TULLOCH

Dean Lake Bridge

Steel Coupon and Concrete Core Testing Results For Rehabilitation Planning

Municipality of Huron Shore

Dean Lake Bridge, Iron Bridge, ON

April 19, 2024

April 19, 2024
23-0862-200

Municipality of Huron Shores
7 Bridge Street,
Iron Bridge, ON
P0R 1H0

Attention: Ms. Natashia Roberts, CAO/Clerk

> Re: Dean Lake Bridge – Steel Coupon Testing and Coring Results with Recommendations

Dear Mrs. Roberts,

TULLOCH Engineering (TULLOCH) is providing you with this report of the steel coupon testing and concrete coring results to give additional information for use in any future planning for reconstruction or rehabilitation of the Dean Lake Bridge.

This report also outlines some of the history of the bridge known to TULLOCH, and also provides a recommendation on the next planning steps necessary to make an informed decision on the repairs or replacement of this bridge. TULLOCH is recommending (1) a rehabilitation feasibility study, (2) another detailed visual inspection and (3) the initiation of a current bridge monitoring survey.

The new monitoring survey should attempt to tie-in old survey data however, given the gap between data collection it will be very difficult to assess current survey information to older surveys without some level of uncertainty. The monitoring survey will provide assurance that previously noted movements to the bridge abutments are not continuing, otherwise, additional investigation works would be necessary to determine feasible foundation repairs that could be implemented at the site. The extent and requirement for additional repairs is important for any rehabilitation planning of this bridge site.

1. PROJECT SUMMARY

Steel coupons were extracted from the bridge while repairs were being completed by a contractor. To save mobilization costs during the repairs, TULLOCH recommended that steel coupons be extracted from the bridge while the repair contractor was onsite. These coupons were taken and sent to a third-party material testing agency for chemical analysis and strength testing. The intent of the steel testing was to develop a site-specific material strength that could be used in analyzing and determining the working capacity of the bridge trusses and individual truss elements. This information was necessary to develop a working yield strength for the truss elements in accordance with the Canadian Highway Bridge Design Code (CHBDC) This would confirm if previous assumed strengths were comparable to determine load capacity and help develop any potential rehabilitation plans for the structure.

Previous bridge inspection records by TULLOCH outlined that there were some localized steel repairs required. These repairs were isolated to end sections of the exterior stringer beams below the bridge deck at the north and south ends of the bridge. There was also a single weld that required repairs where an exterior stringer rested on a cross transom beam. The weld holding the stringer beams onto the shim plates/bearing plates over the transom beam was cracked. These repairs were recommended by TULLOCH from the biennial OSIM inspections.

2. HISTORICAL INFORMATION & PREVIOUS REPORTS

The following is a short historical summary of previous reports and information provided to TULLOCH from old records, discussions with Municipality personnel and TULLOCH biennial inspections. The bridge orientation taken from the TULLOCH reports differs from some of the historical documents as the bridge is on a slight skew. For the purposes of this report, the bridge is generally running in a north-south direction. The north end of the bridge is that closest to highway 17E and the south end of the bridge is the abutment closest to Dean Lake (south side of the Mississauga River).

2.1. Historical Information

The bridge was constructed in 1908 by the province as an access point for the purposes of construction of the provincial railway system. The bridge had previously been inspected and maintained by the Ministry of Transportation of Ontario (MTO) prior to the 1990’s.

Based upon the available information, the following is a summary of repairs known to have been completed over the life of the structure:

- 1908: the bridge is completed and opened to traffic. The original bridge design drawings indicate that the bridge and trusses were designed for 1400 lbs per liner foot of dead load and 1400 lbs per linear foot live load total. The live load is equivalent to an approximate uniform deck live loading of 4.8 kPa (100 psf). This is for reference only, as truck axle loads create local stresses on the deck and structure depending on the truck placement.

- Circa 1915: there is some undermining issues on the north abutment. A drawing by the Ontario Public Works at the time illustrated the repair. This drawing outlined a false cribbing work to be installed outboard of the existing abutment face. The purpose was to protect the foundation from scour and undermining, and to protect and in-fill the undermining that had already occurred since the bridge opened.

- There is limited documentation between 1915 and the 1960's. It is assumed however, that the timber decking would likely have been replaced every 15-20 years. The timber decking likely would have matched the old decking in-kind.

- Circa 1963: there was a drawing package which outlined that the 9" deep I-beam interior stringers and the 9" deep channel external stringers were replaced with 8" W-beams and 8" channels respectively. There was to be a new 75mm (3") creosote coated timber deck installed with new wooden curb. This outlines that there were some minor dimensional (height) changes completed to the floor framing/stringers but the elevation makeup would be eliminated with steel shim plates as required. All bridge steel was to be sandblasted and re-painted.

- At some time (assumed pre 1988) there was steel sheet piling installed around the footings of all abutments and piers. The infill distance between the backside of the new steel sheet piling and face of the original foundations was infilled with concrete. The depth of the sheet piling and any reinforcement details are unknown.

- Circa 1988: Kresin Engineering provided repair drawings that outlined the existing timber deck including the surface treatment was to be removed and replaced with another transverse nail laminated timber deck and new timber curb. The new timber decking was called out to be a 38mm x140mm laminated creosote timber deck. It should be noted that the original seven (7) stringer beams were shown spaced equally across the width of the decking. A loading note on these drawings indicated that the bridge was to remain posted with a 16 / 24 / 28 tonne load posting as per the Ontario Highway Bridge Design Code 1983. In addition to the decking replacement, the drawings also showed the following repairs:

    - The exterior channel stringers were removed and replaced with new W200x31 steel stringers. The inner bottom chord of the trusses at the north end and south ends of the bridge were partially replaced.

    - Temporary end bracing was shown installed within the trusses to facilitate jacking and support the bridge as the repairs to the abutment bearing seats and ballast walls were completed. This indicated that a lifting plan developed by a Contractor and approved by Kresin Engineering was necessary for the replacement of the bearings. Several bearings at the intermediate piers were also shown as being replaced.

    - Concrete repairs - new bearing seat pedestals and new ballast walls complete with new armouring angles were to be completed. At the piers, any change in elevations was shown to be made up with grout pads below bearing plates under each stringer.

    - Some pipe handrails and miscellaneous flex beam guiderail repairs on the bridge, along with replacement of the approach guiderails, were also shown to be completed.

- In 2004: M.R. Wright and Associates tendered the replacement of 30m +/- of nail laminated decking on the bridge, and replacement was to include new steel clips to tie down the decking to the stringers. There was a provisional item for replacement of the entire deck. Assuming that overall construction pricing was too high, none of the decking replacement was completed and postponed until a later date.

- Between 2004 and 2010: there was concerns raised about overall movement of the bridge from previous correspondence and uncertainty when survey monitoring occurred. It appears that the concern was in regards to general movements of the bridge abutments only. M.R. Wright and Associates was retained by the Municipality to review and oversee a bridge survey which was completed by M.F. Tulloch Surveying. The surveying records we were able to find are provided in Appendix E of this report.
    - From this information: the original baseline survey was setup by others (Henderson Paddon) in 1988 at the time of the rehabilitation. Later on, M.F. Tulloch Surveying was retained by either the Municipality or M.R. Wright and Associates to complete surveying checks on the bridge. These select survey checks were completed in 1989, 1996 and in 2004). No further monitoring was known beyond 2004, and the planning for the replacement of the bridge decking and re-painting of the bridge steel continued. It was noted that survey measurements and benchmarks are affected by frost and physical disturbances throughout the years, and can give inaccurate results.

- Around the same time, M.R. Wright and Associates advised to get the decking replaced on the bridge along with cleaning and repainting of the structural steel. Due to assumed overly high project costs, a new proprietary decking system proposed in October 2007. The new decking system is comprised of fiber glass wrapped-wooden deck planks which are infused with epoxy resin. This system is what currently resides on the bridge today. A description of the decking system is as follows:
    - There appears to be an ambiguity from the design drawings as to whether the deck panel system was to have a gravel topping embedded into the top of the exposed fiber glass decking surface or, if there is supposed to be a 20mm thick wearing surface installed. The panels which are onsite have the 20-25mm (3/4 - 1”) +/- thick asphalt type wearing surface.
    - The design, testing and installation of the new deck panels was proposed and completed by the Newton Group. The engineers responsible for the design and

  oversight of the testing to confirm compliance with the Bridge Code and the design parameters was completed by I & F Engineering Corp.

  - Two panels were built and tested in a facility near Guelph Ontario, and the results were documented and deemed acceptable by I & F Engineering Corp, which verified the panels performed to the design intent. However, the wearing surface on the first two panels of the bridge (starting at the north end) had surface defects and issues since being installed at the bridge site. Full depth removal of the defects, primarily in the wheel paths, was completed and replaced with new asphalt material from an Elliot Lake source. It is possible that the two tested panels were installed on the bridge.

  - The deck panel was not designed for a full truck load of the CHBDC S6-06 at the time. The I & F Engineering Corp. drawings indicate that the design was for the 16 / 24 / 28 tonne vehicle loadings. This loading restriction was common with the 1988 rehabilitation drawings and M. R. Wright & Associates documents from 2004.

- In 2019: TULLOCH completed a detailed inspection of the bridge utilizing rope access and maintenance (RAM) crews to get a much better visual review of areas of the bridge that are not readily visible from the abutments and bridge deck. Moderate to severe corrosion areas were noted along the bottom chords, gusset plates and connections nodes below the bridge deck. The decking panels were also visually inspected, and select locations cored. It was found that the two northern deck panels were retaining water.

  - TULLOCH used this field data to estimate the strength of the structural elements and complete a load evaluation of the bridge in its current state. The load evaluation followed the CHBDC S6-14. Serviceability limit states (bridge deflections) and fatigue limit states were not considered in our general load evaluation review.

  - Some assumptions had to be made regarding the bridge deck panels because there is no readily available information on resistances of epoxy infused fiber glass wrapped wooden deck panels. It was assumed that the bridge deck panels were only designed for a maximum 16-tonne single unit vehicle as per the I & F Engineering Corp drawings/documents. When completing our review of the deck panel designs, the testing was completed with only the two (2) interior stringers in place and was assumed to have composite resistance. Conservatively, we assumed the exterior stringers did not provide a composite resistance with the fiber glass deck panels from field bolted connections.

  - The floor system was reviewed to see if the current decking and floor framing could carry the full loadings of the current CHBDC CL-625-ONT truck. The load evaluation determined the current exterior beams in the framing (the floor framing was changed from 7 stingers to 4 stringers) were a potential weak area. The two (2) interior stringers were built as a composite member with the bridge deck and the exterior girders were assumed to be changed W200x31 beams per the 1988 rehabilitation drawings. We found that the current exterior stringers cannot support the full CHBDC design truck. The load evaluation results from 2019 recommended that a load posting of 10 / 17 / 24 tonnes for the single unit, double unit and vehicle train, respectively. To help alleviate the stresses and loading potential on the historic bridge and prolong the remaining service life, the bridge was posted with a single maximum 10 tonne load limit.

    - The abutments and piers were identified as requiring major rehabilitation repairs. The exterior surface of the bridge abutments and piers has significant delaminations, efflorescence staining and wide-spread map cracking. The general concrete conditions were more severe at the abutments, and more moderate along the interior piers. The detailed visual inspection supported these previous observations and outlined that there are some wide cracking and delaminations throughout the piers. The detailed inspection from 2019 supported that significant concrete repairs are required for any long term continued use of this bridge. Concrete core samples were taken and the results are presented in section 4 of this report. Although the concrete abutments and piers are mass concrete, the outer layer of concrete must be restored to provide protection to the inner portion of the abutments and piers.

- In 2023: TULLOCH advised from the 2023 biennial bridge inspection that steel coupon testing, concrete core extraction, testing, and localized steel repairs to the steel stringers be completed. There were localized areas of steel corrosion with perforations to the stringers at the abutment bearing seats, and a single bearing plate connection weld was noted to have a crack. These items were repaired in November 2023.

- As part of the information herein, steel coupon samples of the bridge truss batten plates were taken and tested by Acuren Group (a third-party material testing company), and the results are summarized in Section 3.

- Recently, in order to restrict any larger vehicles on the bridge, the Municipality has installed a height restriction bar on the ends of the bridge. This was completed because during the previous winter (2022/2023) during a highway closure, large transports (fully loaded) were crossing the bridge. In addition to overloading the Municipal bridge they were getting stuck on the narrow local roadway known as Chevis Road as it connects back to Dayton Road and ultimately back to Highway 17E. These trucks were damaging the roadway and causing flow restrictions to the only by-pass road that exists for Highway 17E between Dean Lake Road and Iron Bridge, ON.

3. STEEL COUPON TESTING

In 2023, TULLOCH advised that steel coupon testing be completed to gather more information, as to determine the in-situ strengths of the truss steel, and to confirm the steel grade and weldability. The results would confirm the previously assumed steel strengths. The steel strength results could also be used to re-check if strengthening of specific truss elements would be achievable and a practical solution to the bridge without the need for a load restriction.

Steel Speed Inc. extracted the steel coupon specimens from the Dean Lake Bridge under the supervision of TULLOCH while they were onsite to complete some localized steel repairs. The coupon specimens were extracted from the lattice batten plates along the underside of the top chord. For the ease of access, these specimens were taken about 1.5m (5ft) above the deck surface at the handrail elevation. An equivalent sized plate with structural bolts was reinstalled at each of the extraction locations. A total of 12 samples were taken, one from each quadrant of each truss span.

The coupons were labelled E1 thru E6 and W1 thru W6. The “E” represents the specimen was located in the east truss and the “W” represents that the specimen was located on the west truss. The numbering system started at the south end of the bridge and the samples were taken from both trusses in all three spans and the progress moved northward across the bridge. The sampling locations are depicted on a sampling sketch – Drawing ‘S2’ provided in Appendix D of this report. Table 1 lists all of the samples.

Table 1 - Steel Specimen Sample Location

Specimen ID	Truss on the Overall Bridge Layout	Location on Dean Lake Bridge
E1	East Truss	South end of the south span
E2	East Truss	North end of the south span
W1	West Truss	South end of the south span
W2	West Truss	North end of the south span
E3	East Truss	South end of the middle span
E4	East Truss	North end of the middle span
W3	West Truss	South end of the middle span
W4	West Truss	North end of the middle span
E5	East Truss	South end of the north span
E6	East Truss	North end of the north span
W5	West Truss	South end of the north span
W6	West Truss	North end of the north span

3.1. Coupon Results

TULLOCH retained Acuren Group Inc. to complete third party chemical analysis of the specimens and mechanical testing to develop yield strengths, ultimate strengths, elongation properties, and weldability. Below is a summary of the yield strengths for the entire sample results.

Item	Results
Number of Samples	12
Average Yield Strength (MPa)	281
Standard Deviation	32
Calc Factor ks – Table A14.1.1	1.24
Equivalent Strength from Test Results per Bridge Code (MPa) for Evaluation	211

3.2. Discussion

It should be noted that without any testing results, the Bridge Code assigns assumed values for steel strengths based on date of constructions. The previous load evaluation for this bridge by TULLOCH originally assumed a steel yield strength of 210MPa. The steel coupon testing results confirmed that the strength value used previously is accurate. The results show that there is no additional reserve capacity or redundant safety factor built into the resistances of the members to allow a higher quality steel at this particular bridge site.

The steel testing indicates that the steel from the trusses is a weldable material. TULLOCH would advise that the type of welding repairs and the associated welding procedures would need to be developed by a certified welding Engineer. Also, during the field repairs, material testing procedures and third-party welding inspection services would be required. However, welding repairs to main structural elements of the truss are not desirable given the additional work and avoiding fillet welds which may cause fatigue cracking.

Given all of the above, TULLOCH would advise that any structural repairs should utilize bolted connections, unless welding is absolutely necessary during potential repairs or upgrades to the truss elements.

A summary of these results from the Acuren Group material testing and the calculation for the yield strength permissible by the CHBDC are provided in Appendix B of this report.

4. CONCRETE CORING RESULTS

Along with the steel samples, TULLOCH utilized the same general contractor to conduct concrete coring of the abutments and piers to gather a better understanding of the depths of the delaminations and wide cracking. The coring locations were chosen in locations that were relatively easy to access at the base of the abutments and piers. This allowed the contractor to have a flat surface (concrete footings) to stand on while operating the core drill. All holes from the sampling were infilled with CPD non-shrink grout.

Two samples each were taken of the north abutment, north pier, south pier and south abutment, for a total of 8 concrete core samples. The concrete samples were extracted 300mm (12”) in depth to determine if the delamination and weak concrete extended below the surface delaminations, and to give a better representation of the actual depth of concrete removal that would be required in a rehabilitation of the concrete abutments and piers.

The concrete cores were tested in TULLOCH’s Sault Ste. Marie Testing Facility. A summary of the compressive strength testing results are provided in Appendix C of this report. The minimum compressive strength obtained for the concrete core samples was 15MPa and the high end of the compressive strength results was 29 MPa, with an average strength of 22 MPa. There were two (2) of the samples that did not have adequate length remaining when the cores were extracted from the drill to meet the minimum length to diameter (L/D) ratio for testing. Those samples were either a large pile of broken segments or a few smaller broken cores within the overall 300mm length. It appears that the first third-segment of the concrete was soft, and essentially crumbled during the coring procedure. Also, internal cracking within the core likely intercepted and upon removal of the core from the drill casing, the core broke into small length segments. The core samples were screened for overall lengths that would allow testing to be completed and in one instance, a sample was intentionally cut into two (2) samples to get additional testing results.

4.1. Concrete Rehabilitation Discussions

All the concrete samples were cored to a depth of approximately 300mm (12”) deep and concrete was encountered to this depth at all sampling locations. Some samples crumbled during the core removal, however, upon visual examination of the cored holes, sound concrete was visible in all sample locations.

No reinforcing steel was encountered during any of the concrete core drilling. It is assumed that no reinforcing bars were used in the larger face areas of the abutments and piers. From the rehabilitation drawings, there are localized areas which have reinforcing steel, however, these areas are assumed to be at the tops of the abutments and piers near the bearing seats.

In general, concrete core samples indicated the north abutment was the worst condition. Its closest proximity to highway 17E and tracking of chlorides/salts depositing from the bridge onto this abutment could explain why this abutment is in the worst condition state. The north abutment is estimated to have concrete removal depths from 100mm to 150mm during rehabilitation. The other elements (i.e. piers and south abutment) would be expected to have removal depth of 75mm – 125mm of concrete to reach sound substrate.

Any rehabilitation would require the full removal of any delaminations, and the surfaces of the concrete chipped down to sound concrete substrate prior to installation of any new concrete cover material.

5. REHABILITATION OPTIONS & GENERAL DISCUSSIONS

Any long-term plans for continued use of the Dean Lake Bridge would require the following considerations at a minimum. The items and quantities provided are from the information we have at this time and below is a summary of discussion items:

5.1 The bridge is currently posted for a 10-tonne load restriction, and is based on the bridge deck conditions, limited knowledge of the decking system (proprietary system) and that the decking/panel system was not designed for full loaded highway vehicles. This limits the size and weight of the vehicles that can cross over the bridge.

5.2 If the current bridge deck is to be maintained, the deck surface of the bridge needs the surface to be sealed to help mitigate water infiltration under the wearing surface and into the deck panels. Water has already entered into the two (2) most northern deck panels as seen from previous inspections. It may be necessary to remove the entire wearing surface to expose the top of the deck panels, and allow the placement of a water proofing membrane to prevent further water infiltration.

5.3 The joints and seals must be periodically removed and repaired with new sealants. This will prevent water from entering down into the bearing seat areas and within the joints between sections of the deck panels.

5.4 The concrete bearing seat at the south pier on the west end (upstream side) has spalled out around the edge of the bearing plate and bearing pads for the middle span, and is creating an un-even loading under the truss. This spall/undermining appears to be allowing the truss to settle slightly, which is causing the deck joint and armoring on the west side of the bridge to become elevated relative to the bridge deck on the south span. This undermining and settlement should be investigated, and engineered repairs completed within 2 years.

5.5 There appears to have been settlement or lateral movement issues previously identified by Kresin Engineering back in circa 1989. There are no records of a continuous survey/monitoring program continuing after the rehabilitation. There was a change over in ownership of this bridge at the time and its likely that the monitoring program was never initiated by the Municipality as a regular item.

5.6 The overall rehabilitation of the concrete abutments and piers will involve removal of all the loose, cracked, and weak concrete down to sound substrate. Our preliminary estimation of the quantities would involve removal of 27 cubic meters of vertical surface concrete repairs on the abutments and wingwalls and another 20 cubic meters of vertical surface concrete repairs on the piers. All concrete repairs will require installation of temporary scaffolding, platforms and potentially hoarding/netting to catch and collect the concrete debris as it is removed above the waterway. This will increase the price per unit rate considerably and TULLOCH is estimating that these concrete repair costs would be ~$500,000. The trusses will likely have to be temporarily removed to complete the concrete repairs, which is an additional cost to the concrete work.

5.7 The previous load evaluation identified that many members of the original trusses would have to be strengthened or replaced to accommodate the full CL-625-ONT design truck loading. These members are: (1) the decking system, (2) the floor stringer beams below the decking, (3) the supporting transom beams, (4) many of the truss diagonals, and (5) the bottom chord. Repairs to these elements would require the removal of the deck panels. If a load posting lower than the full CL-625 truck is used, then less members accordingly would be required to be reinforced or replaced. Other elements may require strengthening or replacement once a rehabilitation analysis is completed that would use new or rehabilitation load factors from the CHBDC.

5.8 There are several gusset plates and connections along the bottom chord which have undergone rust jacking. Rust jacking occurs where layers of corrosion build up between layers of plates, and the rust begins to spread the plates apart and creates tension forces into the rivets. Rust jacking forces are very difficult and nearly impossible to accurately estimate. Due to the rust jacking and the overall corrosion of the gusset plates themselves, replacement of the gusset plates and changing of the rivets to structural bolts “in-kind” is required.

5.9 Rehabilitating the trusses will require a temporary jacking and support system to elevate the bridge off the bearing seats to allow the concrete at the top portions of the abutments and piers to be rehabilitated. If the trusses were removed from the bridge and placed on the ground, and not suspended over the water, this would allow an open work area to rehabilitate the concrete piers and abutments. Repairs to the trusses on the ground would be a safer and more effective approach for any rehabilitation strategy.

5.10 Re-installing a corrosion protection system on the exposed structural steel of the trusses would be part of a long-term rehabilitation plan for this bridge. Things to consider when looking at rehabilitation costs would be the in-situ sand blasting and collection of the blast media. These items and planning can become very costly. The trusses must be able to support the added weight of the suspended scaffolding and the lateral loadings from wind due to the entire bridge having to be under tarps, and negative pressure to collect the particulate/dust.

5.11 Some select sampling of the existing coating system discovered that there is a layer of coating which is below the silver coating which contains lead. The hoarding and steel preparation for a new coating system while being suspended over the water is expected to significantly increase the cost of re-installing a new protection system to the bridge steel. TULLOCH has observed coating pricing in the past few years which range from \$35 - \$60 per square foot of steel area, and the price of the zinc rich primers/mid coat and the topcoat products have all had substantial increases of 20-30%.

Coating contractors have indicated that the general labour costs have also increased a reasonable amount in the past couple of years. These costs are only for the steel preparation and coating application. This can have additional difficulties because of general access and there would need to be consideration and loadings from the scaffolding, construction loadings and sand blasting media weights if the bridge was to be rehabilitated in position. TULLOCH estimates that there would be approximately 350 m^2 (3,800 sq. ft) of bridge steel per bridge truss that would need to be cleaned and coated. Based on pricing we obtained in 2023, the coating system could cost in the range of $150,000 - $250,000 per truss span. This does not include hoarding, protection, collection and disposal of the sand blasted material which because of the presence of lead, will require special attention and hazard disposal efforts.

5.12 Any rehabilitation option must consider the type of decking and support system to be used. A feasibility study could outline additional dead loading onto the original truss based on the type of decking to be used. It should be expected that a new deck and wearing surface would be required every 15 to 20 years +/-, depending on the deck used. The truss would have to be analyzed with the select deck system dead load and the CHBDC truck load with the appropriate load factors to determine if an improved load limit can be achieved. The cost for this should be planned and factored into the Municipalities future operating budgets for their bridge structures.

5.13 Construction of a new truss to match the historic appearance and configuration of the existing would also be an option, however, the overall reaction loads from any new truss configuration should be limited to the original truss design loadings.

5.14 The timber piles under the concrete foundations cannot be observed and are below grade or covered in concrete. Any new designs or load ratings should limit the overall reactions at the foundation level to the original design loads with some allowances or reductions as the service life has been over 100 years. Further investigation may be warranted to determine if the exposed sheet piling was installed as scour protection or if the steel sheet piling could be included for in the overall foundation resistances. No information of when the steel sheet piling was installed was available at the time of preparing this report.

6. CONCLUSIONS & RECOMMENDATIONS

The following is a list of our conclusions and recommendations for the Dean Lake Bridge given the information that we have collected to date.

6.1 The bridge has undergone several rehabilitations over the course of its 115 years of service. This has included a number of deck replacements, steel repairs and larger scale steel repairs where partial segments of the bottom and top chords were replaced. The bridge was originally designed for a dead load and live load of 1400 lbs each per linear foot of truss (entire truss system). Our calculations indicate that there is some reserve on the dead load from the original design, which could be added to the live loadings to improve the load posting.

6.2 The most recent deck replacement involved removing the original stringer beams, and installing a proprietary fiber glass wrapped wooden decking system, which was designed to act compositely with reduced steel stringers per bay. The new composite decking was designed for a load posting of 16 / 24 / 28 tonnes. If the trusses were completely rehabilitated, the current decking system would likely limit the load posting on the bridge.

6.3 We reviewed the survey results from 1989, 1996 and 2004. There are survey markers in the same vicinity of the abutments which give inconsistent movement results. The monuments likely shifted over the years due to frost and disturbance. Movement of the monuments would cause inaccurate results when comparing sequential years. Future monitoring programs should have reference benchmarks which can’t be impacted by vehicular damage or frost. The survey can be completed at a local relative datum because we are only interested in understanding of the abutments are moving relative to the bridge piers.

6.4 The coupon testing results indicate that the “Equivalent Strength” from Test Results used to determine evaluation strength properties per the CHBDC is 211 MPa. This steel strength confirms the 210 MPa steel strength from the 2019 TULLOCH load evaluation. Evaluation load factors can vary significantly from those used in rehabilitation or new designs. With the load factors being higher for rehabilitations or new construction (i.e. live load factor of 1.7 versus 1.49 in this case), there can be additional members of the truss which would require replacement or strengthening.

6.5 We advise the current 10 tonne maximum single unit vehicle load posting remain in effect. The load posting is required due to specific members within the floor framing, as well as uncertainties and defects observed in the new deck panels. To extend the remaining service life of the deck panels it may be necessary to reduce the load posting to 5 tonnes in the future to limit the stresses on the decking and floor framing members. The Municipality has observed significantly loaded transports which appeared much higher than the 10-tonne load posting across the bridge.

6.6 The details of a rehabilitation plan for the truss include: reconstruction of various members or strengthening of the existing members, replacement of many bottom chord connections which are exhibiting severe rust jacking, and changing out rivets for structural bolts. Some connections may be permissible to repair in-situ, but there are several critical connections which appear to be very difficult to repair while the bridge is in place. Further exploration would be required to determine if extracting members or connections selectively is possible, or if extensive temporary shoring or stabilization of the bridge trusses would be required. Otherwise, the only option would be to remove the bridge from its bearings and complete the rehabilitation at a staging zone on the ground near the approaches.

6.7 A long-term rehabilitation plan should also include a new coating system for the existing and new structural steel. Re-painting of the truss in-situ is a costly item when factoring in all the scaffolding, hoarding and containment requirements. Applying a new coating on a plated structure with the number of cracks and crevices is less than ideal. The effectiveness of the coating system is reliant on the surface preparation and application procedures, which on a bridge style of this type, is problematic at best and will likely require regular cleanup and touch ups to the coating system at the gusset plates and connections to prevent cracking and bleed through at the joints.

6.8 Rehabilitation of the concrete abutments and piers is required and given the size and location of the abutments and piers adjacent to water, general access and scaffolding will increase the overall cost which we are accustomed to seeing. TULLOCH estimates that it will cost $500,000 – $600,000 to rehabilitate the concrete. This item has the potential to have significant cost increases as the costs are directly correlated to the volume of concrete having to be removed which can vary significantly from our estimate. The depth of the rehabilitation should remove all soft, delaminated, or spalled concrete, and the actual quantities would need to be verified as the contractor progressed to ensure that excessive depths were not occurring without reasonable rationale.

6.9 It has not been confirmed what loads the piers and abutments have been designed for. This must be investigated along with rehabilitation of the concrete and development of concrete repair drawings. The term ‘rehabilitation’ is used somewhat in a broad manner and could involve any of the following:

6.9.1 Repairing the concrete abutments and piers while maintaining the 10-tonne load posted bridge ‘as-is’ and continue repairing the decking and monitoring the bridge with regular detailed inspections.

6.9.2 Repairing the concrete abutments and piers, repairing the trusses, and installing a new decking system consisting of; new steel stringers, new transom beams, a new deck top and wearing surface. This could improve the overall load posting on the bridge, provided a truss and pier analysis is carried out.

6.9.3 Repair the concrete abutments and piers while rehabilitating the trusses in position on the bridge. Consideration will need to be given to permissible construction loadings of the hoarding system onto the original truss structure. During the rehabilitation the strengthening or replacement would be completed to allow the load posting to be improved.

6.9.4 Repair the concrete abutments and piers with the intention of installing a new truss bridge (similar in layout to continue the historic appearance) of the bridge. A comparison of the original design loadings versus the current design loadings from the CHBDC are necessary to determine if significant steel member modifications are necessary and overall loadings at the foundation would be acceptable or not. Alternatively, removing, repairing, and reinstalling the existing trusses would follow a similar procedure.

6.10 Based on the above, TULLOCH recommends the following items be further explored to provide repair options for this bridge. We propose a feasibility study which would request Contractor involvement to better understand the overall costing for each, and to refine the overall expected project costing on a life cycle basis using a 50-year horizon:

6.10.1 An abutment monitoring program is recommended to review foundation movements specifically the abutments which was previously a concern around the 1989 rehabilitation. Surveys of the abutments shall be completed twice a year for the next 2 years to verify any movements. If there are ongoing movements, foundation stabilization repairs would be required, and an engineered repair would need to be included as part of the overall concrete rehabilitation plans. This may require a geotechnical investigation to assess ground conditions which are not visible. More permanent survey monuments or reference points would be required to obtain a more accurate understanding of any pier and abutment movements.

6.10.2 During the next 2 years, engineered repair plans should be developed to rehabilitate the current abutments and piers with the intention of maintaining the bridge at the 10-tonne load capacity.

6.10.3 Determine allowable loadings of the bridge if concrete and steel repairs are completed. This would outline the volume of steel repairs and element replacements needed and compare if construction of a new truss superstructure would be more economical than refurbishing the existing trusses.

6.10.4 Review floor framing and stringer layouts. This would involve looking at wood decking, steel decking and concrete decking options and reviewing their dead loads on the trusses. It would be ideal to install a water proofing membrane and an asphalt wearing surface on a concrete or steel deck for added service life between full deck replacements.

6.10.5 Review construction hoarding requirements, and additional live loadings (vertical and lateral) from sand blasting and containment to ensure that rehabilitating in-situ is an option, or at least outline loading criteria that would need to be considered to allow this option to be achievable. Example: only half of the truss span can be enclosed at a time. Alternatively, the truss would be refurbished on blocks on a laydown area located near the bridge.

6.10.6 Compare rehabilitated bridge and deck options along with a new truss option to determine if there are limitations or a loading scenario causing an overload or adding significant loads onto the foundation level. Limiting the loadings onto the foundations will be necessary, otherwise, expanding and increasing the foundation (adding piles and concrete footings) will be required and this cost will need to be factored into each bridge rehabilitation option.

6.10.7 Research the lifespan of timber piles driven in the ground. The available drawings we have appear to show the pilings to be timber piles. If it is recorded in literature that timber piles have been re-used for many years, then that would support the plan of repairing and re-using the existing abutments and piers for many more years of use.

6.10.8 A detailed inspection within arm’s reach or as close as practical is recommended and should be completed this year (summer 2024). This will give further confirmation that existing conditions have not changed significantly, and no further repairs are required which could not be observed from the bridge deck during our regular OSIM inspections. TULLOCH is working on obtaining pricing for that detailed inspection and will provide it as soon as possible.

7. DISCLAIMER

Conclusions and Recommendations derived are specific to the Dean Lake Bridge within the Municipality of Huron Shores. The details of this report are meant to provide information for the purpose of managing and maintaining this bridge asset. Conclusion and recommendations presented herein are made with the information present at the time of preparing this report. Should new or other relevant information be discovered that would impact the overall capacities or functionality of the bridge, TULLOCH would appreciate the opportunity to amend this report to reflect the new information.

While the Client may release the report to third parties, any use of this report by a party other than the client, or any reliance on or decisions made based on the findings described in this report, are the sole responsibility of such third parties, and TULLOCH accepts no responsibility for damages, suffered by any third party as a result of decisions made or actions conducted based on this report. No other warranties are implied or expressed.

8. CLOSURE

Although the results of the steel coupon testing are similar to that previously assumed, the results do confirm that the current load posting is valid and that no further reduction in load capacity is necessary. The concrete coring indicates that we do have sound concrete below the delaminations and that a minimum 15 MPa could be expected for a concrete compressive strength in any rehabilitation planning.

If the Municipality wishes to continue onto the feasibility study and development of a current bridge monitoring survey, please contact TULLOCH and we will prepare a price and schedule to complete this. Confirmation of the detailed inspection are also being requested and developed and will be shared with the Municipality as soon as possible.

If you have any questions regarding the information contained here in, contact the undersigned at your convenience.

Sincerely yours,

LICENSED PROFESSIONAL ENGINEER
April 19, 2024
M. D. KIRBY
100173482
Matt Kirby
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
2024/04/19
D. W. MACNEILL
100086576
PROVINCE OF ONTARIO

Report prepared by:
Matt Kirby, P. Eng.
Project Manager/ Engineer
Sault Ste. Marie

Report reviewed by:
Danny MacNeill, P. Eng.
Project Manager/Engineer
Sault Ste. Marie

APPENDIX A: Old Bridge Drawings & Information

DEAN LAKE BRIDGE

TOWNSHIP OF THOMPSON

DISTRICT OF ALGOMA

CONTRACT NO. 8812

LIST OF DRAWINGS

42-8617-1 GENERAL ARRANGEMENT
2 ABUTMENTS PIERS & BEARING MODIFICATIONS
3 STRUCTURAL STEEL DETAILS
4 MISCELLANEOUS DETAILS
5 STANDARDS

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS
ONTARIO
M.T.C. STRUCTURE SITE NO. 38S-234
FINAL DESIGN APPROVED FOR STRUCTURAL
ADEQUACY THIS DAY 28 11 1
MUNICIPAL STRUCTURAL ENGINEER

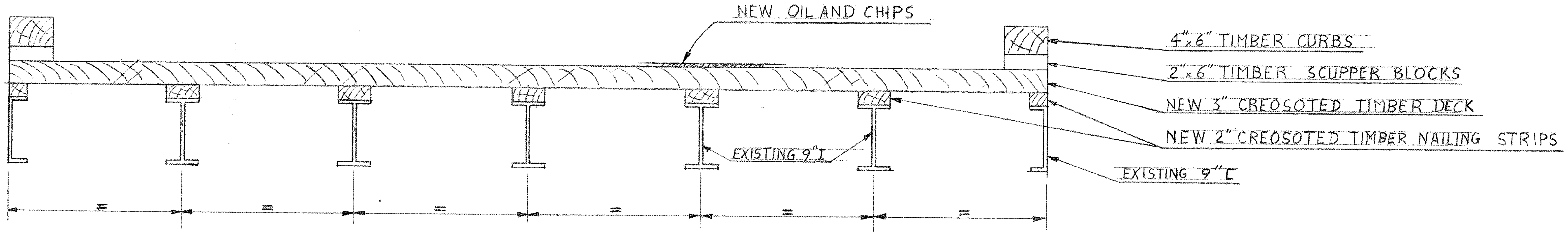
M T C DOWNSVIEW
RECEIVED
[illegible]
STRUCTURAL OFFICE
M. T. C.

