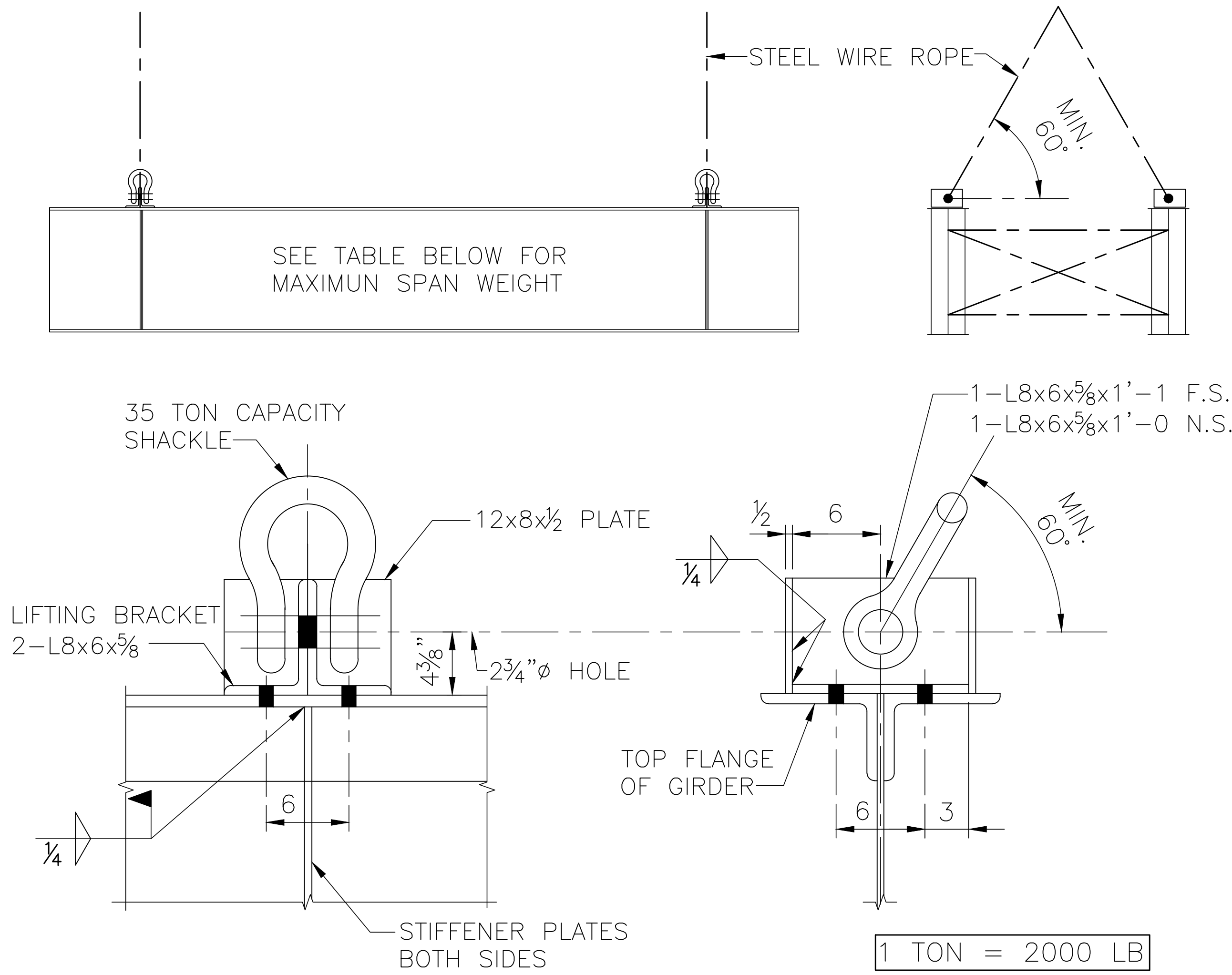
Drawn Dessin	JE	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

LIFTING BRACKETS FOR EXISTING SPANS (1 CRANE LIFT)

METROLINX

Drawing Number
Dessin Numero: MS–14A

Rev.



1 TON = 2000 LB

MAX. WEIGHT OF STEEL SPAN (TON)							
LIFTING BRACKETS	BOLTS DIA.	MIN. FLANGE THICKNESS					STEEL WIRE ROPE MIN. DIA.
		⅝	¾	⅞	1	1⅛	
2–L8x6x⅝	⅞	22	27	34	42	51	1½"

SPECIFICATIONS

STRUCTURAL STEEL: CSA CAN3–G40.21 GRADE 300W.
BOLTS: HIGH STRENGTH ASTM A–325 WITH HEAVY HEXAGONAL NUTS AND HARDENED WASHERS. NUTS SHALL BE TIGHTENED USING TURN OF NUT METHOD.

NOTE

IF CORROSION ON TOP FLANGE EXCEEDS ⅛" OF THICKNESS. THIS STANDARD MUST NOT BE USED WITHOUT THE APPROVAL OF THE STRUCTURAL ENGINEER MAINTENANCE.

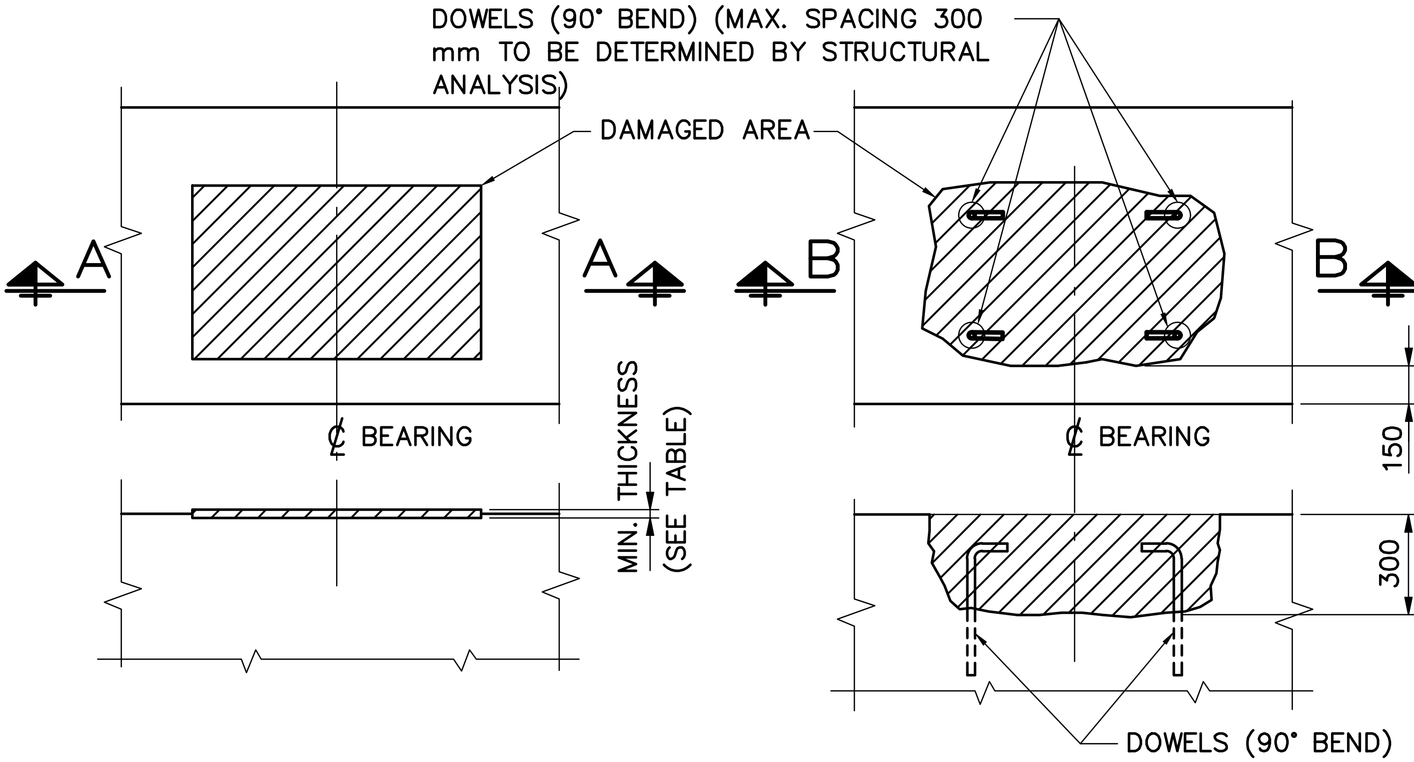
Drawn / Dessin	JE	Checked / Verification	S.Kh.
Scale / Echelle	NONE	Date	March 2019

LIFTING BRACKETS FOR EXISTING SPANS (2 CRANE LIFT)

METROLINX

Drawing Number / Dessin Numero: MS–14B

Rev.



RESURFACING

PROCEDURE

- LIFT SPAN;
- CHIP LOOSE CONCRETE UNDER BEARING;
- ROUGHEN SURFACE TO 6mm;
- INSTALL FORMS AROUND AREA;
- CLEAN CONCRETE SUBSTRATE WITH OIL–FREE COMPRESSED AIR;
- POUR GROUT MIX (SEE TABLE TO ASSIST IN SELECTING THE PROPER PRODUCT);
- LOWER SPAN WHEN GROUT HAS REACHED 1000 psi ;
- RESUME TRAFFIC WHEN GROUT HAS REACHED 2000 psi.

DEEP REPAIR

PROCEDURE

- LIFT SPAN;
- REMOVE BEARING (ONLY IF THE BEARING IS IN THE WAY FOR CHIPPING). BEARINGS CAN BE ATTACHED AND LIFTED WITH THE SPAN;
- CHIP LOOSE CONCRETE UNDER BEARING;
- INSTALL REBAR DOWELS WITH STANDARD 90° BEND;
- CLEAN CONCRETE SUBSTRATE WITH OIL–FREE COMPRESSED AIR;
- POUR GROUT MIXED WITH AGGREGATES (SEE TABLE FOR SELECTING THE PROPER PRODUCT);
- INSTALL BEARING;
- LOWER SPAN WHEN GROUT HAS REACHED 1000 psi ;
- RESUME TRAFFIC WHEN GROUT HAS REACHED 2000 psi.

SIKA® PRONTO 11	SET® 45
MIN. THICKNESS 6mm	MIN. THICKNESS 13mm
MIN. TEMPERATURE: –10°C REGULAR –25°C (WITH SUB ZERO COMPONENT)	MIN. TEMPERATURE: 2°C

SEE MANUFACTURER'S RECOMMANDATIONS FOR UTILISATION

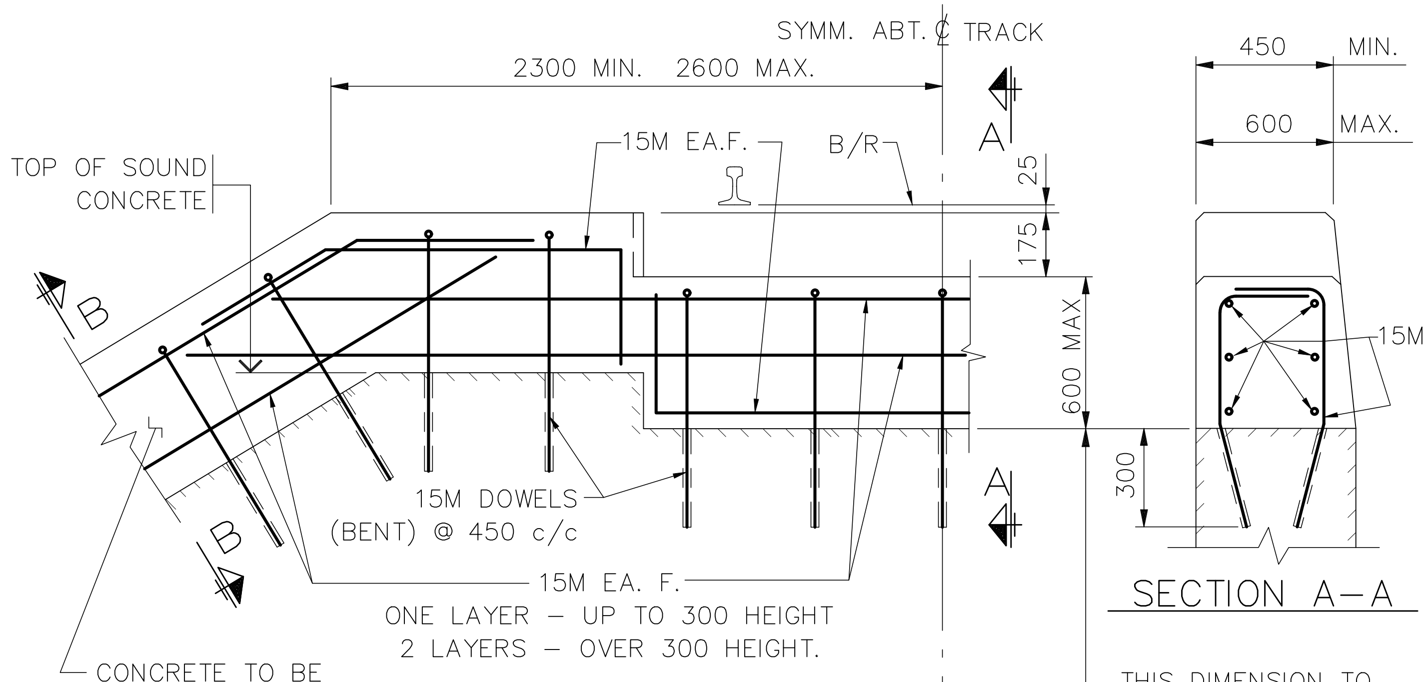
Drawn Dessin	PS	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	May 2020

STANDARD CONCRETE REPAIRS UNDER BEARING

METROLINX

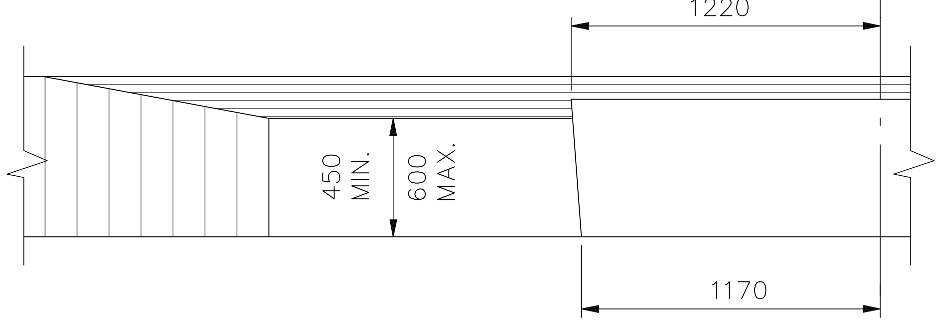
Drawing Number
Dessin Numero: MC–01

Rev.

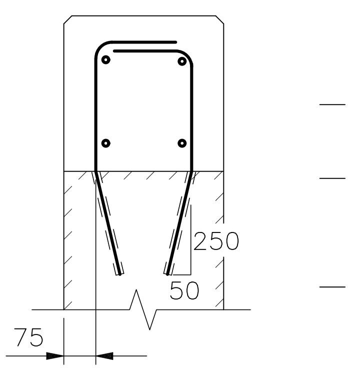


ELEVATION

SECTION A–A



P L A N



SECTION B–B

NOTES:
- CHIP TOP OF WALL TO SOUND CONCRETE.
- SURFACES IN CONTACT WITH NEW CONCRETE TO BE ROUGHENED.
- MIN. LAP FOR BARS = 450

SPECIFICATIONS

CONCRETE

C.S.A. CAN3 A23.1 & A23.2 MINIMUM OF 30 MPa @ 28 DAYS.

REINFORCING STEEL

C.S.A. G30.18. ALL BARS TO BE DEFORMED AND OF GRADE 400W.

COVER

MINIMUM COVER FOR MAIN REINFORCING STEEL TO BE 50 UNLESS NOTED OTHERWISE.

CHAMFER

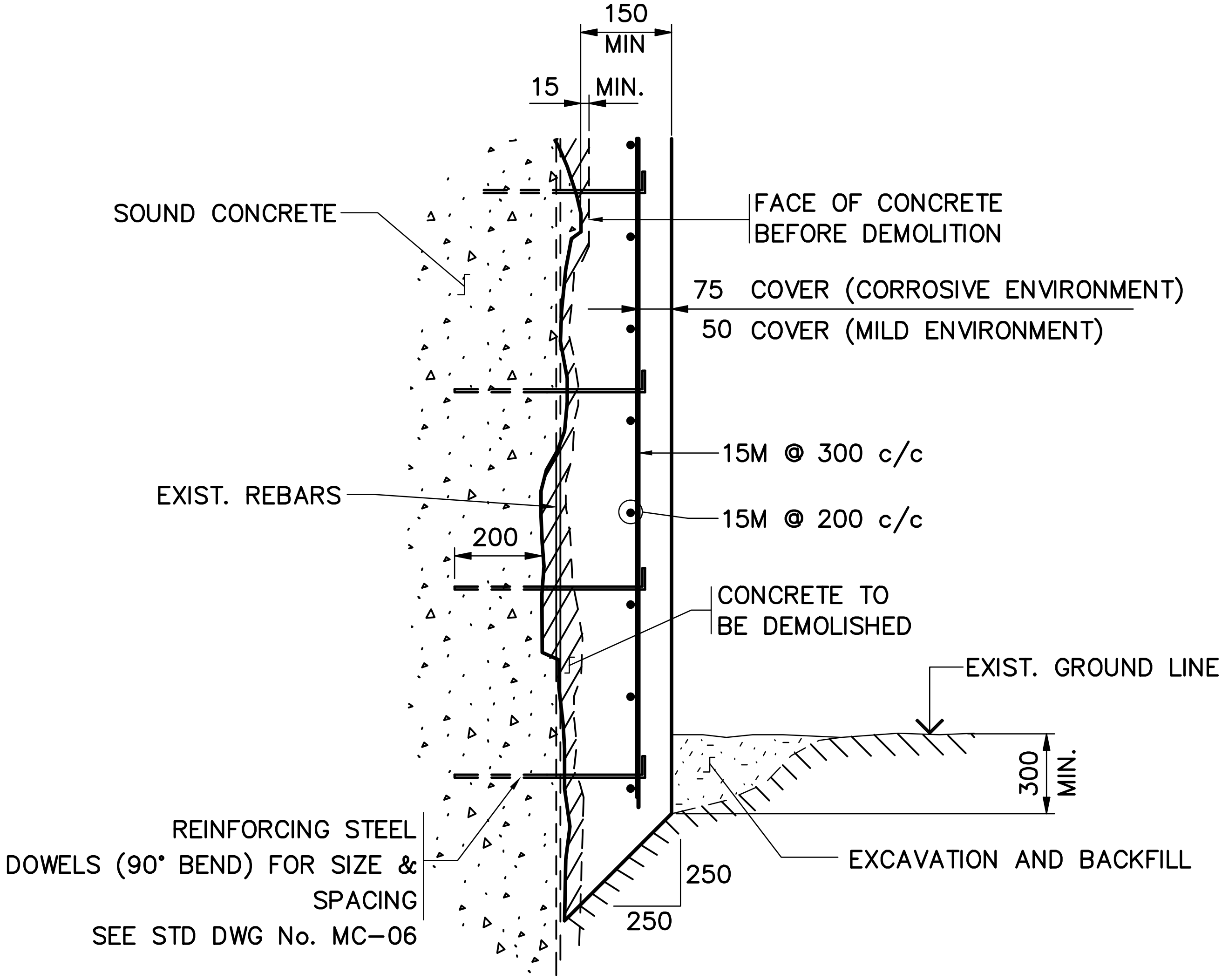
20mm FOR ALL EXPOSED EDGES.