STRUCTURE SITE No 38S-234

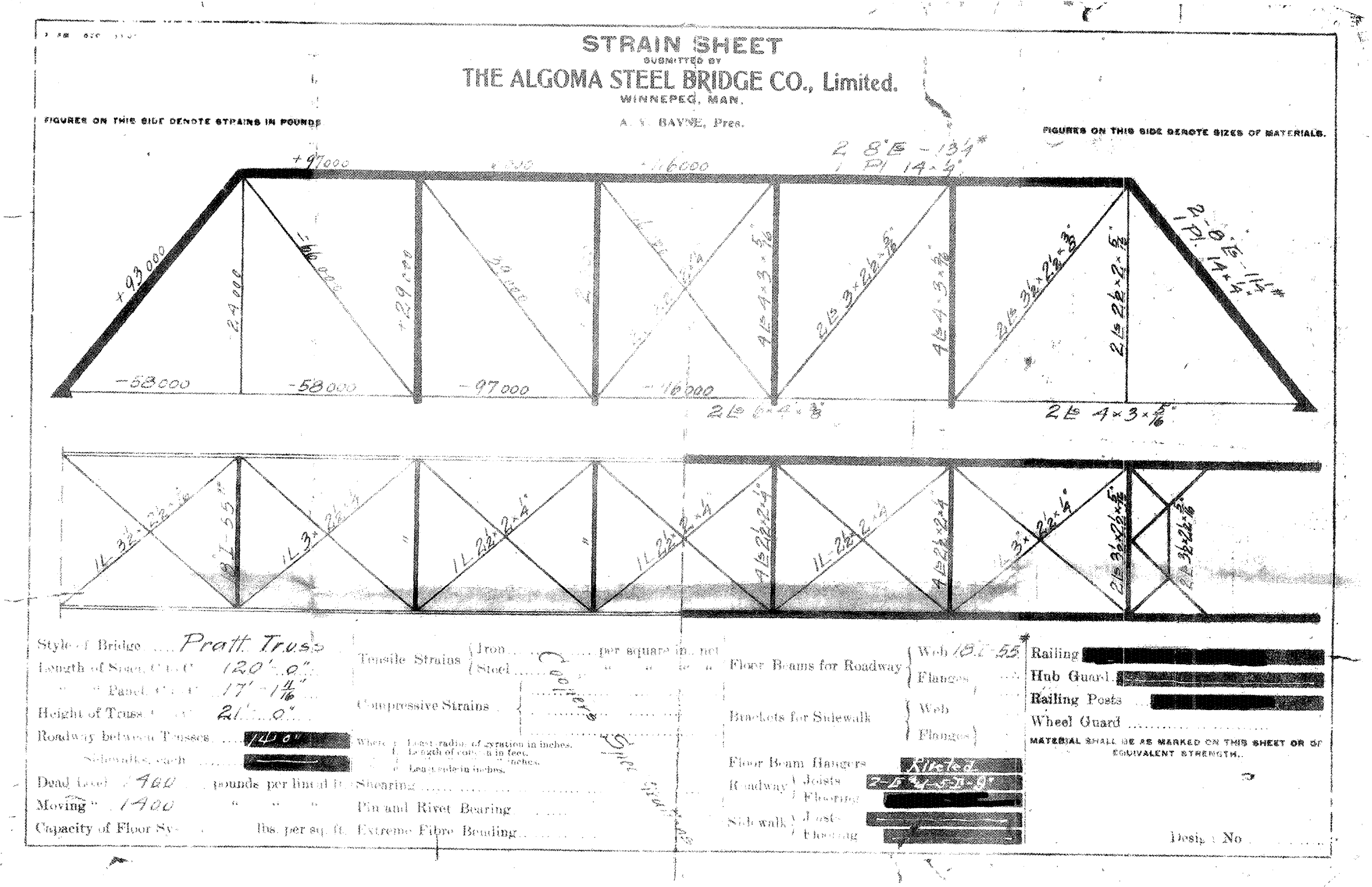
FOR APPROVAL

KRESIN
ENGINEERING & PLANNING

SITE No. 38S-234 MISSISSAUGI RIVER PROJECT No. 42-8617



SECTION THROUGH DECK N.T.S.



STRAIN SHEET
SUBMITTED BY
THE ALGOMA STEEL BRIDGE CO., Limited.
WINNEPEG, MAN.
A. Y. BAYNE, Pres.

FIGURES ON THIS SIDE DENOTE STRAINS IN POUNDS

FIGURES ON THIS SIDE DENOTE SIZES OF MATERIALS.

Style of Bridge ..... Pratt Truss
Length of Span, C. to C. 120'-0"
" " Panel, C. to C. 17'-1 11/16"
Height of Truss C. to C. 21'-0"
Roadway between Trusses ..... 14'0"
Sidewalks, each ..... —
Dead Load 1400 pounds per lineal ft.
Moving " 1400 " " "
Capacity of Floor Sy... lbs. per sq. ft.

Tensile Strains { Iron ..... per square in. net
{ Steel ..... " " " "
Compressive Strains { .....
Where r = Least radius of gyration in inches.
l = Length of column in feet.
" " " inches.
d = Least side in inches.
Shearing .....
Pin and Rivet Bearing .....
Extreme Fibre Bending .....

Floor Beams for Roadway { Web 18 I-55
{ Flanges .....
Brackets for Sidewalk { Web
{ Flanges
Floor Beam Hangers Riveted
Roadway { Joists 2-[s 4 - 9 I - 9"
{ Flooring [redacted]
Sidewalk { Joists —
{ Flooring —

Railing [redacted]
Hub Guard —
Railing Posts [redacted]
Wheel Guard .....
MATERIAL SHALL BE AS MARKED ON THIS SHEET OR OF EQUIVALENT STRENGTH.

Design No .....

ELEVATION AND PLAN OF ONE TRUSS N.T.S.

NOTES

1/ THE PLANS ARE FOR GUIDANCE ONLY AND DO NOT NECESSARILY REPRESENT EXISTING CONDITIONS AND DIMENSIONS.

2/ THE STRUCTURE CONSISTS OF 3 EQUAL SPANS, OF WHICH ONE IS SHOWN ON THIS PLAN.

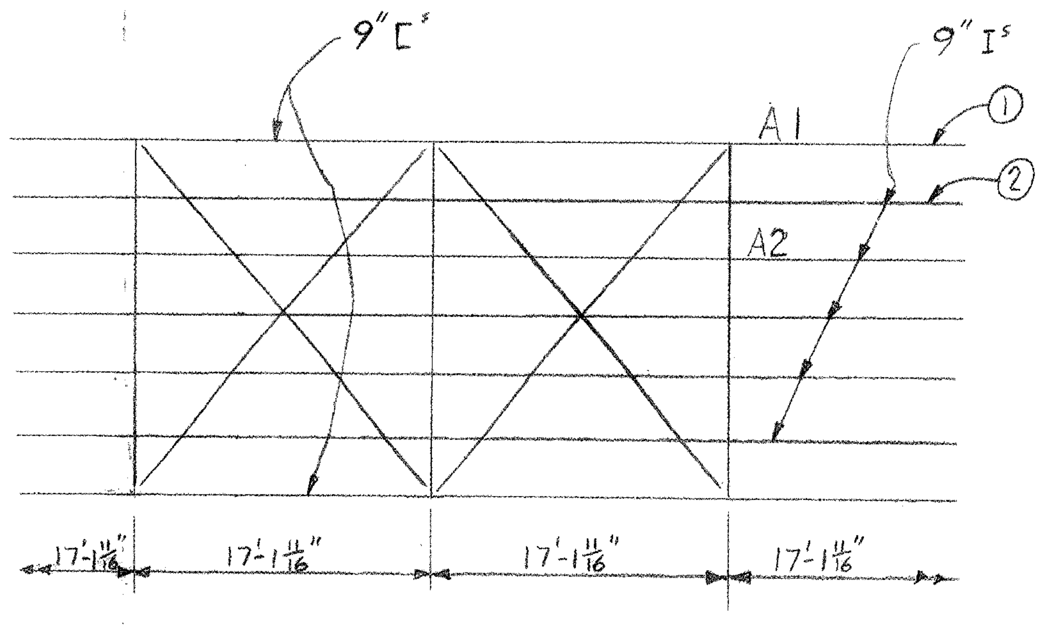
3/ REMOVE EXISTING TIMBER DECK AND REPLACE WITH NEW 3" CREOSOTED TIMBER DECK.

4/ SANDBLAST AND REPAINT ALL STRUCTURAL STEEL.

5/ REPAIRS TO CONCRETE NOT SHOWN. REPAIRS TO CONCRETE SHOULD BE CARRIED OUT AS DIRECTED BY THE ENGINEER.

① REPLACE 'A1' WITH NEW 8"[s WHERE NECESSARY, EQUIVALENT OR BETTER THAN THE EXISTING 9"[s, AS DIRECTED BY THE ENGINEER. NEW [s HAVE TO BE BROUGHT UP TO THE TOP ELEVATION OF THE EXISTING [s WITH SHIM PLATES ETC.

② REPLACE 'A2' WITH NEW 8"Is OR 8"WF WHERE NECESSARY, EQUIVALENT OR BETTER THAN THE EXISTING 9"Is, AS DIRECTED BY THE ENGINEER. NEW Is (OR WF) HAVE TO BE BROUGHT UP TO THE TOP ELEVATION OF THE EXISTING Is WITH SHIM PLATES ETC.



DETAIL OF FLOOR SYSTEM N.T.S.

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

MISSISSAUGA RIVER BRIDGE

DIST. ALGOMA
TWP THOMPSON NE ¼ SECTION 8

PLAN ELEVATION & SECTIONS

SITE No. 38S-234

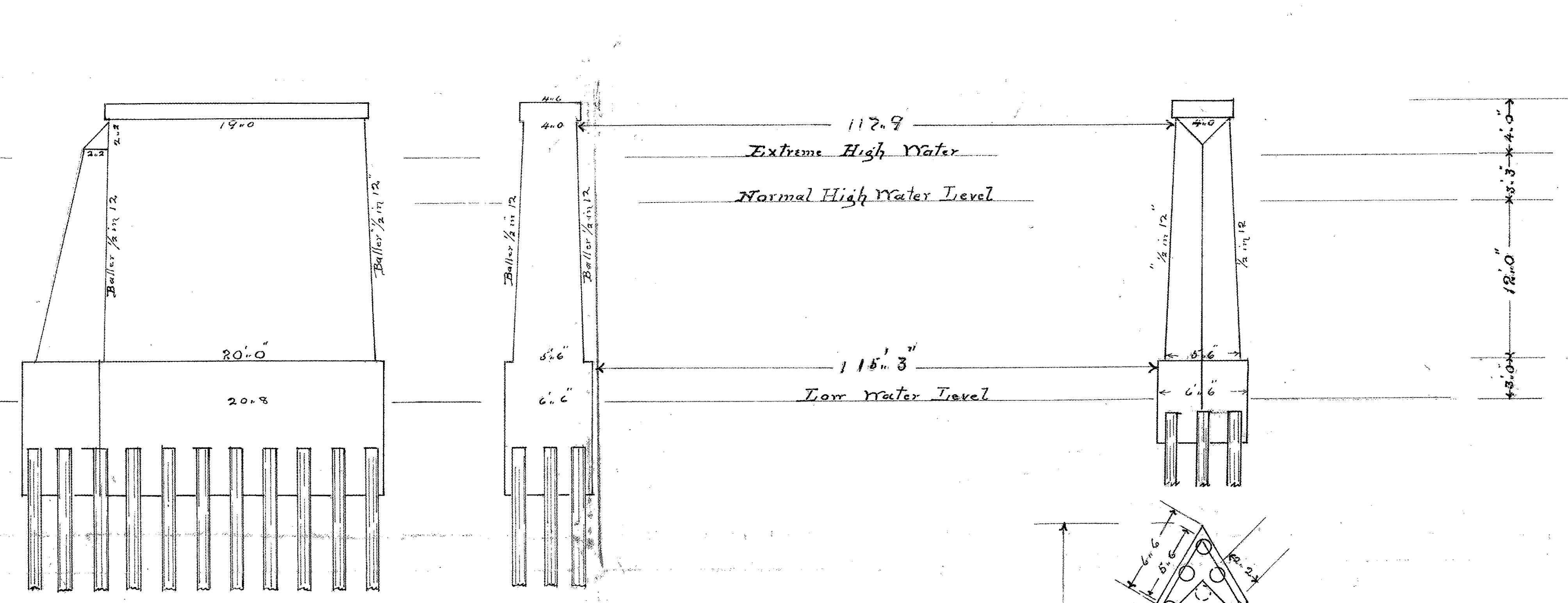
DRAWING PW CHECK
DATE FEB 1963 DRAWING NO

1-0

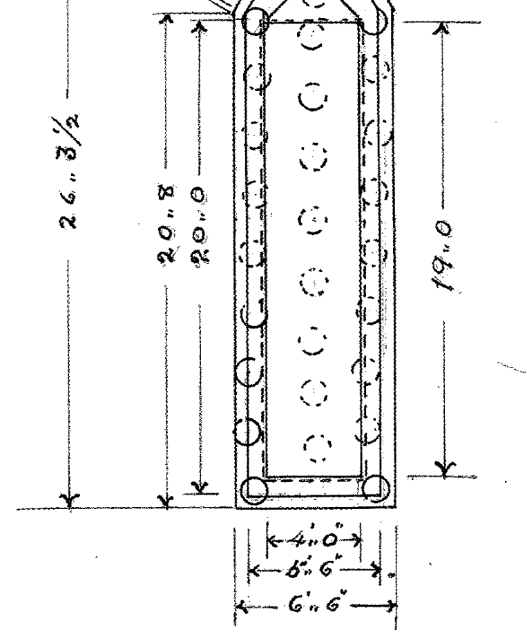
ELEVATION OF BRIDGE

Scale 20 ft

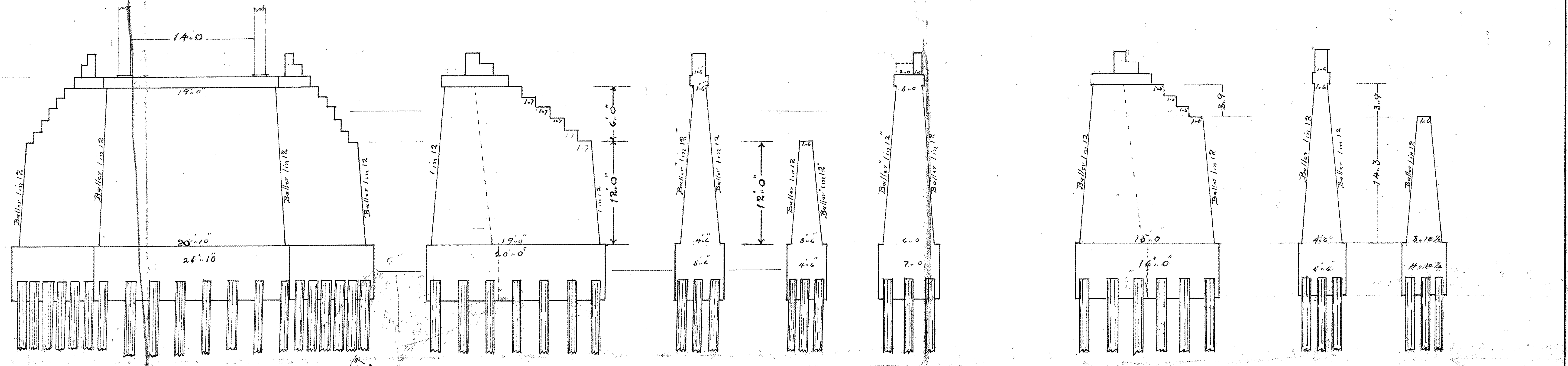
PLAN OF BRIDGE



ELEVATION OF PIERS Nos 2 & 3



PLAN OF PIERS

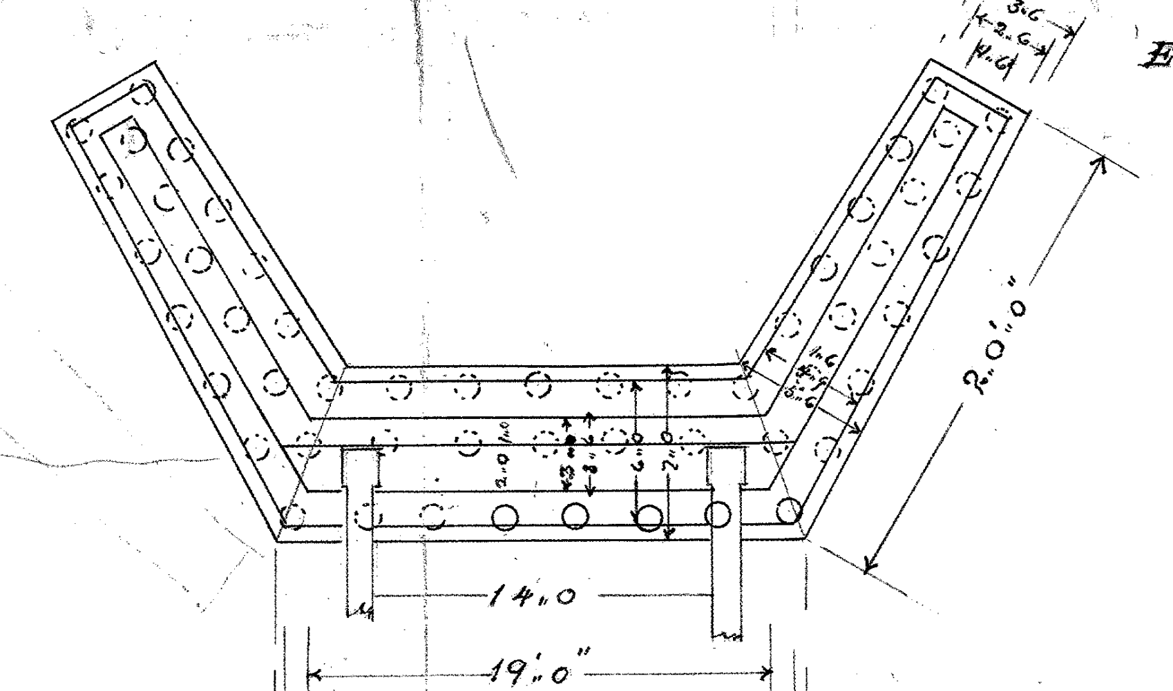


ELEVATION OF WING WALLS

SECTIONS OF WING WALLS

SECTION of ABUTMENTS

ELEVATION & SECTIONS OF WING WALLS OF No 6



PLAN OF ABUTMENTS

Scale 8 ft per inch

MISSISSAUGA RIVER BRIDGE

AT SIMPSONS ISLAND

TOWNSHIP OF THOMPSON

DISTRICT OF ALGOMA

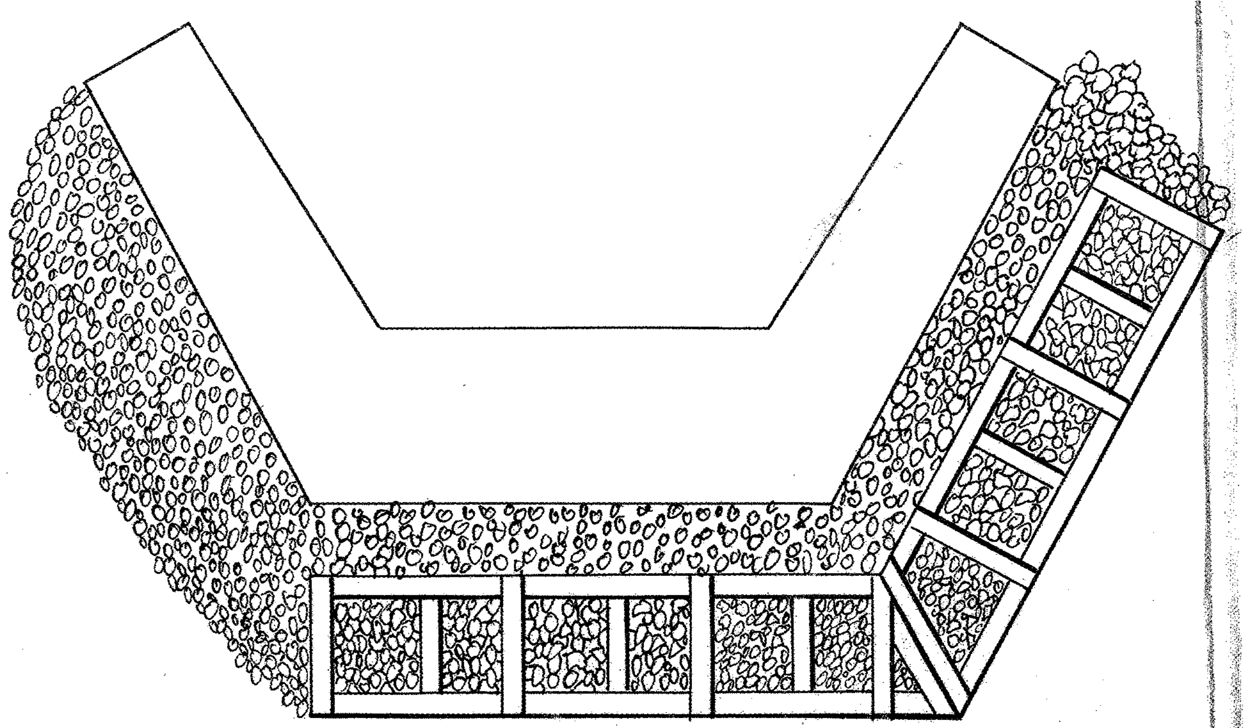
38s-234

Department of Public Works Ontario.
September 24th 1907.

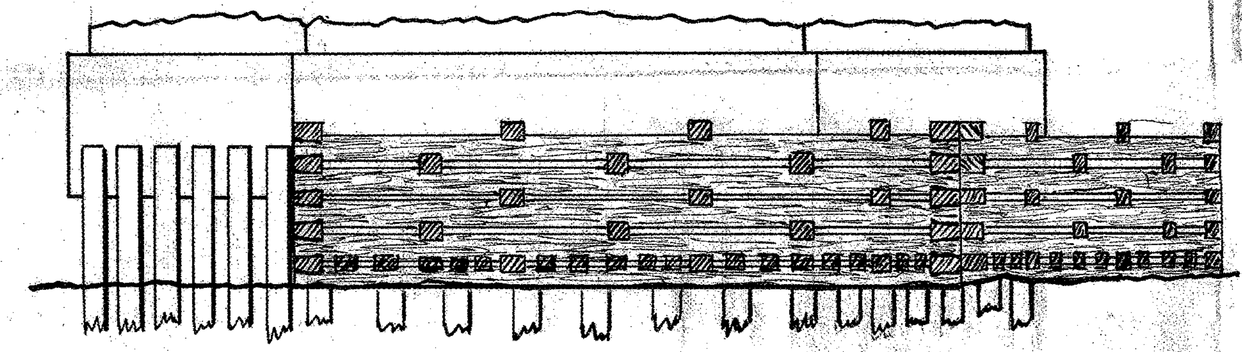
R. P. Fairbairn
Engineer Public Works

our me
two
us
7th December
A.D. 1907
Witness
The Algoma Steel Bridge Co Ltd
By N. T. Bayne President

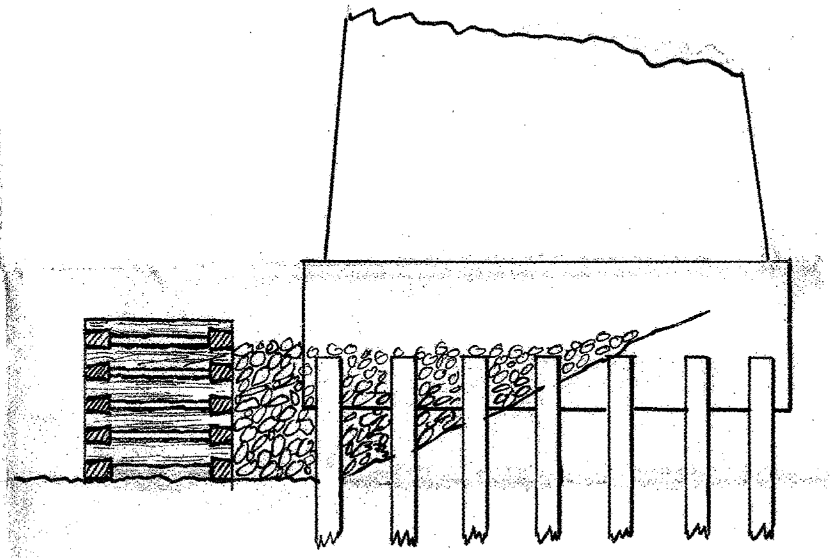
B 144-1



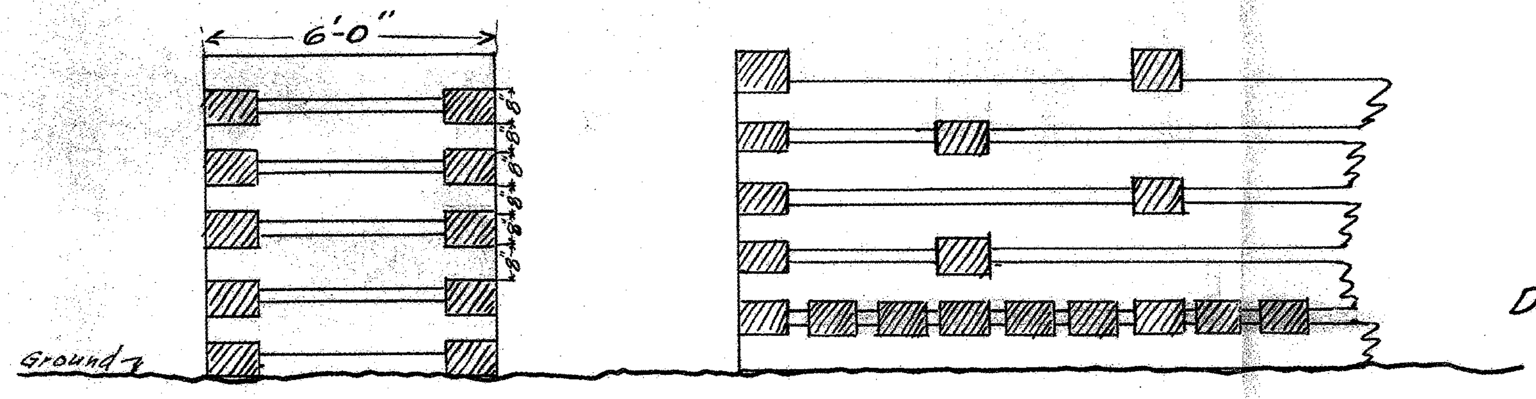
PLAN OF PIER SHEWING CRIB & RIP RAP



ELEVATION



END VIEW



END VIEW

FRONT ELEVATION

DETAIL OF CRIB WORK
1/4 IN. SCALE

MISSISSAUGA RIVER BRIDGE
TRUNK ROAD
REPAIRS TO EAST ABUTMENT

DEPARTMENT OF PUBLIC WORKS ONTARIO
TORONTO JAN 5th 1915.

38s-234
ENGINEER OF PUBLIC WORKS

FORM 5 500. 10-07

STRAIN SHEET

SUBMITTED BY

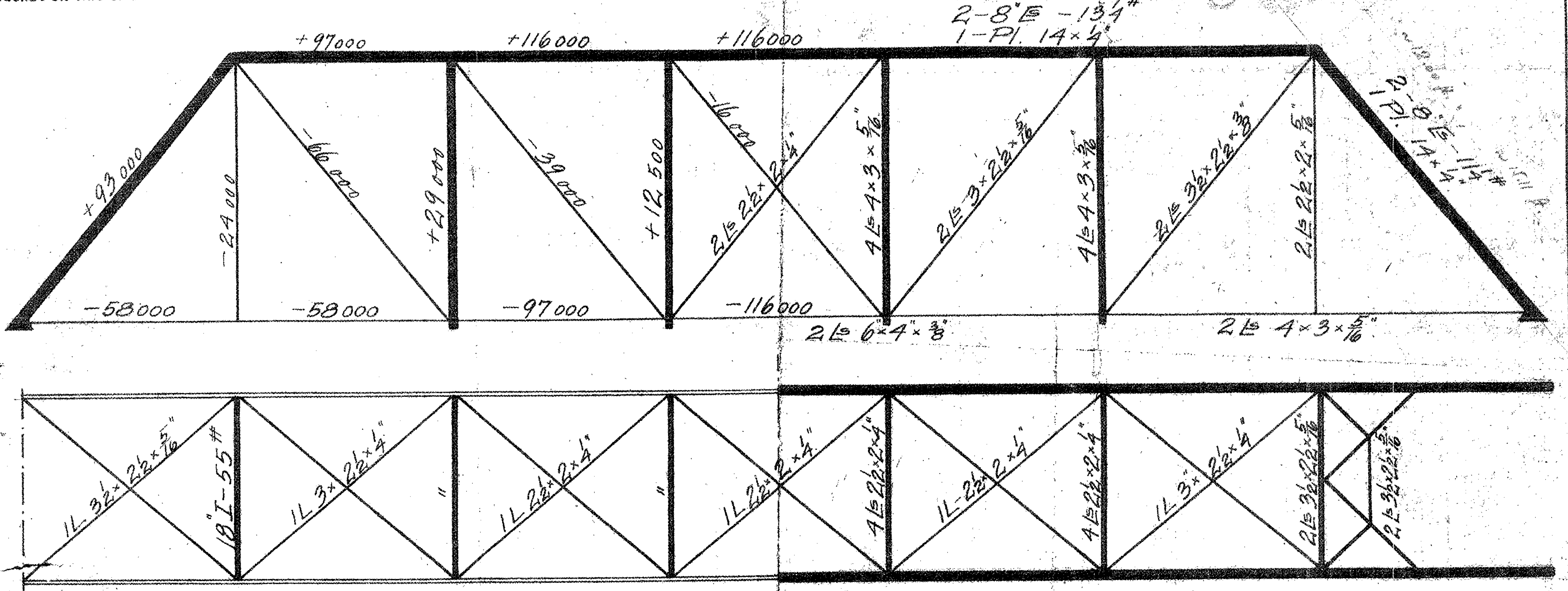
THE ALGOMA STEEL BRIDGE CO., Limited.

WINNEPEG, MAN.

A. Y. BAYNE, Pres.

FIGURES ON THIS SIDE DENOTE STRAINS IN POUNDS.

FIGURES ON THIS SIDE DENOTE SIZES OF MATERIALS.



Style of Bridge *Pratt Truss*

Length of Span, C to C *120'-0"*

" " Panel, C to C *17'-1 11/16"*

Height of Truss, C to C *21'-0"*

Roadway between Trusses *14'-0"*

Sidewalks, each

Dead Load *1400* pounds per lineal ft.

Moving " *1400* " " "

Capacity of Floor System lbs. per sq. ft.

Tensile Strains { Iron per square in., net
{ Steel " " " "

Compressive Strains

Where r=Least radius of gyration in inches.
" L=Length of column in feet.
" l=" " " inches.
" d=Least side in inches.

Shearing

Pin and Rivet Bearing

Extreme Fibre Bending

Coopers Specifications

Floor Beams for Roadway { Web *18" I-55#*
{ Flanges

Brackets for Sidewalk { Web
{ Flanges

Floor Beam Hangers *Riveted*

Roadway { Joists *2-Ls [illegible]-5 I-9"*
{ Flooring *3" Pine or Tamarac*

Sidewalk { Joists
{ Flooring

Railing *3 Lines 1¼" Gas Pipe*

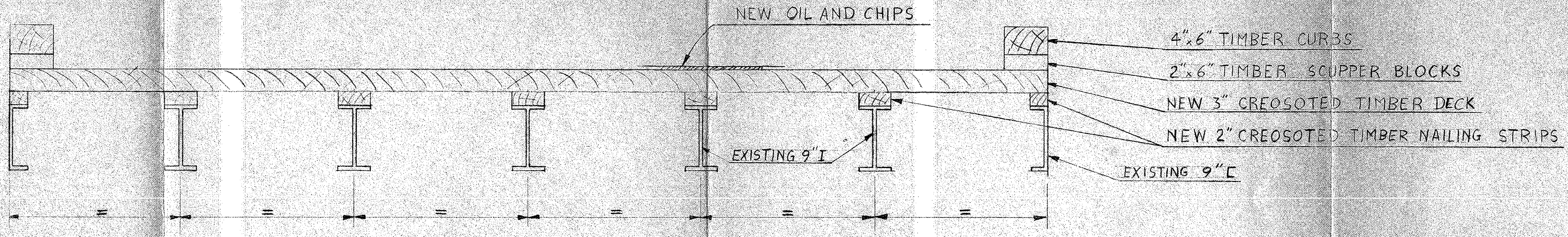
Hub Guard

Railing Posts *3½"×3"×5/16" Ls*

Wheel Guard

MATERIAL SHALL BE AS MARKED ON THIS SHEET OR OF EQUIVALENT STRENGTH.

Design No.



SECTION THROUGH DECK N.T.S.

STRAIN SHEET
SUBMITTED BY
THE ALGOMA STEEL BRIDGE CO., Limited.
WINNEPEG, MAN.
A. Y. BAYNE, Pres.

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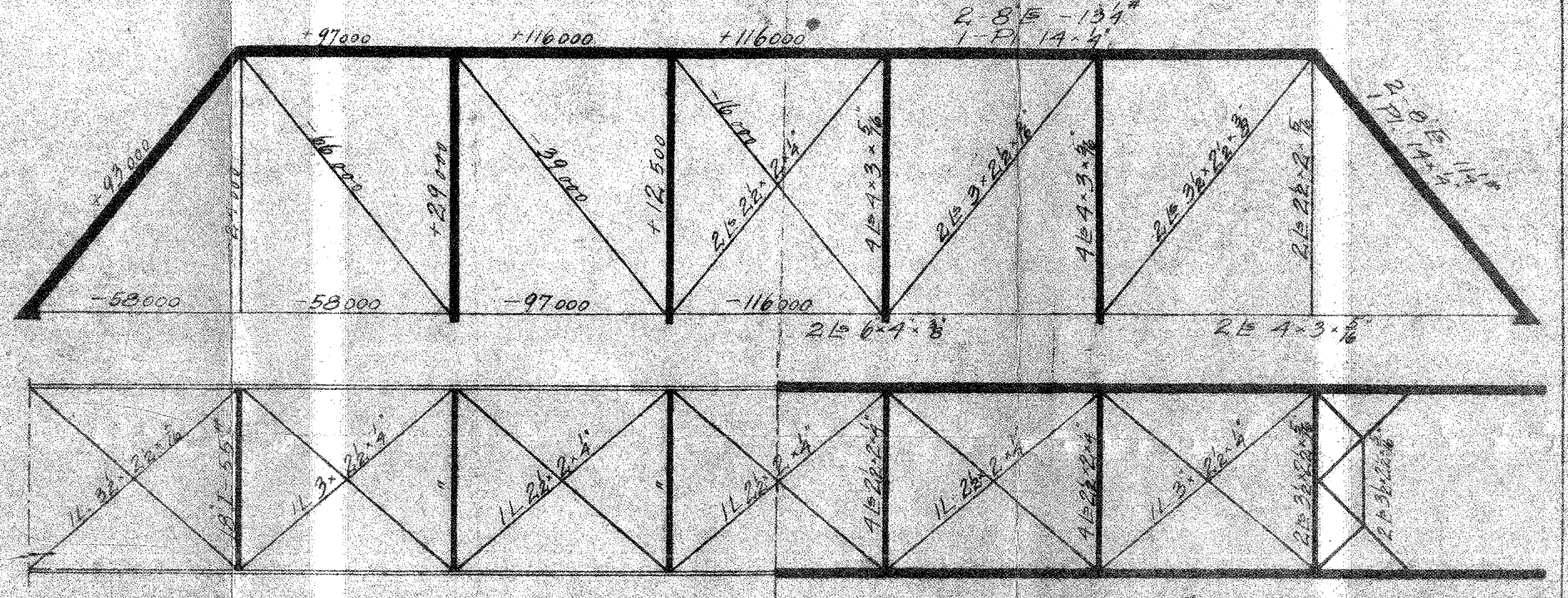


Style of Bridge	Pratt Truss
Length of Span, C to C	120'-0"
" Panel, C to C	17'-1 11/16"
Height of Truss, C to C	21'-0"
Roadway between Trusses	[redacted]
Sidewalks, each	[redacted]
Dead Load	1400 pounds per lineal ft.
Moving "	1400 " " "
Capacity of Floor System	lbs. per sq. ft.

Tensile Strains { Iron .... per square in., net
{ Steel .... " " " "

Compressive Strains { ....

Where r = Least radius of gyration in inches.
" L = Length of column in feet.
" l = " " " " inches.
" d = Least side in inches.

Shearing ....

Pin and Rivet Bearing ....

Extreme Fibre Bending ....

Coopers Specifications

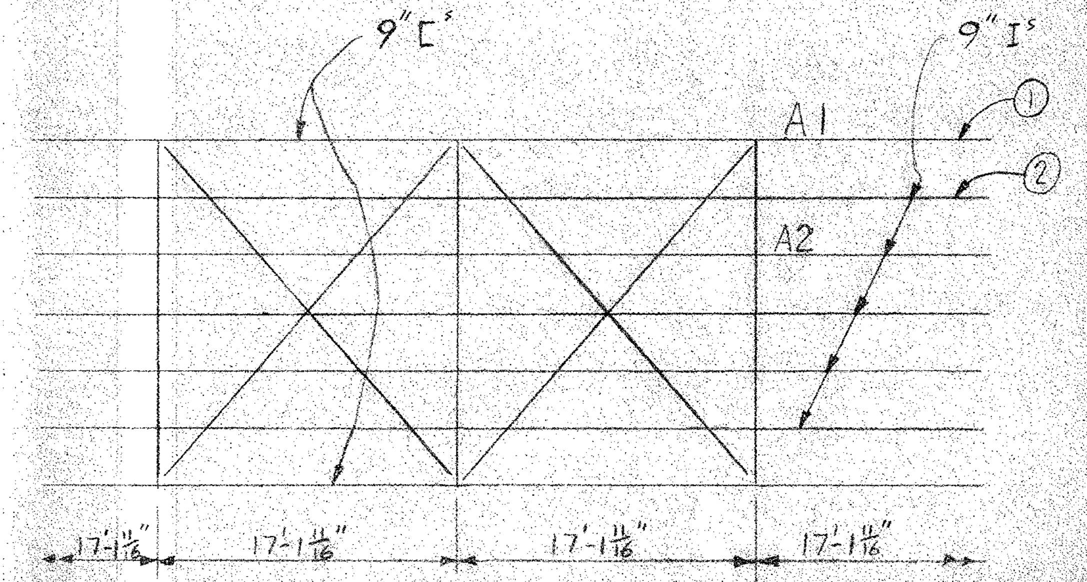
Floor Beams for Roadway { Web	18 I 55#
Flanges	
Brackets for Sidewalk { Web	
Flanges	
Floor Beam Hangers	[redacted]
Roadway { Joists	[redacted]
Flooring	[redacted]
Sidewalk { Joist	[redacted]
Flooring	[redacted]

Railing	[redacted]
Hub Guard	[redacted]
Railing Posts	[redacted]
Wheel Guard	

MATERIAL SHALL BE AS MARKED ON THIS SHEET OR OF EQUIVALENT STRENGTH.

Design No.

ELEVATION AND PLAN OF ONE TRUSS N.T.S.



DETAIL OF FLOOR SYSTEM N.T.S.

NOTES

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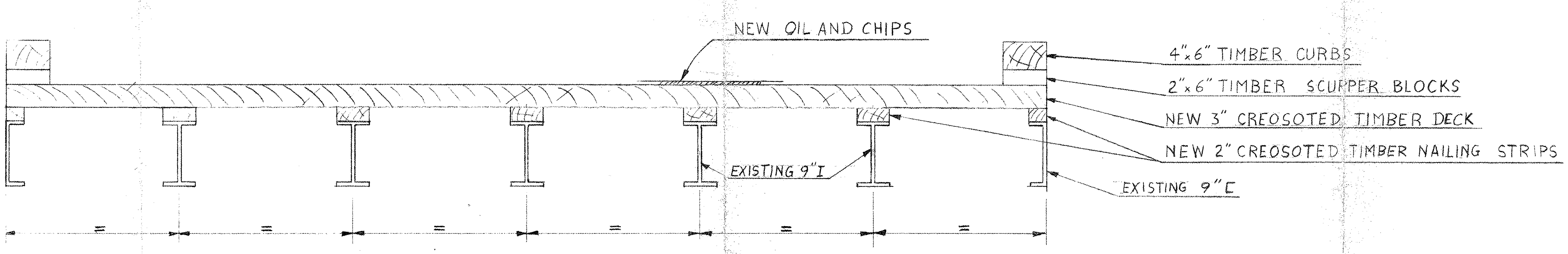
DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

MISSISSAUGA RIVER BRIDGE

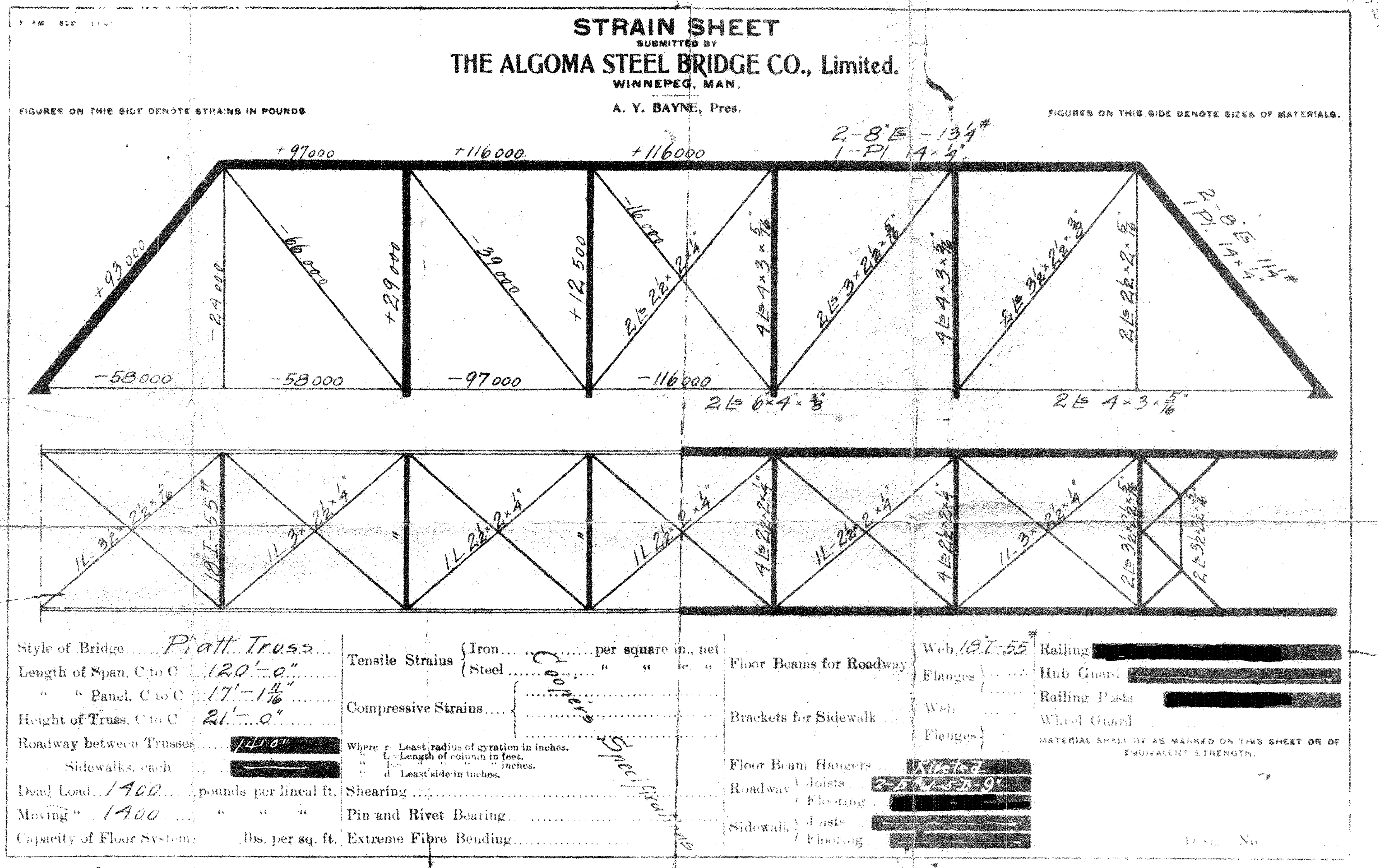
DIST. ALGOMA
TWP THOMPSON NE ¼ SECTION 8

PLAN ELEVATION & SECTIONS

				SITE No 50-234	
DRAWING	PW	CHECK		Rev. Drgs B 1680 B 144	
DATE	FEB 1963			DRAWING NO	M 12320-1



SECTION THROUGH DECK N.T.S.



STRAIN SHEET

SUBMITTED BY

THE ALGOMA STEEL BRIDGE CO., Limited.

WINNEPEG, MAN.

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FIGURES ON THIS SIDE DENOTE STRAINS IN POUNDS

FIGURES ON THIS SIDE DENOTE SIZES OF MATERIALS.

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Height of Truss, C to C 21'-0"
Roadway between Trusses 14'0"
Sidewalks, each —
Dead Load 1400 pounds per lineal ft.
Moving " 1400 " " "
Capacity of Floor System lbs. per sq. ft.

Tensile Strains { Iron .... per square in., net
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Compressive Strains ....

Where r Least radius of gyration in inches.
" L Length of column in feet.
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Shearing ....
Pin and Rivet Bearing ....
Extreme Fibre Bending ....

Cooper's Specification

Floor Beams for Roadway { Web 18I-55# Flanges
Brackets for Sidewalk { Web Flanges
Floor Beam Hangers Riveted
Roadway { Joists 2-5"I-5I-9' Flooring [redacted]
Sidewalk { Joists — Flooring —

Railing [redacted]
Hub Guard —
Railing Posts [redacted]
Wheel Guard

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Test No.

ELEVATION AND PLAN OF ONE TRUSS N.T.S.

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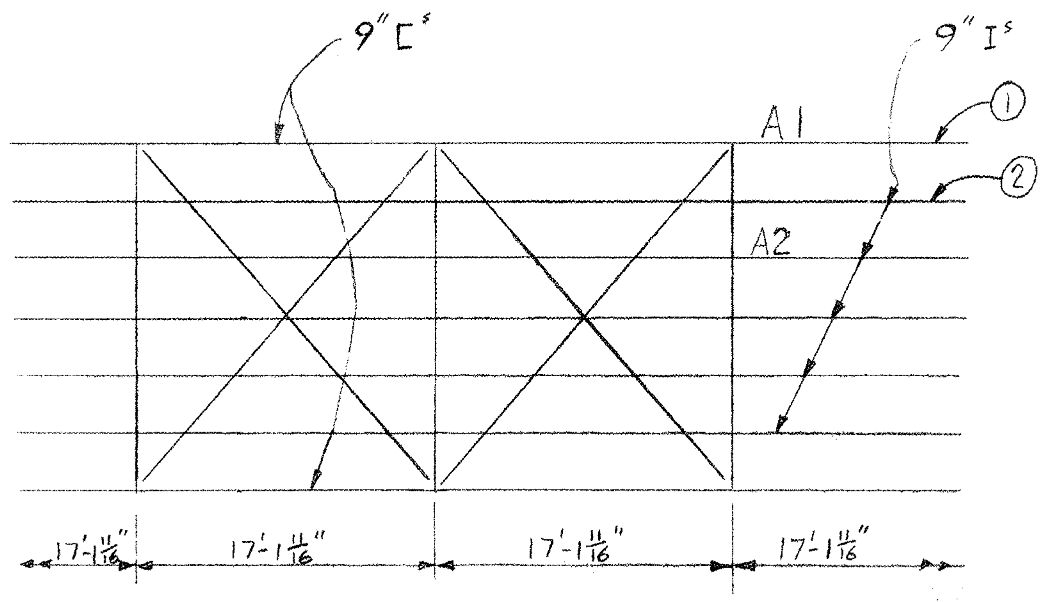


DETAIL OF FLOOR SYSTEM N.T.S.

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

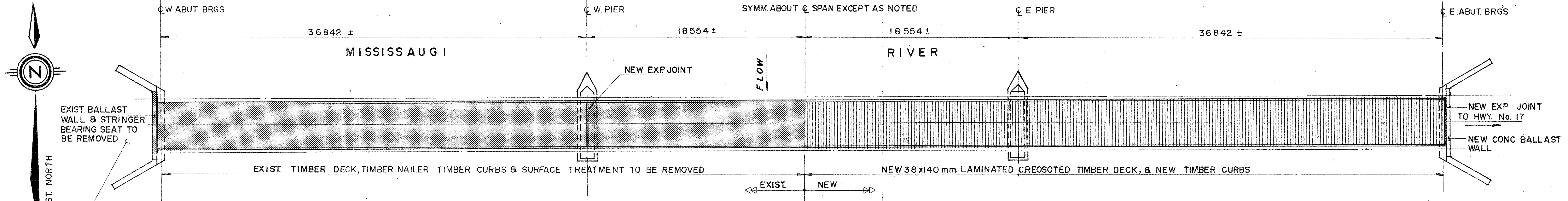
MISSISSAUGA RIVER BRIDGE

DIST. ALGOMA
TWP THOMPSON NE ¼ SECTION 8

PLAN ELEVATION & SECTIONS

				SITE No	50-234
DRAWING	PW	CHECK			
DATE	FEB 1963			DRAWING NO	M 12320-1

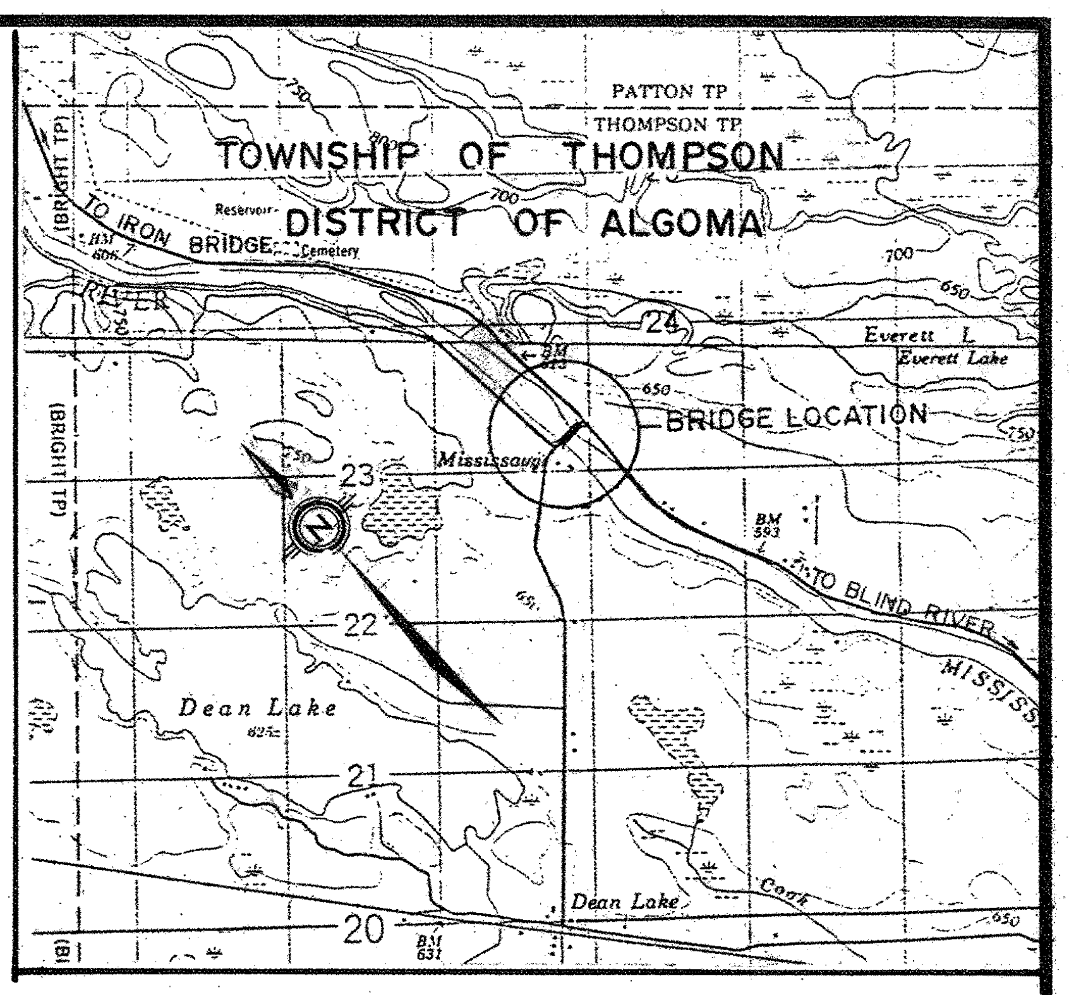
TOWNSHIP OF THOMPSON
DISTRICT OF ALGOMA



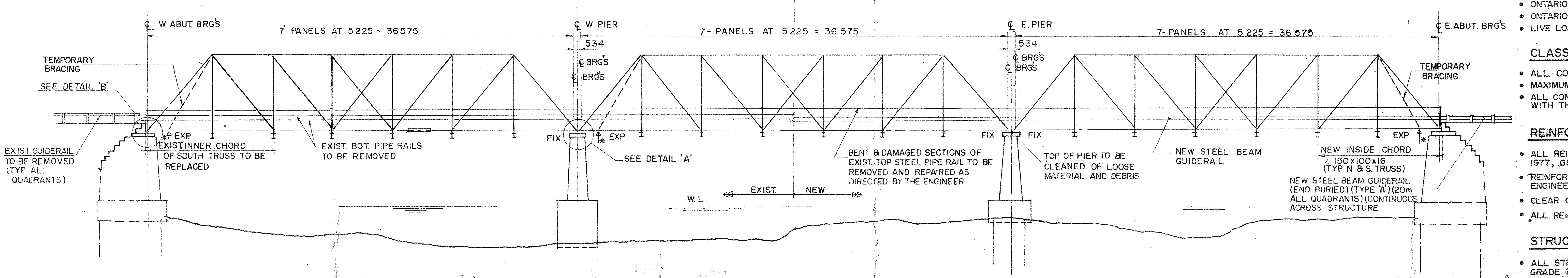
PLAN
SCALE 1:200

NOTES TO CONTRACTOR

- CONTRACTOR TO CHECK ALL RELEVANT STRUCTURE DIMENSIONS AND ELEVATIONS SHOWN ON THE DRAWINGS AND ADJUST DIMENSIONS AND ELEVATIONS AS REQUIRED TO MATCH EXISTING STRUCTURE AND AS APPROVED BY THE ENGINEER.
- ANY DAMAGE TO THE EXISTING STRUCTURE DURING REMOVALS OR RECONSTRUCTION SHALL BE REPAIRED BY THE CONTRACTOR TO THE SATISFACTION OF THE ENGINEER AND AT NO COST TO THE OWNER.



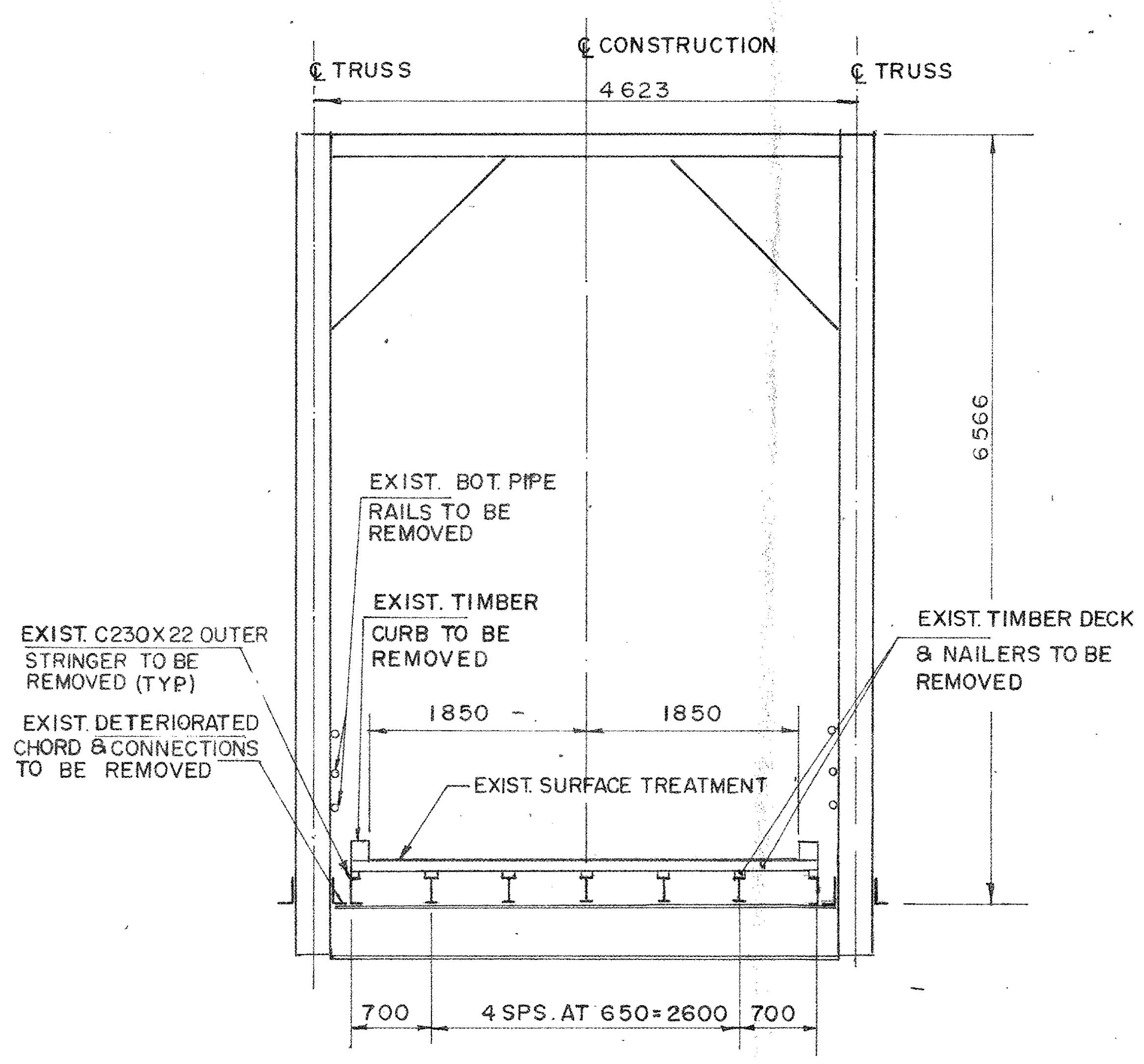
KEY PLAN
SCALE 1:50000



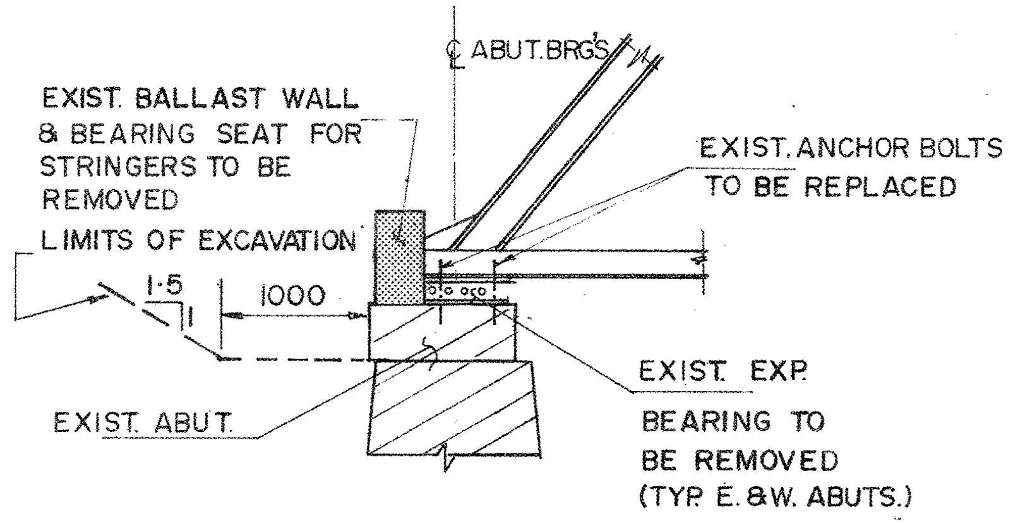
ELEVATION
SCALE 1:200

NOTES

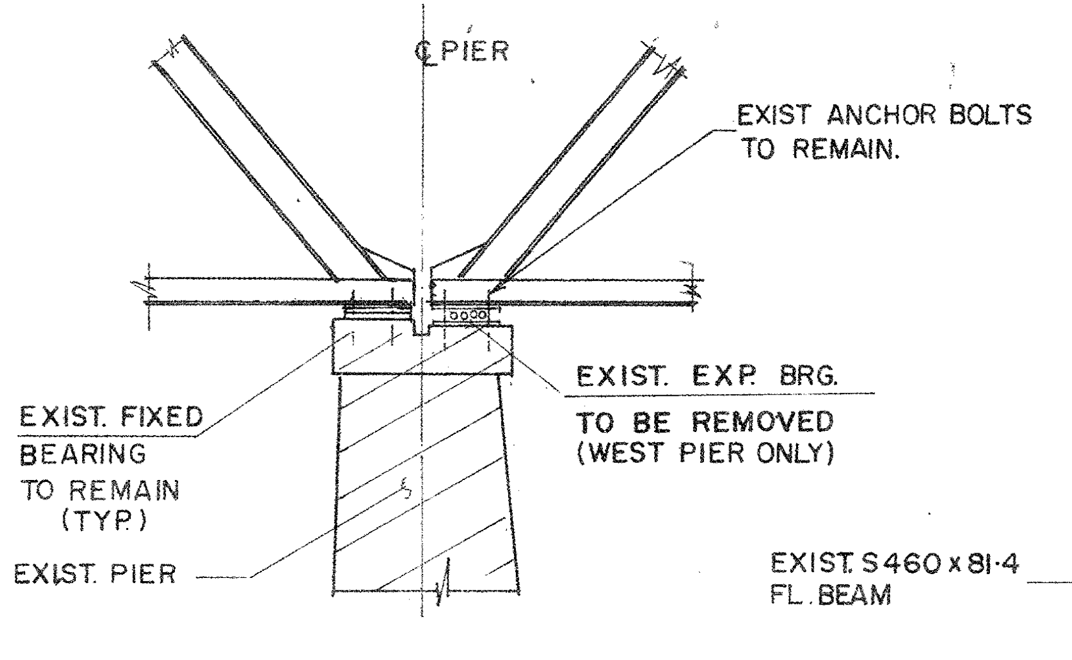
- MISSING & DETERIORATED RIVETS TO BE REPLACED
- MISSING HANDRAIL ANCHOR BOLTS FOR TOP RAIL TO BE REPLACED



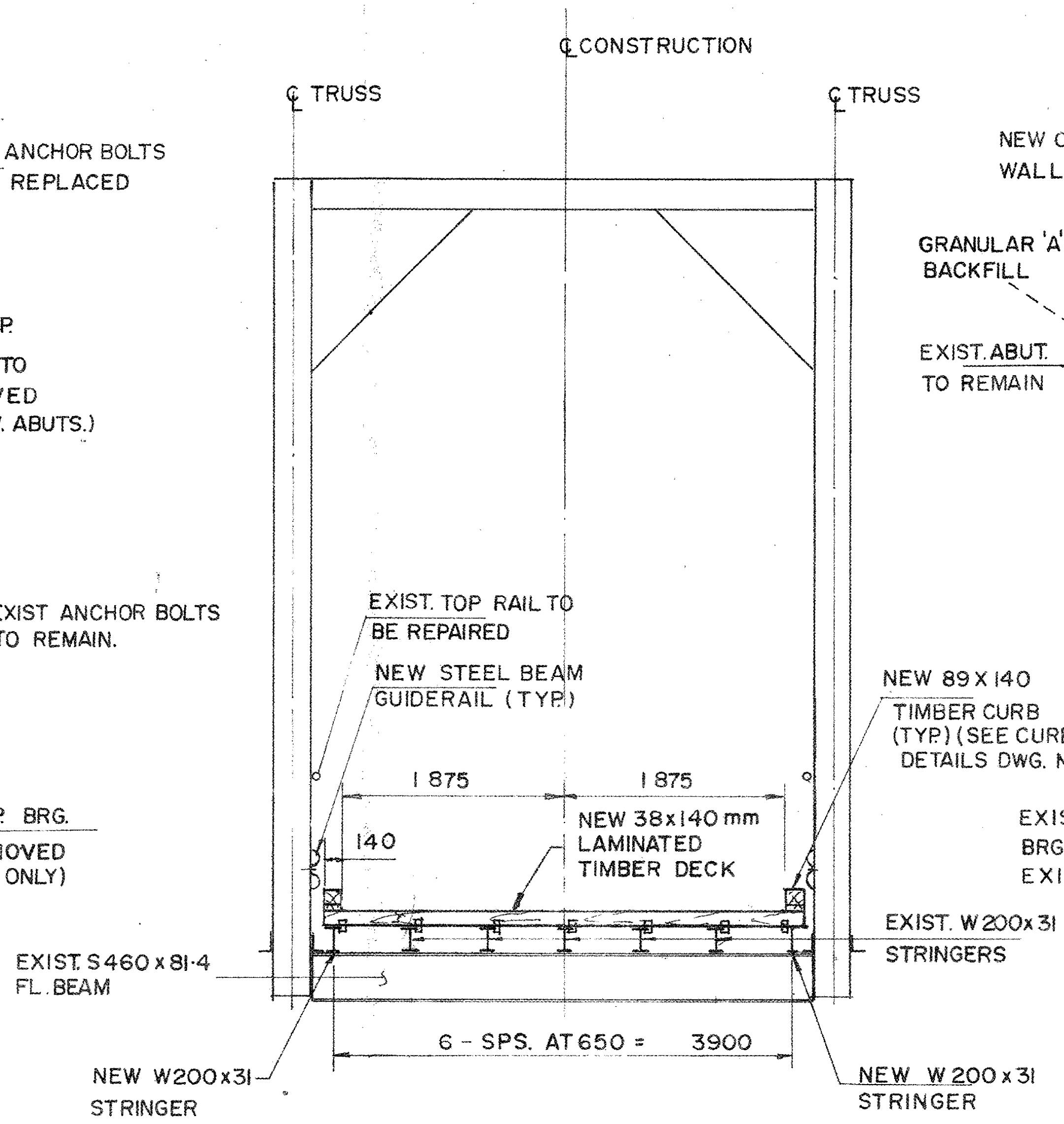
EXIST. DECK SECTION



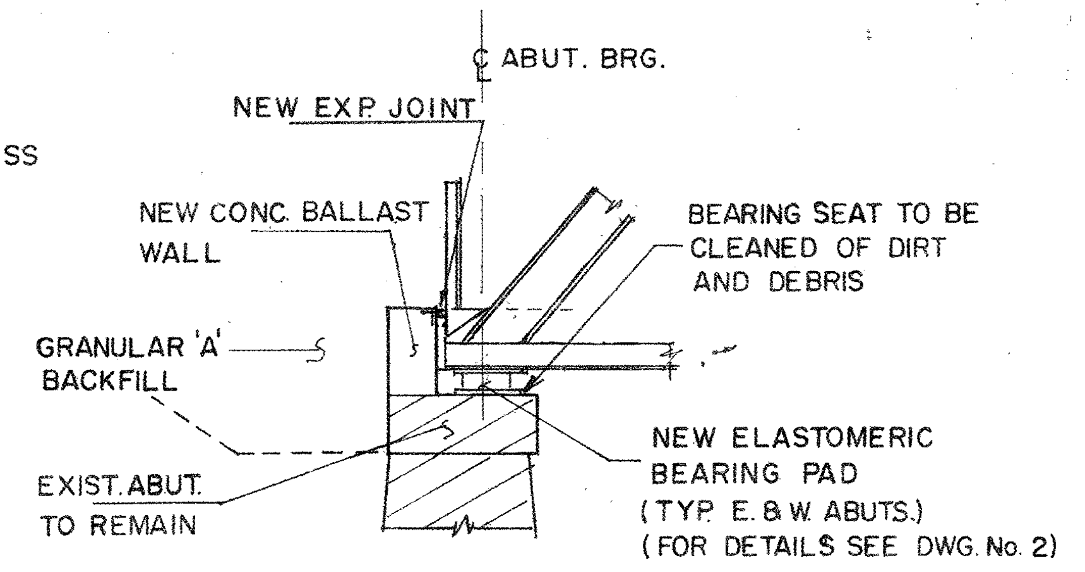
DETAIL 'B'



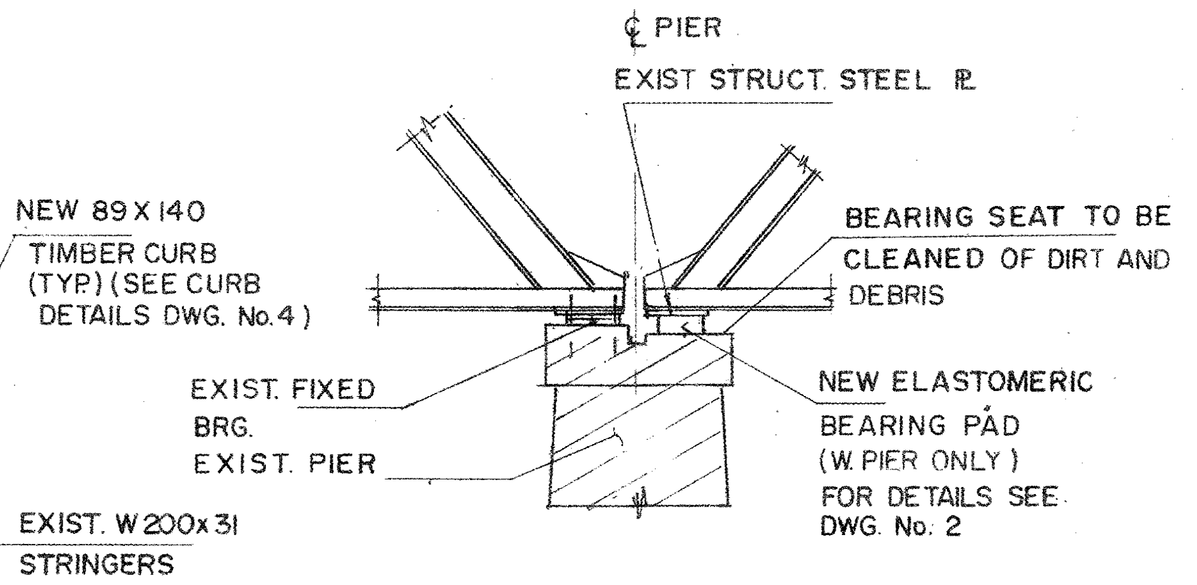
DETAIL 'A'



NEW DECK SECTION



NEW BALLAST WALL & EXP. BEARING



NEW EXP. BEARING PIER

REFERENCE DRAWINGS

DETAILS OF EXISTING STRUCTURE HAVE BEEN DERIVED FROM DRAWINGS OF THE MISSISSAUGI RIVER BRIDGE BY THE DEPT. OF PUBLIC WORKS, ONTARIO DATED SEPT. 1907

NOTES

SPECIFICATIONS

- ONTARIO PROVINCIAL STANDARD SPECIFICATIONS, DIVISION 9.
- ONTARIO HIGHWAY BRIDGE DESIGN CODE, 1983.
- LIVE LOAD O.H.B.D.C. L1 = 16 TONNES, L2 = 24 TONNES, L3 = 28 TONNES

CLASS OF CONCRETE

- ALL CONCRETE . . . . 30 MPa
- MAXIMUM AGGREGATE SIZE. . . . . 19 mm
- ALL CONCRETE REPAIR MATERIALS SHALL BE APPLIED IN ACCORDANCE WITH THE MANUFACTURER'S RECOMMENDATIONS.