Drawn Dessin	PS	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

STANDARD FOR RAISING WINGWALLS AND BACKWALLS

METROLINX

Drawing Number Dessin Numero	Rev.
MC–02	



NOTES:

- FOR OTHER NOTES SEE DWG No. MC-07
- FOR CONCRETE MIX SEE DWG No. MC-09
- ALL SURFACES TO BE REPAIRED SHALL BE DEMOLISHED TO A MINIMUM DEPTH OF 15mm. THE PROFILE OF THE DEMOLISHED SURFACE SHALL BE SUCH THAT THICKNESS OF CONCRETE IS RELATIVELY UNIFORM (SEE FIGURE 2 ON MC-07B). IF A UNIFORM PROFILE CANNOT BE ACHIEVED, THEN THE THICKNESS OF THE REPAIR SHOULD BE INCREASED. THICKNESS TO BE DETERMINED BY THE STRUCTURAL ENGINEER-MAINTENANCE.
- MINIMUM LAP FOR NEW REINFORCING STEEL SHALL BE 450mm.

SPECIFICATIONS:

- CONCRETE: CAN3-A23.1 & A23.2
- REINFORCING STEEL: CAN3-G30.18

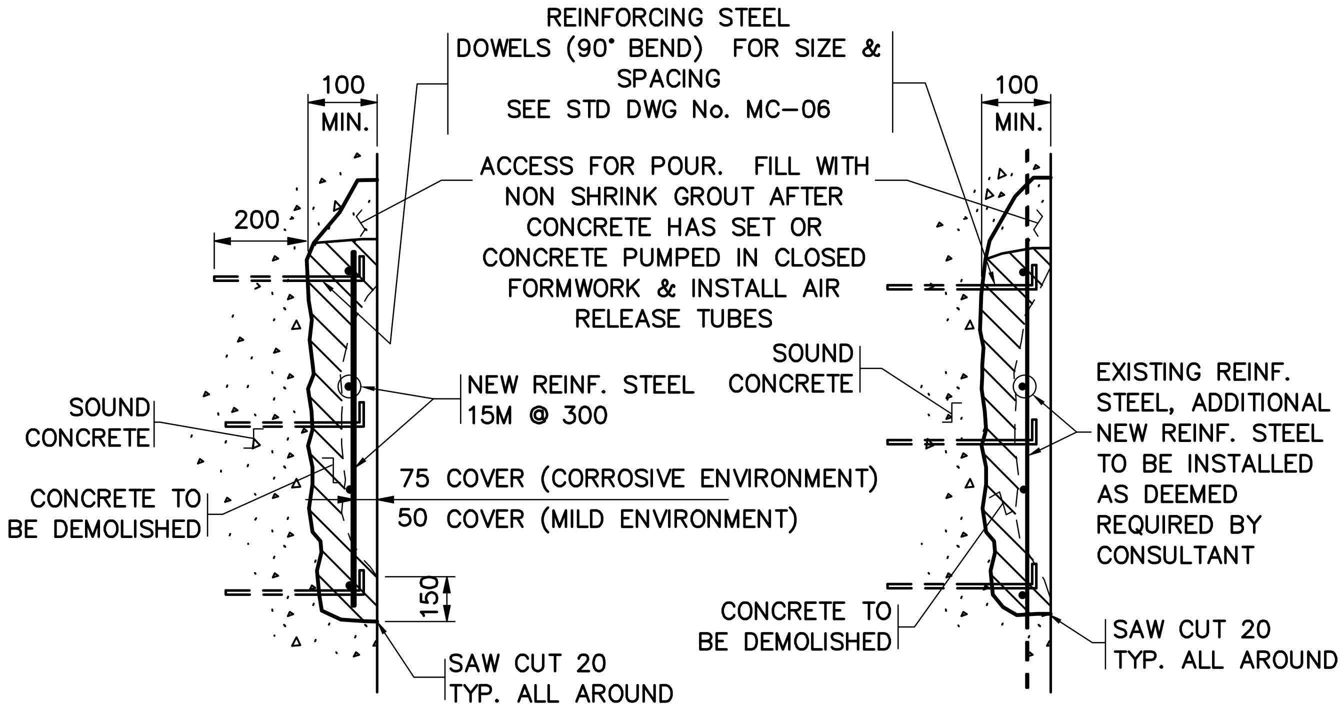
Drawn Dessin	PS	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	May 2020

STANDARD CONCRETE REPAIR WITH ADDITIONAL CONCRETE

METROLINX

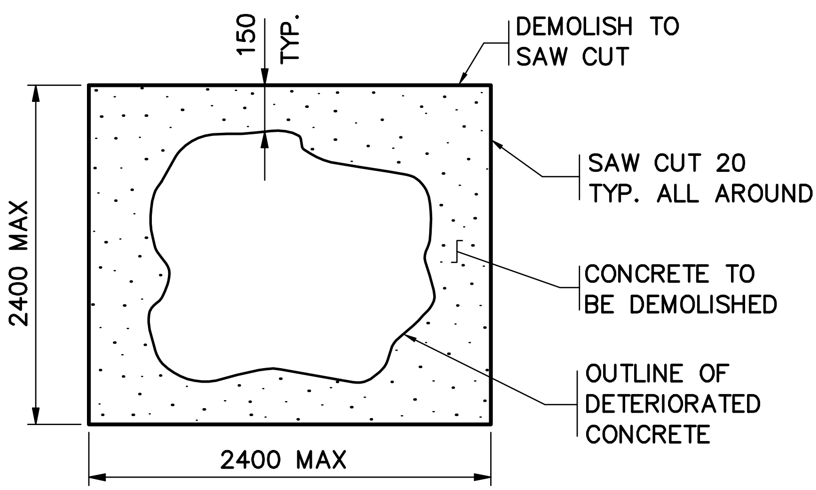
Drawing Number
Dessin Numero: MC-03

Rev.



UNREINFORCED CONCRETE SUBSTRATE

REINFORCED CONCRETE SUBSTRATE



ELEVATION

NOTES:

- FOR REPAIRS LARGER THAN 2400x2400, THE DETAIL SHOWN ON MC-03 SHOULD BE CONSIDERED
- FOR OTHER NOTES SEE DWG No. MC-07
- FOR CONCRETE MIX SEE DWG No. MC-09

SPECIFICATIONS:

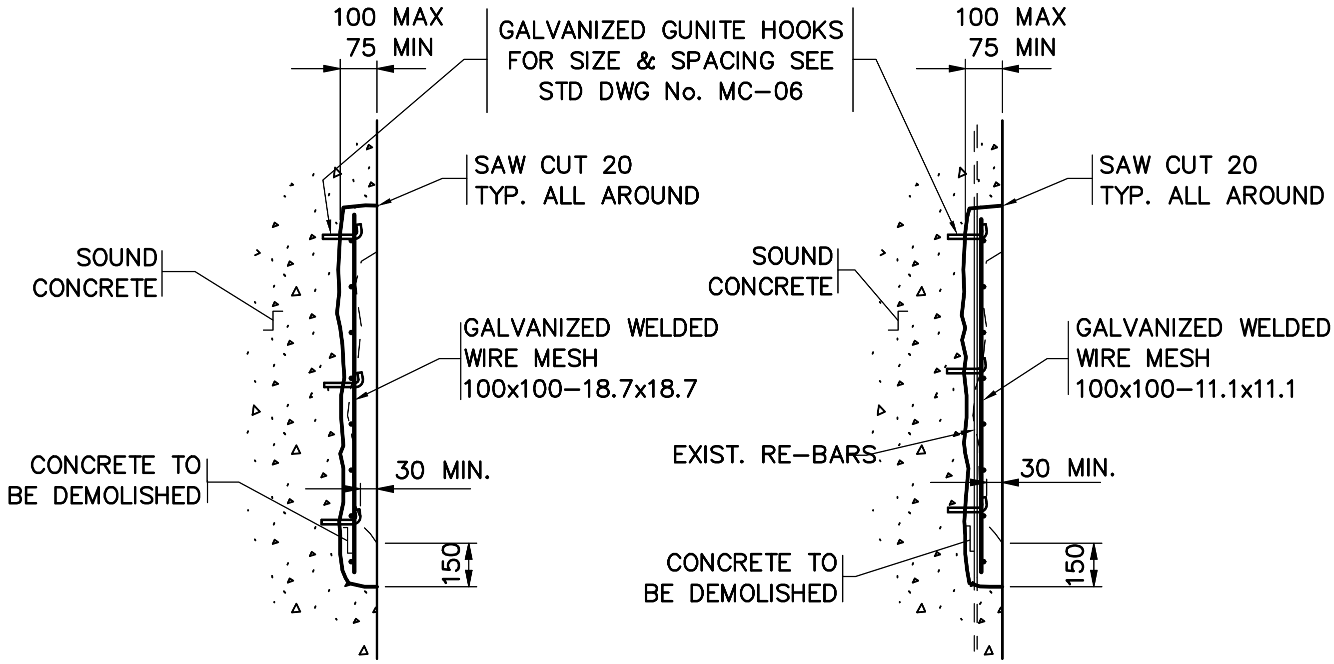
- CONCRETE: CAN3-A23.1 & A23.2
- REINFORCING STEEL: CAN3-G30.18

Drawn Dessin	PS	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	May 2020

STANDARD CONCRETE REPAIR
WITHOUT ADDITIONAL CONCRETE
(SMALL PATCH-MINIMUM DEPTH OF 100)

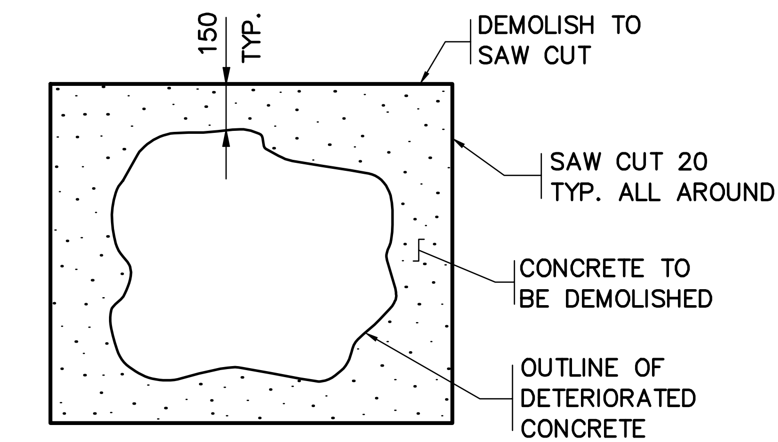
METROLINX

Drawing Number Dessin Numero	Rev.
MC-04	



UNREINFORCED CONCRETE SUBSTRATE

REINFORCED CONCRETE SUBSTRATE



ELEVATION

SPECIFICATIONS:

- CONCRETE: CAN3-A23.1 & A23.2
- REINFORCING STEEL: CAN3-G30.18

NOTES:

- NOTES ON SURFACE PREPARATION SHOWN ON DRAWING MC-07 ARE APPLICABLE FOR THIS REPAIR
- SHOTCRETE MIX SPECIFICATION TO BE DETERMINED BY STRUCTURAL ENGINEER-MAINTENANCE
- MINIMUM LAP FOR WELDED WIRE MESH SHALL BE 200.

Drawn / Dessin	PS	Checked / Verification	S.Kh.
Scale / Echelle	NONE	Date	May 2020

STANDARD CONCRETE REPAIR
USING SHOTCRETE OR PUMPED CONCRETE
(DEPTH OF 75 TO 100)

METROLINX

Drawing Number / Dessin Numero: MC-05

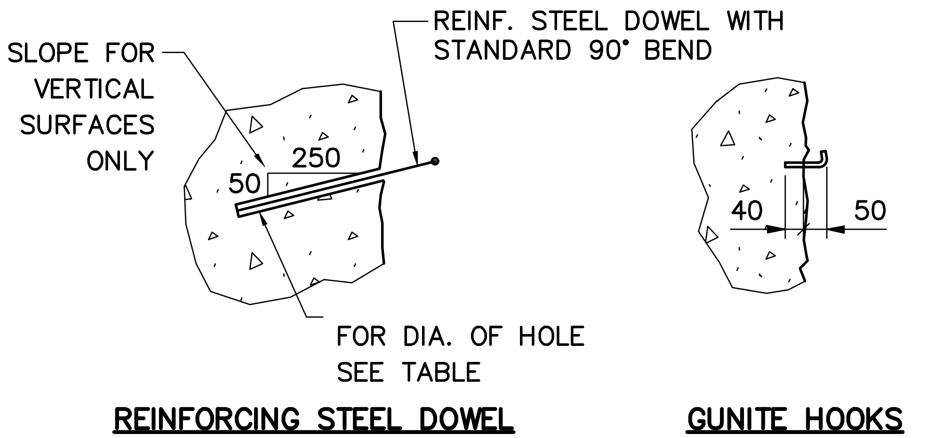
Rev.

THICKNESS OF NEW CONCRETE mm	GUNITE HOOKS DIAM. mm	REINF. STEEL DOWELS DIAM. mm	SPACING IN BOTH DIRECTIONS (mm)		
			SUSPENDED SURFACES	VERTICAL SURFACES	TOP SURFACES
75	10	10M	@ 550	@ 650	@ 915
100	10	10M	@ 500	@ 600	@ 915
125	10	10M	@ 425	@ 525	@ 915
150	–	10M	@ 400	@ 500	@ 915
175	–	10M	@ 350	@ 450	@ 915
200	–	15M	@ 475	@ 575	@ 915
225	–	15M	@ 450	@ 560	@ 915
250	–	15M	@ 425	@ 525	@ 600
275	–	15M	@ 400	@ 500	@ 600
300	–	15M	@ 375	@ 450	@ 600

REINFORCING STEEL DOWELS SHALL BE INSTALLED IN HOLES OF DIAMETER SHOWN IN TABLE. THE HOLES SHALL BE FILLED WITH NON–SHRINK GROUT. ALTERNATIVELY, DOWELS CAN BE SET USING "HILTI HIT HY–150" RESIN. HOLE DIAMETER SHOULD THEN BE REDUCED BY 15mm.

GUNITE HOOKS CAN BE USED FOR THE REPAIR SHOWN ON MC–05 IF THE CONCRETE SUBSTRATE IS SOLID AND IF THE THICKNESS OF THE REPAIR DOES NOT EXCEED 125.

DOWEL SIZE	HOLE DIAM. mm
10M	35
15M	40
20M	45
25M	50
30M	55
35M	60



ANCHORAGE DETAILS

Drawn Dessin	PS	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	May 2020

ANCHORS
FOR CONCRETE REPAIRS

S.Kh.

METROLINX

Drawing Number Dessin Numero: MC–06

Rev.

PROCEDURE

- DETERMINE BOUNDARIES OF AREA TO BE REPAIRED AS SHOWN ON FIGURE 1 AND DWGS MC–04 AND MC–05;
- SAW CUT 20mm DEEP ALL AROUND AND PERPENDICULAR TO SURFACE. CARE SHALL BE TAKEN TO AVOID CUTTING EXISTING REINFORCING STEEL;
- USING A 15 LBS JACKHAMMER, CHIP DAMAGED CONCRETE SUBSTRATE TO SOUND CONCRETE TO GET UNIFORM THICKNESS AND PERPENDICULAR FACES ON PERIMETER AS SHOWN ON FIGURE 2. HEAVIER CHIPPING EQUIPMENT SHALL BE USED ONLY WITH PRIOR CONSENT OF STRUCTURAL ENGINEER – MAINTENANCE;
- IF PERIMETER OF EXISTING REINFORCING STEEL BAR IS EXPOSED BY MORE THAN 50% THEN CHIP UNDER BAR TO GET 25mm CLEAR AS SHOWN ON FIGURE 3;
- ALL BARS CORRODED BY MORE THAN 30% IN DIAMETER SHALL BE REPLACED. SPLICE LENGTH SHALL BE AS SHOWN ON TABLE 1. CHIP ADDITIONAL CONCRETE TO OBTAIN SPLICE LENGTH IF NECESSARY AS SHOWN ON FIGURE 4;
- SAND BLAST ALL EXISTING REINFORCING STEEL;
- REMOVE LOOSE CONCRETE AND/OR DEBRIS USING OIL–FREE COMPRESSED AIR OR LIGHT SANDBLAST;
- APPLY TWO COATS OF "SIKATOP ARMATEC–110 EPOCEM " OR APPROVED EQUIVALENT ON ALL REINFORCING STEEL;
- INSTALL FORMWORK;
- WET CONCRETE SUBSTRATE TO OBTAIN A SATURATED SURFACE AND REMOVE EXCESS WATER;
- APPLY BONDING AGENT "SIKATOP ARMATEC–110 EPOCEM " OR APPROVED EQUIVALENT TO PERIMETER WALLS;
- POUR CONCRETE MIX CONFORMING TO STANDARD DRAWING MC–09; (SEE NOTES)
- KEEP CONCRETE SURFACE MOIST FOR A PERIOD OF SEVEN DAYS OR ALTERNATIVELY, USE "SIKA CUREHARD" CURING COMPOUND OR APPROVED EQUIVALENT;
- FILL OPENING AT TOP OF FORM USING NON SHRINK GROUT SUCH AS "SET 45" OR APPROVED EQUIVALENT;
- APPLY SEALANT "MASTERSEAL SL40" OR APPROVED EQUIVALENT.

NOTES:

- THIS PROCEDURE APPLIES TO REPAIRS SHOWN ON DWGS MC–03, AND MC–04 USING PORTLAND CEMENT CONCRETE AS PER DWG MC–09. SURFACE PREPARATION FOR REPAIRS SHOWN ON DWG MC–05 SHALL ALSO BE DONE AS PER THIS PROCEDURE.
- SEE DWG MC–07B FOR FIGURES 1 TO 4 AND TABLE 1.
- CONCRETE MIX TO BE SELECTED FROM DRAWING MC–09A TO D BASED ON ENVIRONMENT AND TOTAL THICKNESS OF REPAIRS. IF CONCRETE IS CHIPPED BEHIND REBAR AS SHOWN ON FIGURE 3,THEN THE MIXES SHOWN ON DRAWING MC–09A AND C SHALL BE USED REGARDLESS OF THE TOTAL THICKNESS OF THE REPAIR.