REINFORCING STEEL

- ALL REINFORCING STEEL SHALL BE IN ACCORDANCE WITH C.S.A. G30.12M 1977, GRADE 400, UNLESS OTHERWISE NOTED.
- REINFORCING STEEL SHALL BE FIELD CUT AS DIRECTED BY THE ENGINEER AND AS A NON-PAY ITEM.
- CLEAR COVER TO REINFORCING BARS 70 ± 20mm UNLESS NOTED.
- ALL REINFORCING BARS SHALL BE COATED BARS

STRUCTURAL STEEL

- ALL STRUCTURAL STEEL SHALL BE IN ACCORDANCE WITH C.S.A. G40.21, GRADE 300W OR EQUAL.
- ⚠ ALL BOLTS SHALL BE ASTM A325 TYPE I, M22 UNLESS NOTED
- ALL NEW STRUCTURAL STEEL, EXCEPT AS OTHERWISE NOTED, SHALL BE PAINTED WITH EPOXY ZINC PRIMER IN ACCORDANCE WITH OPSS 912. CONTRACTOR TO SUPPLY ALL PAINT.
- ⚠ ALL EXISTING STRUCTURAL STEEL IN CONTACT WITH NEW STRUCTURAL STEEL OR TIMBER SHALL BE CLEANED. SURFACES IN CONTACT WITH TIMBER SHALL BE PAINTED WITH EPOXY ZINC PRIMER IN ACCORDANCE WITH OPSS 912
- CONTRACTOR SHALL FIELD DRILL HOLES FOR ALL CONNECTIONS.
- ALL WELDING SHALL BE IN ACCORDANCE WITH C.S.A. SPECIFICATION W59 AND SHALL BE DONE BY A WELDER QUALIFIED UNDER C.S.A. W47.
- ALL NEW STRUCTURAL STEEL NOSING ANGLES SHALL BE HOT DIP GALVANIZED IN ACCORDANCE WITH C.S.A. STANDARD G164-M OR ZINC METALLIZED AFTER FABRICATION IN ACCORDANCE WITH C.S.A. STANDARD G189.

TIMBER

- THE DECK LAMS SHALL BE PLACED WITH DRESSED EDGE BEARING ON STRINGER BEAMS VERTICALLY AND TIGHTLY AGAINST PRECEEDING LAM.
- LAMS SHALL BE FINISHED S2S1E.
- ⚠ ALL TIMBER TO BE STRUCTURAL GRADE № 2 OR BETTER, JACK PINE OR APPROVED EQUAL UNLESS NOTED OTHERWISE
- LAMS TO BE NAILED AT 345 mm O.C. USING 115 mm ARDOX NAILS, STAGGERED.
- ALL TIMBER TO BE CREOSOTED IN ACCORDANCE WITH C.S.A. SPECIFICATION 080 APPENDIX 'A' LATEST REVISIONS, WITH MINIMUM RETENTION OF 130 Kgs PRESERVATIVE PER m^3. CARE MUST BE TAKEN IN HANDLING CREOSOTED TIMBER TO AVOID DEFACEMENT OF SURFACE. NO HOOKS, CHAINS OR PEAVIES TO BE USED IN HANDLING.
- ALL HOLES DRILLED IN FIELD IN TREATED TIMBER SHALL BE TREATED WITH HOT CREOSOTED OIL APPLIED WITH A BOLT HOLE TREATER SUPPLIED BY CONTRACTOR.
- WHEREVER A FRESH SURFACE IS EXPOSED BY CUTTING IN THE FIELD, IT MUST BE TREATED WITH THREE BRUSH COATS OF HOT CREOSOTED OIL SUPPLIED BY CONTRACTOR.
- ALTERNATIVE METHOD OF TIMBER TREATMENT WILL BE CONSIDERED SUBJECT TO APPROVAL OF THE ENGINEER.

NOTE

* DENOTES POINT OF BRIDGE TO BE JACKED UP TO ACCOMMODATE NEW BEARING PAD INSTALLATION. (JACKING FORCE = 30 TONNES / TRUSS) (EXCLUDING TIMBER DECK)

THE METHOD OF JACKING UP THE BRIDGE SHOWN ON THIS DRAWING IS SCHEMATIC ONLY ALTERNATE METHODS FOR JACKING WILL BE CONSIDERED SUBJECT TO APPROVAL OF THE ENGINEER.

M.T.C. DOWNSVIEW
RECEIVED
[illegible] 1988
STRUCTURAL OFFICE
M. T. C.

FOR APPROVAL

M.T.O SITE No. 38S-234

KRESIN ENGINEERING & PLANNING

523 WELLINGTON ST. EAST
SAULT STE. MARIE, ONTARIO
942-2612

G. L. ALEONG

No.	DATE	BY	REVISIONS
⚠	OCT, 88	G.L.A	REVISIONS AS PER M.T.O. REQUEST

DESIGNED: G. O. L.
DRAWN: R. A. S.
CHECKED: D. L. B.
APPROVED: G. L. A.
SCALE: 1:50 EXCEPT AS NOTED

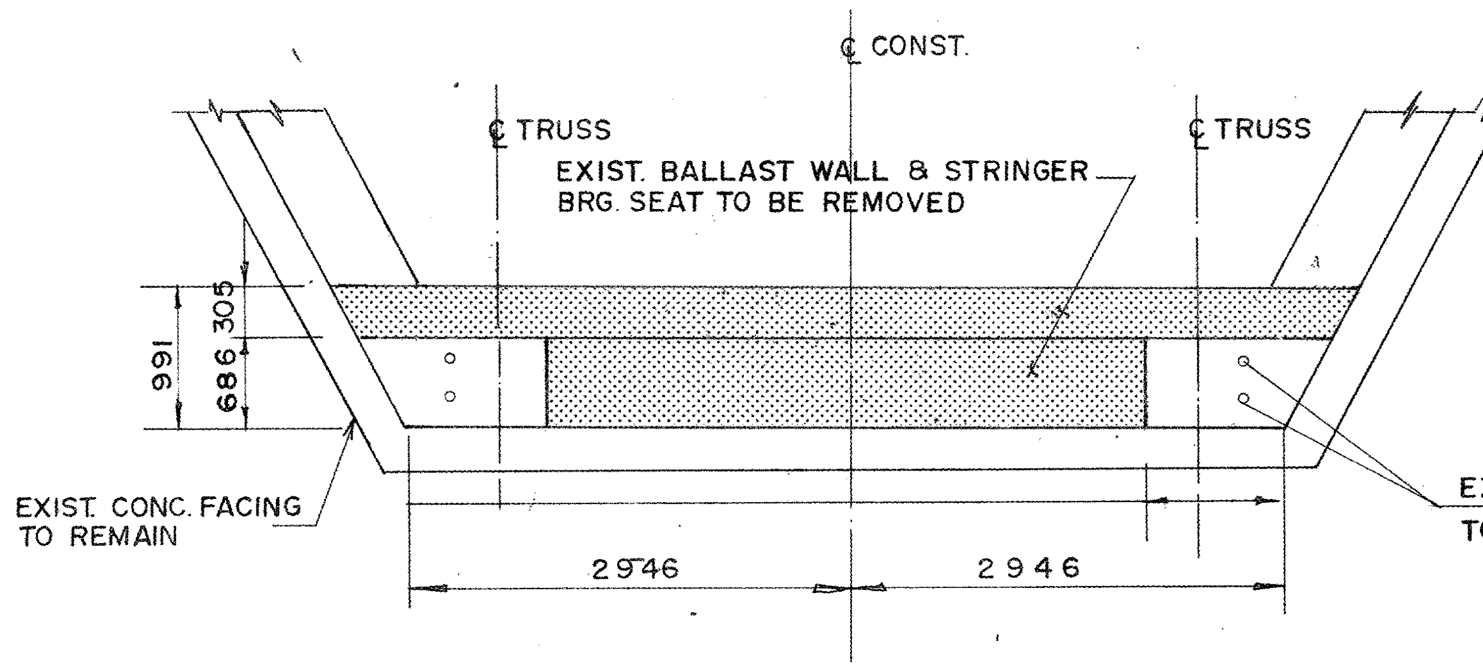
DEAN LAKE BRIDGE
TOWNSHIP OF THOMPSON
DISTRICT OF ALGOMA
LOT
MISSISSAUGI RIVER
CON. VI

STRUCTURE SITE No

GENERAL ARRANGEMENT

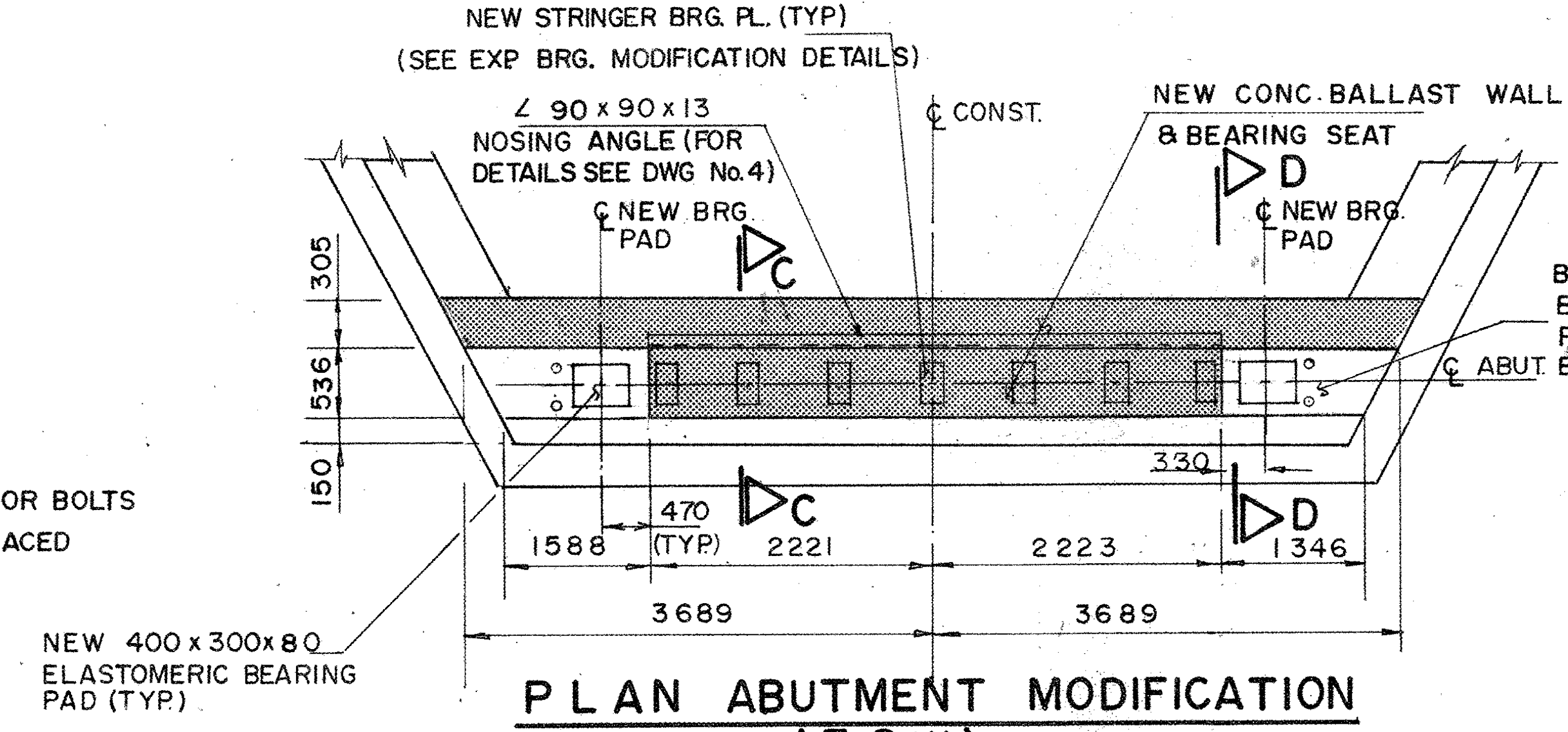
DATE: AUG, 1988
PROJECT: 42-8617
DRAWING: 385-234

1



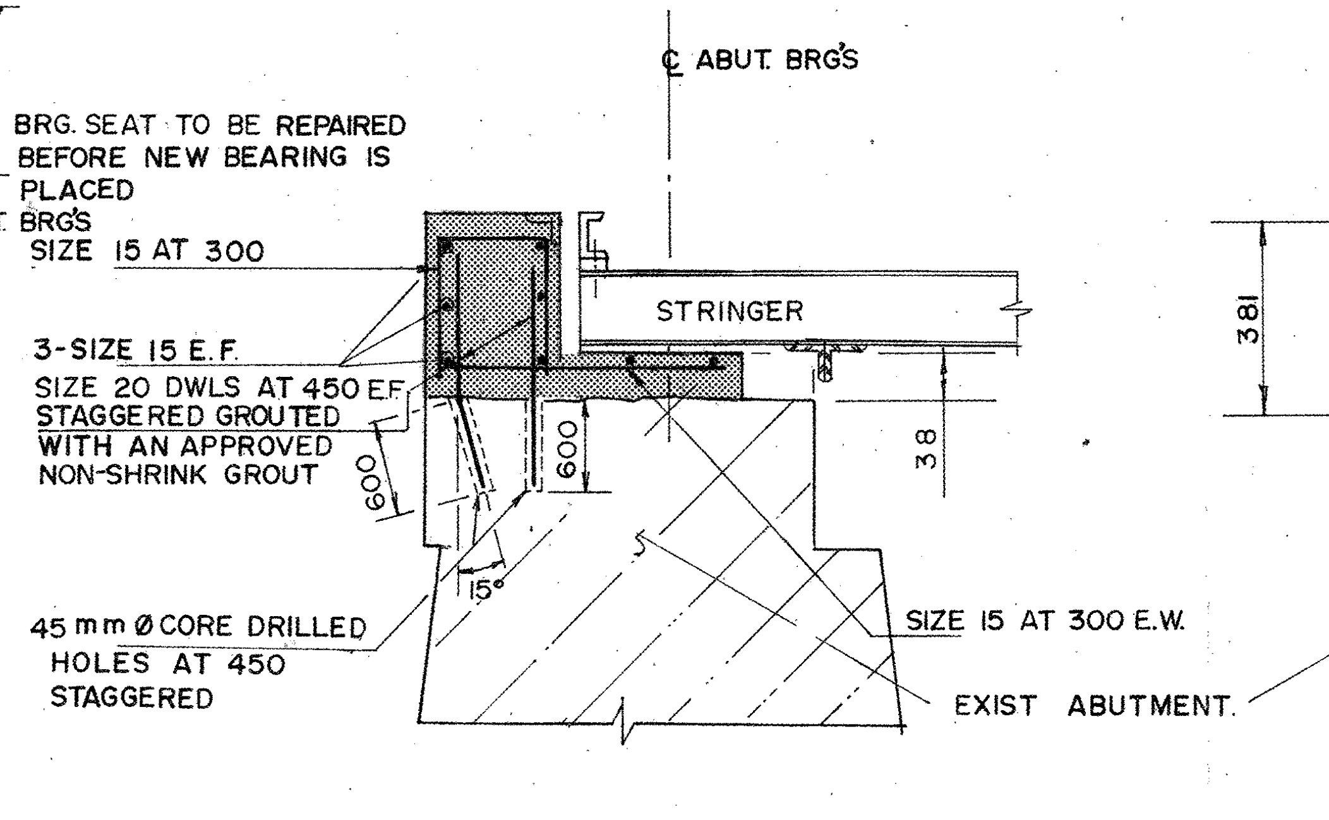
PLAN EXIST. ABUT. (E & W)

SCALE 1:50



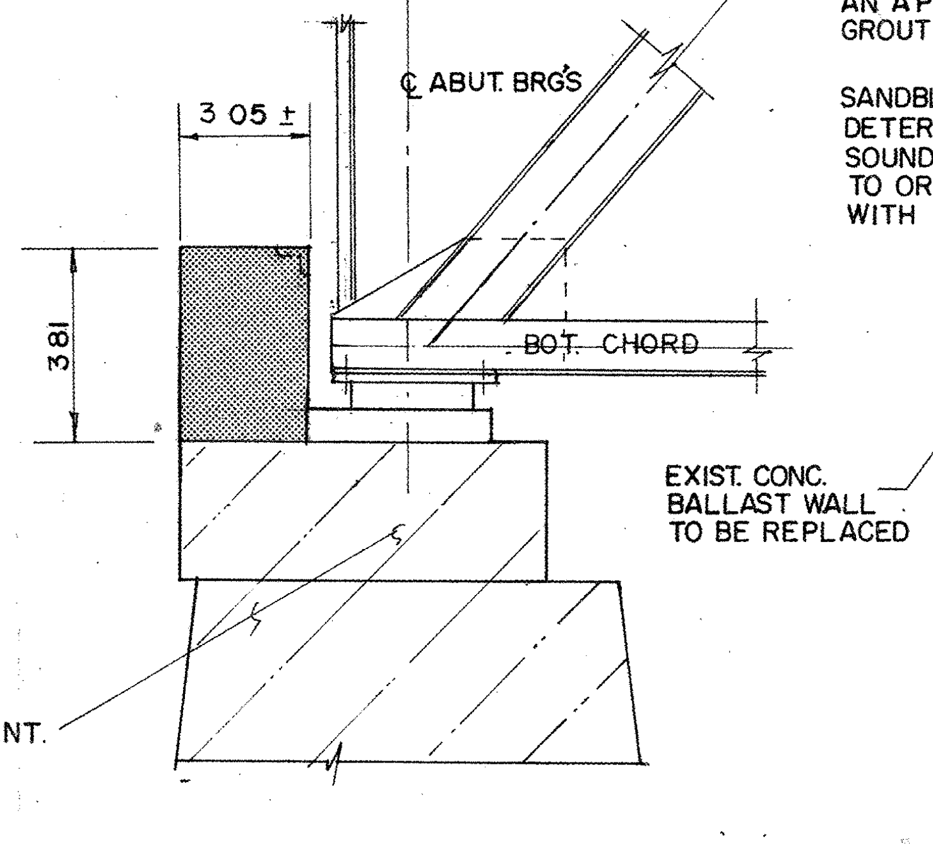
PLAN ABUTMENT MODIFICATION (E & W)

SCALE 1:50



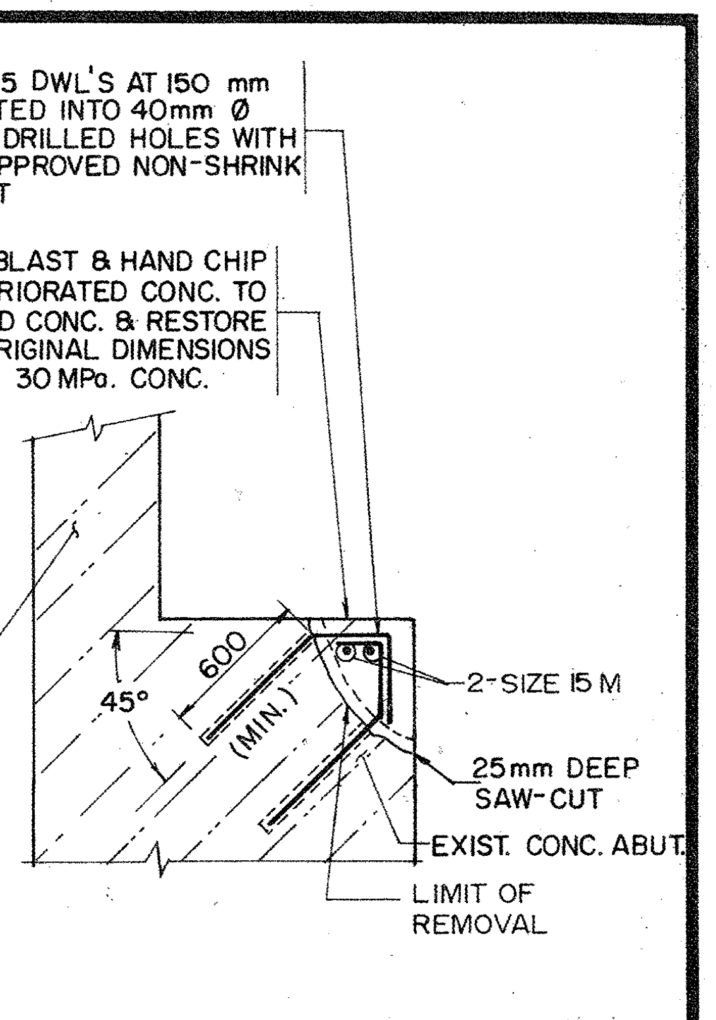
SECTION C-C

SCALE 1:20



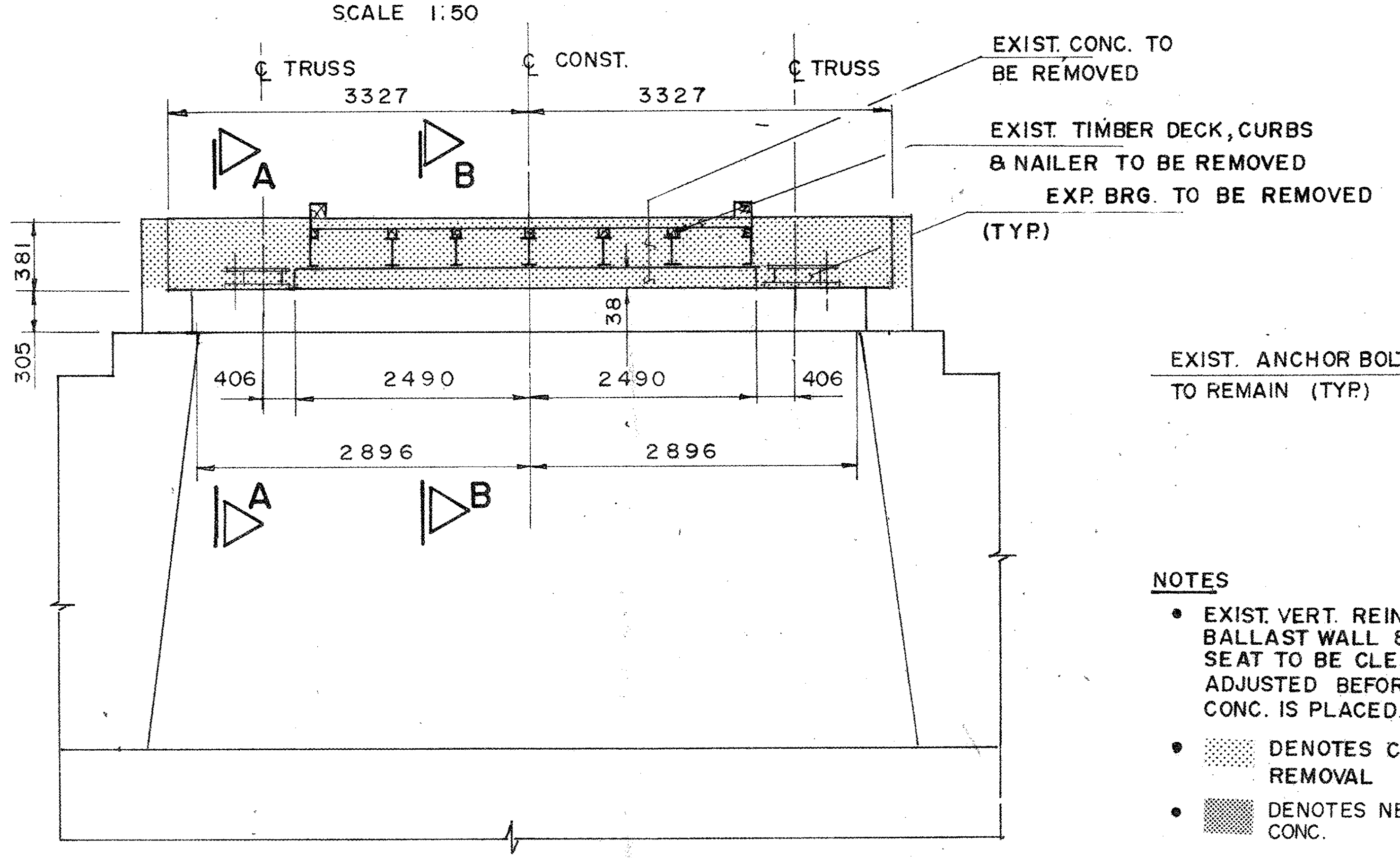
SECTION D-D

SCALE 1:20



TYPICAL ABUTMENT BEARING SEAT REPAIR DETAIL

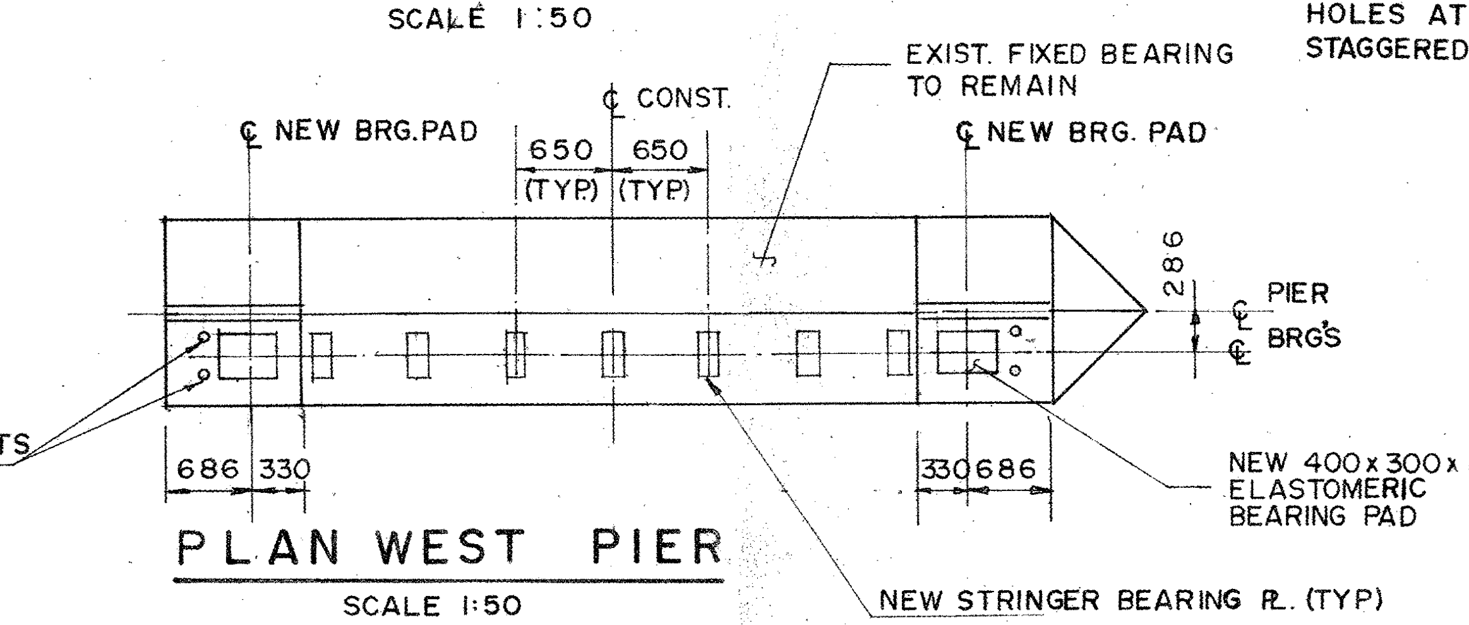
N.T.S.



ELEVATION EXIST. ABUT. (E & W)

SCALE 1:50

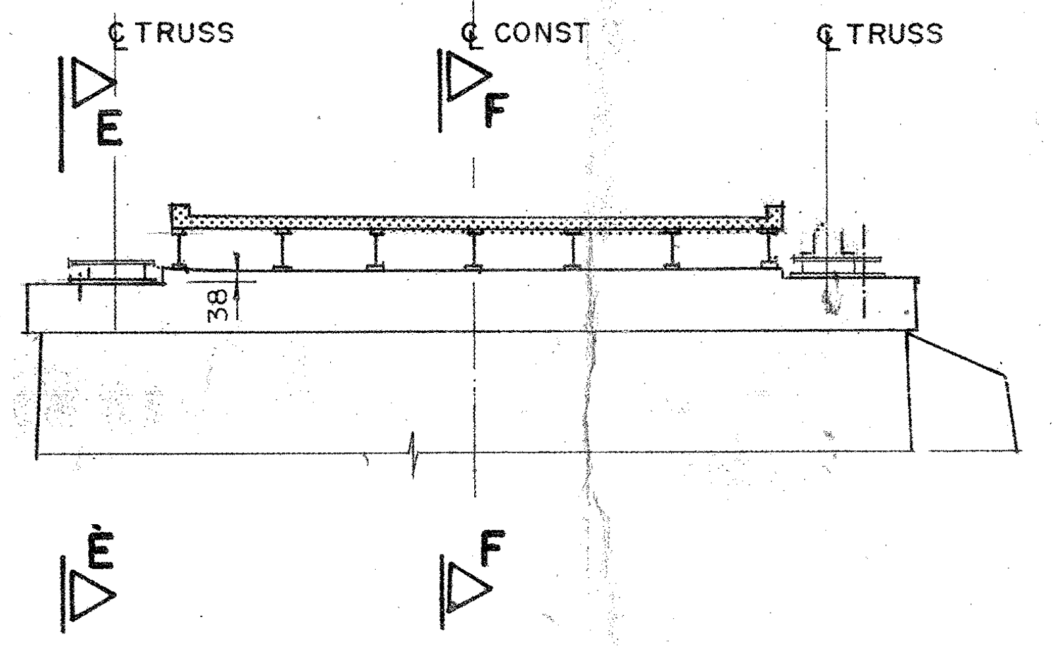
NOTE
TRUSS MEMBERS NOT SHOWN FOR CLARITY



PLAN WEST PIER

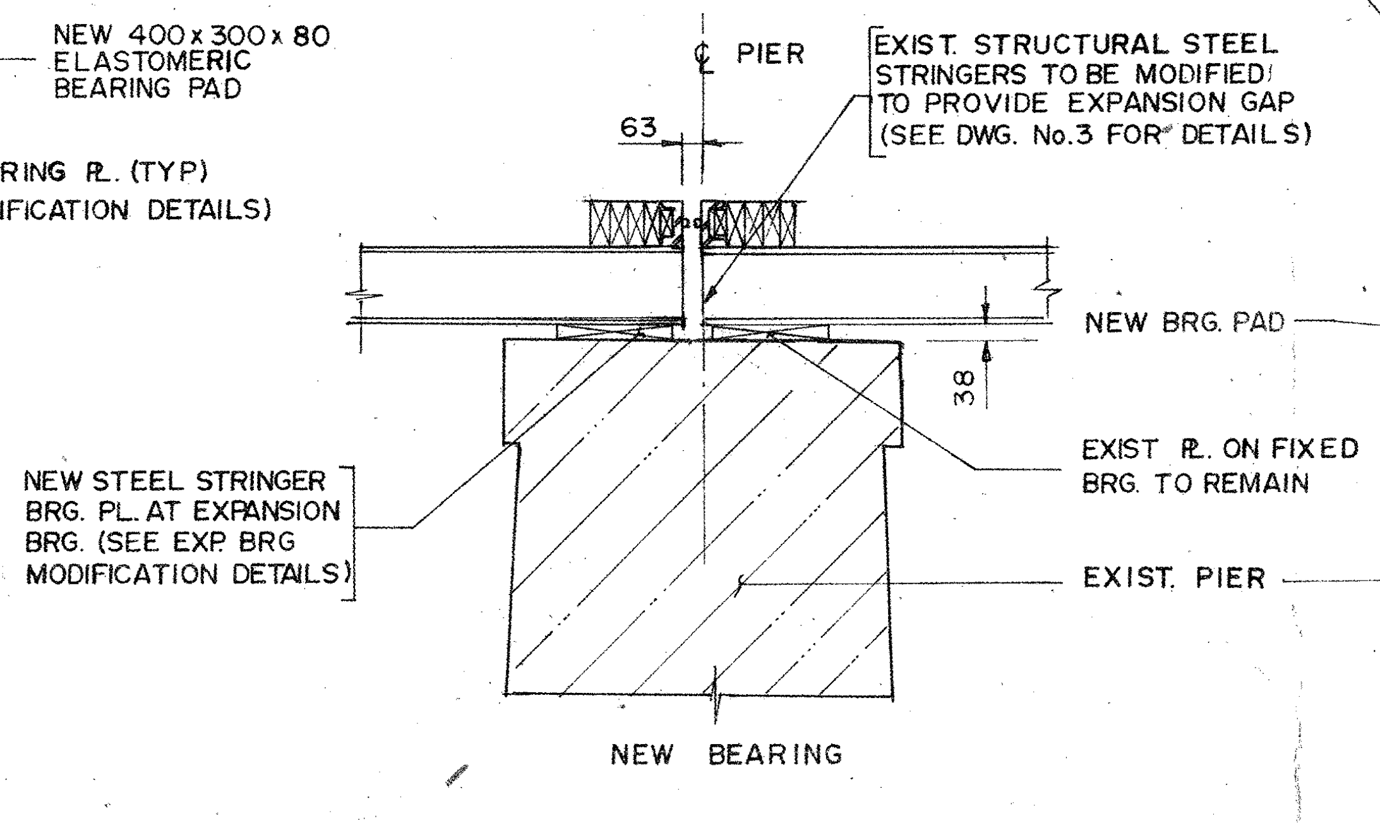
SCALE 1:50

NOTES
- EXIST. VERT. REINF. BARS IN BALLAST WALL & BRG. SEAT TO BE CLEANED & ADJUSTED BEFORE NEW CONC. IS PLACED.
- ▨ DENOTES CONC. REMOVAL
- ▩ DENOTES NEW CONC.