Drawn / Dessin	FM	Checked / Verification	S.Kh.	STANDARD PROCEDURE FOR CONCRETE REPAIRS
Scale / Echelle	NONE	Date	March 2019	SHEET 1 OF 2

METROLINX	Drawing Number / Dessin Numero	Rev.
	MC–07A	

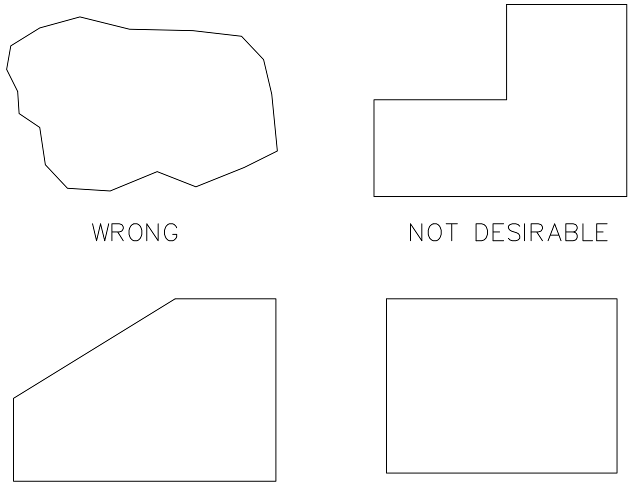


FIGURE 1
SAW CUT OUTLINE

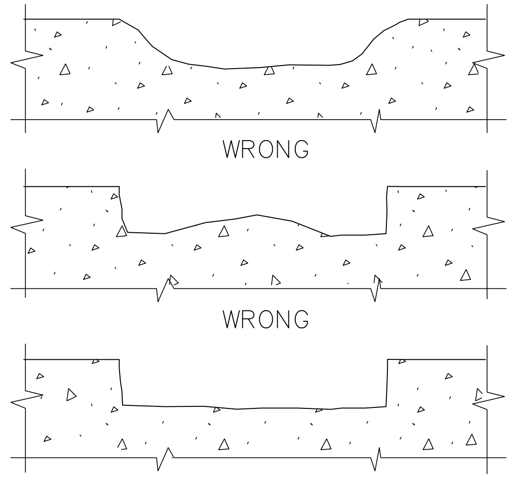


FIGURE 2
CHIPPING

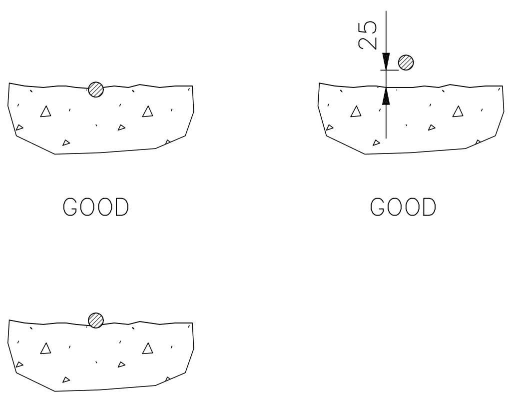


FIGURE 3
EXIST. REINF. STEEL

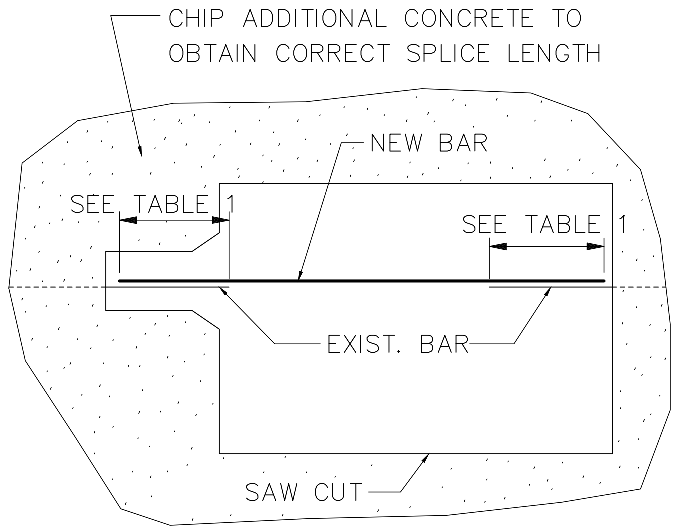


FIGURE 4
REINF. STEEL SPLICE

TABLE 1
(SPLICE LENGTH)

10M – 350	20M – 600	30M – 1400
15M – 500	25M – 1000	35M – 2000

THESE SPLICE LENGTHS SHOULD BE MULTIPLIED BY 1.3 IF THERE IS 300mm OF FRESH CONCRETE BELOW THE SPLICE

THIS DRAWING TO BE USED IN CONJUNCTION WITH STD DRAWING MC-07A

Drawn Dessin	FM	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

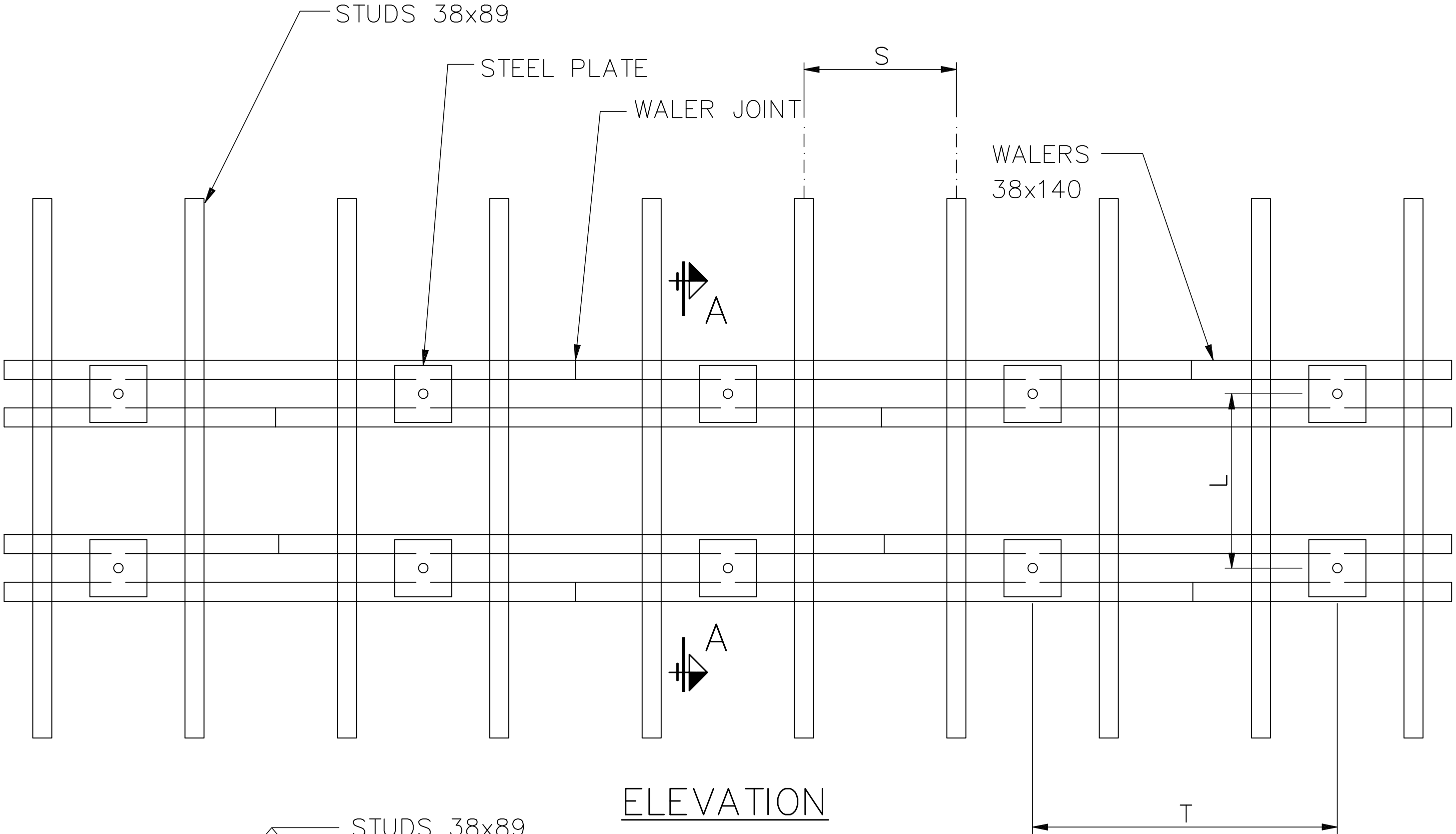
STANDARD PROCEDURE FOR CONCRETE REPAIRS

SHEET 2 OF 2

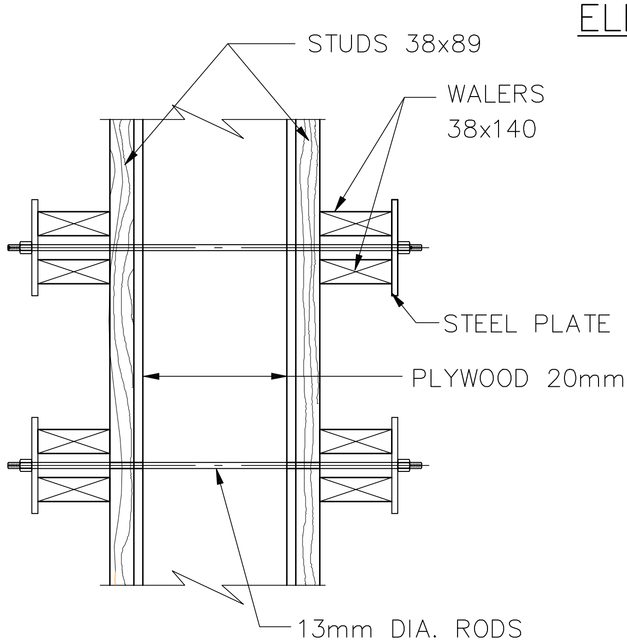
METROLINX

Drawing Number Dessin Numero: MC-07B

Rev.



ELEVATION



SECTION "A–A"

NOTES:

- THIS DWG. TO BE READ IN CONJUNCTION WITH DWG. MC–08B
- WALERS MUST EXTEND OVER 4 STUDS MINIMUM
- SPLICES ARE NOT ALLOWED NEXT TO TIE RODS
- SPLICES MUST BE STAGGERED

SPECIFICATIONS:

- TIMBER: C.S.A. 086.1 SPF No.1
- RODS: 13mm DIA. ALLOWABLE LOAD 13 kN(3000lbs)
- STEEL PLATES: C.S.A. G30.12 GRADE 300W

Drawn Dessin	JT	Checked Verification	S.Kh.
Scale Echelle	N.T.S.	Date	March 2019

STANDARD FORMWORK DETAILS

METROLINX

Drawing Number Dessin Numero	Rev.
MC–08A	

Rate of placement meter per hr.		Temperature of concrete in forms deg.C 32	25	20	15	10	5
0.3	S	410	410	410	410	410	410
	L	710	710	710	710	710	710
	T	765	765	765	765	765	765
0.6	S	410	355	355	355	355	355
	L	710	710	710	710	710	710
	T	765	765	765	765	765	765
0.9	S	355	355	310	310	310	310
	L	710	710	635	635	635	635
	T	765	765	635	635	635	635
1.2	S	310	310	310	254	254	254
	L	635	635	635	560	560	560
	T	635	635	635	510	510	510
1.5	S	310	310	255	255	255	*
	L	635	635	560	560	560	*
	T	635	635	510	510	510	*
1.8	S	255	255	255	255	*	*
	L	560	560	560	560	*	*
	T	510	510	510	510	*	*
2.1	S	255	255	255	*	*	*
	L	560	560	560	*	*	*
	T	510	510	510	*	*	*
2.4	S	255	255	255	*	*	*
	L	560	560	560	*	*	*
	T	510	510	510	*	*	*
2.7	S	255	255	255	*	*	*
	L	560	560	560	*	*	*
	T	510	510	510	*	*	*
3.0	S	255	255	255	*	*	*
	L	560	560	560	*	*	*
	T	510	510	510	*	*	*
More than 3 m/hr		* To be determined by the engineer					

S: 38x89 stud spacing(mm)
L: 38x140 waler spacing(mm)
T: Horizontal spacing of anchors(mm)

Step by step instructions:

1: For height of formwork less than 1500mm use S=355, L=635, T=635
2: Verify that all conditions for simple form are satisfied
3: Calculate rate of placement
4: Measure temperature of concrete
5: For forms higher than 1500mm with rate of placement of more than 3m, the design shall be done by the engineer.

Simple form conditions

Concrete
1 – Type GU or HE cement
2 – Maximum slump 100mm
3 – Internally vibrated concrete
4 – Depth of vibration limited to 1200mm below concrete surface

Formwork
5 – Maximum batter of formwork 1 in 6
6 – Plywood is used in the strong direction
7 – Plywood deflection is not critical
8 – Plywood 20mm
9 – Studs 38x89
10– Walers 38x140 double
11– Ties 13mm rods with 13 kN(3000 lbs) allowable load
12– Not for column formwork

Drawn Dessin	JT	Checked Verification	S.Kh.
Scale Echelle		Date	March 2019

STANDARD FORMWORK DESIGN TABLE

METROLINX

Drawing Number Dessin Numero: MC–08B

Rev.

QUANTITY: _____________ m^3

CONCRETE CLASS: CLASS C-1 EXPOSURE

f'c : 35 MPa @ 28 DAYS

CEMENT: PORTLAND CEMENT TYPE GU CONFORMING TO CSA-A3001 & A3002

WATER: CONFORMING TO CSA-A23.1

COARSE AGGREGATE: 10mm CONFORMING TO CSA-A23.1

FINE AGGREGATE: CONFORMING TO CSA-A23.1

MIN. CEMENT CONTENT: 365kg PER CUBIC METER

MAX. WATER/CEMENT RATIO: 0.40

SLUMP: 75mm $\pm$ 25mm

AIR ENTRAINMENT: 7-10%, CONFORMING TO ASTM-C260

WATER REDUCING ADMIXTURE: CONFORMING TO ASTM-C494, TYPE A

NO SUPERPLASTICIZERS, CALCIUM CHLORIDE OR OTHER ADMIXTURES ALLOWED

MAXIMUM CONCRETE TEMPERATURE: 32°C

COPY OF MIX DESIGN INCLUDING SIEVE ANALYSIS OF AGGREGATES AND A CERTIFICATE INDICATING THAT THE AGGREGATES ARE NOT REACTIVE SHALL BE SUPPLIED TO THE ENGINEER

QUALITY CONTROL FOR AIR, SLUMP AND STRENGTH AS PER CSA-A23.2, TO BE PROVIDED BY CONCRETE SUPPLIER

APPLICATION IN CORROSIVE ENVIRONMENT:
THICKNESS < 200mm

Drawn Dessin	FM	Checked Vérification	S.Kh.
Scale Échelle		Date	May 2020

STANDARD CONCRETE SPECIFICATION FOR REPAIRS (SHEET 1 OF 4)

METROLINX

Drawing Number
Dessin Numero
MC-09A

Rev.

QUANTITY: ______________ m^3

CONCRETE CLASS: CLASS C-1 EXPOSURE

f'c : 35 MPa @ 28 DAYS

CEMENT: PORTLAND CEMENT TYPE GU CONFORMING TO CSA-A3001 & A3002

WATER: CONFORMING TO CSA-A23.1

COARSE AGGREGATE: 20mm CONFORMING TO CSA-A23.1

FINE AGGREGATE: CONFORMING TO CSA-A23.1

MIN. CEMENT CONTENT: 365kg PER CUBIC METER

MAX. WATER/CEMENT RATIO: 0.40

SLUMP: 75mm ± 25mm

AIR ENTRAINMENT: 5-8%, CONFORMING TO ASTM-C260

WATER REDUCING ADMIXTURE: CONFORMING TO ASTM-C494, TYPE A

NO SUPERPLASTICIZERS, CALCIUM CHLORIDE OR OTHER ADMIXTURES ALLOWED

MAXIMUM CONCRETE TEMPERATURE: 32°C

COPY OF MIX DESIGN INCLUDING SIEVE ANALYSIS OF AGGREGATES AND A CERTIFICATE INDICATING THAT THE AGGREGATES ARE NOT REACTIVE SHALL BE SUPPLIED TO THE ENGINEER

QUALITY CONTROL FOR AIR, SLUMP AND STRENGTH AS PER CSA-A23.2, TO BE PROVIDED BY CONCRETE SUPPLIER

APPLICATION IN CORROSIVE ENVIRONMENT:
THICKNESS > 200mm

Drawn Dessin	FM	Checked Vérification	S.Kh.
Scale Échelle		Date	May 2020

STANDARD CONCRETE SPECIFICATION FOR REPAIRS (SHEET 2 OF 4)

METROLINX

Drawing Number Dessin Numero	Rev.
MC-09B	

QUANTITY: _____________ m^3

f'c : 30 MPa @ 28 DAYS

CEMENT: PORTLAND CEMENT TYPE GU CONFORMING TO CSA–A3001 & A3002

WATER: CONFORMING TO CSA–A23.1

COARSE AGGREGATE: 10mm CONFORMING TO CSA–A23.1

FINE AGGREGATE: CONFORMING TO CSA–A23.1

MIN. CEMENT CONTENT: 365kg PER CUBIC METER

MAX. WATER/CEMENT RATIO: 0.45

SLUMP: 75mm $\pm$ 25mm

AIR ENTRAINMENT: 5–8%, CONFORMING TO ASTM–C260

WATER REDUCING ADMIXTURE: CONFORMING TO ASTM–C494, TYPE A

NO SUPERPLASTICIZERS, CALCIUM CHLORIDE OR OTHER ADMIXTURES ALLOWED

MAXIMUM CONCRETE TEMPERATURE: 32°C

COPY OF MIX DESIGN INCLUDING SIEVE ANALYSIS OF AGGREGATES AND A CERTIFICATE INDICATING THAT THE AGGREGATES ARE NOT REACTIVE SHALL BE SUPPLIED TO THE ENGINEER

QUALITY CONTROL FOR AIR, SLUMP AND STRENGTH AS PER CSA–A23.2, TO BE PROVIDED BY CONCRETE SUPPLIER

APPLICATION IN MILD ENVIRONMENT:
THICKNESS < 200mm

Drawn Dessin	JE	Checked Vérification	S.Kh.
Scale Échelle		Date	March 2019

STANDARD CONCRETE SPECIFICATION FOR REPAIRS (SHEET 3 OF 4)

METROLINX

Drawing Number
Dessin Numero: MC–09C

Rev.