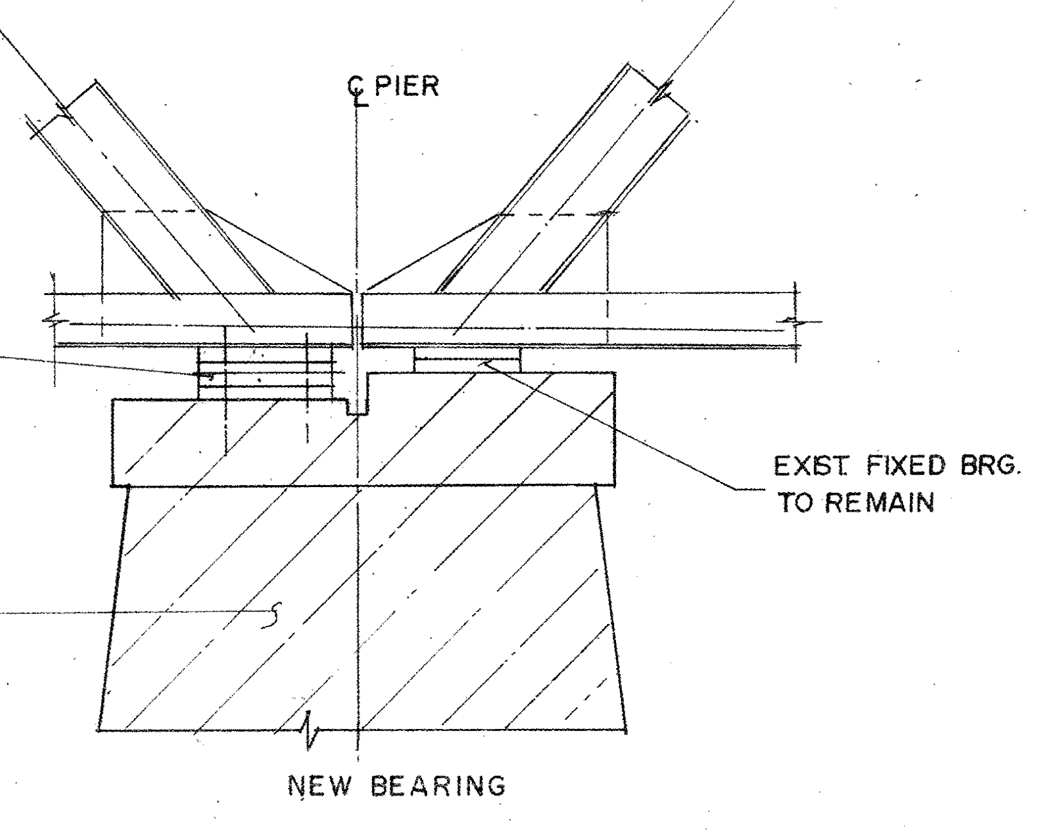
ELEVATION WEST PIER

SCALE 1:50



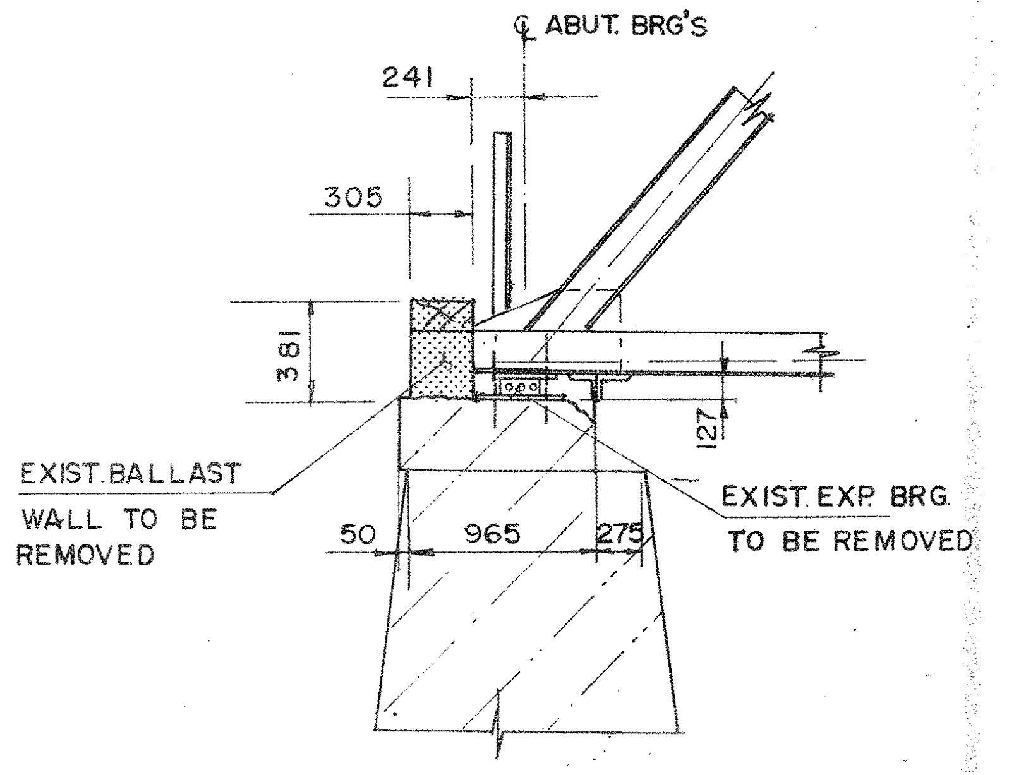
⚠ SECTION F-F

SCALE 1:20



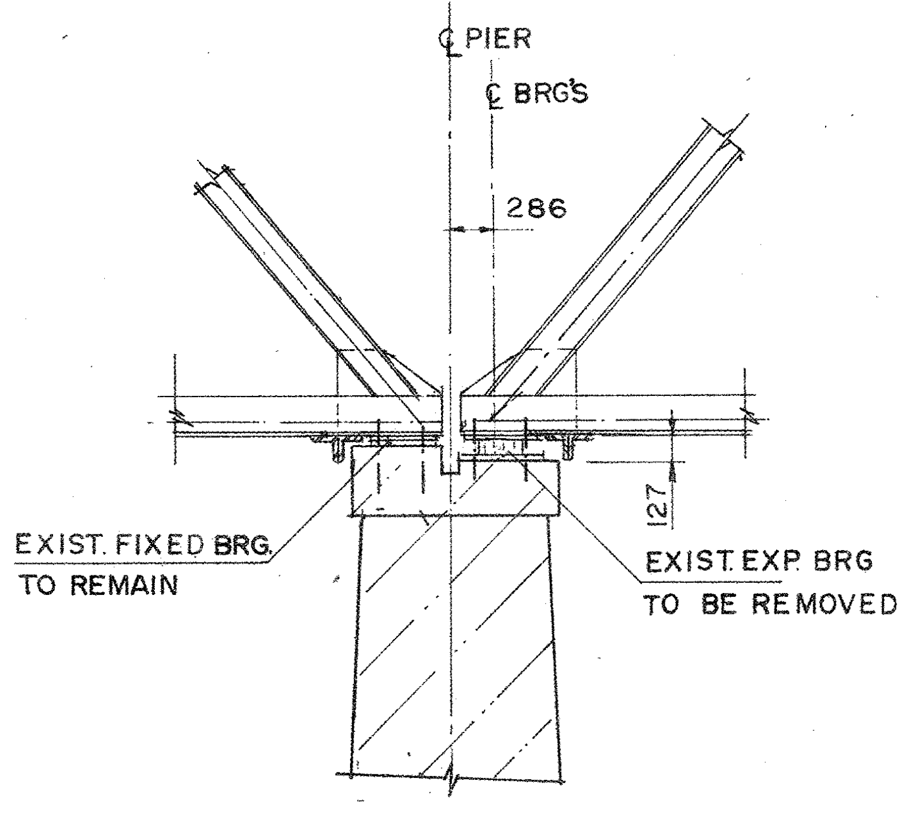
SECTION E-E

SCALE 1:20



SECTION A-A (EXIST.)

SCALE 1:40



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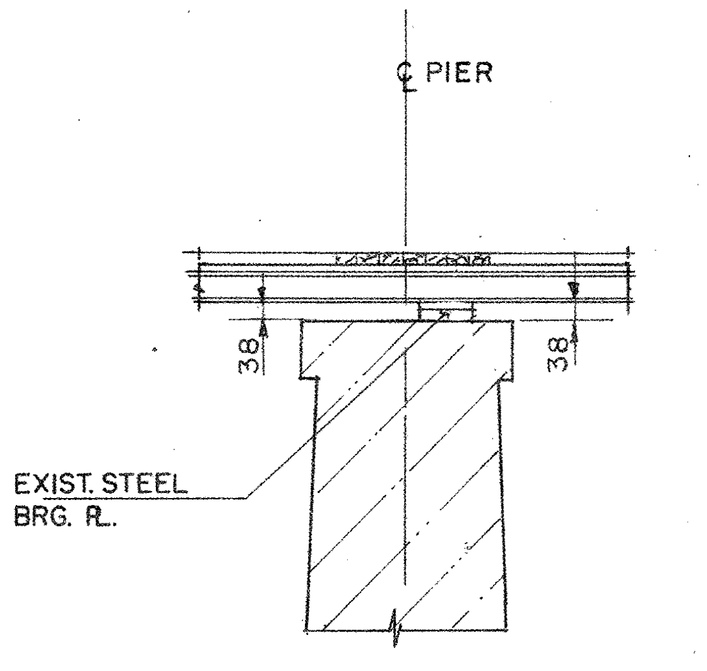

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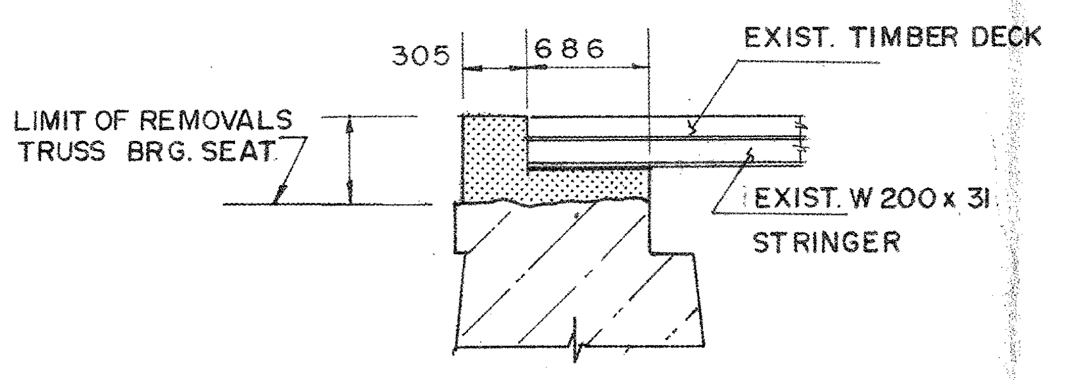
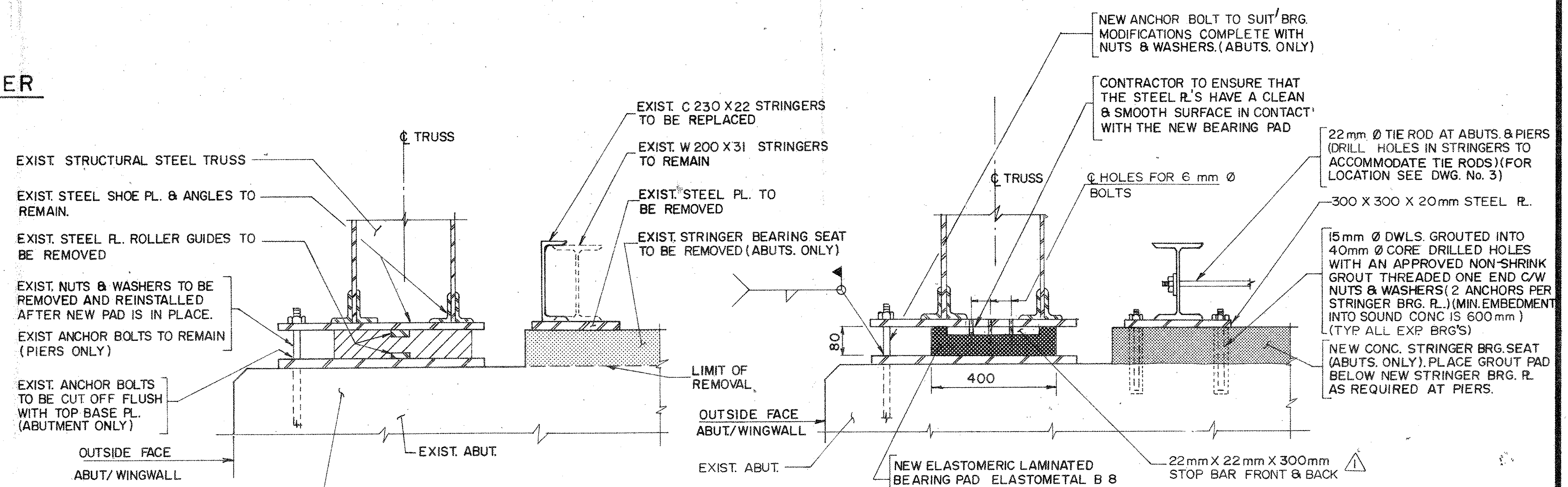


SECTION B-B (EXIST.)

SCALE 1:40



DETERIORATED TRUSS BEARING SEAT TO BE REPAIRED (SEE TYP. BRG. SEAT REPAIR DETAIL.)

EXISTING BEARING

NEW BEARING

⚠ NOTES FOR STOP BARS:
- BOLT HOLE LOCATIONS TO BE DETERMINED IN FIELD
- FIELD DRILL TOP PLATE FOR NEW BOLTS

EXPANSION BEARING MODIFICATION DETAILS

N.T.S.

MTC DOWNSVIEW
RECEIVED
NOV 01 1988
STRUCTURAL OFFICE
M.T.C.

STRUCTURE SITE No. 38S-234

FOR APPROVAL

M.T.O SITE No. 38S-234

KRESIN
ENGINEERING & PLANNING

523 WELLINGTON ST. EAST
SAULT STE. MARIE, ONTARIO
942 - 2612

G. L. ALEONG

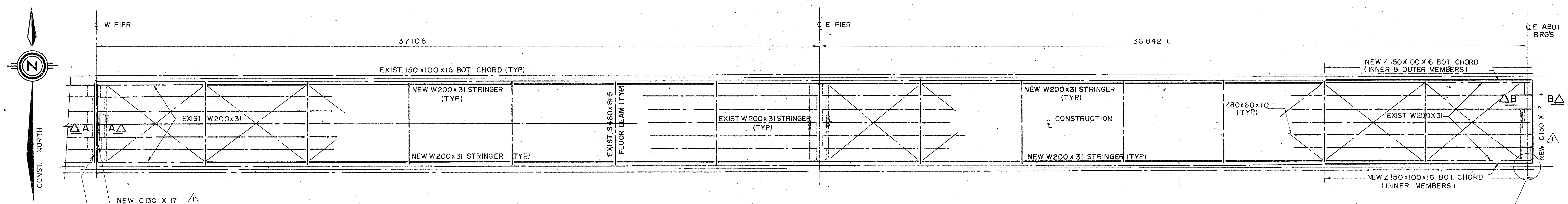
No.	DATE	BY	REVISIONS
⚠	OCT., 88	G.L.A.	REVISIONS AS PER M.T.O. REQUEST

DESIGNED: G.O.L.
DRAWN: R.A.S
CHECKED: D.L.B.
APPROVED: G.L.A.
SCALE: AS NOTED

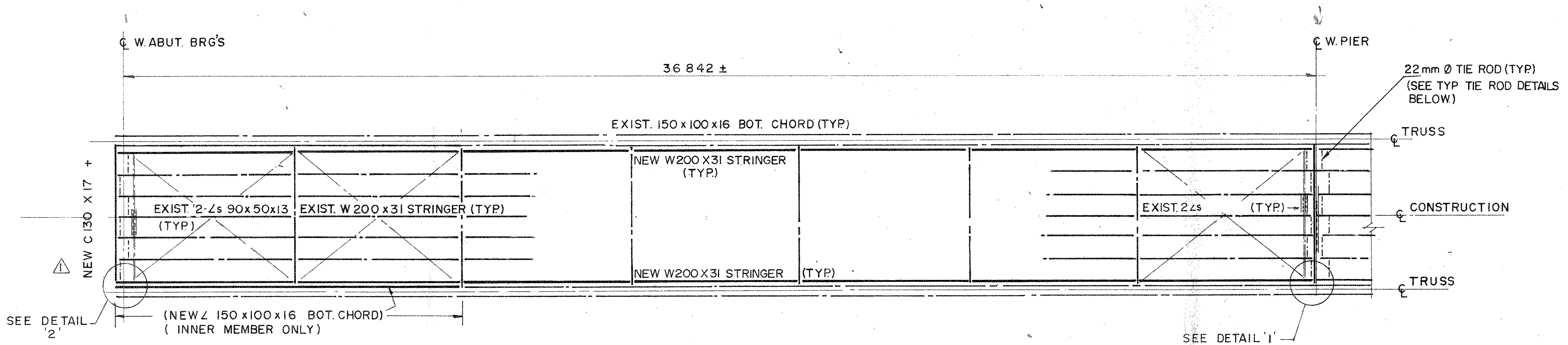
DEAN LAKE BRIDGE
TOWNSHIP OF THOMPSON
DISTRICT OF ALGOMA
LOT
MISSISSAUGI RIVER
CON. VI

ABUTMENTS PIERS & BEARING MODIFICATIONS

DATE: AUG., 1988
PROJECT: 42-8617
DRAWING: 2

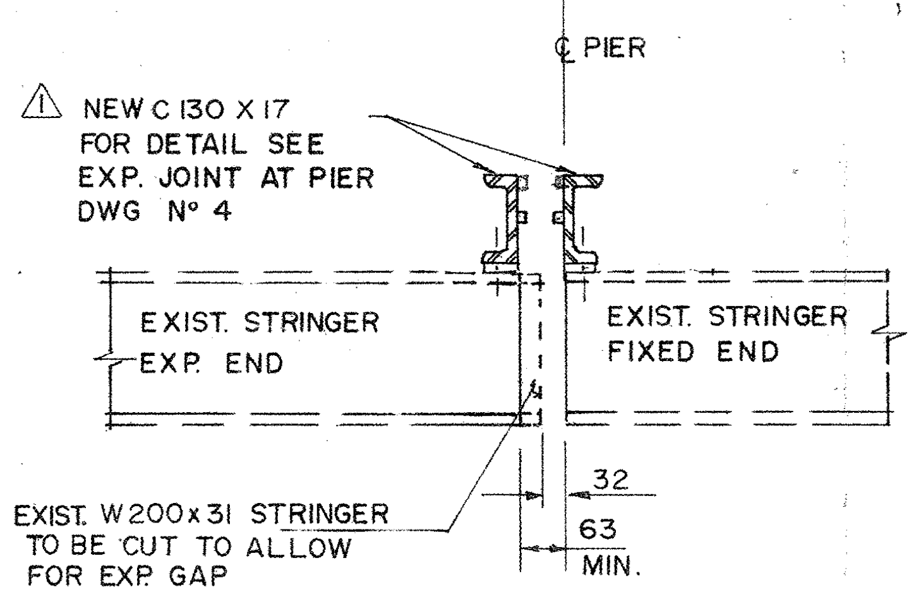


PART PLAN



PART PLAN

NOTES
- ─── DENOTES NEW MEMBER
- — - — DENOTES EXIST. MEMBER



SECTION A-A

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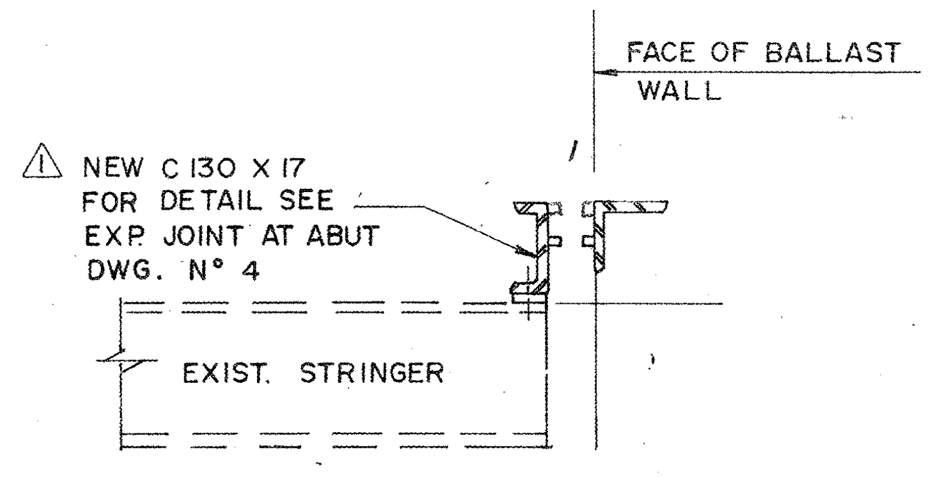

SECTION B-B

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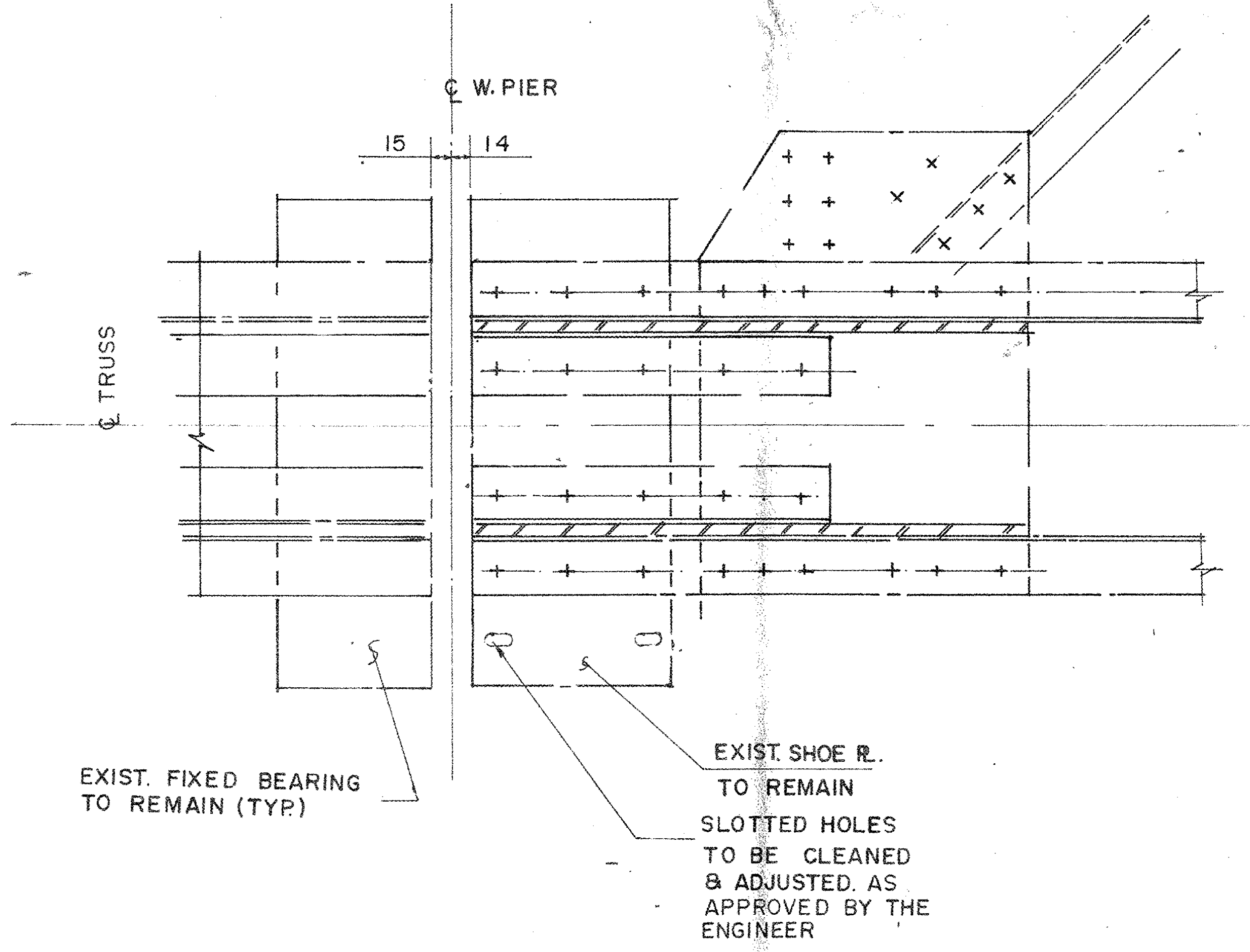

DETAIL '1'

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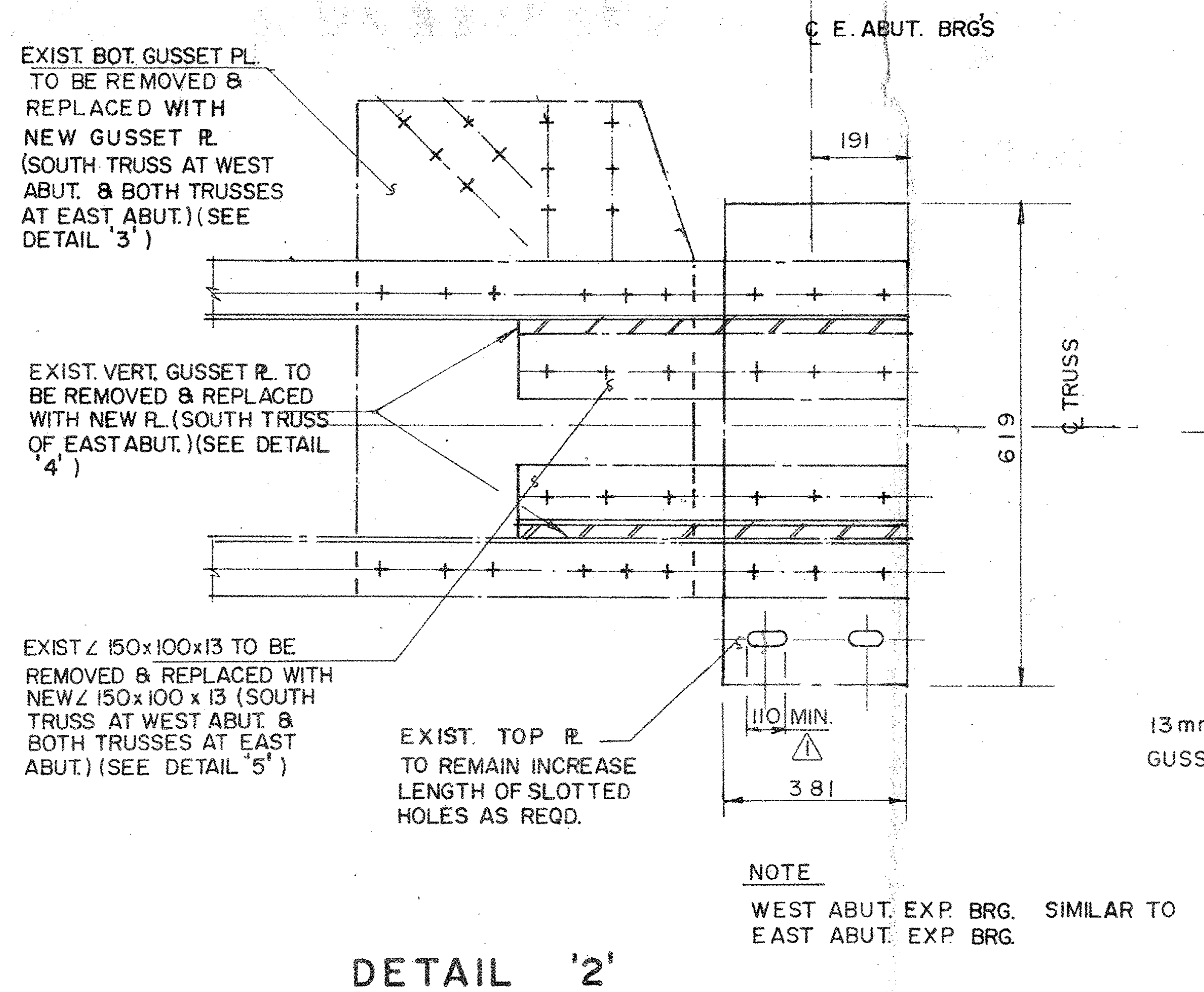
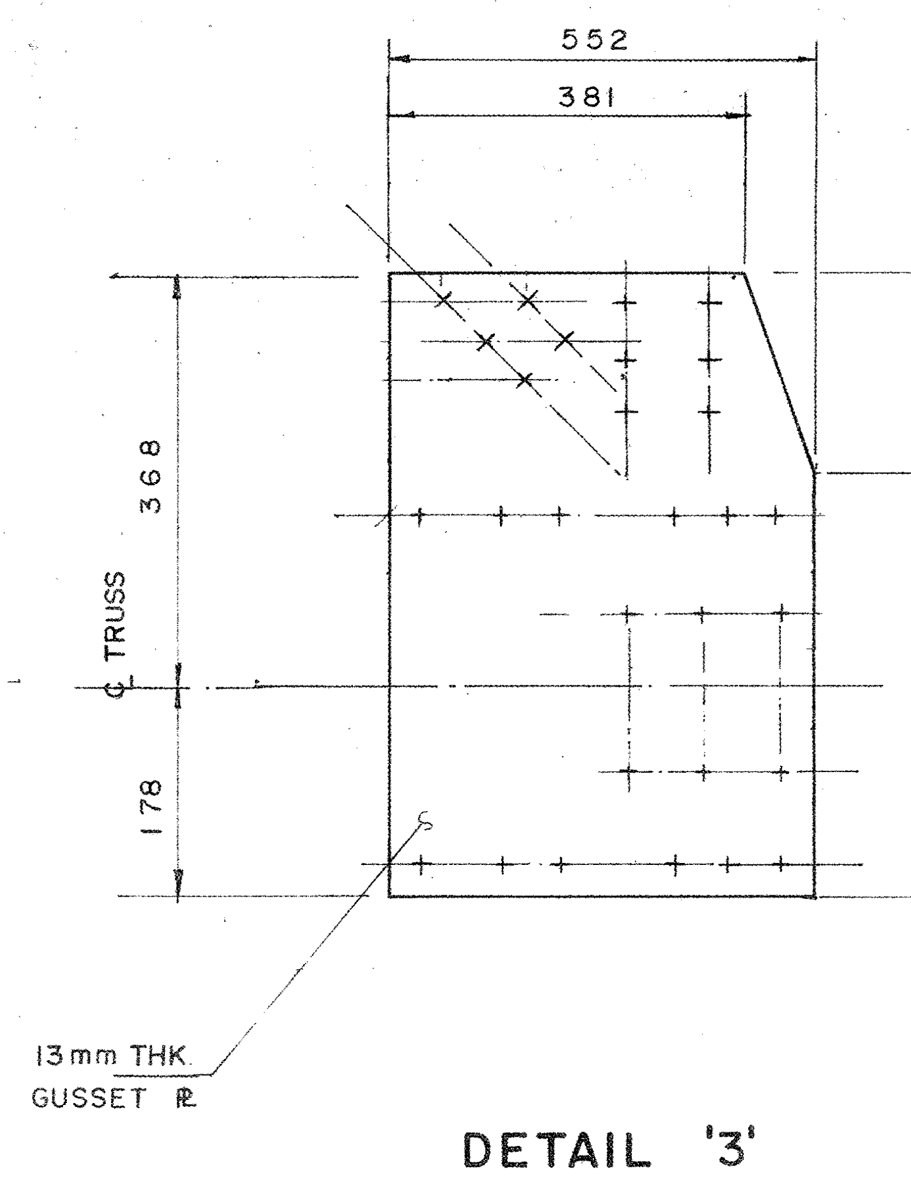


NOTE
WEST ABUT. EXP. BRG. SIMILAR TO EAST ABUT. EXP. BRG.

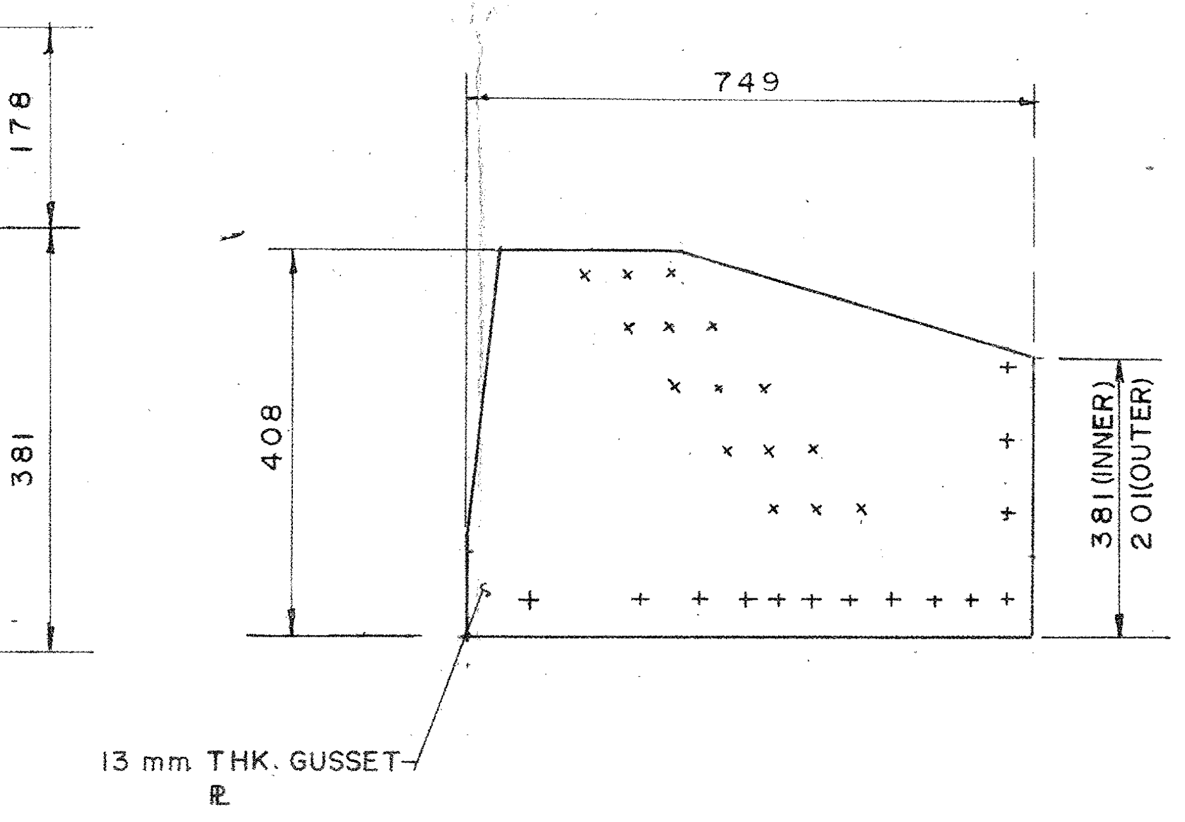
DETAIL '2'

N.T.S.



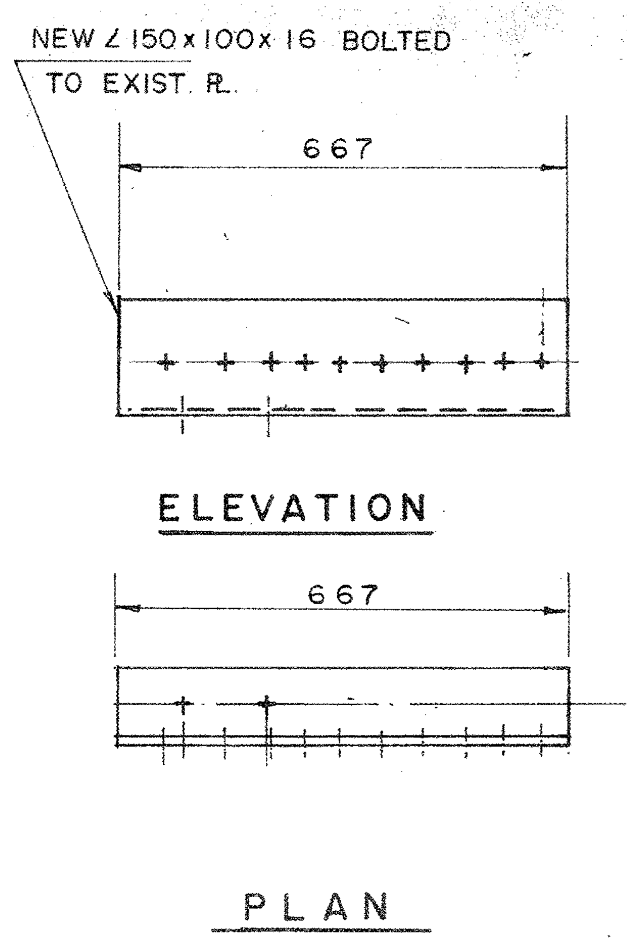
DETAIL '3'

N.T.S.
(3 Nos. REQ'D)



DETAIL '4'

(2 Nos. REQ'D)



DETAIL '5'

N.T.S.
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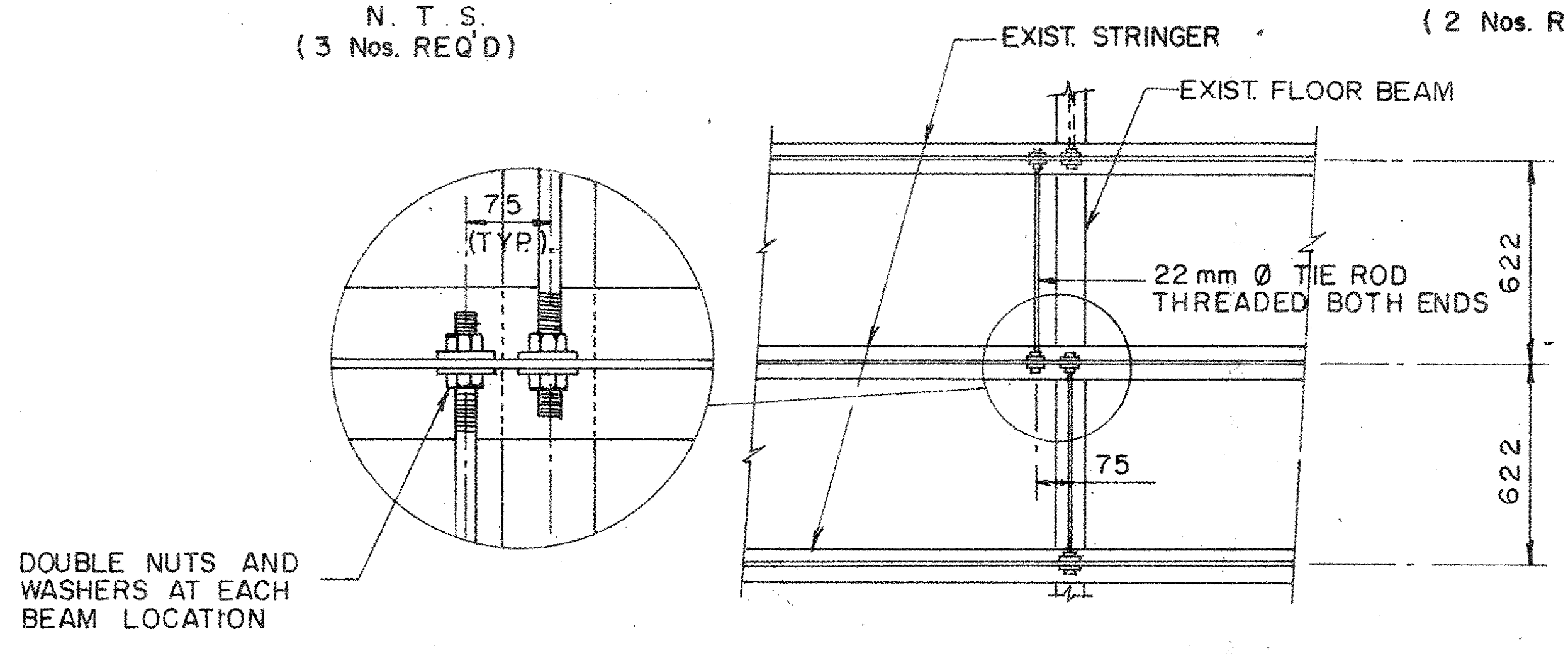


TYPICAL TIE ROD DETAILS

N.T.S.