QUANTITY: _____________ m^3

f'c : 30 MPa @ 28 DAYS

CEMENT: PORTLAND CEMENT TYPE GU CONFORMING TO CSA-A3001 & A3002

WATER: CONFORMING TO CSA-A23.1

COARSE AGGREGATE: 20mm CONFORMING TO CSA-A23.1

FINE AGGREGATE: CONFORMING TO CSA-A23.1

MIN. CEMENT CONTENT: 365kg PER CUBIC METER

MAX. WATER/CEMENT RATIO: 0.45

SLUMP: 75mm ± 25mm

AIR ENTRAINMENT: 4-7%, CONFORMING TO ASTM-C260

WATER REDUCING ADMIXTURE: CONFORMING TO ASTM-C494, TYPE A

NO SUPERPLASTICIZERS, CALCIUM CHLORIDE OR OTHER ADMIXTURES ALLOWED

MAXIMUM CONCRETE TEMPERATURE: 32°C

COPY OF MIX DESIGN INCLUDING SIEVE ANALYSIS OF AGGREGATES AND
A CERTIFICATE INDICATING THAT THE AGGREGATES ARE NOT REACTIVE
SHALL BE SUPPLIED TO THE ENGINEER

QUALITY CONTROL FOR AIR, SLUMP AND STRENGTH AS PER CSA-A23.2,
TO BE PROVIDED BY CONCRETE SUPPLIER

APPLICATION IN MILD ENVIRONMENT:
THICKNESS > 200mm

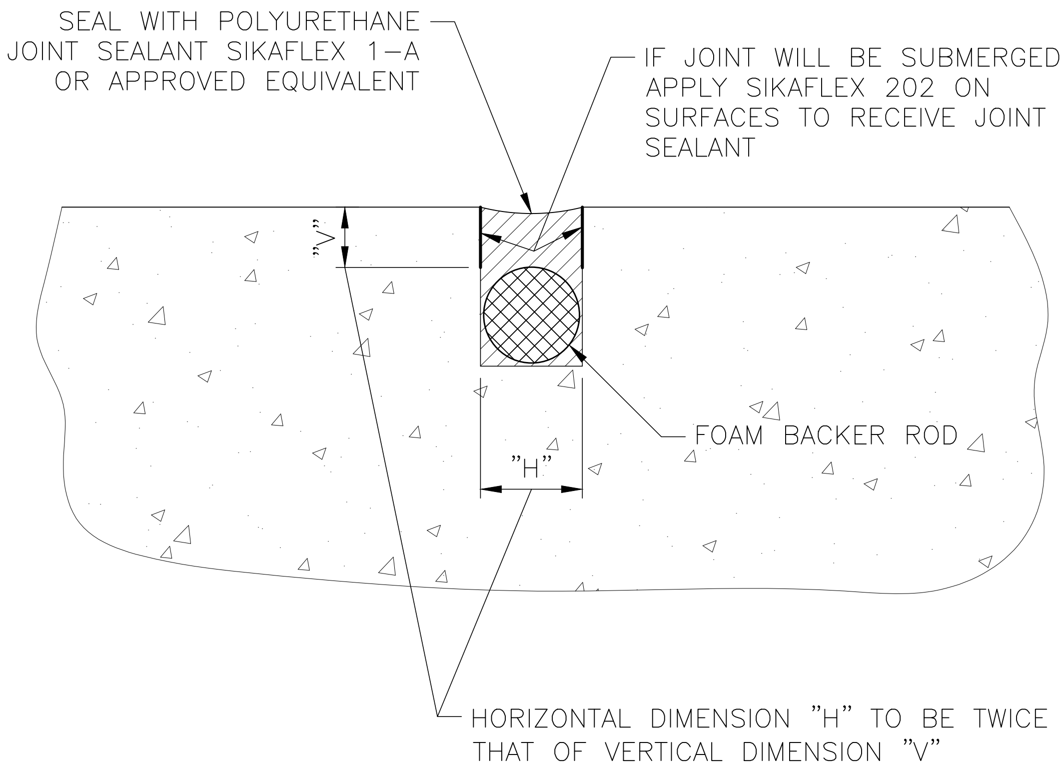
Drawn Dessin	JE	Checked Vérification	S.Kh.
Scale Échelle		Date	March 2019

STANDARD CONCRETE SPECIFICATION FOR REPAIRS (SHEET 4 OF 4)

METROLINX

Drawing Number
Dessin Numero: MC-09D

Rev.



RECOMMENDED BACKER ROD SIZE PER JOINT:

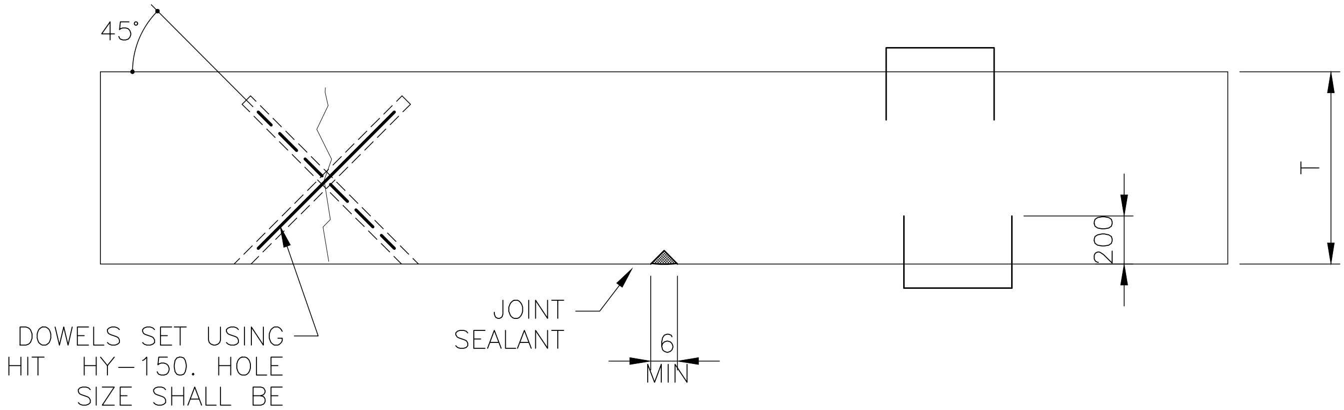
JOINT WIDTH	BACKER ROD DIAMETER
10mm	13mm
13mm	16mm
16mm	19mm
19mm	26mm
26mm	32mm
32mm	38mm

Drawn / Dessin	JE	Checked / Verification	S.Kh.
Scale / Echelle	N.T.S.	Date	March 2019

STANDARD JOINT SEALING DETAIL

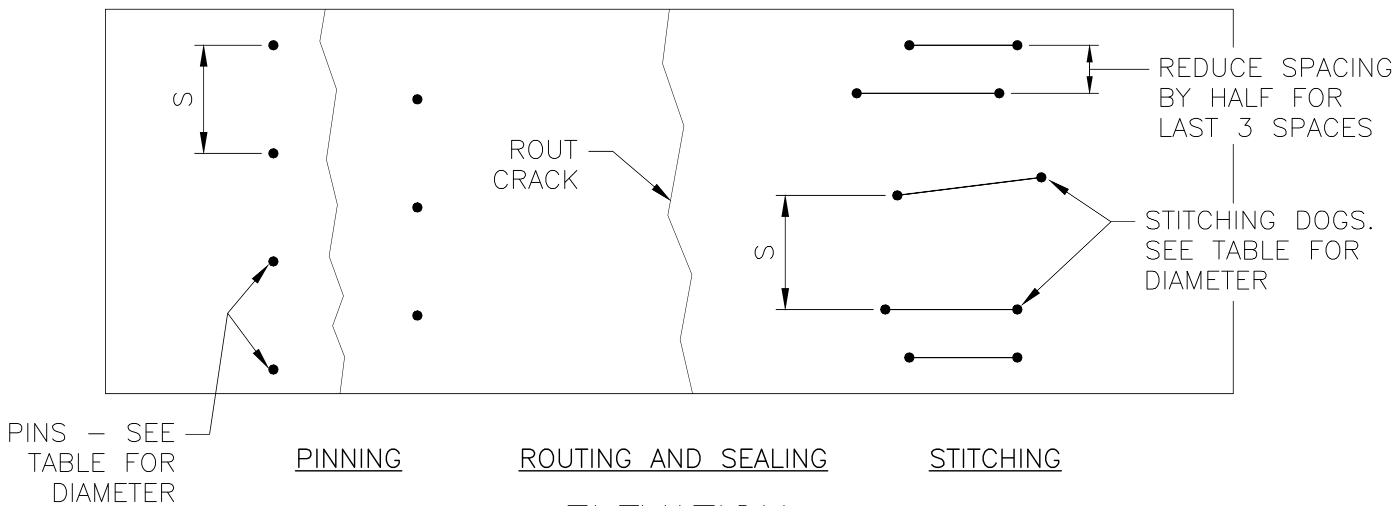
METROLINX

Drawing number / Dessin Numero	Rev.
MC-10	



SECTION

N.B.
VARIATE LENGTH, LOCATION AND ORIENTATION OF DOGS AS SHOWN.



ELEVATION

T	S	DIAMETER
300	200	10M
400	250	15M
500	200	15M
600	250	20M

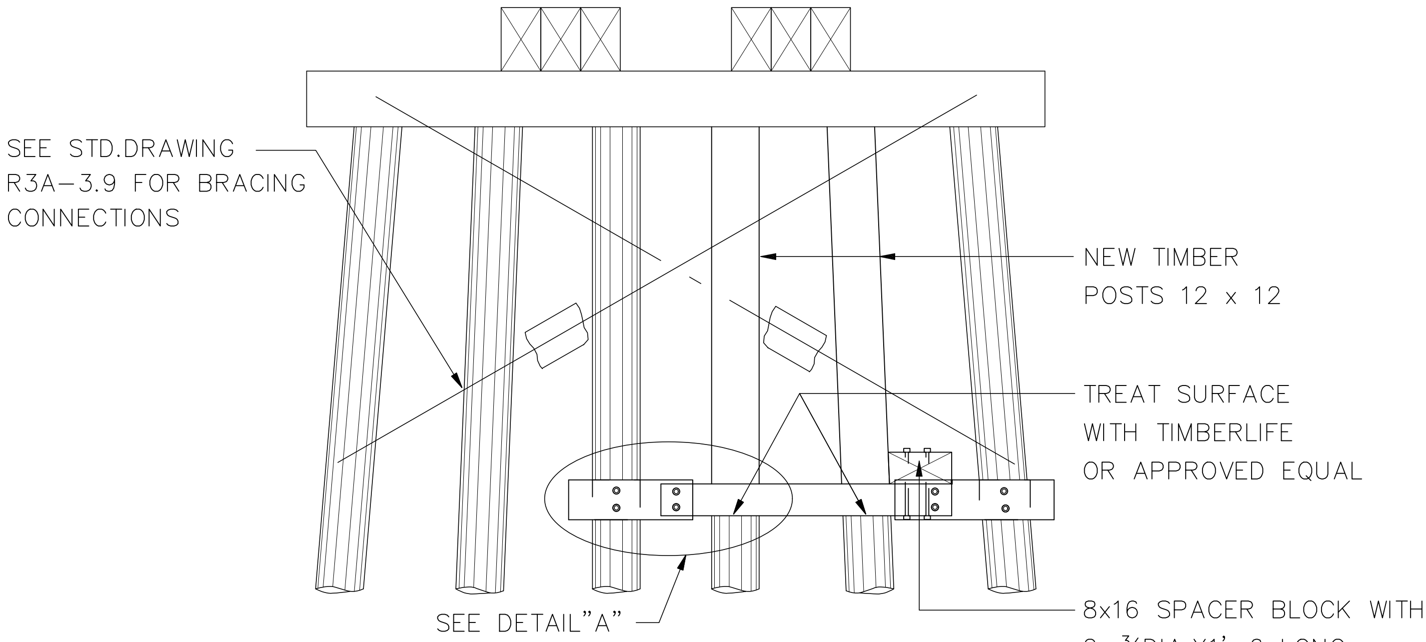
PROCEDURE FOR ROUTING AND SEALING

- ROUT CRACK TO A "V" SHAPE WITH A SAW OR CHIPPING TOOL AS SHOWN ON SECTION.
- CLEAN AND DRY CRACK WITH OIL-FREE COMPRESSED AIR.
- APPLY SIKAFLEX 202 (OR APPROVED EQUIVALENT) WHERE SEALANT WILL BE SUBMERGED.
- APPLY POLYURETHANE JOINT SEALANT SIKAFLEX –1A (OR APPROVED EQUIVALENT) IN ROUTED CRACK.

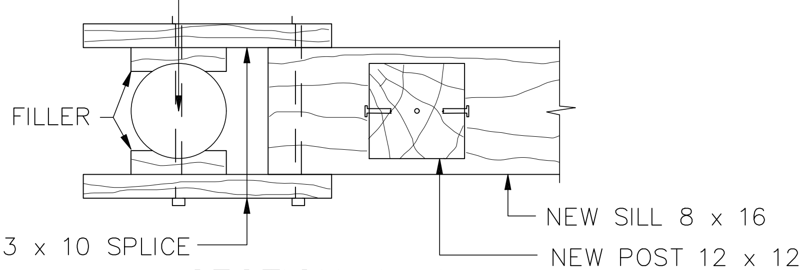
Drawn / Dessin	JE	Checked / Verification	S.Kh.	STANDARD CRACK REPAIRS
Scale / Echelle	N.T.S.	Date	March 2019	

METROLINX

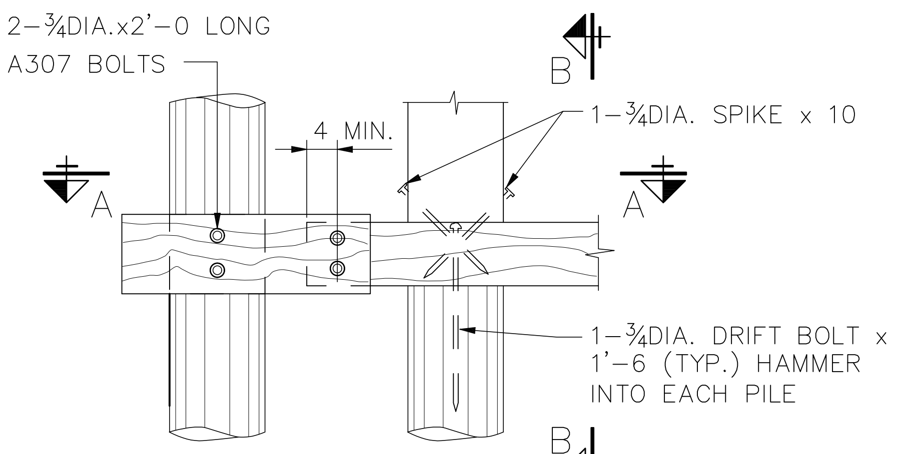
Drawing number / Dessin Numero	Rev.
MC-11	



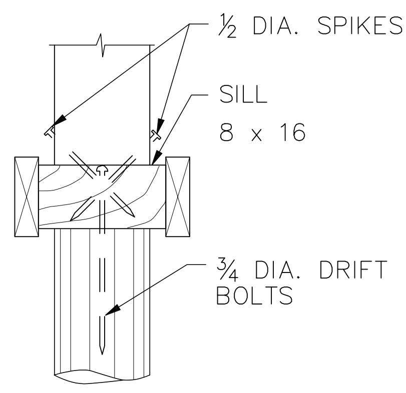
PILE BENT ELEVATION



SECTION A–A



DETAIL"A"



SECTION B–B

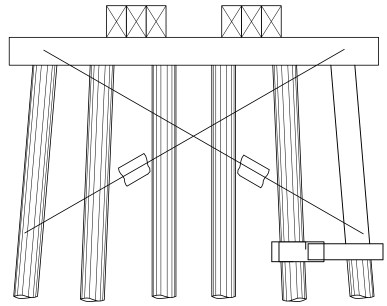
Drawn Dessin	JE	Checked Vérification	S.Kh.
Scale Échelle	N.T.S.	Date	March 2019

POSTING OF TIMBER PILES

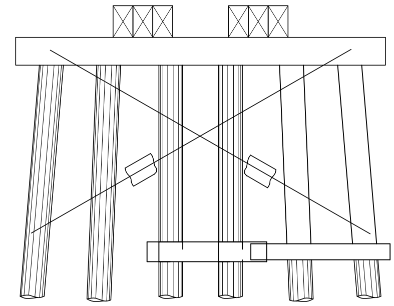
METROLINX

Drawing Number
Dessin Numero
MT–01A

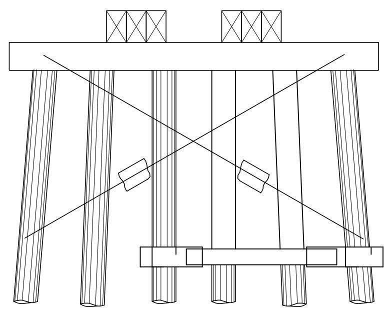
Rev.