STRUCT. [illegible] No. 38S-234

FOR APPROVAL

M.T.O SITE No. 38S-234

KRESIN ENGINEERING & PLANNING

1523 WELLINGTON ST. EAST
SAULT STE. MARIE, ONTARIO
942-2612

G. L. ALEONG

No.	DATE	BY	REVISIONS
⚠	OCT. 88	G.L.A.	REVISIONS AS PER M.T.O REQUEST

DESIGNED: G.O.L.
DRAWN: R.A.S.
CHECKED: D.L.B.
APPROVED: G.L.A.
SCALE: 1:20 EXCEPT AS NOTED

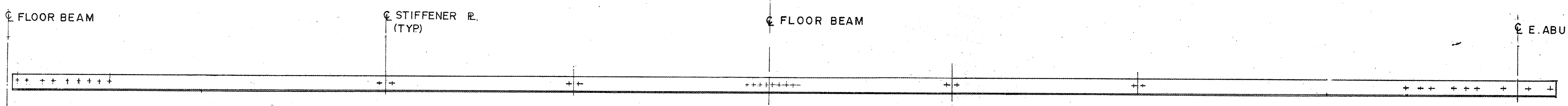
DEAN LAKE BRIDGE
TOWNSHIP OF THOMPSON
DISTRICT OF ALGOMA

LOT
MISSISSAUGI RIVER
CON. VI

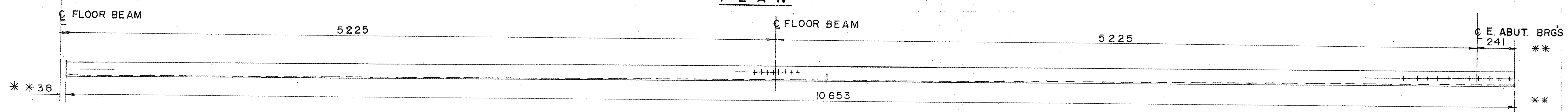
STRUCTURAL STEEL DETAILS

DATE: AUG., 1988
PROJECT: 42-8617
DRAWING: 3

NOTE
✱ ✱ DENOTES DIMENSIONS TO BE FIELD CHECKED

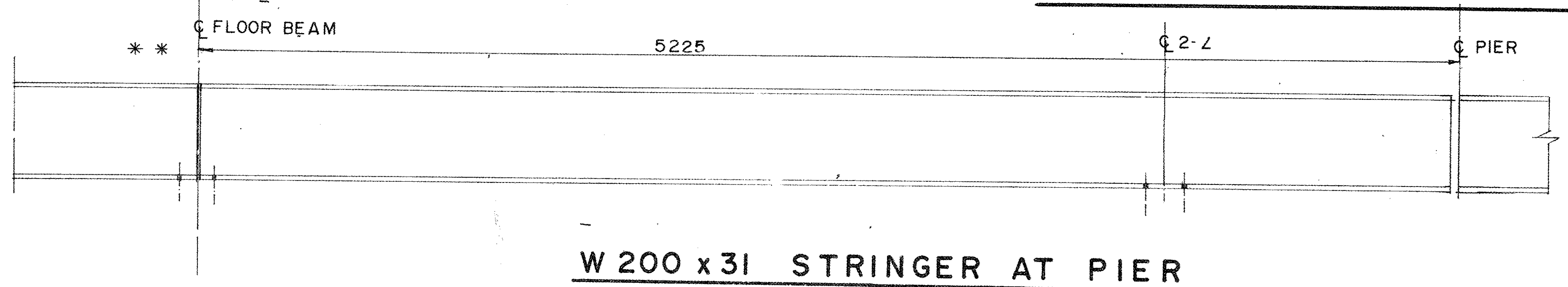


PLAN



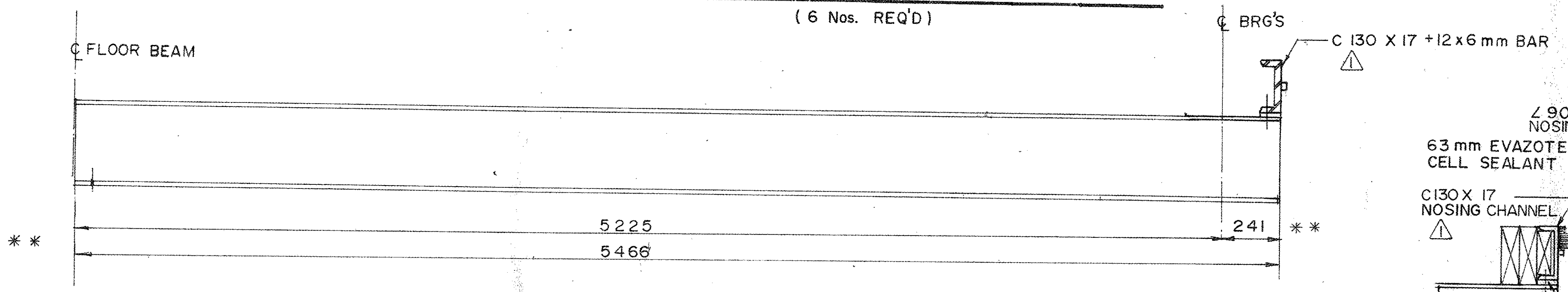
ELEVATION

∠ 150x100x16 BOT. CHORD DETAILS



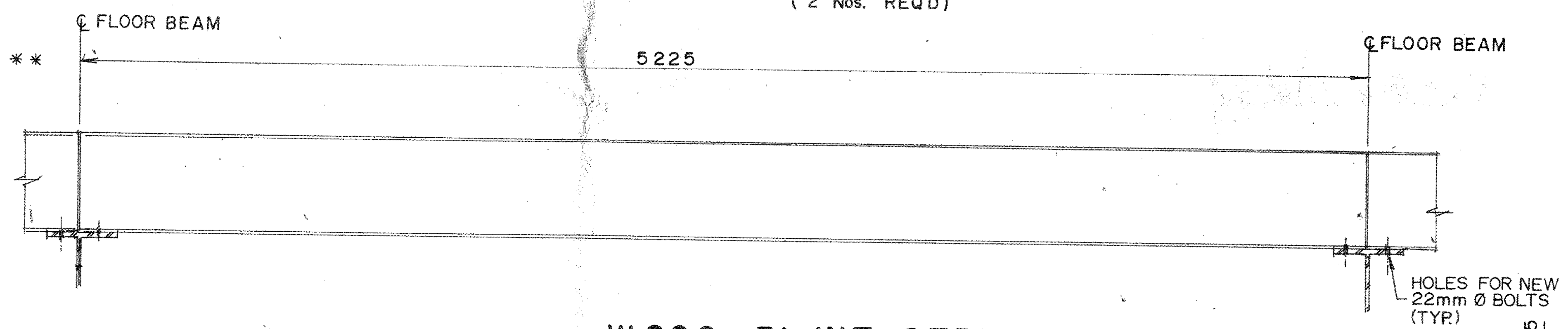
W 200 x 31 STRINGER AT PIER

(6 Nos. REQ'D)



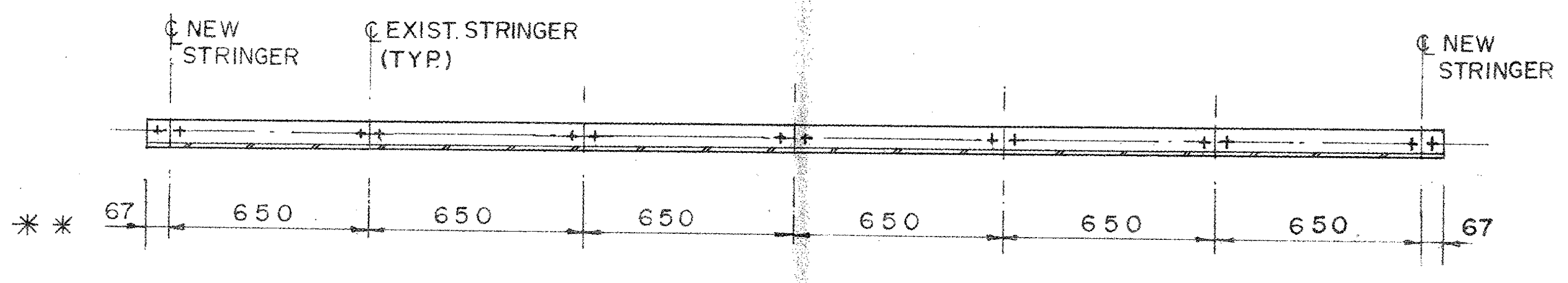
W 200 x 31 STRINGER AT ABUTS.

(2 Nos. REQ'D)

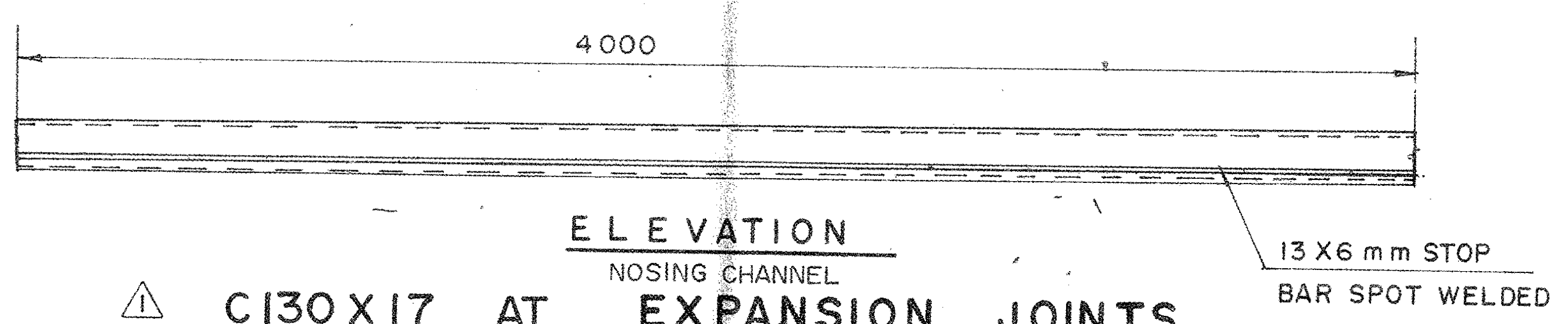


W 200 x 31 INT. STRINGERS

(30 Nos. REQ'D)



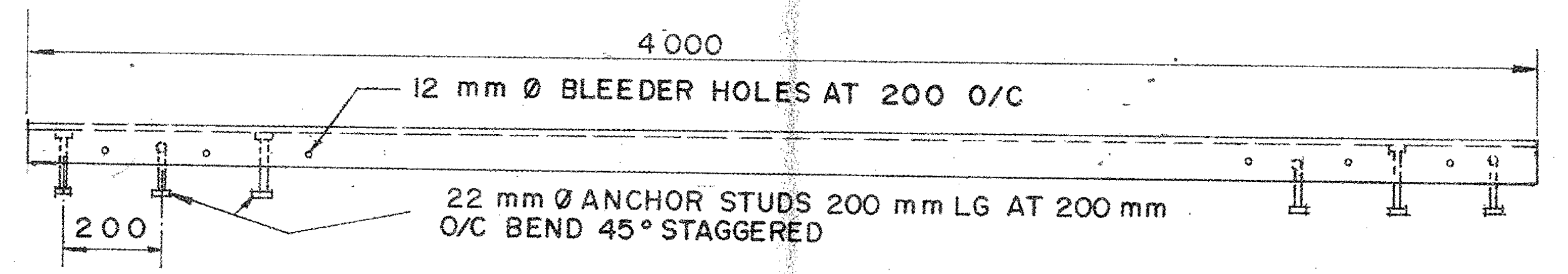
PLAN



ELEVATION

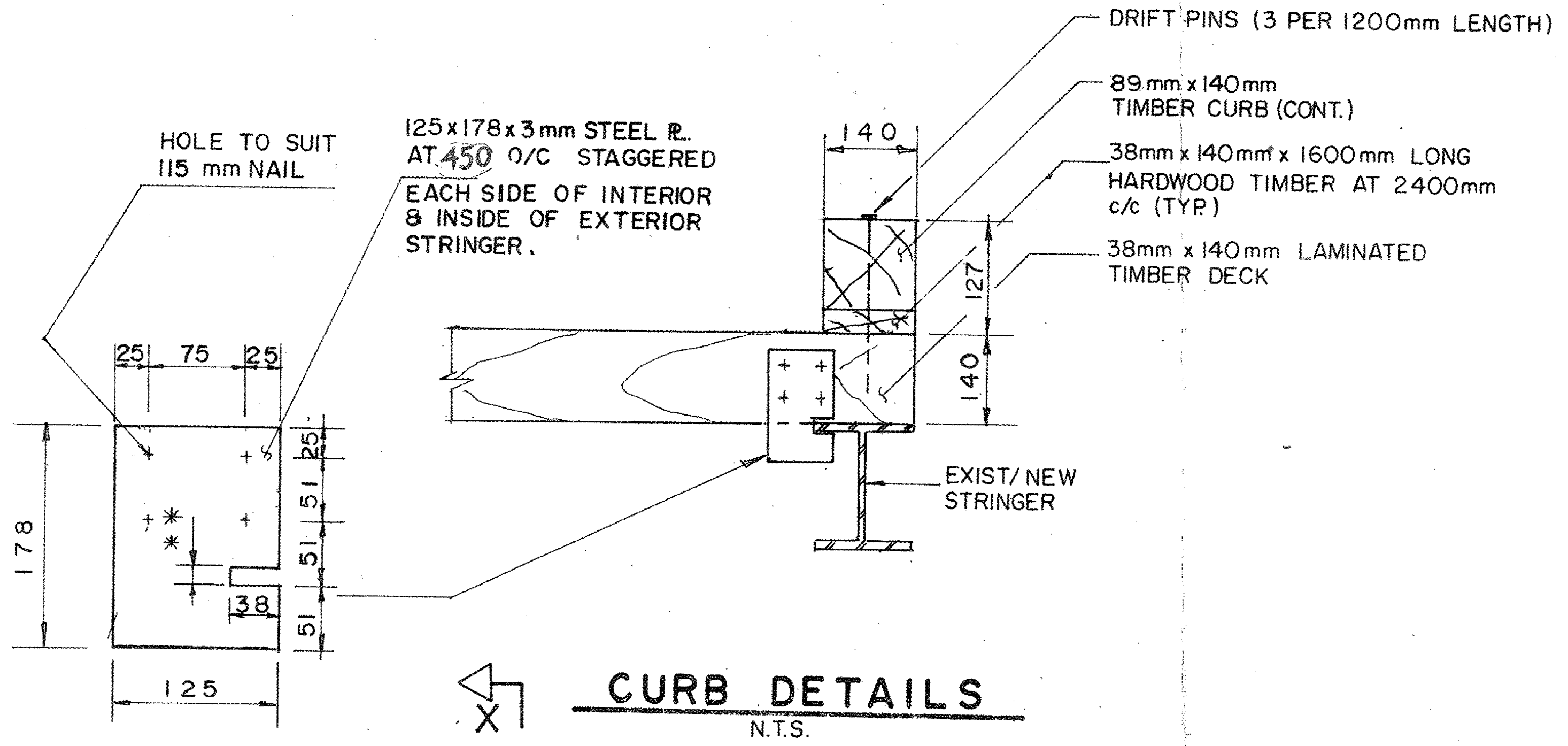
NOSING CHANNEL

⚠ C130 X 17 AT EXPANSION JOINTS



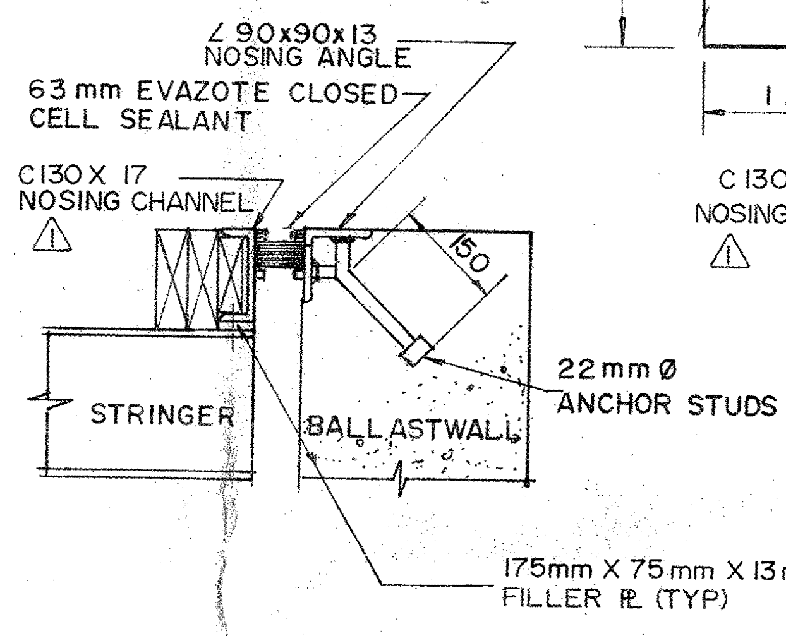
∠ 90 x 90 x 13 EXPANSION JOINT AT ABUT.

NOSING ANGLE



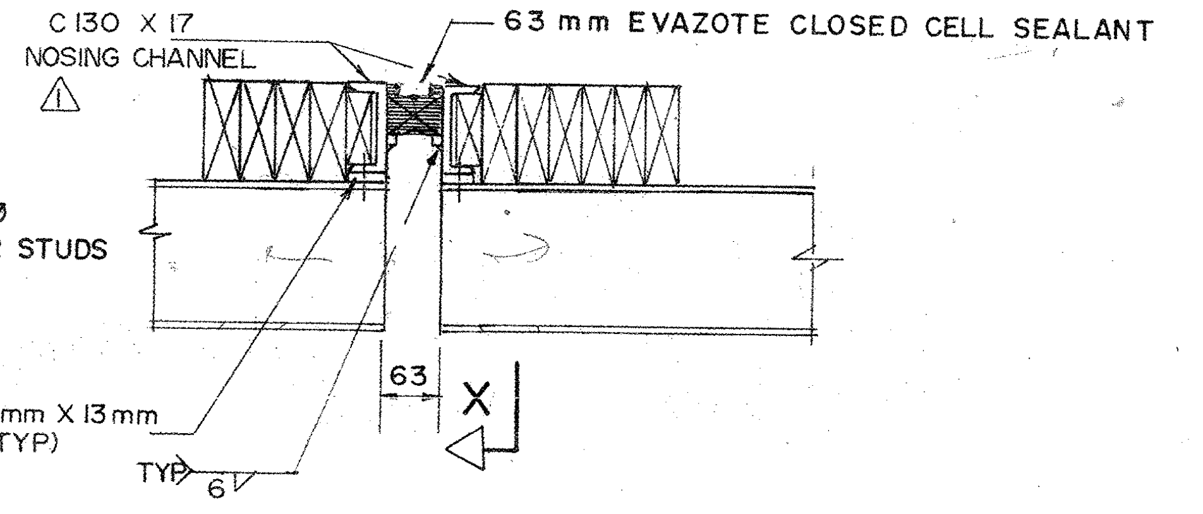
CURB DETAILS

N.T.S.



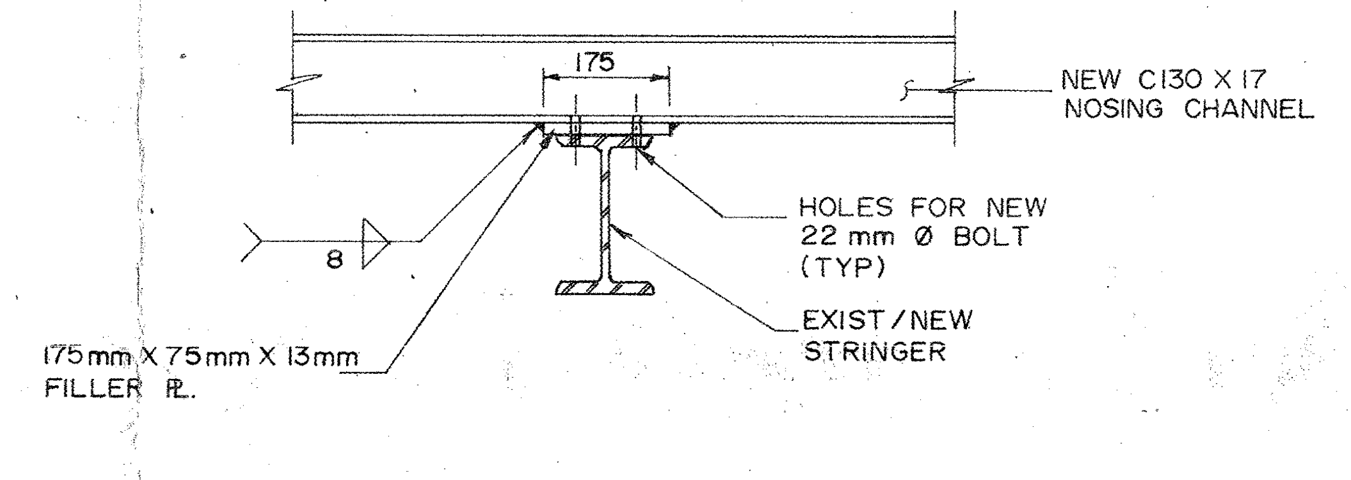
ABUT. EXPANSION JOINT

SCALE 1:10



PIER EXPANSION JOINT

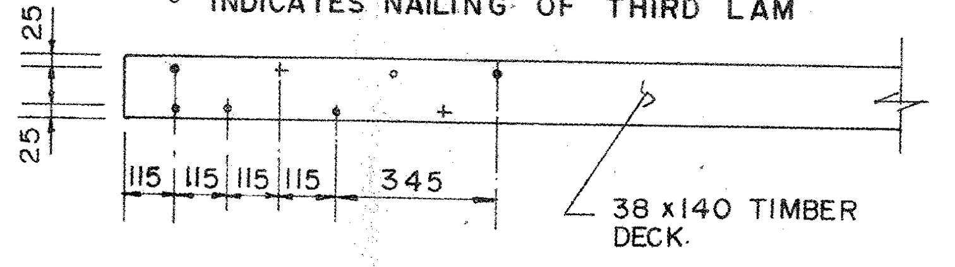
SCALE 1:10



⚠ SECTION X-X

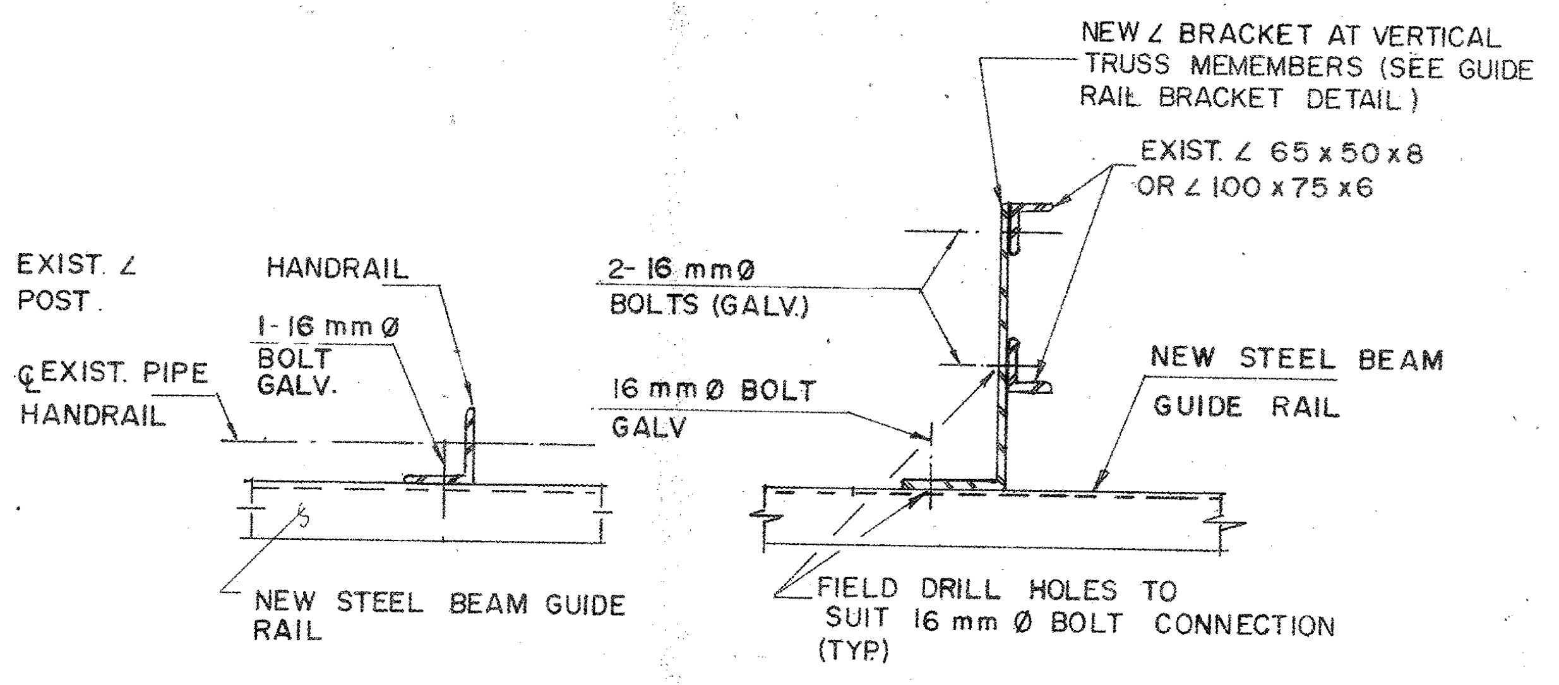
SCALE 1:10

- ● INDICATES NAILING OF FIRST LAM.
- + INDICATES NAILING OF SECOND LAM
- ○ INDICATES NAILING OF THIRD LAM



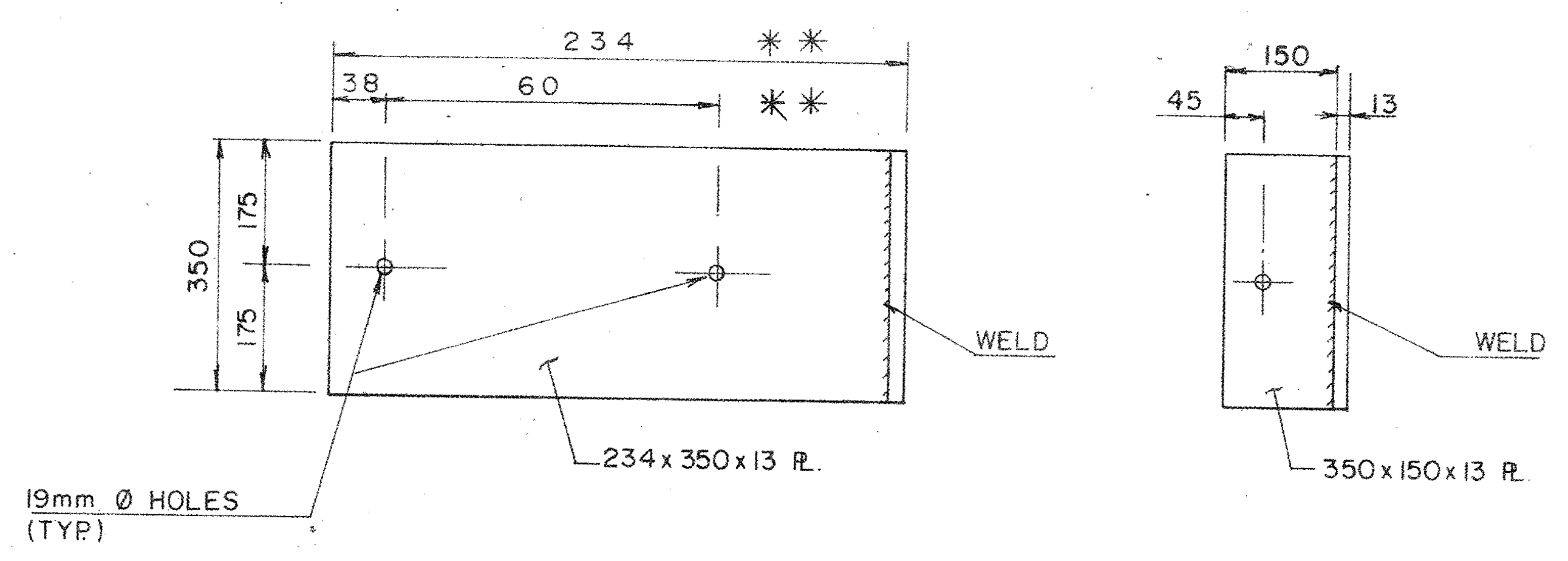
DECK NAILING PATTERN

N.T.S.



STEEL BEAM GUIDE RAIL ANCHORAGE TO EXIST. STRUCTURE

N.T.S.



GUIDERAIL BRACKET

N.T.S.

(36 Nos. REQ'D)

FOR APPROVAL
M.T.O SITE No. 38S-234

KRESIN
ENGINEERING & PLANNING

523 WELLINGTON ST. EAST
SAULT STE. MARIE, ONTARIO
942-2612

G. L. ALEONG

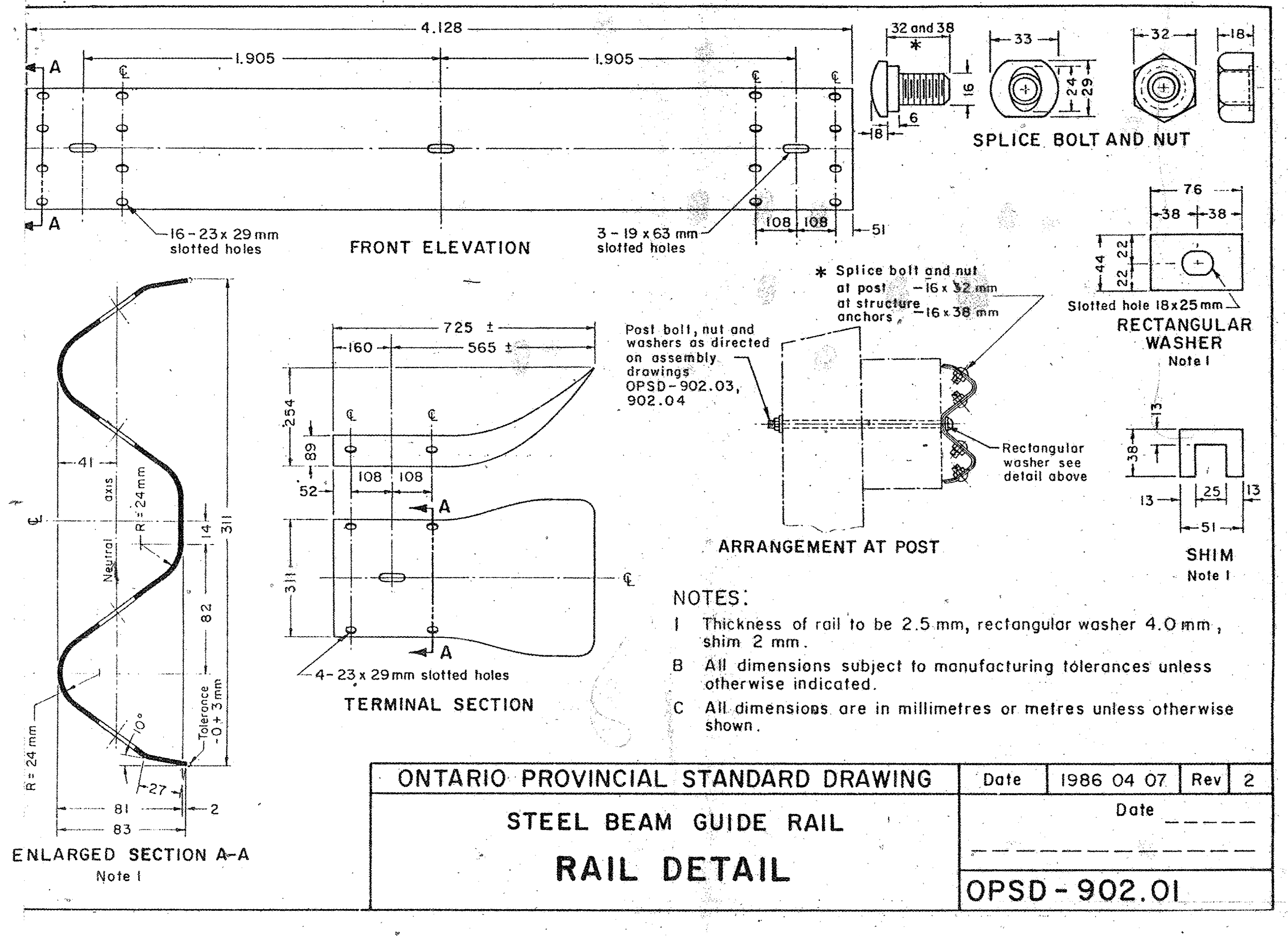
No.	DATE	BY	REVISIONS
⚠	OCT., 88	G.L.A	REVISIONS AS PER M.T.O. REQUEST

DESIGNED: G.O.L.
DRAWN: R.A.S.
CHECKED: D.L.B.
APPROVED: G.L.A.
SCALE: 1:20 EXCEPT AS NOTED

DEAN LAKE BRIDGE
TOWNSHIP OF THOMPSON
DISTRICT OF ALGOMA
LOT
MISSISSAUGI RIVER
CON. VI

MISCELLANEOUS DETAILS

DATE: AUG., 1988
PROJECT: 42-8617
DRAWING: 4



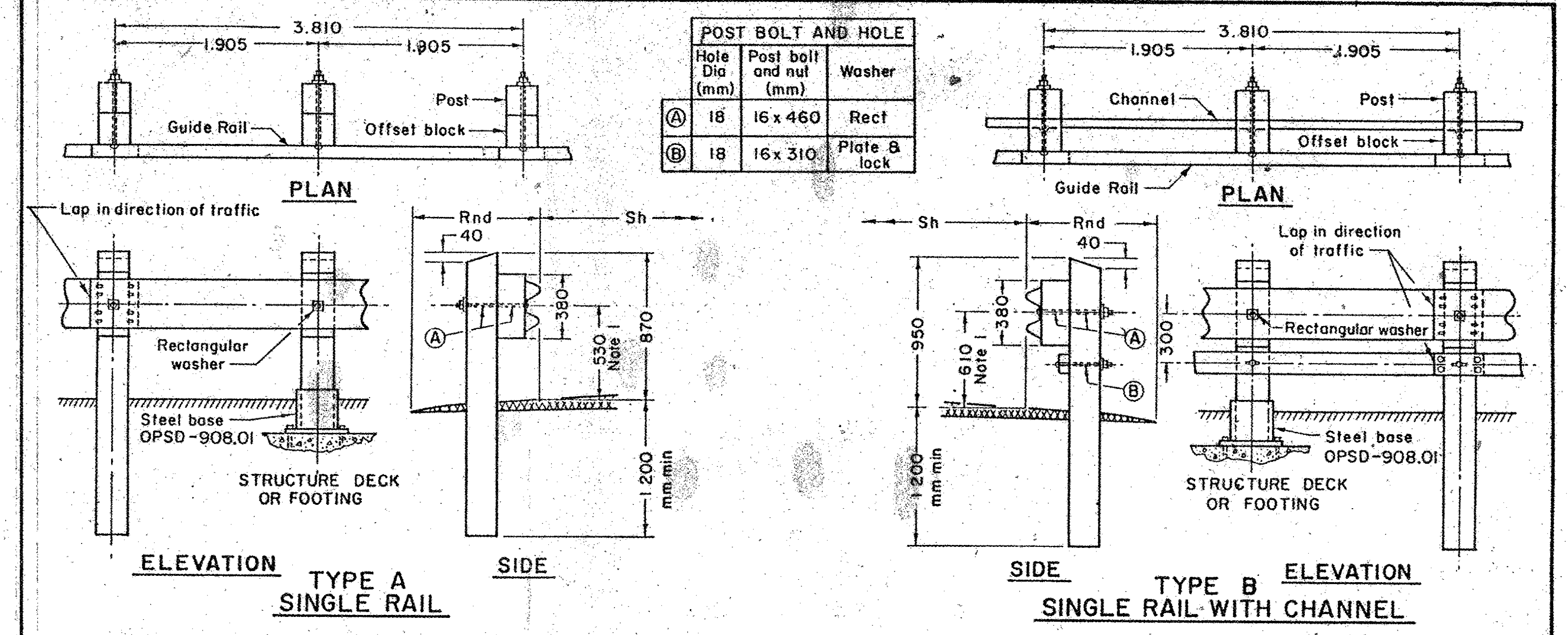
NOTES:

I Thickness of rail to be 2.5 mm, rectangular washer 4.0 mm, shim 2 mm.