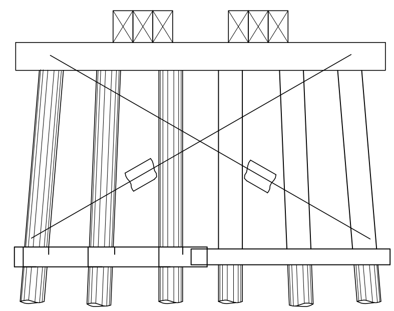
PILE BENT ELEVATION



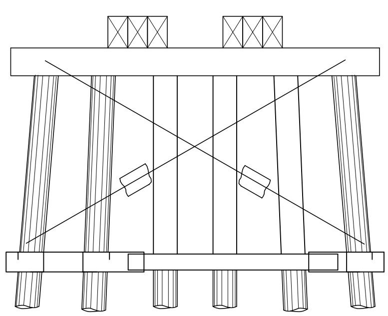
PILE BENT ELEVATION



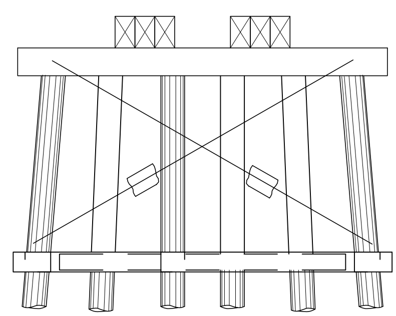
PILE BENT ELEVATION



PILE BENT ELEVATION



PILE BENT ELEVATION



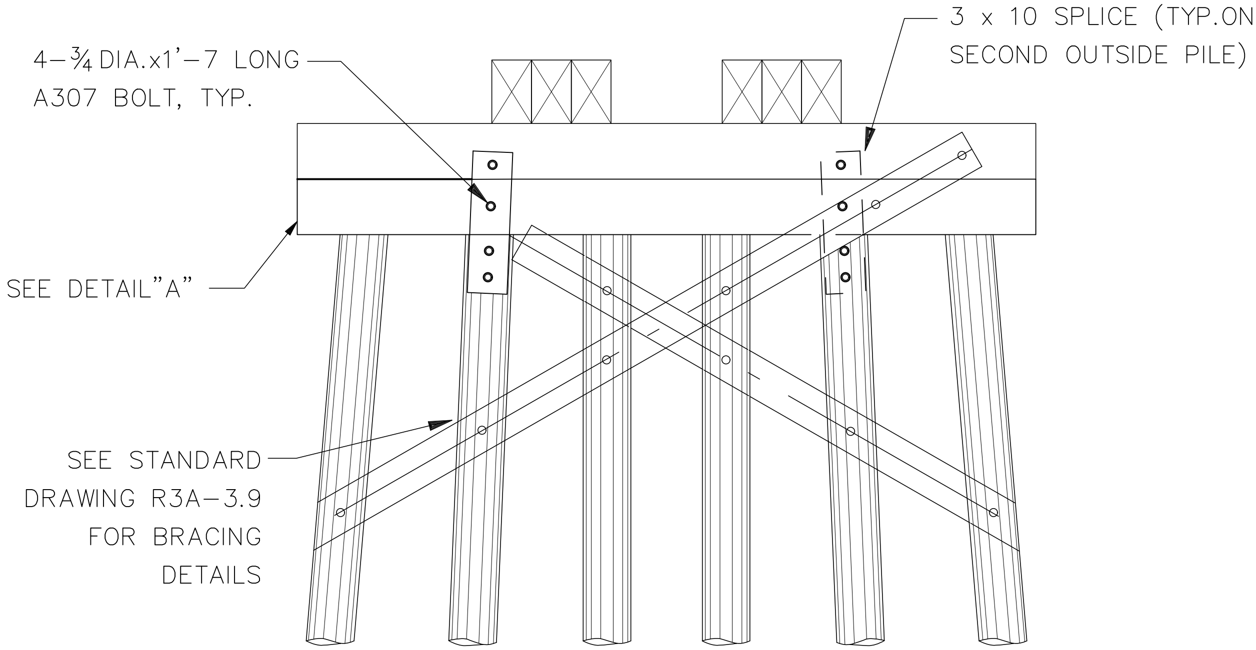
PILE BENT ELEVATION

Drawn Dessin	JE	Checked Vérification	S.Kh.
Scale Échelle	N.T.S.	Date	March 2019

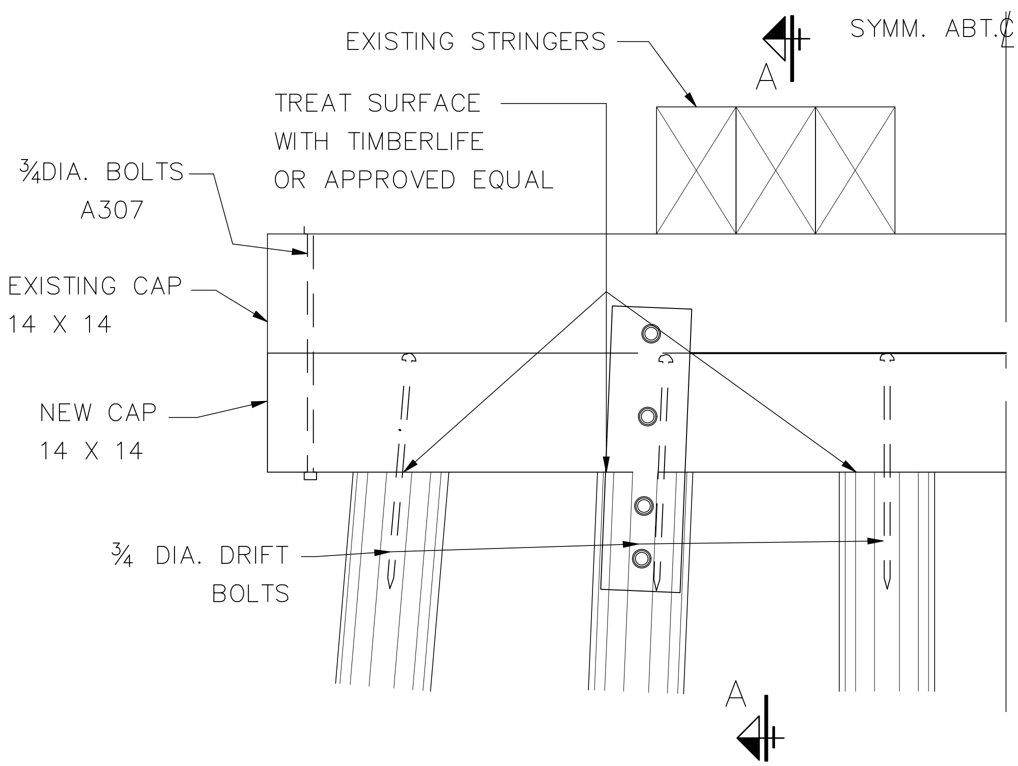
CONTOUR ARRANGEMENT FOR POSTING PILES

METROLINX

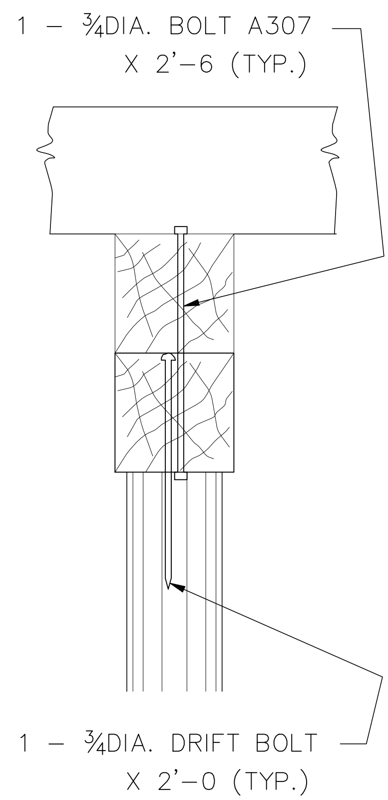
Drawing Number Dessin Numero	Rev.
MT–01B	



PILE BENT ELEVATION



DETAIL"A"



SECTION A–A

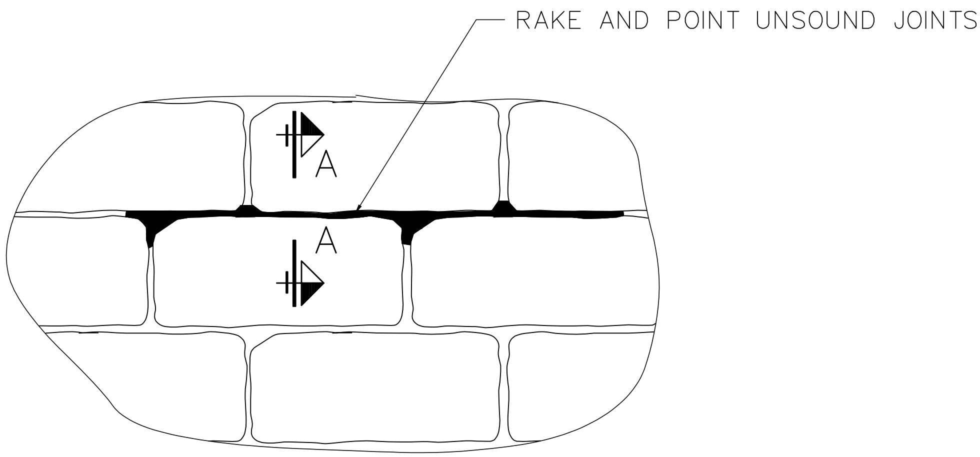
Drawn / Dessin	JE	Checked / Vérification	S.Kh.
Scale / Échelle	N.T.S.	Date	March 2019

DOUBLE CAPPING OF
TIMBER PILE BENTS

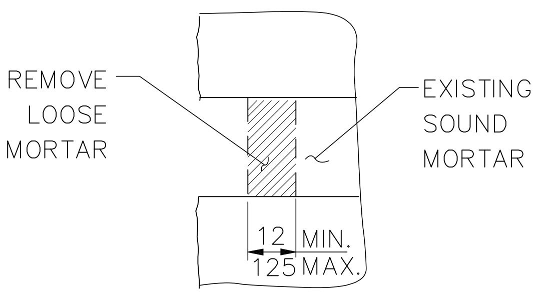
METROLINX

Drawing Number / Dessin Numero: MT–02

Rev.

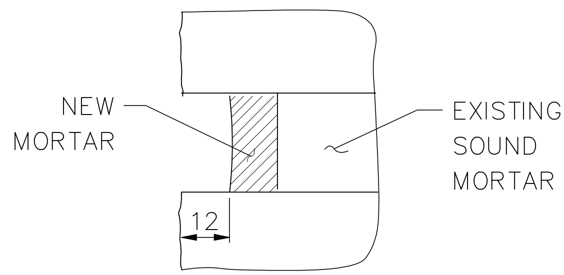


MASONRY WALL ELEVATION



SECTION "A–A"

(RAKING OF JOINTS)



SECTION "A–A"

(POINTING)

PROCEDURE:

- CUT OUT ALL DISINTEGRATED MORTAR TO A DEPTH OF 12 MIN. USING COLD CHISEL AND HAMMER, 15 LBS CHIPPING HAMMER WITH HOOKED CHISEL OR MEDIUM TO HIGH PRESSURE WATER JET.
- CLEAN OUT JOINTS USING WIRE BRUSH, AIR OR WATER.
- WET DOWN JOINTS TO BE POINTED.
- PREPARE MORTAR ACCORDING TO STANDARD MIX.
- PACK MORTAR TIGHTLY IN JOINTS.
- FINISH TO A SMOOTH CONCAVE SURFACE.
- CURE JOINTS FOR 3 DAYS BY THE WET CURING PROCESS OR BY USING Sikagard CUREHARD CURING AGENT

STANDARD MIX (BY VOLUME):

- 1 PART CEMENT PORTLAND.
- 1 PART CEMENT MASONRY (TYPE H).
- 5 PARTS OF SAND
- WATER UNTIL A STIFF PLASTIC MASS IS OBTAINED.

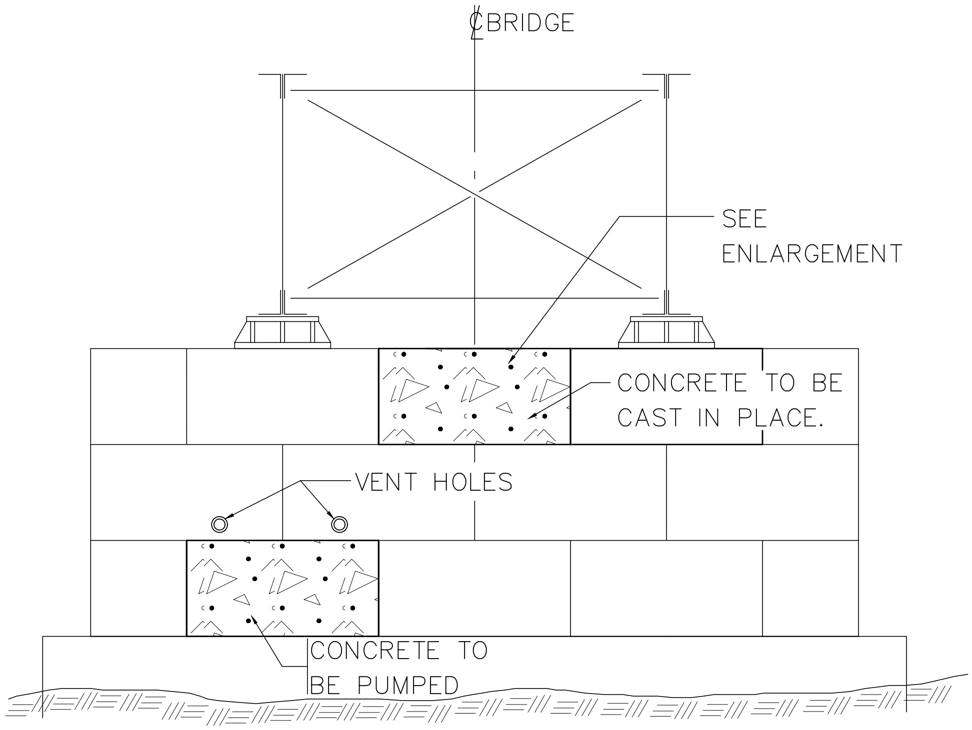
Drawn / Dessin	JE	Checked / Vérification	S.Kh.
Scale / Échelle	N.T.S.	Date	March 2019

POINTING OF STONE MASONRY JOINTS (MAXIMUM DEPTH 125)

METROLINX

Drawing Number / Dessin Numero: MM–01

Rev.



ELEVATION: PIER OR ABUTMENT

SPECIFICATIONS

CONCRETE

C.S.A. CAN3 A23.1 &
A23.2 MINIMUM OF
30 MPa @ 28 DAYS.

REINFORCING STEEL

C.S.A. G30.18 – M92. ALL
BARS TO BE DEFORMED
AND OF GRADE 400
MPa –15M – MIN. LAP 400

COVER

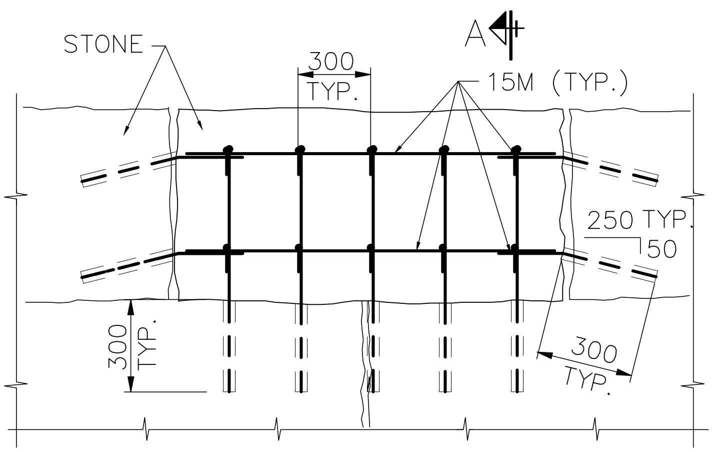
MINIMUM COVER FOR
MAIN REINFORCING STEEL
TO BE 50 UNLESS NOTED
OTHERWISE.

CHAMFER

20 FOR ALL EXPOSED
EDGES.

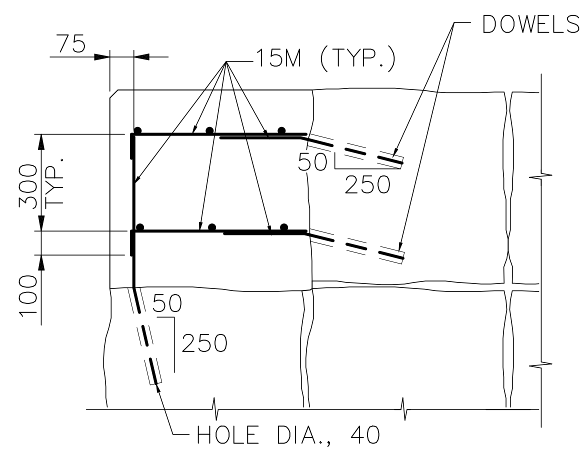
PROCEDURE:

- REMOVE BROKEN STONE WITH A MAX. 15 POUNDS HAMMER.
- REMOVE MORTAR.
- PLACE DOWELS WITH NON SHRINK GROUT.
- PLACE CONCRETE.
- CURE THE CONCRETE FOR 3 DAYS BY THE WET CURING PROCESS OR BY USING Sikagard CUREHARD CURING AGENT.



ELEVATION

ENLARGEMENT



SECTION A–A

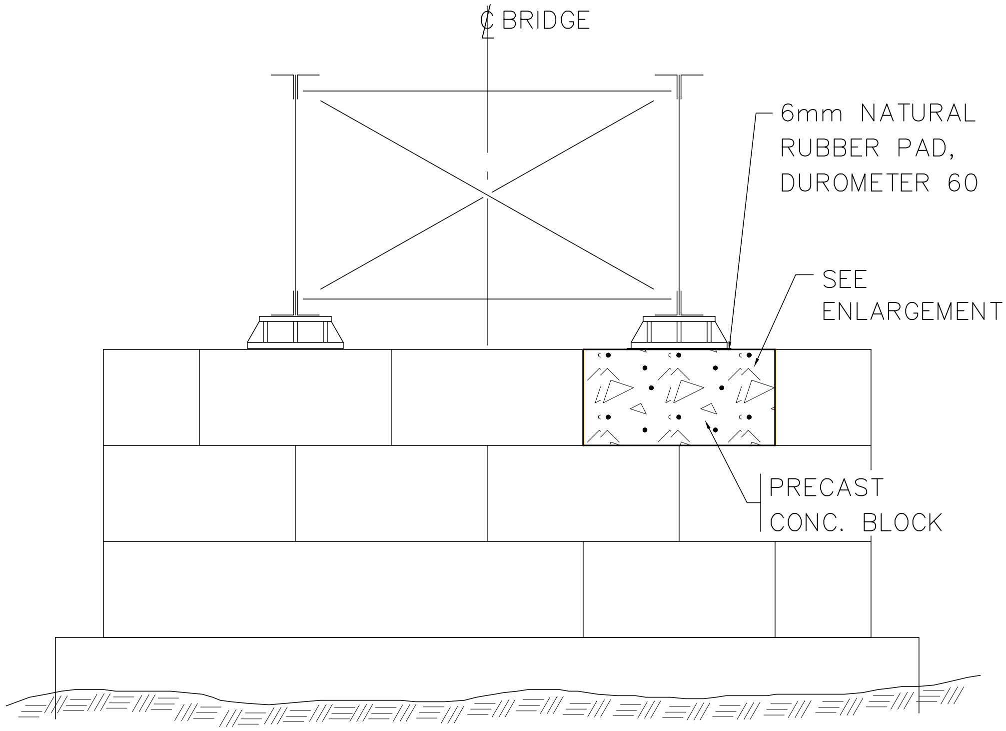
Drawn Dessin	JE	Checked Vérification	S.Kh.
Scale Échelle	NONE	Date	March 2019

STANDARD FOR REPLACEMENT OF STONES IN MASONRY STRUCTURES (SHEET 1 OF 3)

METROLINX

Drawing Number Dessin Numero: MM–02A

Rev.



ELEVATION: PIER OR ABUTMENT

PRECAST CONCRETE BLOCK

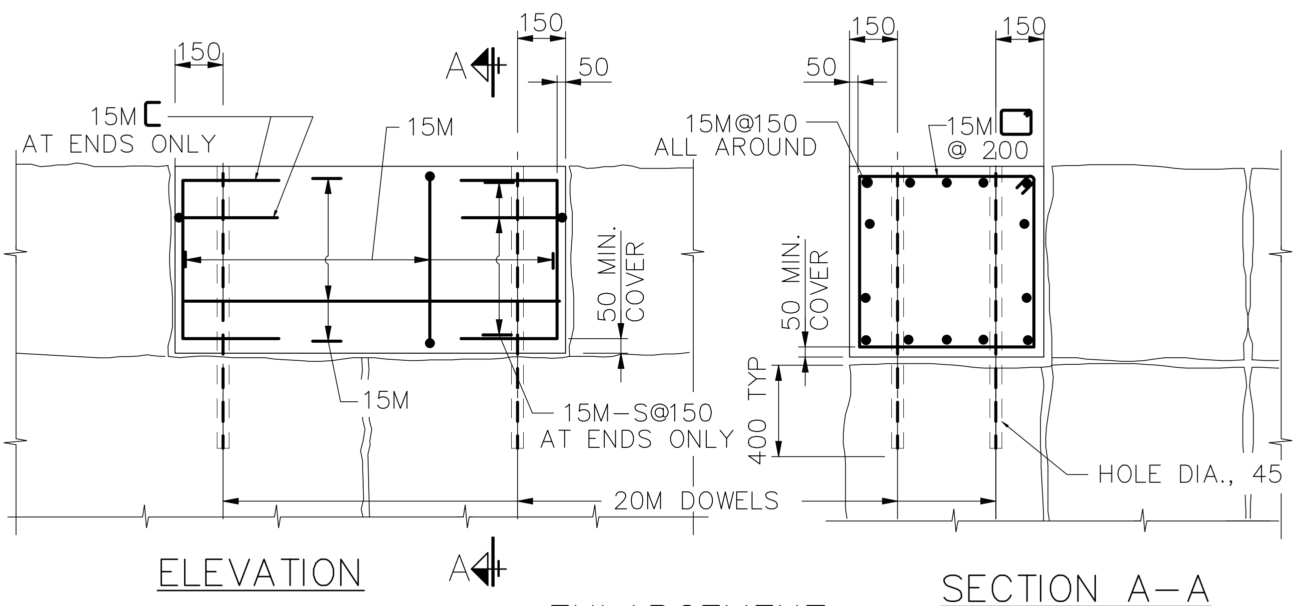
SPECIFICATIONS

CONCRETE
C.S.A. CAN3 A23.1 & A23.2
MINIMUM OF 30 MPa @ 28 DAYS.

REINFORCING STEEL
C.S.A. G30.18 – M92.
ALL BARS TO BE DEFORMED AND OF GRADE 400 MPa – 15M – MIN. LAP 400

COVER
MINIMUM COVER FOR MAIN REINFORCING STEEL TO BE 50 UNLESS NOTED OTHERWISE.

PROCEDURE:
- SUPPORT THE SPAN.
- REMOVE BROKEN STONE WITH A MAX. 15 POUNDS HAMMER.
- REMOVE EXIST. MORTAR.
- PLACE 75 X 75 STEEL SHIM PLATES TO PROVIDE A LEVEL SUPPORT FOR THE PRECAST CONC. BLOCK.
- GROUT SURFACE OF REMAINING STONE TO REQUIRED ELEVATION AS PER STD DWG MC-01.
- POSITION THE PRECAST CONCRETE BLOCK IN PLACE.
- PLACE RUBBER PAD.
- LOWER SPAN.
- RESUME TRAFFIC.
- DRILL HOLES FOR DOWELS AND FILL WITH NON-SHRINK GROUT.
- INJECT ALL JOINTS WITH NON-SHRINK GROUT.



ELEVATION

ENLARGEMENT

SECTION A-A

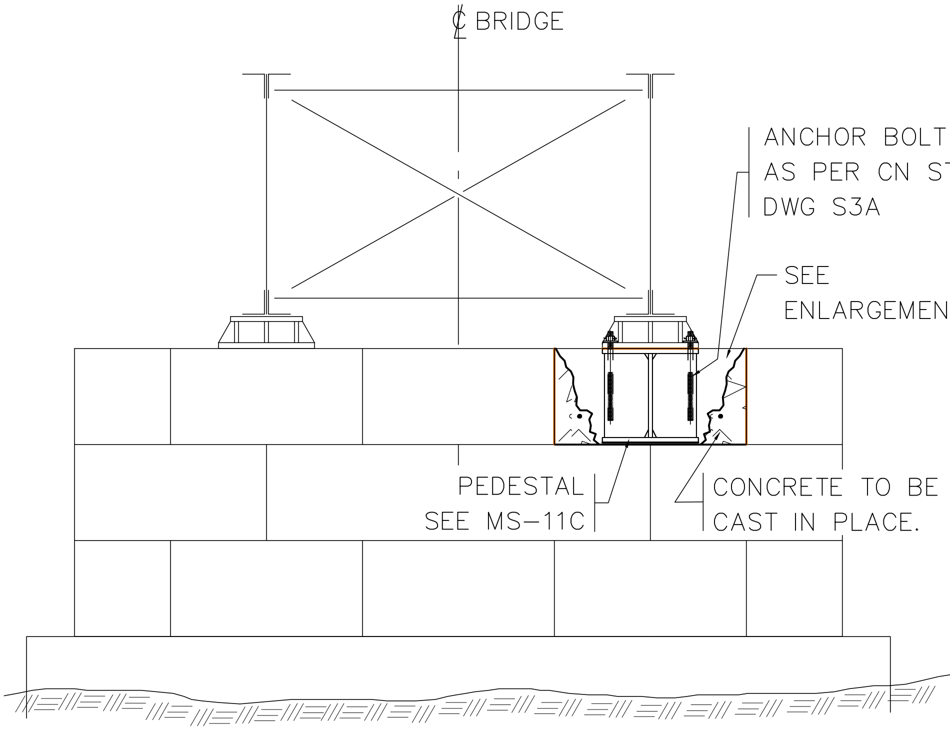
Drawn / Dessin	JE	Checked / Vérification	S.Kh.
Scale / Échelle	NONE	Date	March 2019

STANDARD FOR REPLACEMENT OF BEARING STONES IN MASONRY STRUCTURES (SHEET 2 OF 3)

METROLINX

Drawing Number / Dessin Numero: MM-02B

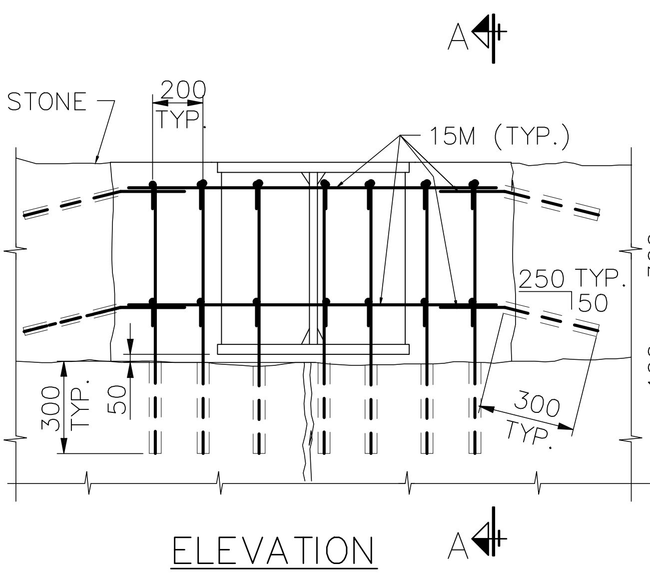
Rev.



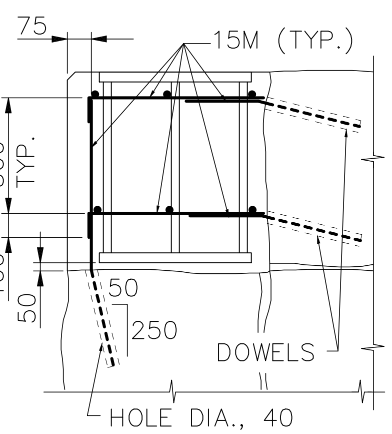
ELEVATION: PIER OR ABUTMENT

PEDESTAL AND CONCRETE

PROVIDE HOLES THROUGH
PEDESTAL FOR REBARS



ELEVATION



SECTION A–A

ENLARGEMENT

SPECIFICATIONS

CONCRETE
C.S.A. CAN3 A23.1 & A23.2
MINIMUM OF 30 MPa
@ 28 DAYS.

REINFORCING STEEL
C.S.A. G30.18 – M92. ALL
BARS TO BE DEFORMED
AND OF GRADE 400 MPa
– 15M – MIN. LAP 400

COVER
MINIMUM COVER FOR MAIN
REINFORCING STEEL
TO BE 50 UNLESS NOTED
OTHERWISE.

CHAMFER
20 FOR ALL EXPOSED EDGES.

PROCEDURE:
- SUPPORT THE SPAN.
- REMOVE BROKEN STONE WITH A MAX. 15 POUNDS HAMMER.
- REMOVE MORTAR.
- PLACE DOWELS WITH NON SHRINK GROUT.
- PLACE 75 X 75 SHIM PLATES TO PROVIDE A LEVEL SUPPORT FOR THE PEDESTAL.
- GROUT SURFACE OF REMAINING STONE TO REQUIRED ELEVATION AS PER STD DWG MC–01
- POSITION THE PEDESTAL IN PLACE.
- POSITION ANCHOR BOLTS.
- PLACE PLANKS AS SHOWN.
- RESUME TRAFFIC.
- SPEED RESTRICTION TO 25 MPH FOR 24 HOURS.
- PLACE REBARS AND POUR CONCRETE.
- CURE THE CONCRETE FOR 3 DAYS BY THE WET CURING PROCESS OR BY USING Sikagard CUREHARD CURING AGENT.

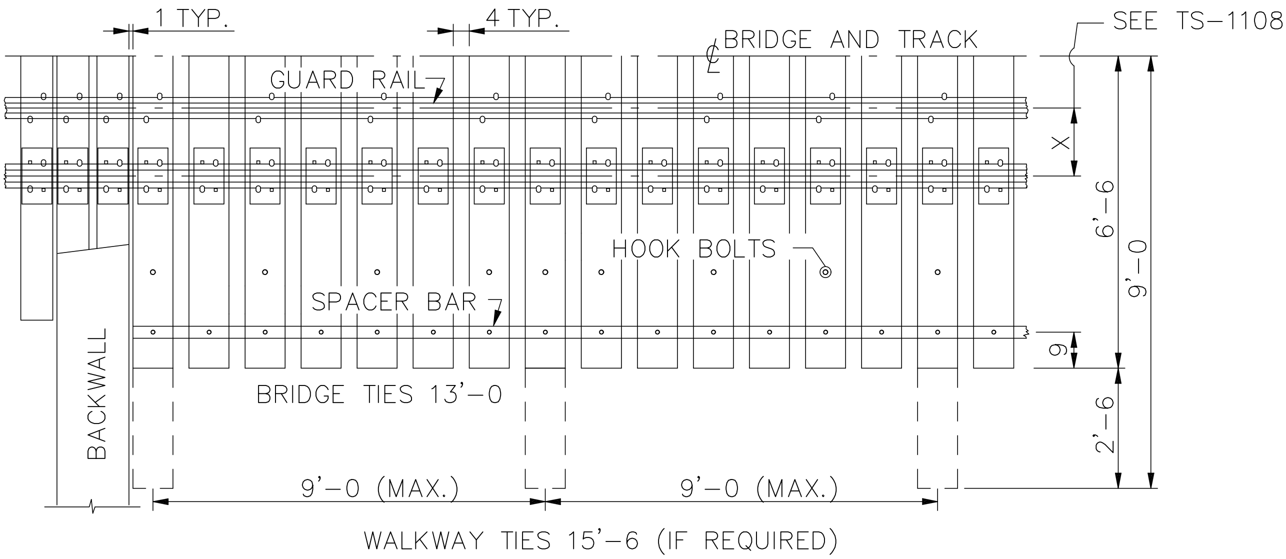
Drawn Dessin	JE	Checked Vérification	S.Kh.
Scale Échelle	NONE	Date	March 2019

STANDARD FOR REPLACEMENT OF BEARING STONES IN MASONRY STRUCTURES (SHEET 3 OF 3)

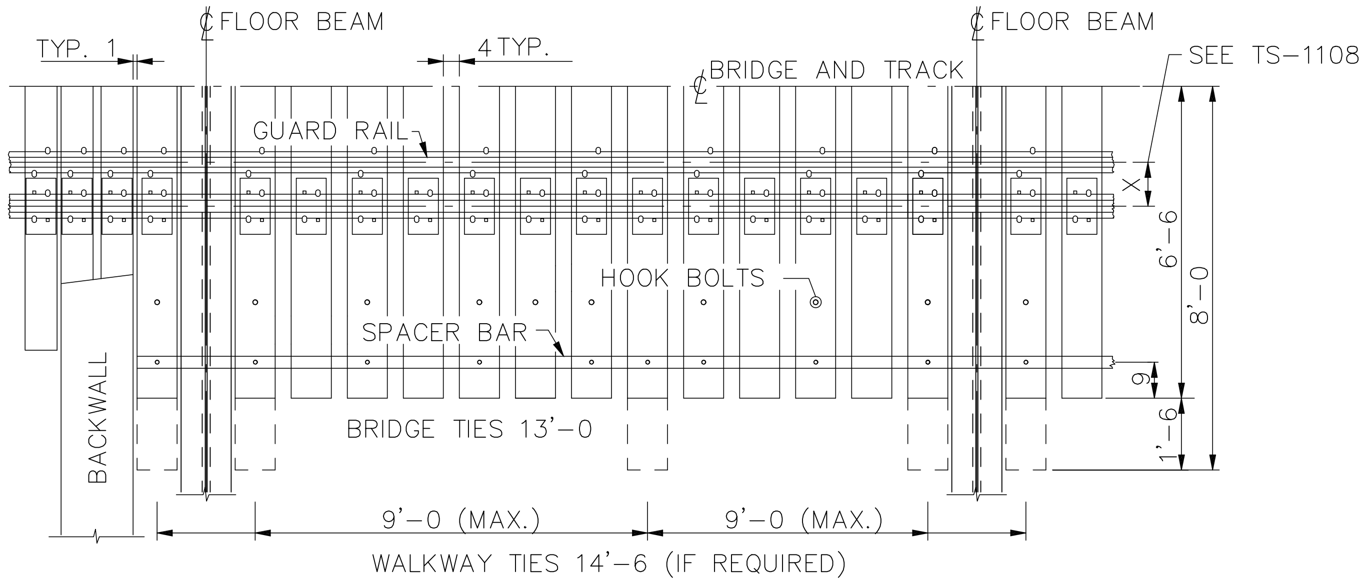
METROLINX

Drawing Number
Dessin Numero
MM–02C

Rev.



DECK BRIDGES



THROUGH BRIDGES

NOTES:
THIS DRAWING IS APPLICABLE
ON TANGENT TRACK ONLY

HOOK BOLTS:
EVERY SECOND TIE: SMOOTH TOP
EVERY FOURTH TIE: RIVETTED TOP
HOOK BOLTS REQUIRED IN ALL
WALKWAY TIES.

TIE SPACER BAR: ⅝" x 3"
WITH LAG SCREW ¾" DIA. x 6"

REFERENCE DRAWINGS:
R9A–1.3 AND R9A–1.4

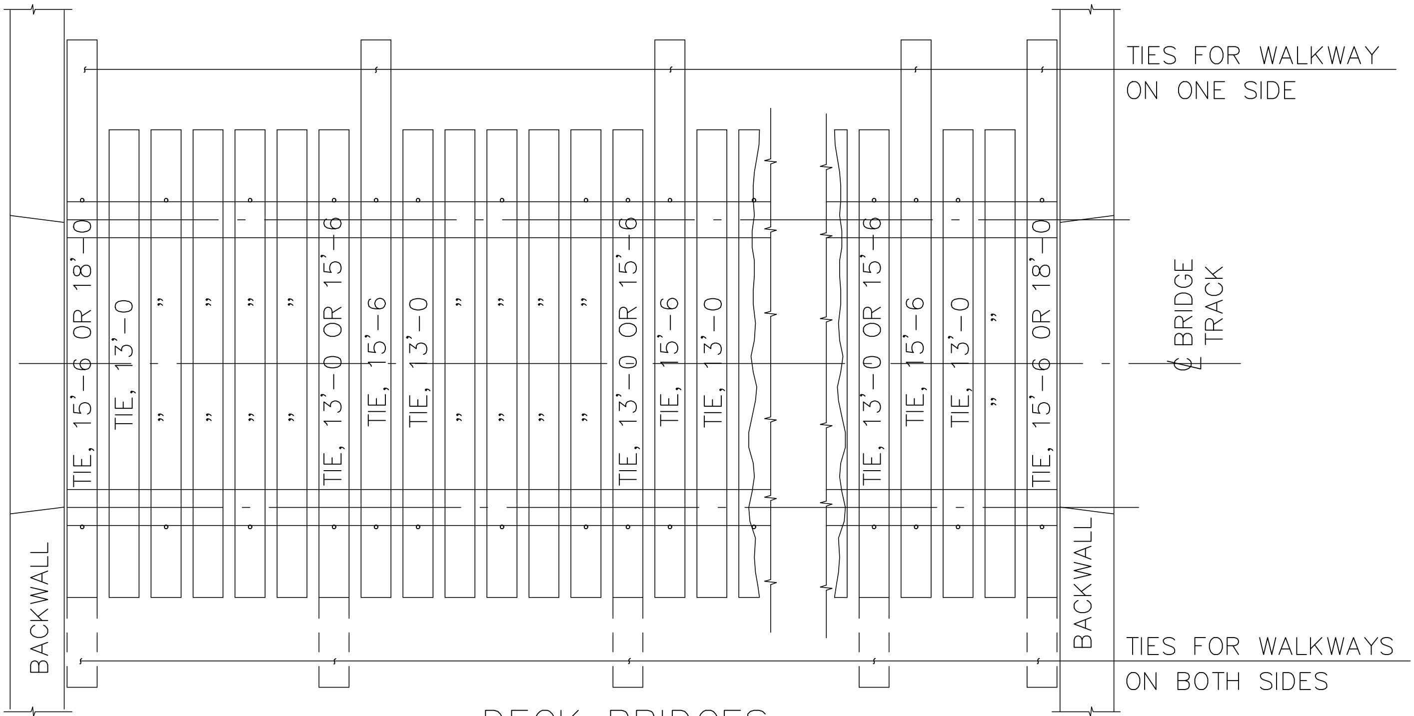
Drawn Dessin	JE	Checked Vérification	S.Kh.
Scale Échelle	¼"=1'–0	Date	March 2019

INSTALLATION OF TIMBER DECKS FOR STEEL BRIDGES

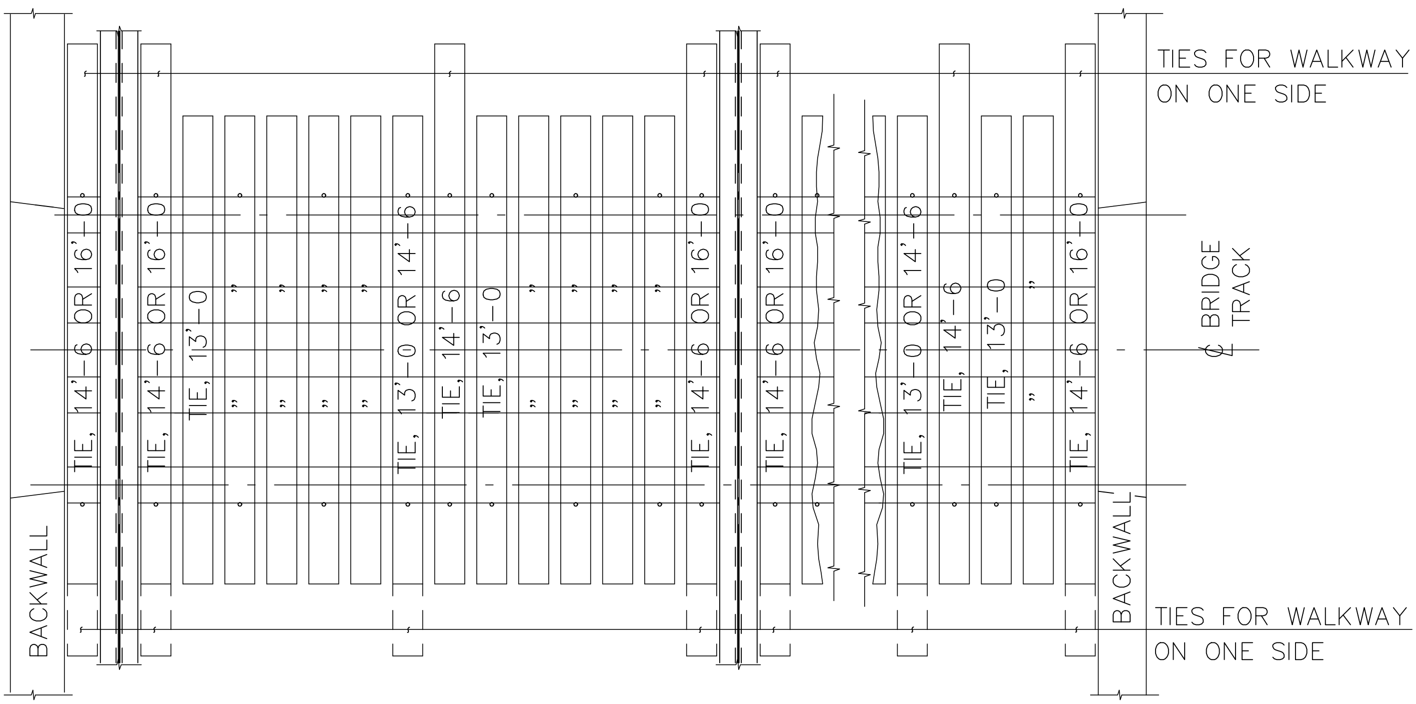
METROLINX

Drawing Number Dessin Numero: MD–01

Rev.



DECK BRIDGES



THROUGH BRIDGES

NOTE: FOR TIE SIZES AND CONNECTIONS, SEE DRAWINGS R9A-1.3 AND R9A-1.4

Drawn Dessin	JE	Checked Vérification	S.Kh.
Scale Échelle	N.T.S.	Date	March 2019

GEOMETRY OF TIMBER DECKS
FOR STEEL BRIDGES

METROLINX

Drawing Number
Dessin Numero
MD-02

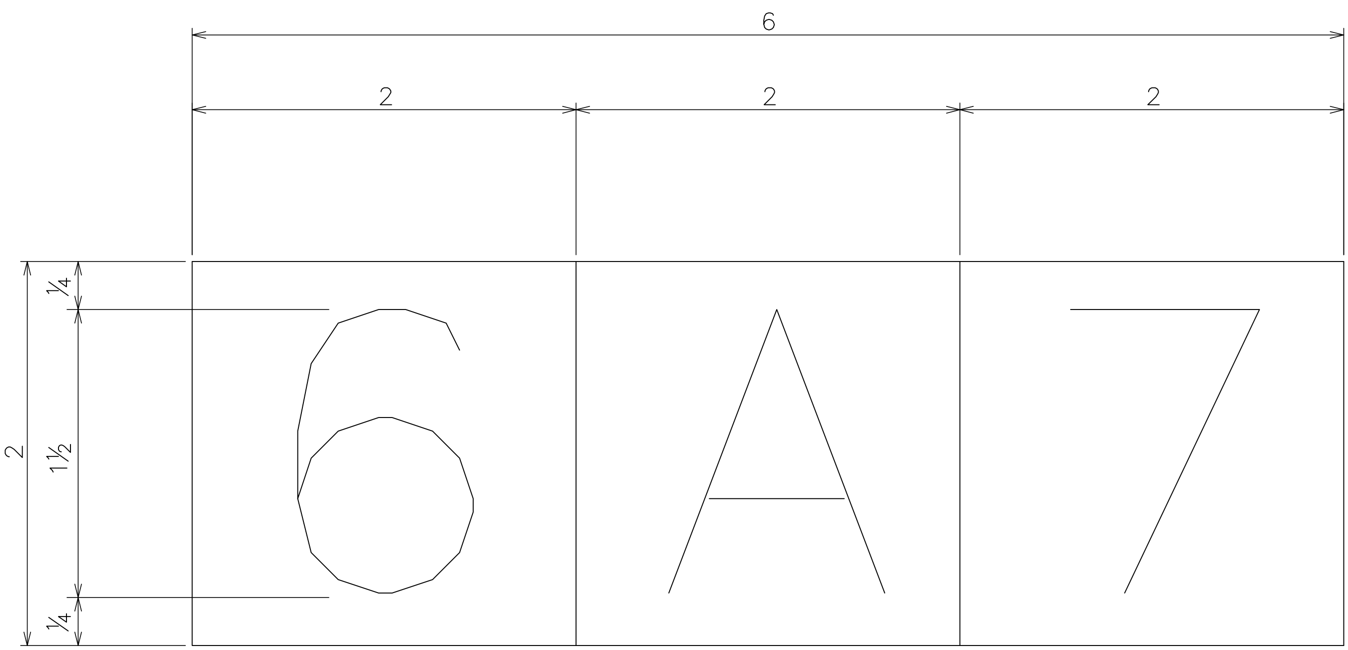
Rev.

LEGEND:

ERECTION MARK "6A7"

- 6 = 1996, YEAR OF THE TIE PROGRAM.
- A = FIRST BRIDGE OF THE PROGRAM FOR THE YEAR.
- 7 = TYPE OF DAP FOR THE SAME BRIDGE.

END OF TIE



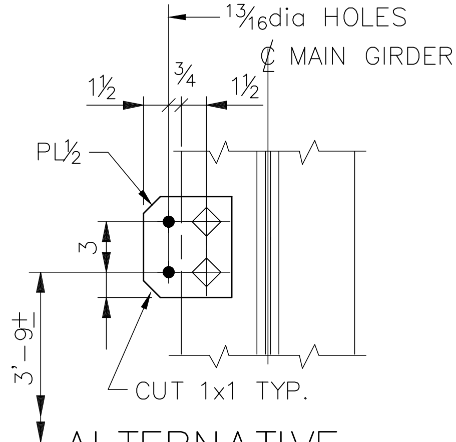
NOTES:

- EACH PLATE TO BE 2" X 2" AND GA. 32.
- THESE PLATES TO BE NAILED TO THE END OF EACH TIE.
- MARK SHALL BE SHOWN ON EACH PLATE AS INDICATED.

Drawn Dessin	JE	Checked Vérification	S.Kh.	IDENTIFICATION PLATES FOR BRIDGE TIES
Scale Échelle	N.T.S.	Date	March 2019	

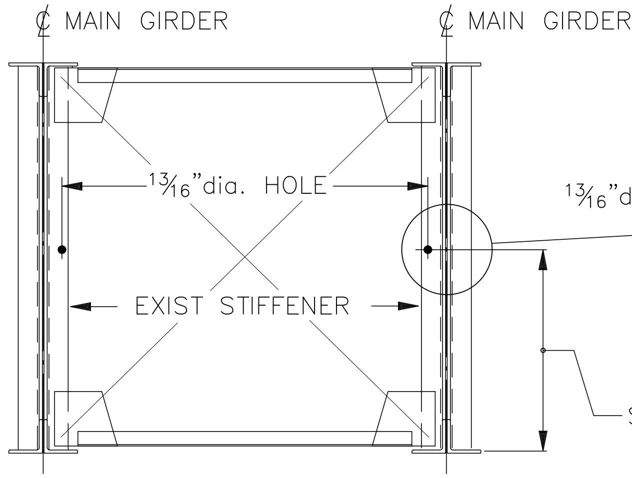
METROLINX

Drawing Number Dessin Numero	Rev.
MD-03	



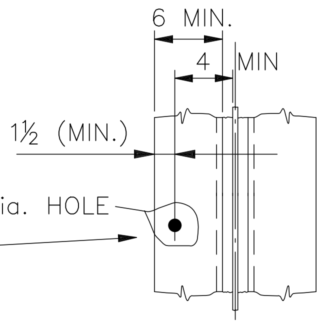
ALTERNATIVE

ON SMALLER STIFFENERS
(5" O.S.L. OR LESS)
TWO BAR HOLES ARE PROVIDED
IN PLATE FOR SPLICE LOCATIONS.
AT OTHER LOCATIONS, LEAVE
ONE HOLE OPEN.

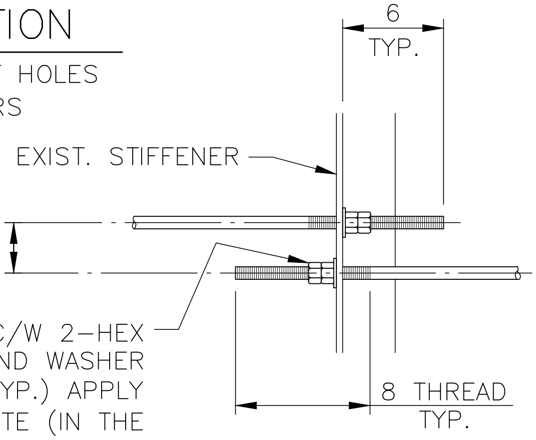


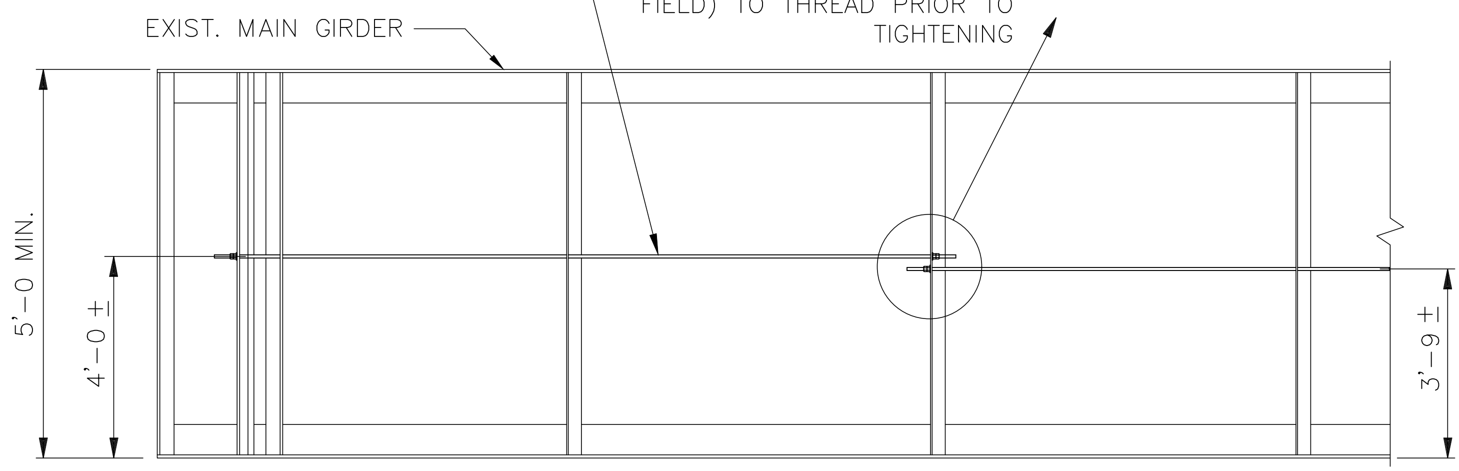
TYPICAL SECTION

SHOWING LOCATION OF HOLES
FOR SAFETY BARS



TYPICAL





TYPICAL ELEVATION

INSIDE

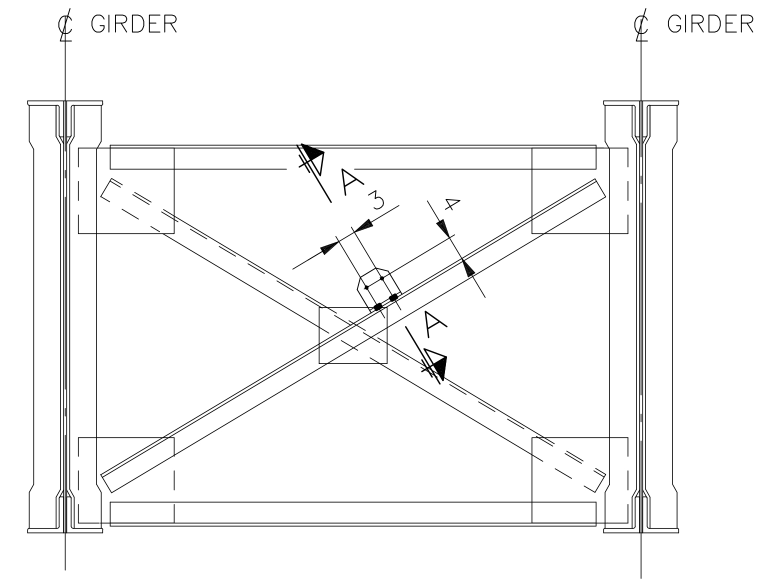
NOTES

— UNLESS SHOWN OTHERWISE, SAFETY BARS SHALL BE INSTALLED ON BOTH MAIN GIRDERS OF ALL SPANS.
— THE USE OF THE EQUIPMENT SHOWN ON THIS DRAWING SHALL BE IN ACCORDANCE WITH THE METROLINX FALL PROTECTION MANUAL.

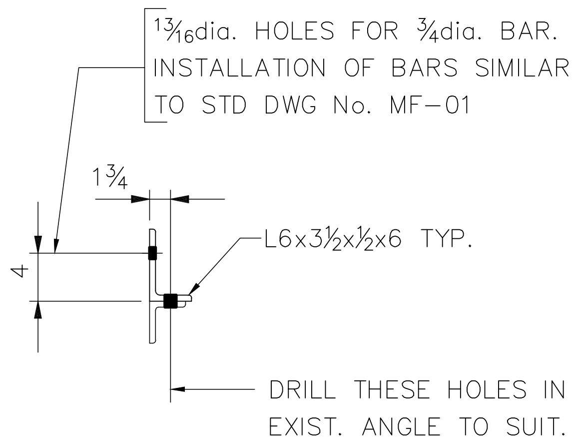
Drawn: Dessin: PS	Checked: Vérification: S.Kh.	SAFETY BARS FOR D.P.G. SPANS
Scale: Echelle: NONE	Date: March 2019	(ON STIFFENERS)

METROLINX

Drawing number
Dessin Numero
MF-01



TYPICAL SECTION AT BRACE FRAME



SECTION "A-A"

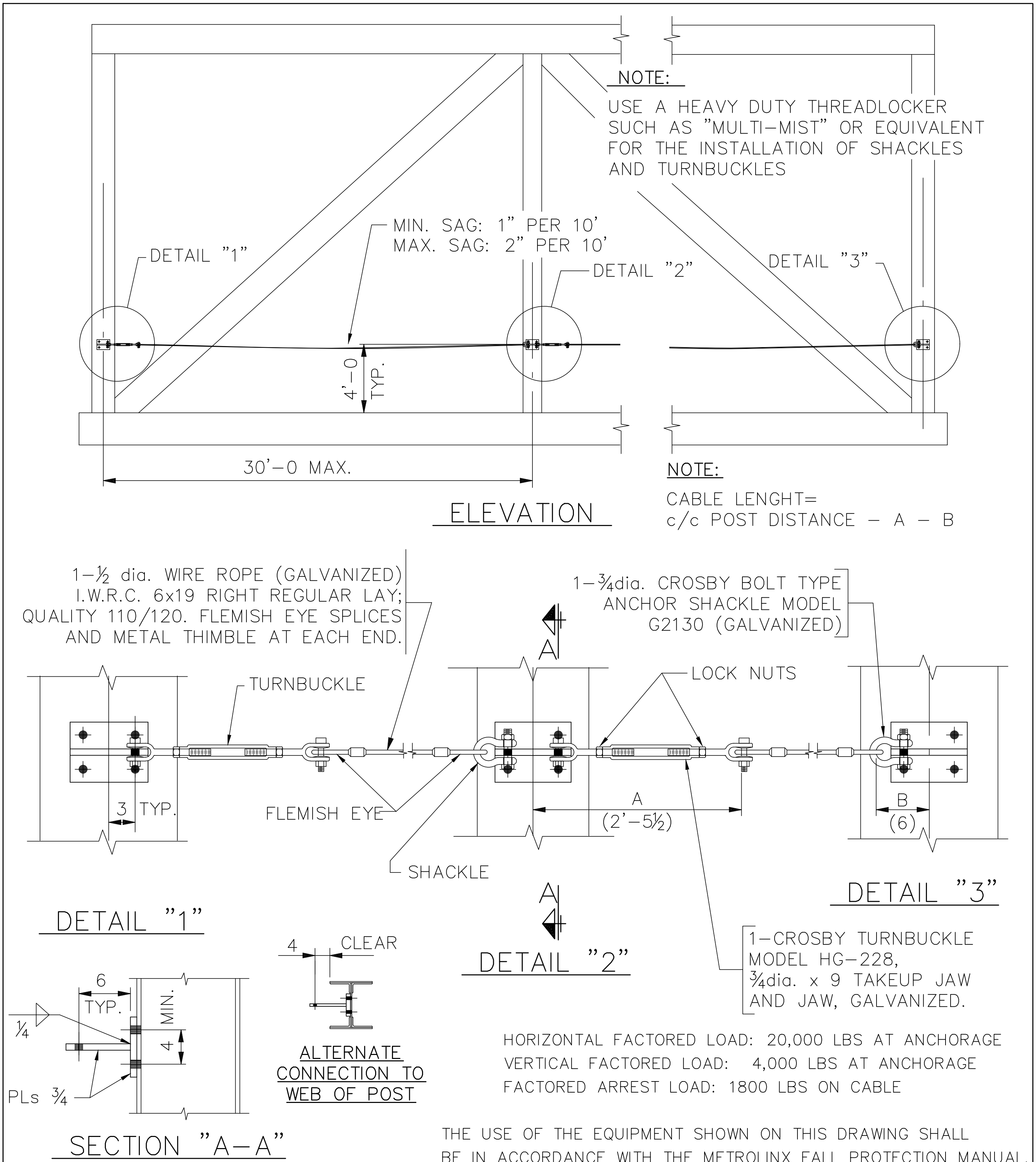
NOTES:

- SAFETY BARS SHALL BE INSTALLED ON BRACE FRAMES WHERE MAIN GIRDERS ARE SHALLOWER THAN 5'-0 AS SHOWN ON STD DWG No. MF-01.
- THIS SYSTEM SHOULD BE USED ONLY WHEN SPAN IS BRACED LONGITUDINALLY AT BOTTOM (TO PERMIT INSPECTORS TO WALK ON BRACING).
- THIS SYSTEM SHALL NOT BE USED IF DISTANCE BETWEEN BRACE FRAMES EXCEEDS 11'-0.
- THE USE OF THE EQUIPMENT SHOWN ON THIS DRAWING SHALL BE IN ACCORDANCE WITH THE METROLINX FALL PROTECTION MANUAL.

Drawn / Dessin	PS	Checked / Verification	S.Kh.	SAFETY BARS FOR D.P.G. SPANS (ON BRACE FRAMES)
Scale / Echelle	NONE	Date	March 2019	

METROLINX

Drawing Number / Dessin Numero	Rev.
MF-02	



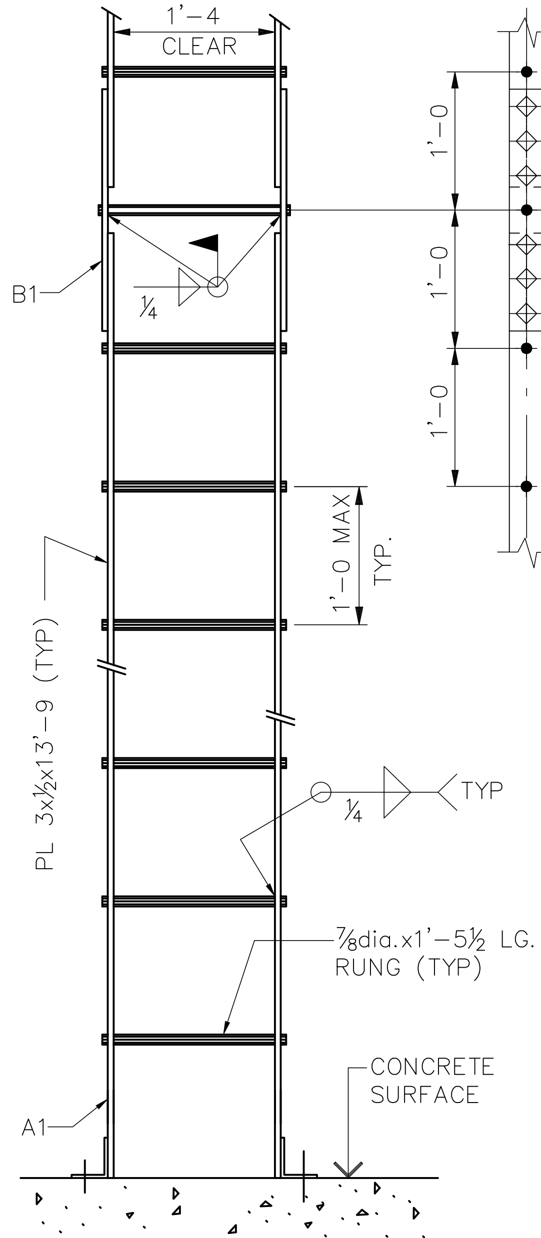
Drawn Dessin	PS	Checked Verification	S.Kh.	SAFETY CABLE – TRUSS SPANS
Scale Echelle	NONE	Date	March 2019	

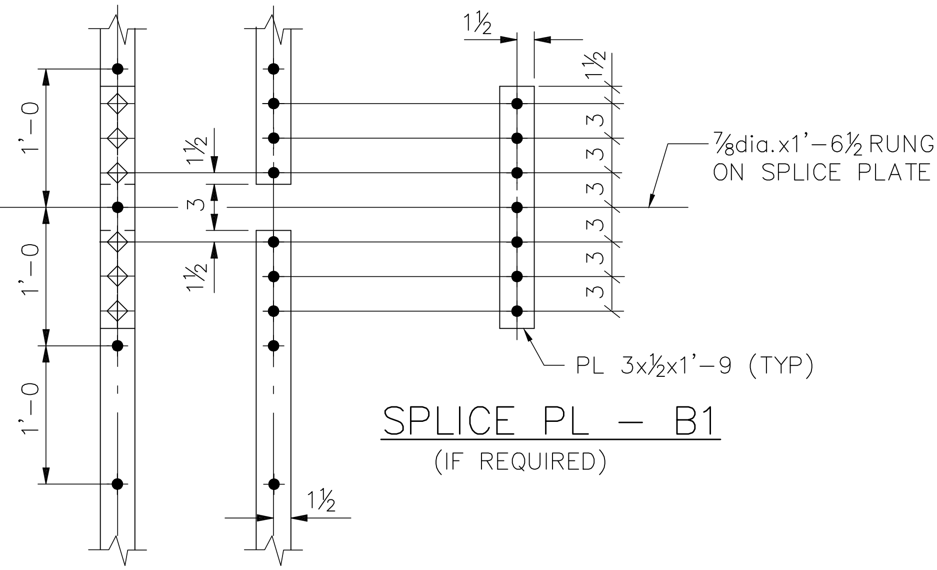
METROLINX

Drawing Number
Dessin Numero

MF–03

Rev.



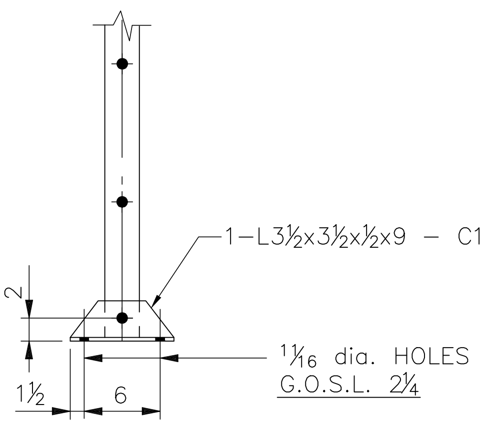


SPLICE PL – B1

(IF REQUIRED)

NOTES:

- MAKE PROVISION FOR SAFE ACCESS TO TOP OF LADDER
- SEE DWG No. MF–04B FOR METHOD OF FASTENING LADDER.



SPECIFICATIONS:

STEEL: CAN/CSA–G40.21, GRADE 300W
WELDING: CSA W59
HOLES: 15/16 dia.
BOLTS: 7/8 dia. ASTM A325
GALVANIZATION: CSA–G164

NOTE:

ALL STEEL TO BE HOT–DIP GALVANIZED ACCORDING TO CSA STANDARD G164

THE USE OF THE EQUIPMENT SHOWN ON THIS DRAWING SHALL BE IN ACCORDANCE WITH THE METROLINX FALL PROTECTION MANUAL.

Drawn Dessin	PS	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

TYPICAL LADDER

METROLINX

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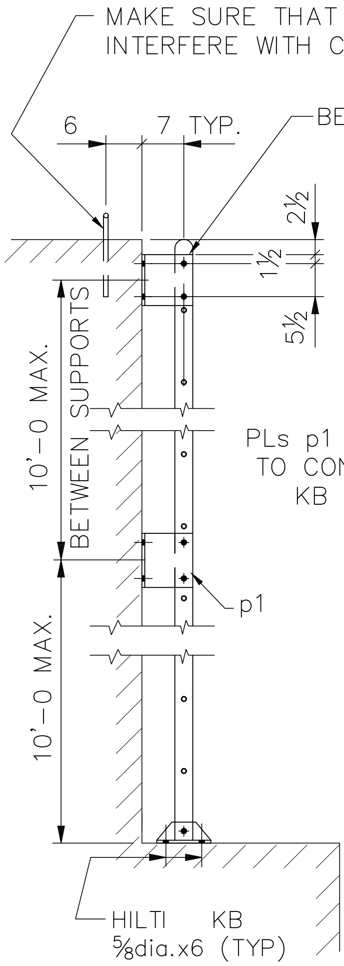
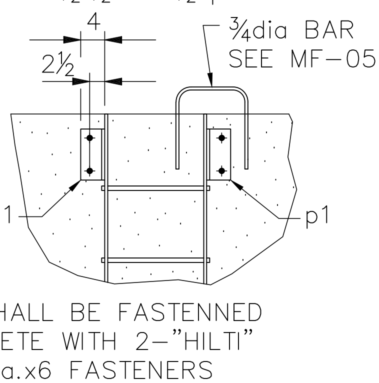
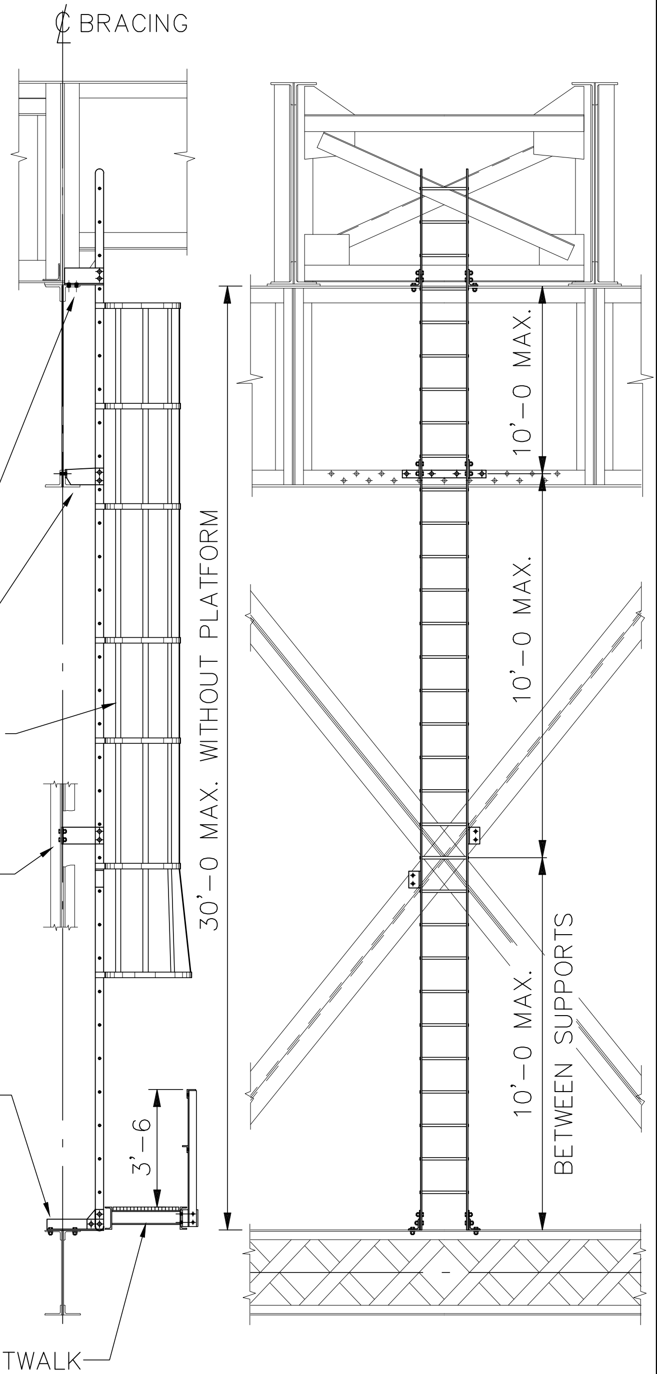
MF–04A	





TYPICAL LADDER

(AT ABUTMENT)



TYPICAL LADDER

(AT D.T. SPANS)

NOTES:

- FOR FABRICATION OF LADDER SEE STD DWG MF-04A
- LADDER SHALL BE CUT IF TOO LONG OR SPLICED USING STANDARD SPLICE BARS IF TOO SHORT
- THE USE OF THE EQUIPMENT SHOWN ON THIS DRAWING SHALL BE IN ACCORDANCE WITH THE METROLINX FALL PROTECTION MANUAL.

Drawn Dessin	PS	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

TYPICAL LADDERS

VARIOUS CONNECTIONS

METROLINX

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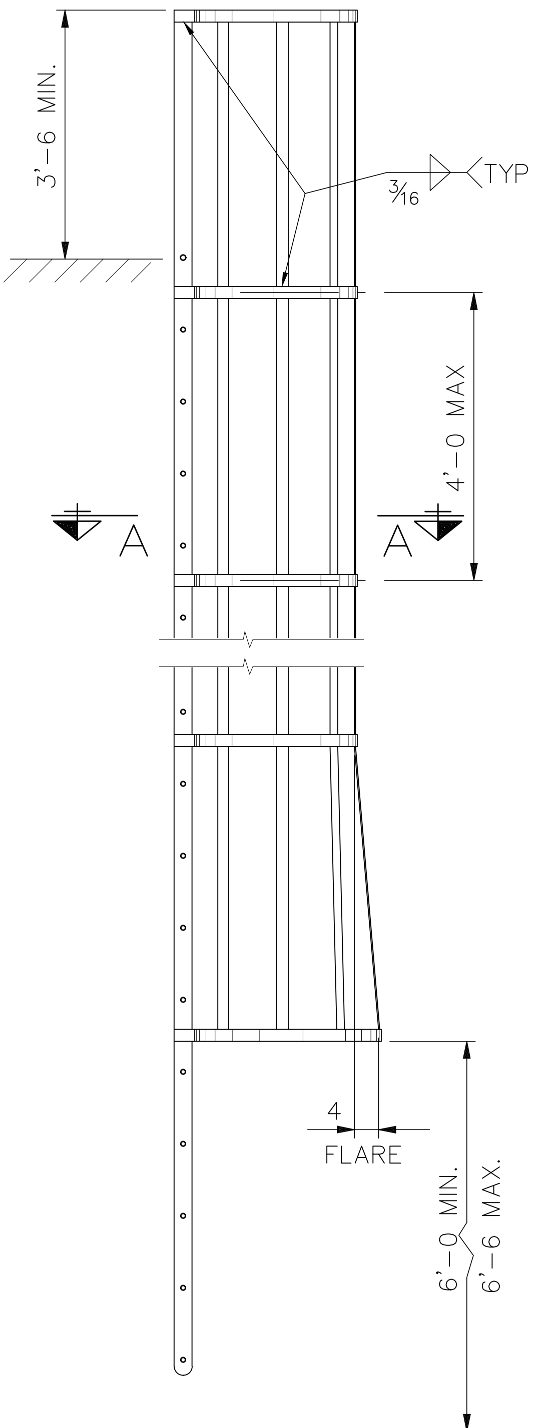
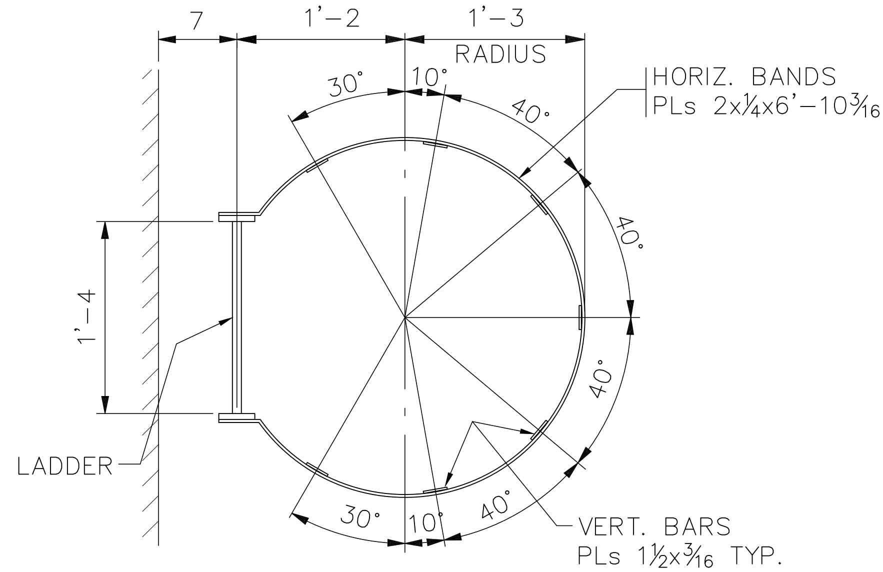
Rev.



TYPICAL LADDER CAGE



SECTION "A–A"

NOTES:

- FOR ADDITIONAL NOTES SEE STD DWG No. MF–04A
- THE USE OF THE EQUIPMENT SHOWN ON THIS DRAWING SHALL BE IN ACCORDANCE WITH THE METROLINX FALL PROTECTION MANUAL.
- A CAGE SHALL BE PROVIDED WHERE A SINGLE LENGTH OF CLIMB IS GREATER THAN 20'–0.
- A CAGE SHALL ALSO BE PROVIDED WHERE THE LENGTH OF CLIMB IS LESS THAN 20'–0, BUT THE TOP OF THE LADDER IS AT A GREATER DISTANCE THAN 20'–0 ABOVE GROUND LEVEL.
- SEE CANADA LABOR CODE AND ANSI A14.3–1984 (SAFETY REQUIREMENTS FOR FIXED LADDERS) FOR MORE SPECIFICATIONS.

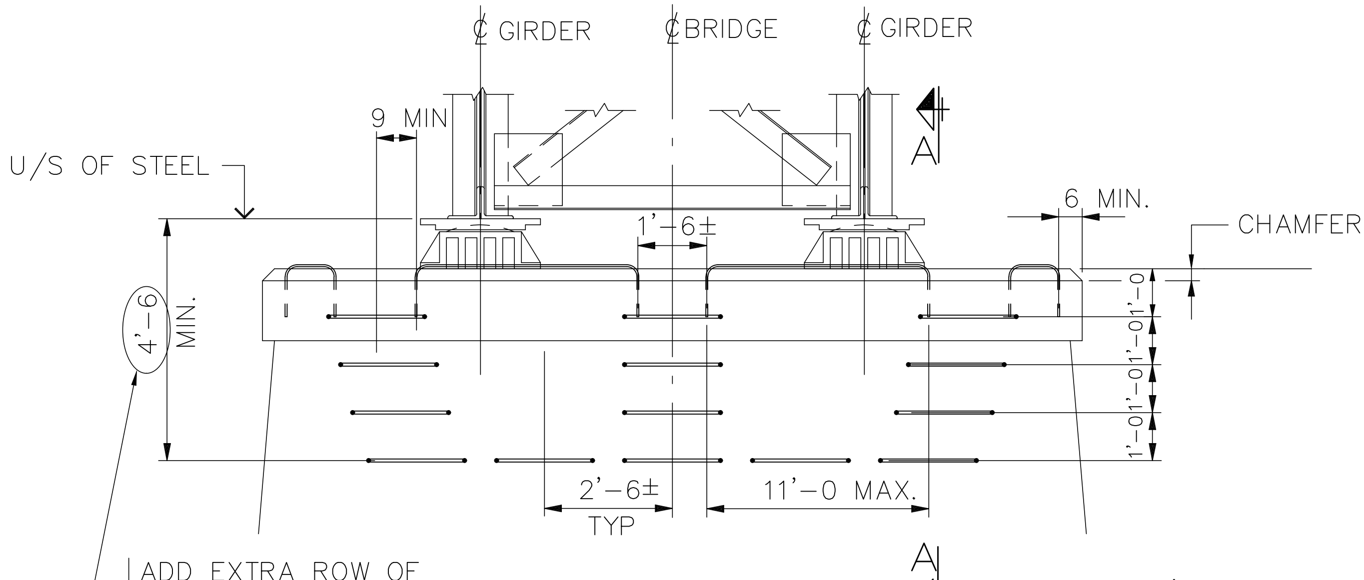
Drawn Dessin	PS	Checked Verification	S.Kh.
Scale Echelle	NONE	Date	March 2019

TYPICAL LADDER

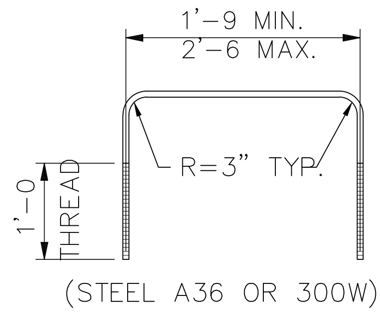
SECURITY CAGE

METROLINX

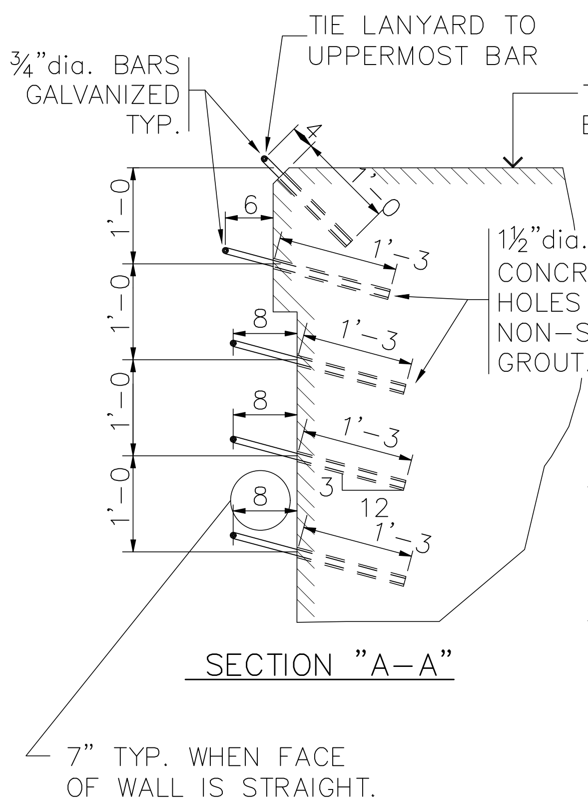
Drawing Number Dessin Numero	Rev.
MF–04C	



ELEVATION



TYPICAL DETAIL OF RUNG



SECTION "A-A"

NOTES:

- IN ORDER TO AVOID INTERFERENCE WITH FUTURE FIELD GANG OPERATIONS, PRECAUTIONS SHOULD BE TAKEN TO AVOID INSTALLING SAFETY BARS DIRECTLY ON BRIDGE SEAT.
- DURING ALL MOVES, THE USER SHALL BE TIED TO THE UPPERMOST BAR AS SHOWN ON SECTION "A-A" OR TO AN APPROVED ANCHOR POINT.
- THE USE OF THE EQUIPMENT SHOWN ON THIS DRAWING SHALL BE IN ACCORDANCE WITH THE METROLINX FALL PROTECTION MANUAL.
- ALTERNATIVELY, RODS CAN BE SET IN 1 INCH HOLES USING "HILTI HIT-HY 200-R".

Drawn / Dessin	PS	Checked / Verification	S.Kh.	LADDER RUNGS ON PIERS AND ABUTMENTS
Scale / Echelle	NONE	Date	March 2019	

METROLINX

Drawing Number / Dessin Numero	Rev.
MF-05	