B All dimensions subject to manufacturing tolerances unless otherwise indicated.

C All dimensions are in millimetres or metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Date	1986 04 07	Rev	2
STEEL BEAM GUIDE RAIL **RAIL DETAIL**	Date _____			
	OPSD - 902.01			



POST BOLT AND HOLE			
	Hole Dia (mm)	Post bolt and nut (mm)	Washer
Ⓐ	18	16 x 460	Rect
Ⓑ	18	16 x 310	Plate & lock

NOTES:

I Where guide rail is adjacent to curb, mounting height shall be measured:
- a Vertically at face of guide rail when face of guide rail is more than 300 mm beyond gutter line.
- b Vertically at gutter line when face of guide rail is 300 mm or less beyond gutter line.

A Posts and offset blocks:
Size 200 x 200 mm nominal, 190 x 190 mm ± 1.5 mm dressed, tops to have 25 mm chamfer.

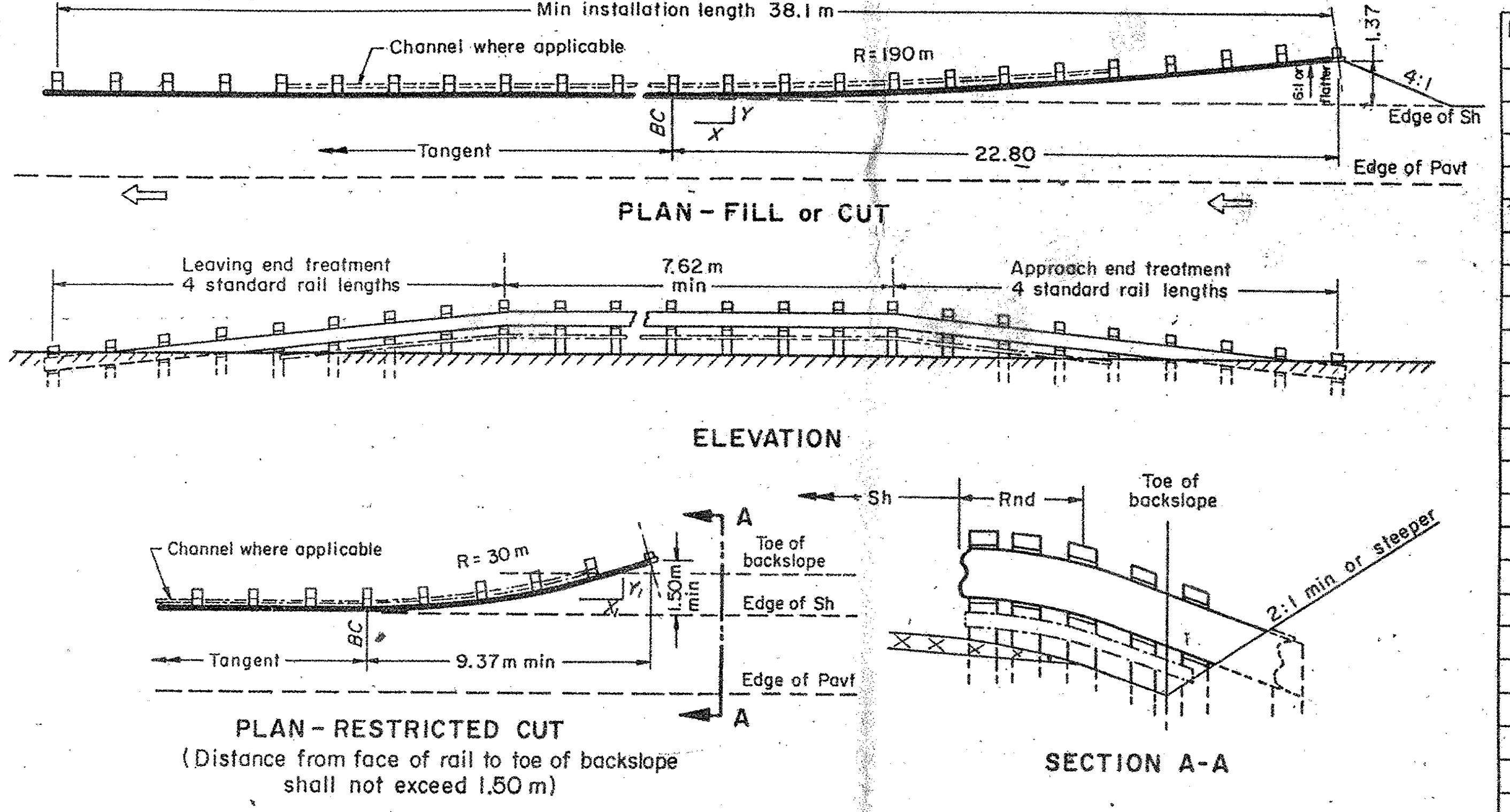
B Posts to be set by instrument for alignment and grade, with tops parallel to pavement grade.

C To produce an even alignment, shim beam element where necessary.

D This standard to be read in conjunction with OPSD-902.01 and 902.02

E All dimensions are in millimetres or metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Date	1986 04 07	Rev	2
STEEL BEAM GUIDE RAIL **ASSEMBLY DETAIL** SINGLE RAIL WITH OR WITHOUT CHANNEL	Date			
	OPSD - 902.03			



(Distance from face of rail to toe of backslope shall not exceed 1.50 m)

POST OFFSET TABLE	
Fill or Cut	
X	Y
1.91	0.01
3.81	0.04
5.71	0.09
7.62	0.15
9.52	0.24
11.42	0.34
13.32	0.47
15.22	0.61
17.12	0.77
19.02	0.95
20.91	1.15
22.80	1.37
Alternate cut	
X_1	Y_1
1.90	0.06
3.80	0.24
5.68	0.54
7.54	0.96
9.37	1.50
11.16	2.15
12.90	2.92
14.59	3.79
16.23	4.77

NOTES:

A All lateral dimensions measured from face of rail.

B Straight guide rail panels to be placed to fit arc when radius exceeds 45 m. Curved guide rail panels to be placed to fit arc when radius is 45 m or less.

C This standard to be read in conjunction with OPSD-902.03.

D Cross section must be widened to incorporate terminal end.

E All dimensions are in millimetres or metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Date	1988 04 15	Rev	1
STEEL BEAM GUIDE RAIL **APPROACH AND LEAVING END TREATMENTS**	Date _____			
	OPSD - 902.05			

MTC DOWNSVIEW
RECEIVED
NOV 01 1988
STRUCTURAL OFFICE
M. T. C.

STRUCTURE SITE No. 385-234

FOR APPROVAL

M.T.O SITE No. 38S-234

KRESIN
ENGINEERING & PLANNING

523 WELLINGTON ST. EAST
SAULT STE. MARIE, ONTARIO
942-2612

G. L. ALEONG

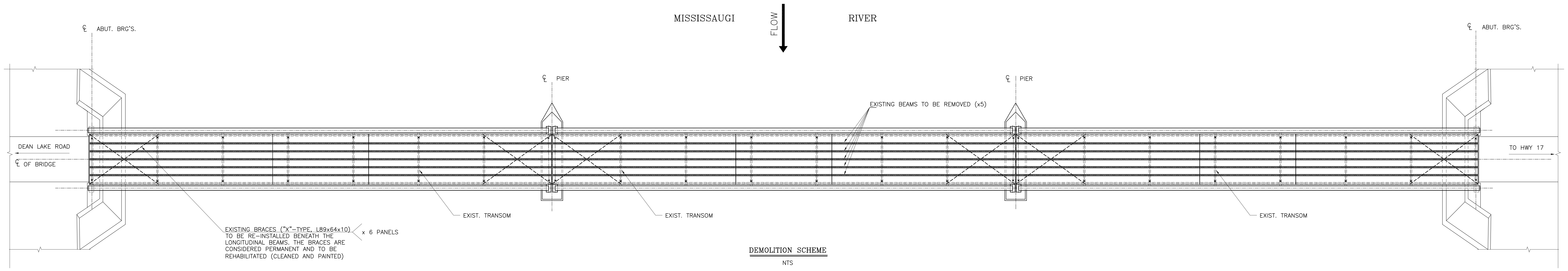
No.	DATE	BY	REVISIONS

DESIGNED	ADAPTED
DRAWN	W. P.
CHECKED	D. L. B.
APPROVED	G. L. A.
FIELD BOOK №	
SCALE	N.T.S.

DEAN LAKE BRIDGE
TOWNSHIP OF THOMPSON
DISTRICT OF ALGOMA
LOT MISSISSAUGI RIVER CON. VI

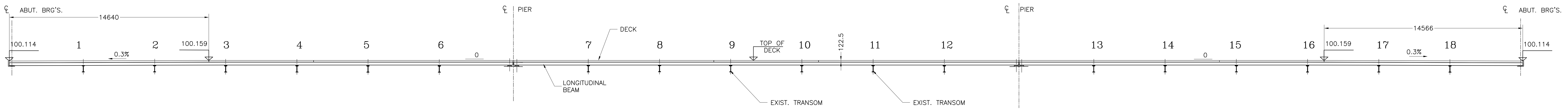
STANDARDS

DATE AUG. 1988
PROJECT 42-8617
DRAWING
5



DEMOLITION SCHEME

NTS



LONGITUDINAL "NEWTON GFRP COMPOSITE DECK" PROFILE

NTS

NOTES:

ALL DIMENSIONS ARE GIVEN IN MILLIMETERS UNLESS OTHERWISE SHOWN.

GENERAL:

- CANADIAN HIGHWAY BRIDGE DESIGN CODE CAN/CSA S6–06, EXCEPT BARRIERS AND RAILING SYSTEM

STANDARDS:

- MTO STANDARD SPECIFICATIONS FOR BRIDGE CONSTRUCTION AND MAINTENANCE

SURVEY:

- THE DESIGN COMPLY WITH "PLAN OF SURVEY OF DEAN LAKE BRIDGE", OCTOBER 10, 2007 BY "TULLOCH"

RESPONSIBILITY FOR DESIGN:

- THE PROJECT UNDERTAKEN CONCERNS TO THE NEW BRIDGE DECK DESIGN ONLY. I&F ENGINEERING CORP. HAS NO ANY RESPONSIBILITY FOR THE EXISTING BRIDGE SUPERSTRUCTURE AND FOOTINGS.

DESIGN CRITERIA (Letter from Mr.R.G.H. Wood P.Eng MRW Consulting Comp., Sept. 17, 2007):

- LEVEL 3 – 16 tn
- LEVEL 2 – 24 tn
- LEVEL 1 – 28 tn
- 1 LANE
- DESIGN SPEED: 50 km/h
- AADT: 200

MATERIALS:

- STRUCTURAL STEEL: CSA G40.21M, GRADE 350WT, CATEGORY 2.
- WELDING SHALL CONFORM TO CSA W59–03.
- ALL BUTT WELDS IN FLANGE AND WEB SHOP SPLICES SHALL BE FINISHED FLUSH OR SMOOTH AS INDICATED, BY GRINDING WHERE NECESSARY IN THE DIRECTION OF APPLIED STRESSES. IF RELOCATION OR ADDITIONAL SHOP SPLICES ARE REQUIRED, THEIR LOCATION SHALL BE APPROVED BY THE ENGINEER.
- UNLESS OTHERWISE NOTED THE MINIMUM FILLET WELD SHALL BE AS FOLLOWS:

MATERIAL THICKNESS OF THICKER PART JOINED (mm)	MINIMUM SIZE OF SINGLE PASS FILLET WELD (mm)
TO 12 INCLUSIVE	5
OVER 12 TO 12	6
OVER 20 TO 40	8
OVER 40 TO 60	10
OVER 60 TO 120	12

- BOLTS: ASTM A325M, TYPE 1, 22 mm DIA. (U/N).
- THREADED RODS: ASTM A449 Fu=120 ksi.
- ANCHOR BOLTS: ASTM A449. HOT DIP GALVANIZED.
- DECK: "NEWTON GFRP COMPOSITE DECK".

COATING SYSTEM FOR SUPERSTRUCTURE:

GFRP CASING:
- TOP 8mm THICK
- BOTTOM AND SOFFITS –6.5mm THICK
- WEARING SURFACE – PEBBLE STONES LAYER EMBEDDED TO GFRP CASING

- GFRP ENCAPSULATION IN ACCORDANCE WITH THE MANUFACTURER PROPRIETARY SYSTEM DEVELOPMENT.

- INORGANIC EPOXY URETHANE ALUMINIUM FLAKE COATING "SILVER A" MANUFACTURED BY GUARDIAN COATING & CHEMICAL OR SIMILAR APPROVED.

- ALL STRUCTURAL STEEL SURFACES EXCLUDING THE CONTACT SURFACES OF BOLTED JOINTS SHALL BE PAINTED.

- INORGANIC EPOXY URETHANE ALUMINIUM FLAKE COATING "SILVER A" MANUFACTURED BY GUARDIAN COATING & CHEMICAL, OR SIMILAR APPROVED

- ALL IN–DECK EMBEDDED ELEMENTS TO BE "WHITE SANDBLASTED"

CONSTRUCTION NOTES:

THE CONTRACTOR SHALL ESTABLISH THE ACTUAL BEARING SEAT ELEVATIONS. IF THE ACTUAL BEARING THICKNESS ARE DIFFERENT FROM THOSE GIVEN WITH THE DESIGN DATA THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL AND ELEVATION OF THE CONCRETE PEDESTALS TO SUIT.
THE EXISTING HORIZONTAL BRACING (X–TYPE, L89x64x10) UNDERNEATH THE EXISTING DECK TO BE RE–INSTALLED LOWER BENEATH THE NEW LONGITUDINAL BEAMS, PROVIDING LATERAL STABILITY FOR INSTALLATION PERIOD. TO BE REMOVED AFTER THE DECK INSTALLATION IS FINISHED, EXCEPT SIX PANELS NOTED ON DEMOLITION SCHEME, WHICH ARE PERMANENT.

SCOPE OF WORK:

1. TEMPORARY RELOCATE AND/OR PROTECT EXISTING SERVICES
2. REMOVE EXISTING BRIDGE DECK
3. CLEAN UP AND PAINT EXISTING UNDERLYING STRUCTURE
4. ASSEMBLE AND INSTALL NEW DECK USING BOLTED CONNECTIONS AS PER DRAWINGS OR SUBSTITUTE WELDING ON SITE BY QUALIFIED CWB PERSONNEL.
5. CARRY OUT REHABILITATION WORKS AND ROAD ADJUSTMENT (BY OTHERS)

LEGEND:

REF. – DIMENSIONS TO BE CHECKED ON SITE BEFORE WORK COMMENCEMENT.

LIST OF DRAWINGS:

S00	GENERAL NOTES. DEMOLITION SCHEME. LONGITUDINAL PROFILE
S01	GENERAL ARRANGEMENT
S02	DECK PLAN. DETAILS
S03	BEARING. DETAILS
S04	DECK PANEL D1a
S05	DECK PANEL D2a
S06	DECK PANEL D3a
S07	DECK PANEL D1b
S08	DECK PANEL D2b
S09	DECK PANEL D3b
S10	DECK PANEL D1c
S11	DECK PANEL D2c
S12	DECK PANEL D3c
S13	DECK DETAILS

METRIC PRINTING COLOR SETTINGS: 1=0.150, 2=0.250, 3=0.350, 4=0.500, 5=0.350, 6=0.600, 7=0.350, 8=0.150, 9=0.100, 10–>255=0.130
IMPERIAL PRINTING COLOR SETTINGS: 1=0.005, 2=0.010, 3=0.014, 4=0.020, 5=0.014, 6=0.024, 7=0.014, 8=0.005, 9=0.010, 10–>255=0.005

NO.	DATE:	REVISION	CH'D
1	10.10.2007	ISSUED FOR REVIEW	V.R.
2	10.24.2007	ISSUED FOR FINAL COORDINATION	V.R.
3	10.31.2007	ISSUED FOR CONSTRUCTION	V.R.

● ISSUED FOR CONSTRUCTION
○ ISSUED FOR PRICING
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○ ISSUED FOR GENERAL REVIEW
○ ISSUED FOR PROGRESS REVIEW
○ ISSUED FOR FINAL COORDINATION

The undersigned has reviewed and takes responsibility for this design, and has the qualifications and meets the requirements set out in the Ontario Building Code to be a designer.

QUALIFICATION INFORMATION

Required unless design is exempt under Division C–3.2.5.1 of the 2006 Ontario Building Code (or 2.17.5.1 of the 1997 Ontario Building Code)

NAME	SIGNATURE	BCIN/BCDN
Lyakhovsky M.		29491

REGISTRATION INFORMATION

Required unless design is exempt under Division C–3.2.4.1 of the 2006 Ontario Building Code (or 2.17.4.1 of the 1997 Ontario Building Code)

FIRM NAME	BCIN/BCDN
I & F Engineering Corp.	29493

Orientation

N

Stamp

I. FINKELSHTEYN
10.31.2007

Newton Bridge Solutions Ltd.

41 MASSEY ROAD
GUELPH, ONTARIO
N1H 7M6
TEL. 519–341–8944
FAX. 519–822–6159

I & F ENGINEERING CORP
CONSULTING ENGINEERS & MANAGERS
100 DRUMLIN CIRCLE, UNIT 203
CONCORD, ON, CANADA
(905) 669–8445
FAX: (905) 669–4803

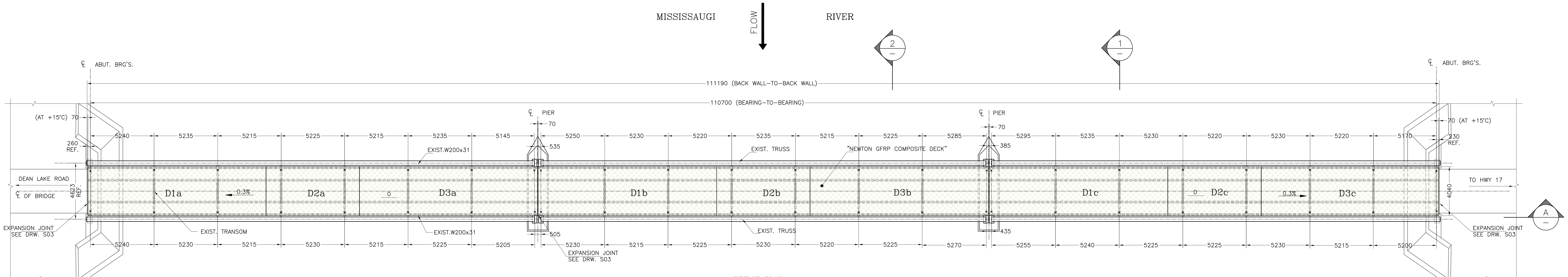
Client:
MUNICIPALITY OF HURON SHORES

Project:
DEAN LAKE BRIDGE
MTO SITE 385–234

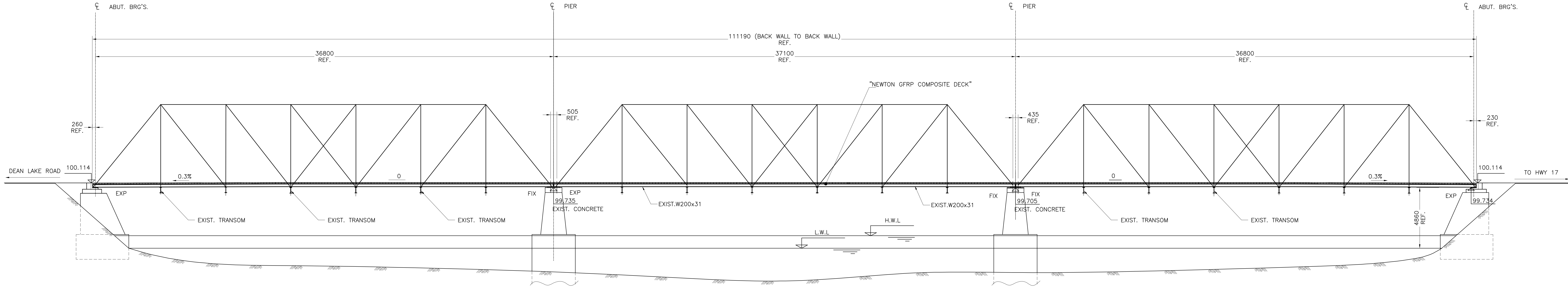
Drawing Title:
GENERAL NOTES
DEMOLITION SCHEME
DECK LONGITUDINAL PROFILE

Drawn by: V.R.	Date: 10.10.2007
Checked by: M.L.	Scale: NTS
Project Nr. 07M05	Drawing Nr. S00

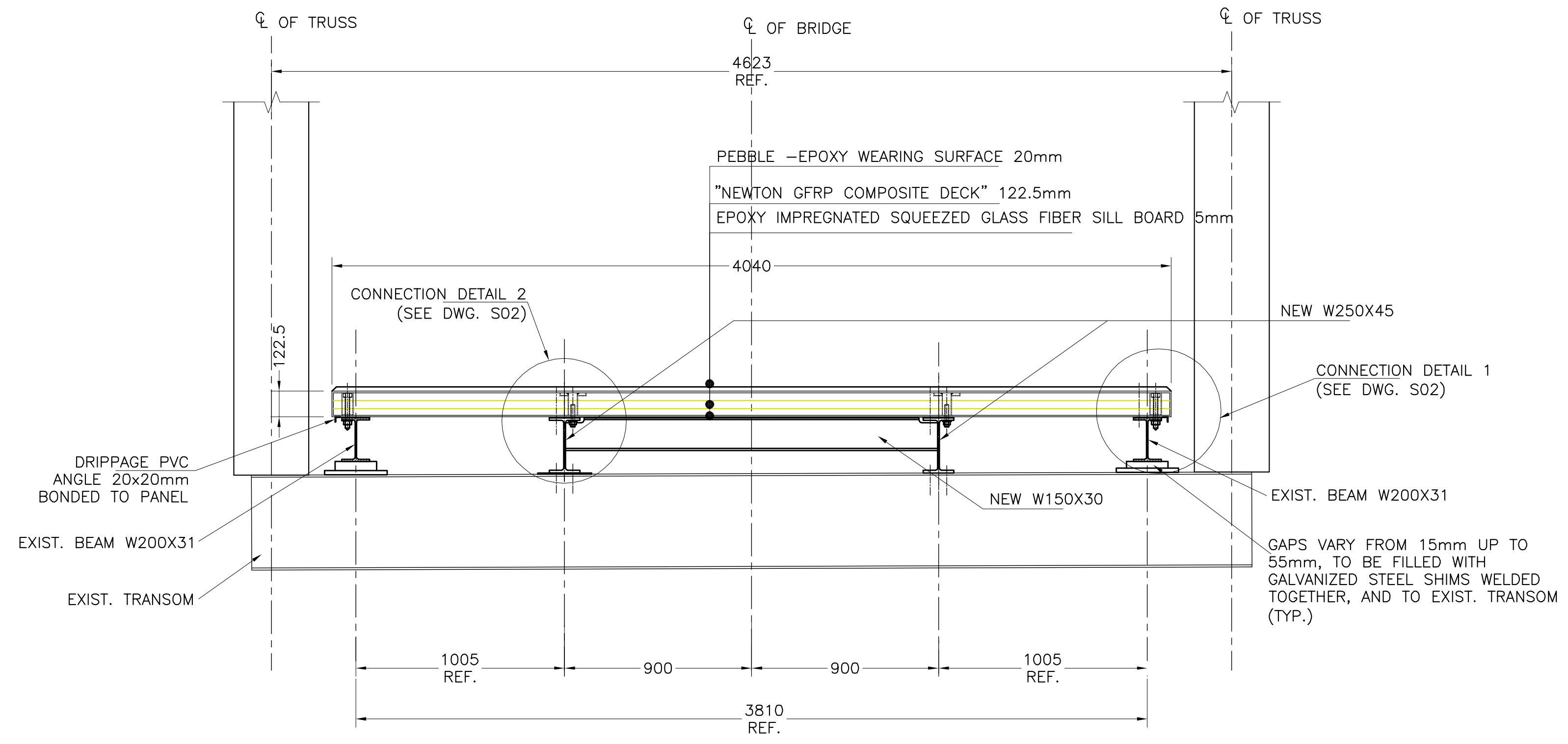
CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY
SIZE 1178x841



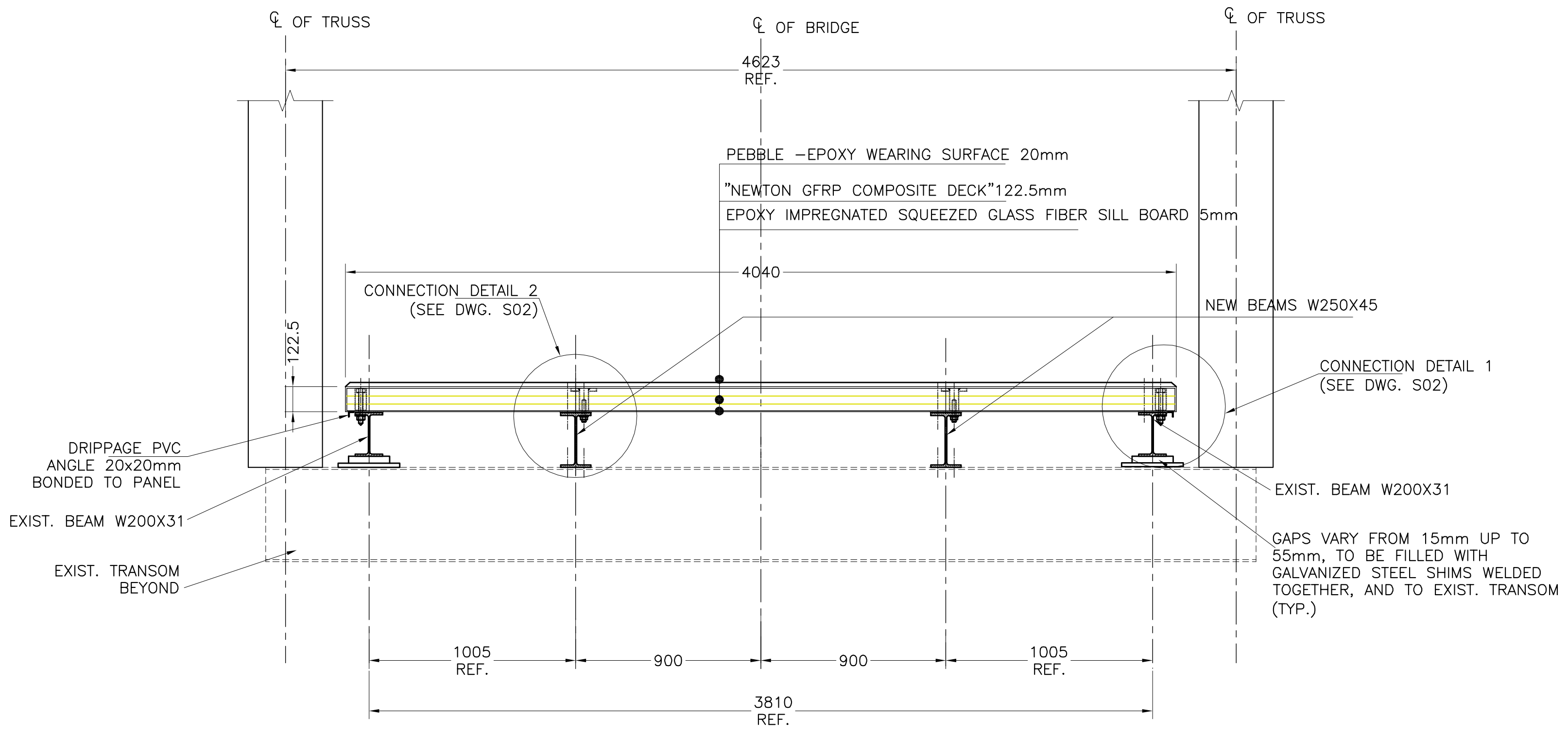
BRIDGE PLAN
SCALE 1:125



ELEVATION (SECTION A)
SCALE 1:125



SECTION 1
NTS



SECTION 2
NTS

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QUALIFICATION INFORMATION

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NAME	SIGNATURE	BCIN/BCDN
Lyakhovsky M.		29491

REGISTRATION INFORMATION

Required unless design is exempt under Division C-3.2.4.1 of the 2006 Ontario Building Code (or 2.17.4.1 of the 1997 Ontario Building Code)

FIRM NAME	BCIN/BCDN
I & F Engineering Corp.	29493

Orientation

Stamp
LICENSED PROFESSIONAL ENGINEER
I. FINKELSHTEYN
10.31.2007
PROVINCE OF ONTARIO

Newton Bridge Solutions Ltd.

41 MASSEY ROAD
GUELPH, ONTARIO
N1H 7M6

TEL. 519-341-8944
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CONSULTING ENGINEERS & MANAGERS

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CONCORD, ON, CANADA

(905) 669-8445
FAX: (905) 669-4803

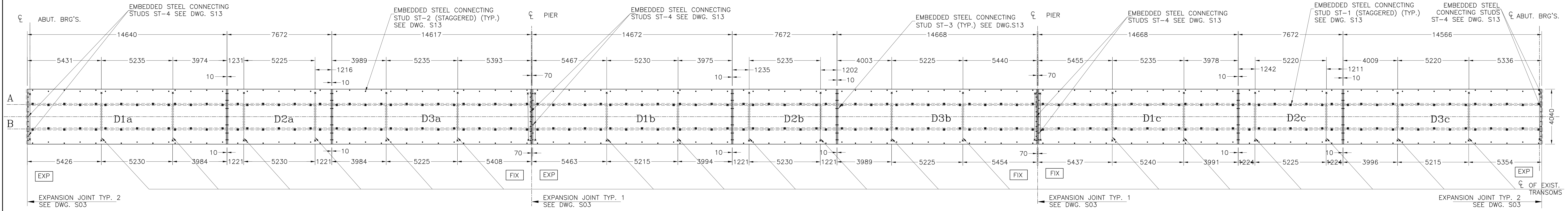
Client:
MUNICIPALITY OF HURON SHORES

Project:
DEAN LAKE BRIDGE
MTO SITE 385-234

Drawing Title:
GENERAL ARRANGEMENT

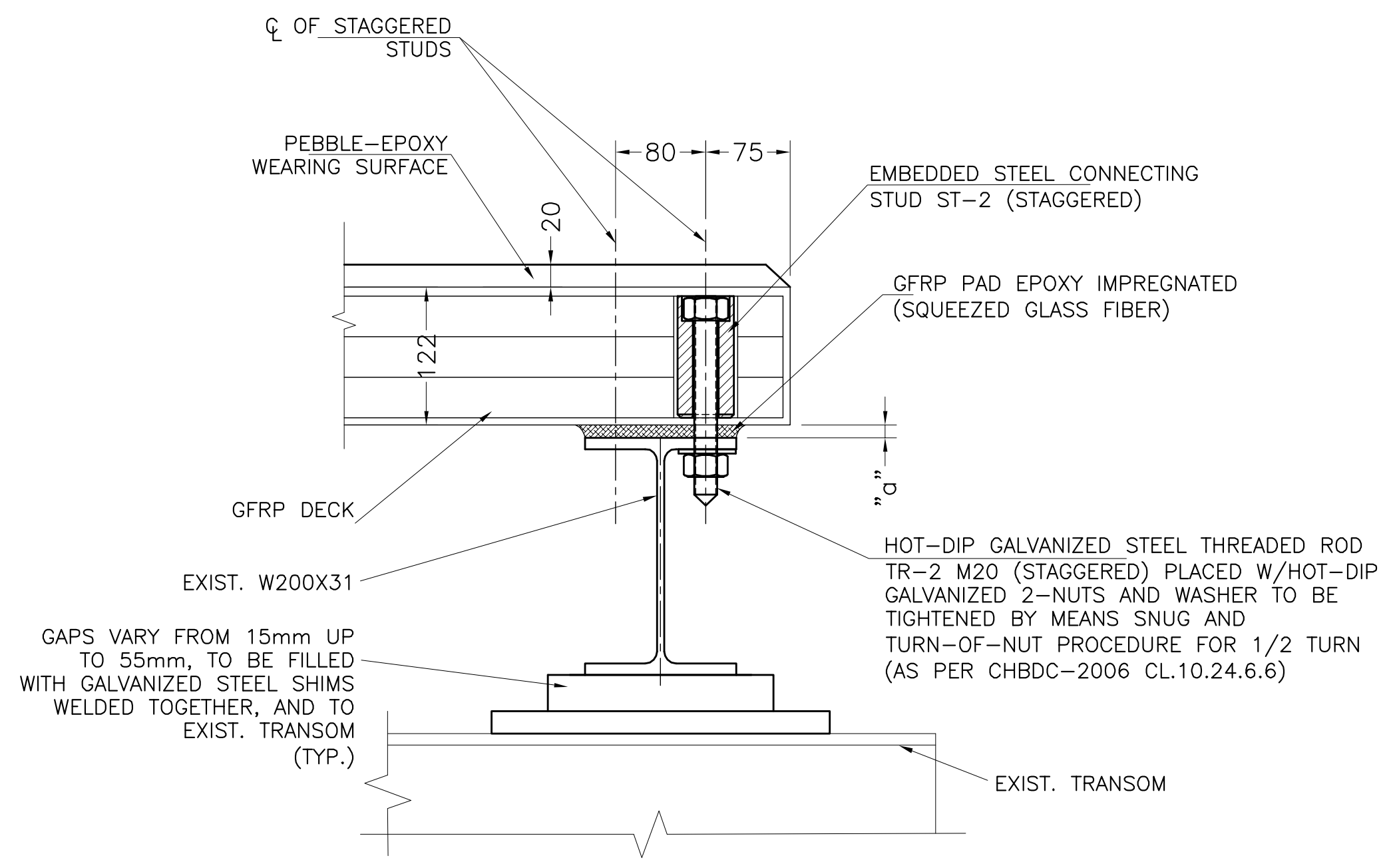
Drawn by: V.R.	Date: 10.10.2007
Checked by: M.L.	Scale: 1:125
Project Nr. 07M05	Drawing Nr. S01

CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY
SIZE 1178x841



DECK PLAN

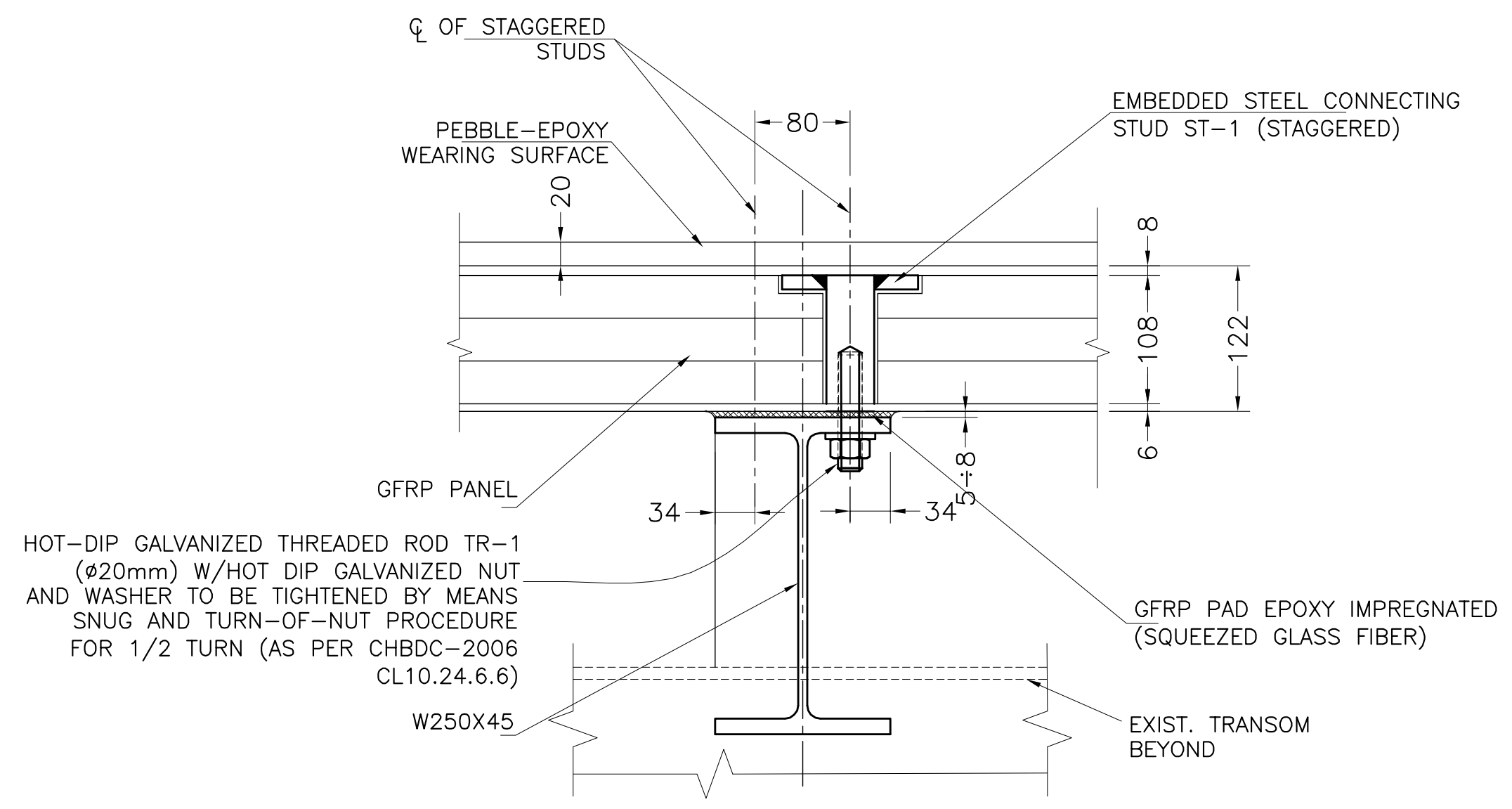
SCALE 1:125



DETAIL 1

DETAIL OF CONNECTION PANEL TO EXIST. BEAM W200X31

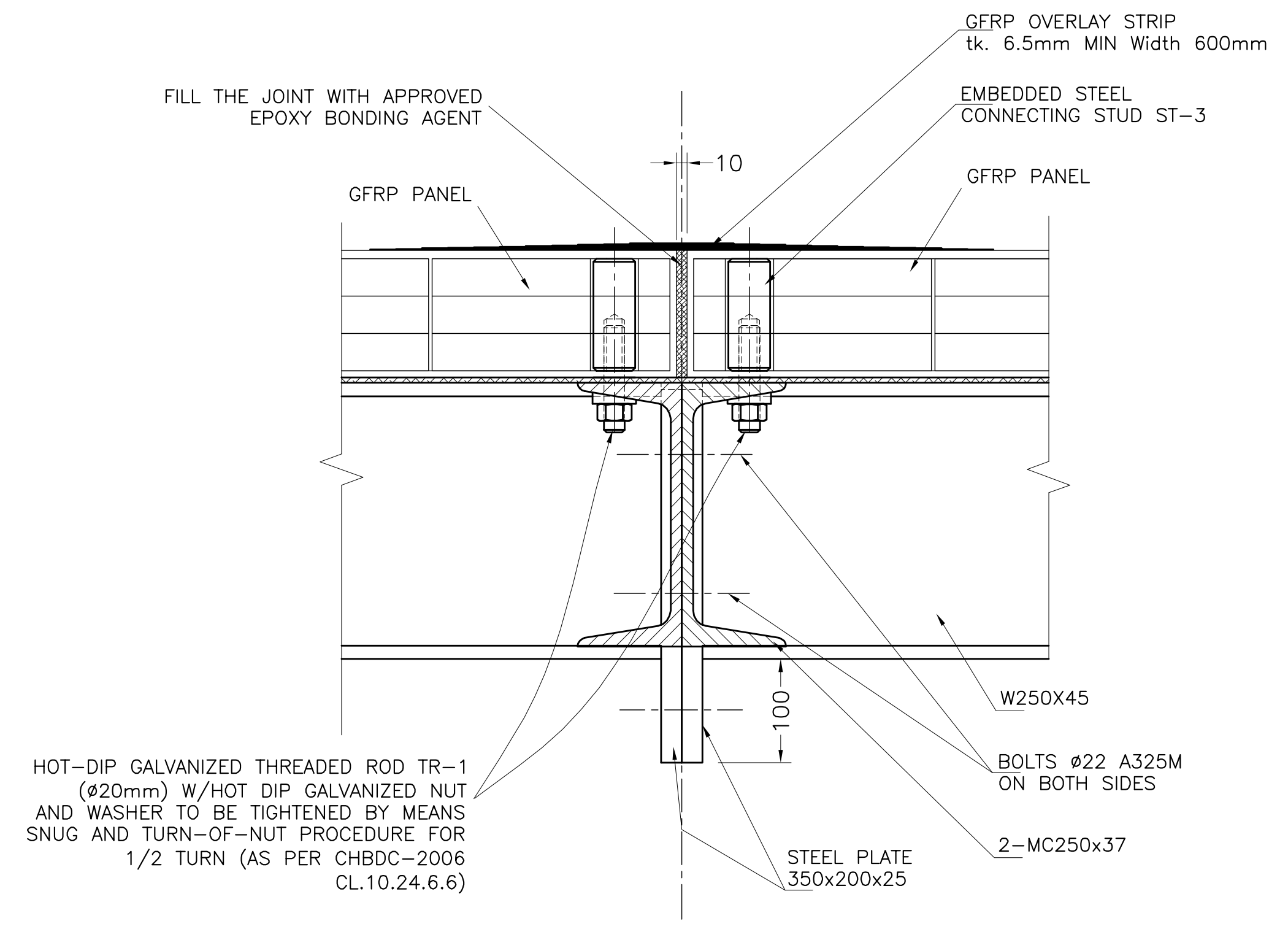
NTS



DETAIL 2

DETAIL OF CONNECTION PANEL TO LONGITUDINAL BEAM W250X45

NTS



DETAIL 3

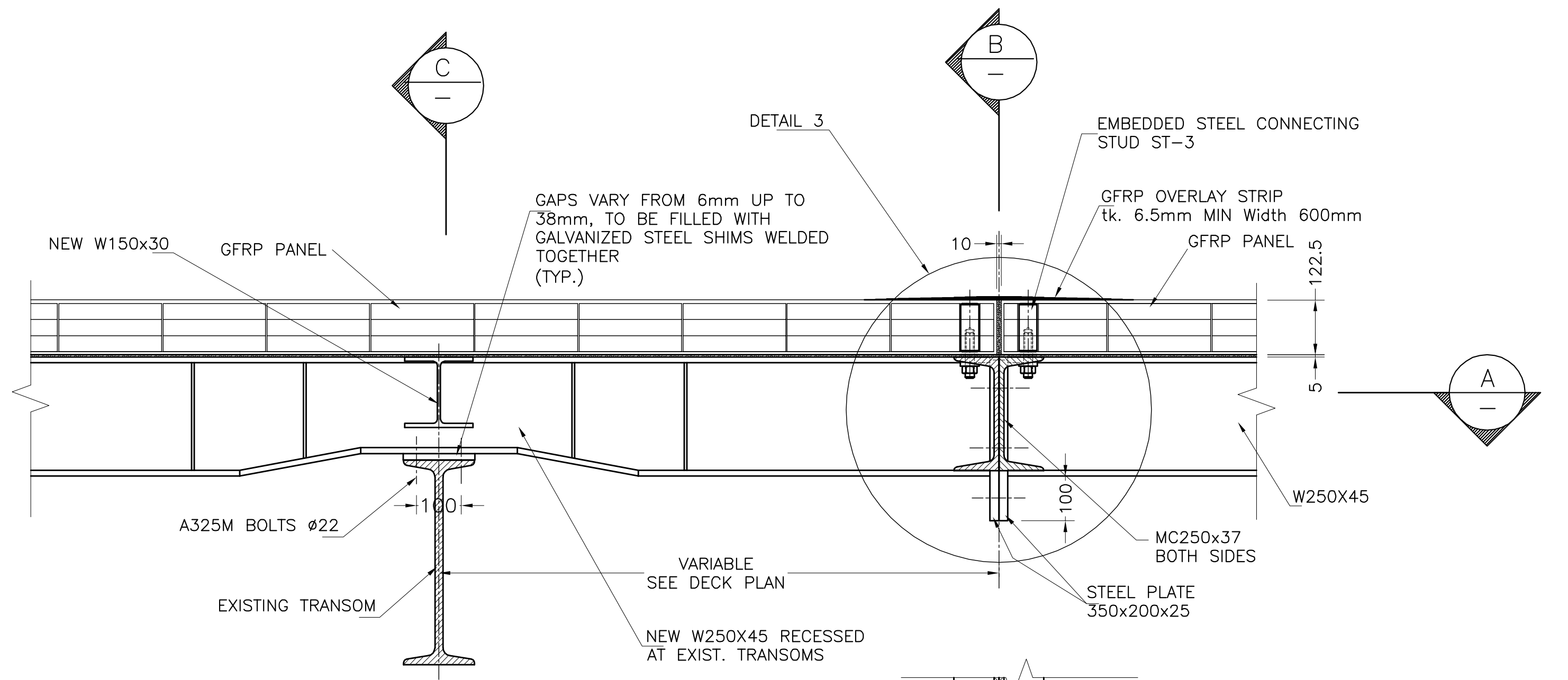
DETAIL OF CONNECTION PANEL TO PANEL

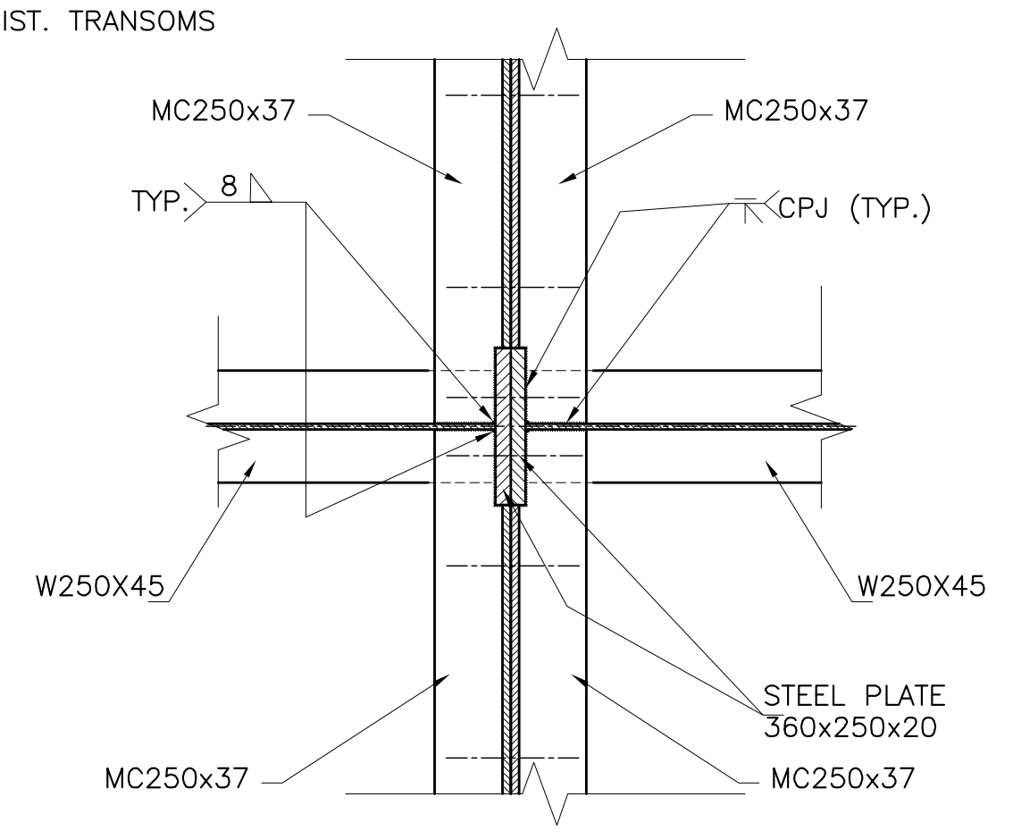
NTS

NOTES:

ALL IN-DECK EMBEDDED ELEMENTS TO BE "WHITE SANDBLASTED"

"a" – VARIABLE



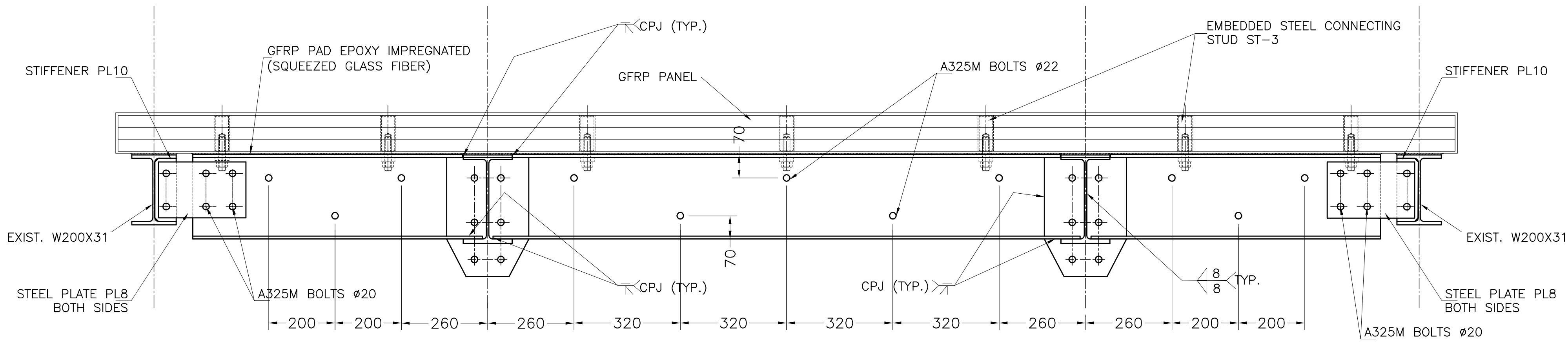


SECTION A

NTS

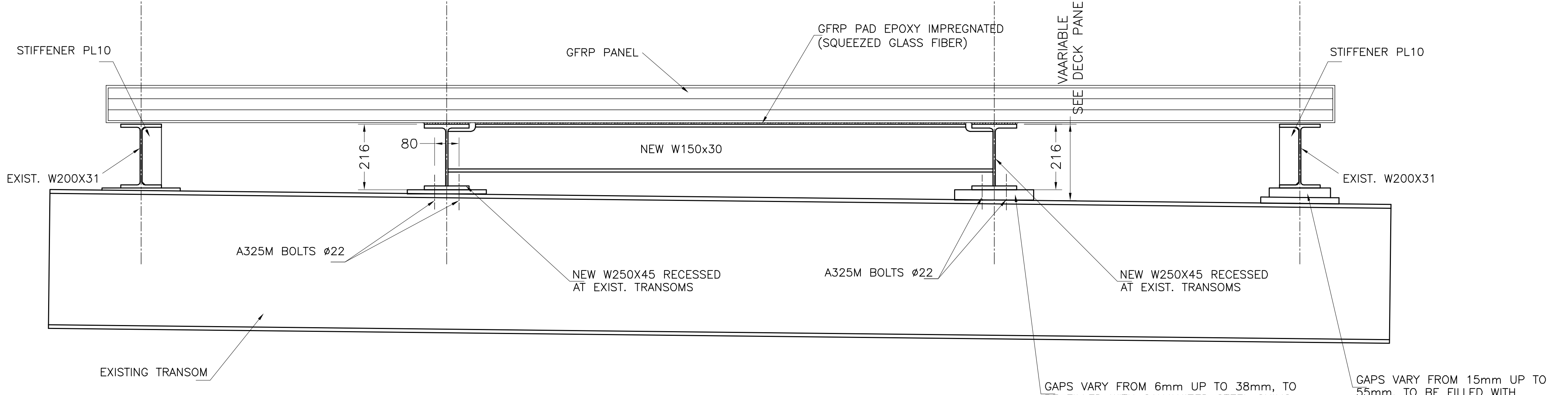
DETAIL CONNECTION "DECK TO DECK"

NTS



SECTION B

NTS



SECTION C

NTS

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QUALIFICATION INFORMATION

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NAME	SIGNATURE	BCIN/BCDN
Lyakhovsky M.		29491

REGISTRATION INFORMATION

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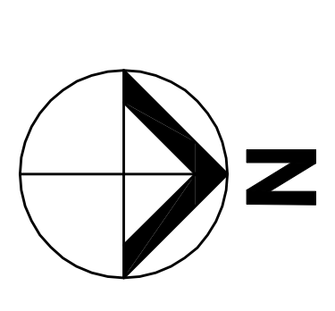
FIRM NAME	BCIN/BCDN
I & F Engineering Corp.	29493

Orientation



Stamp

LICENSED PROFESSIONAL ENGINEER
I. FINKELSHTEYN
10.31.2007
PROVINCE OF ONTARIO

Newton Bridge Solutions Ltd.

41 MASSEY ROAD
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Client:

MUNICIPALITY OF HURON SHORES

Project:

DEAN LAKE BRIDGE
MTO SITE 385-234

Drawing Title:

DECK PLAN & DETAILS

Drawn by: V.R.	Date: 10.10.2007
Checked by: M.L.	Scale: 1:125
Project Nr. 07M05	Drawing Nr. S02

CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY
SIZE 1178x841

METRIC PRINTING COLOR SETTINGS: 1=0.150, 2=0.250, 3=0.350, 4=0.500, 5=0.350, 6=0.600, 7=0.350, 8=0.150, 9=0.100, 10->255=0.130
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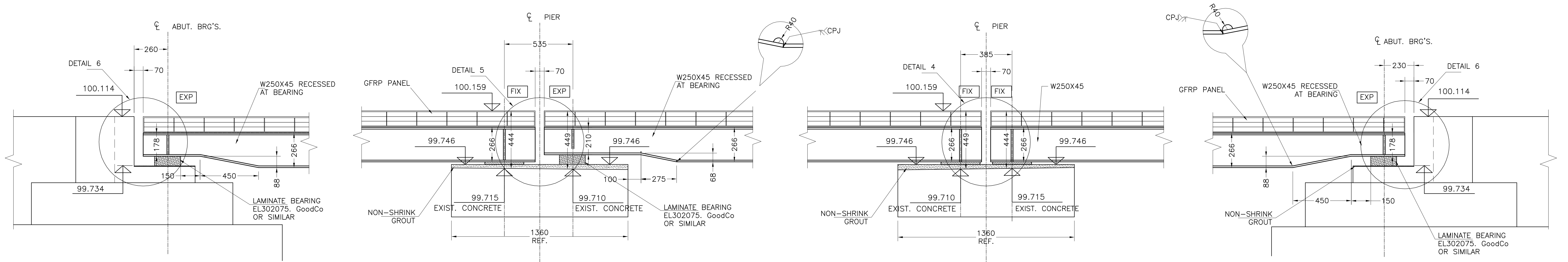
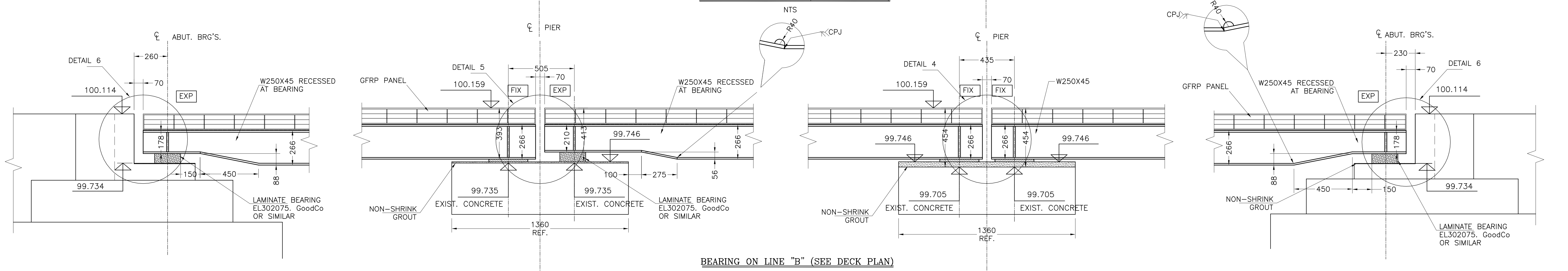


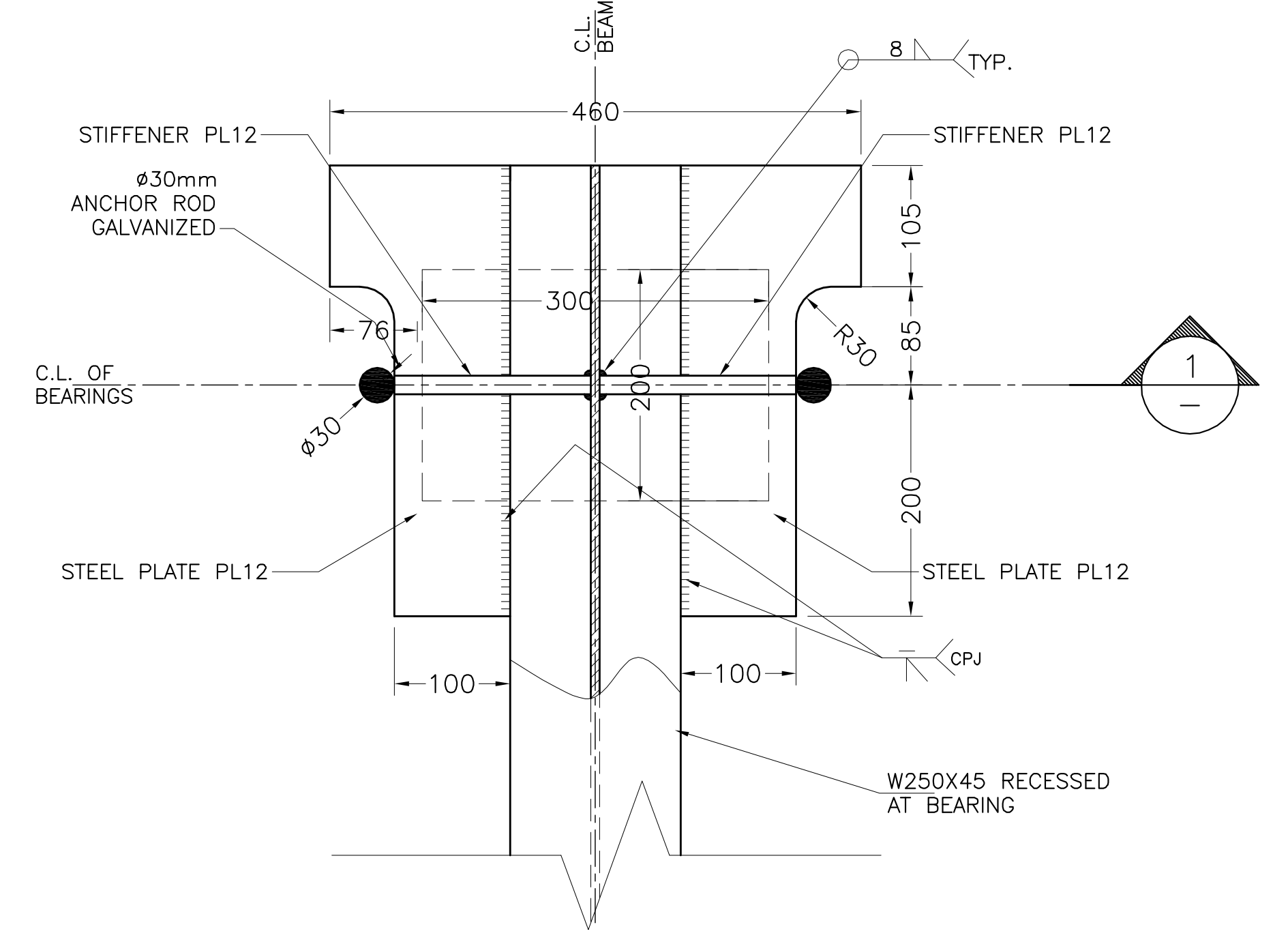
BEARING ON LINE "A" (SEE DECK PLAN)

NTS

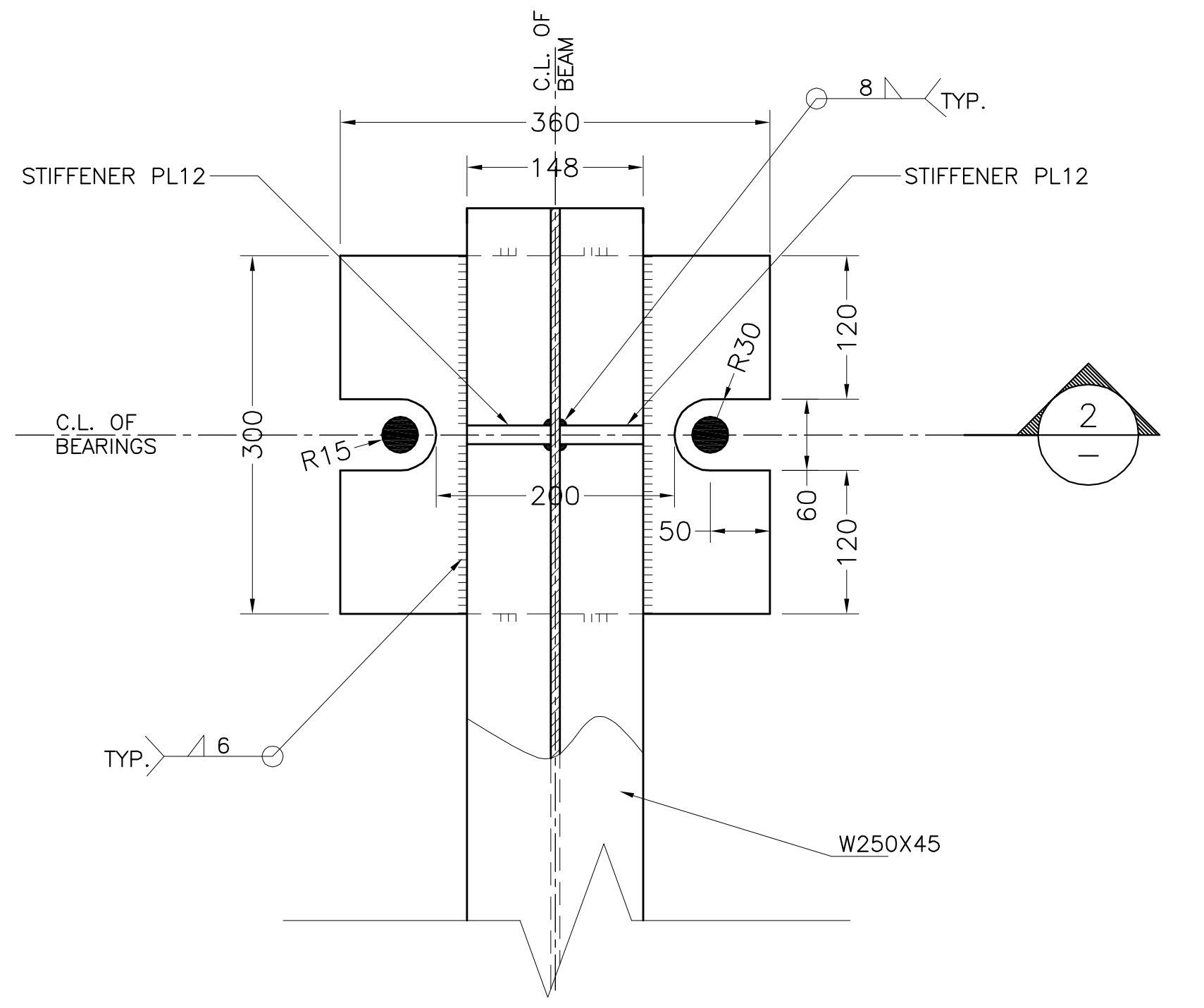


BEARING ON LINE "B" (SEE DECK PLAN)

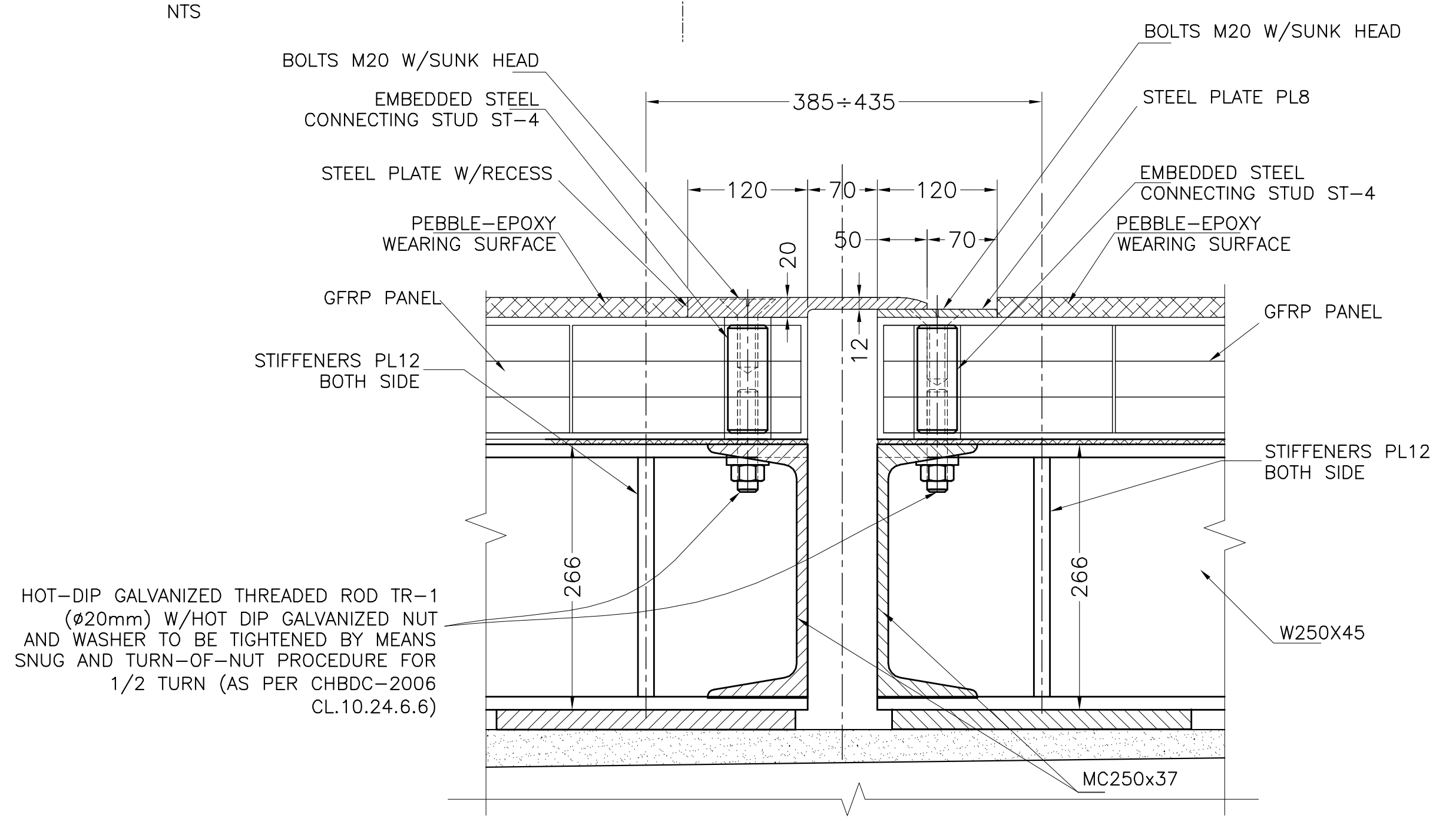
NTS



PLAN



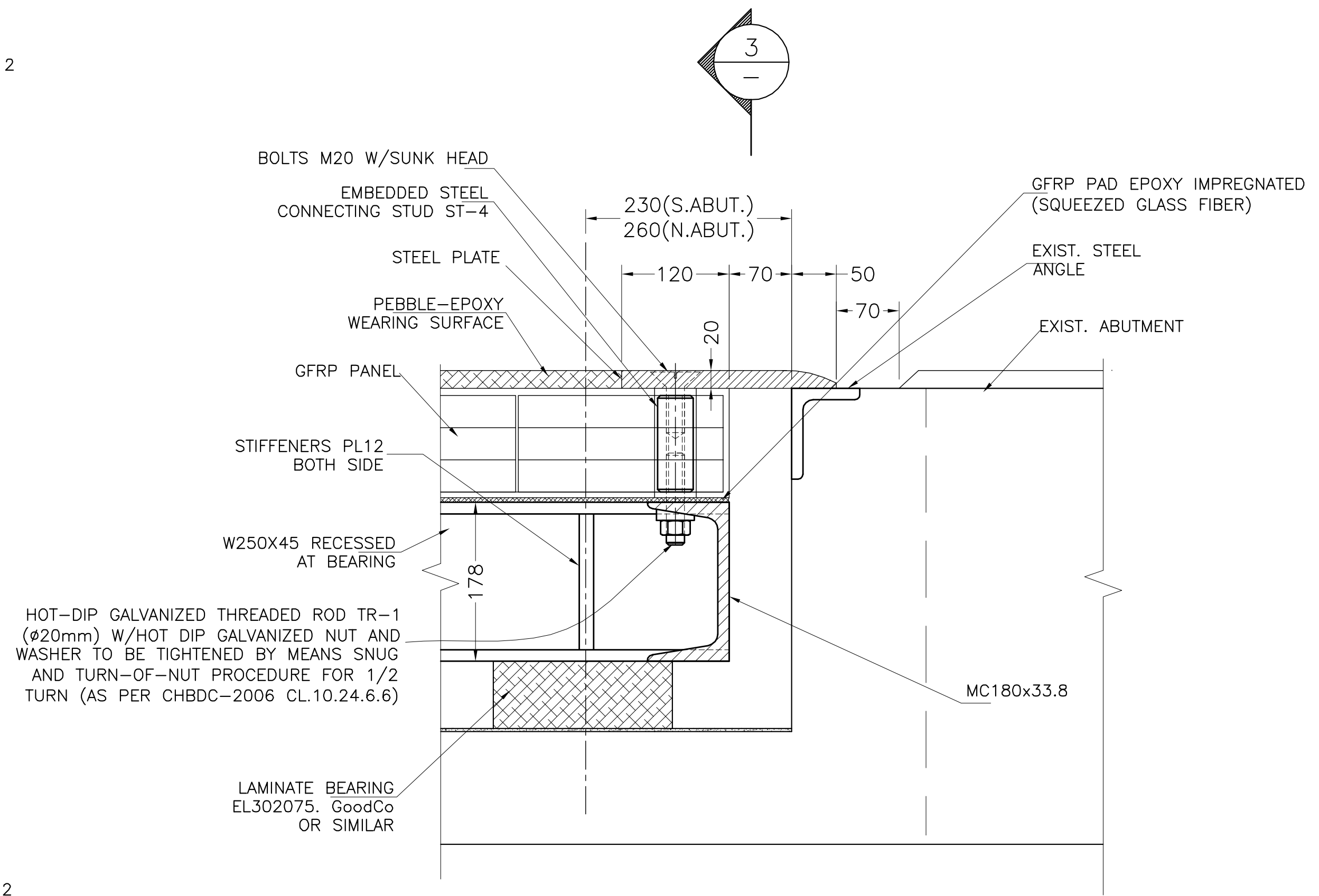
PLAN



DETAIL 4

EXPANSION JOINT TYP. 1 (SEE DECK PLAN)

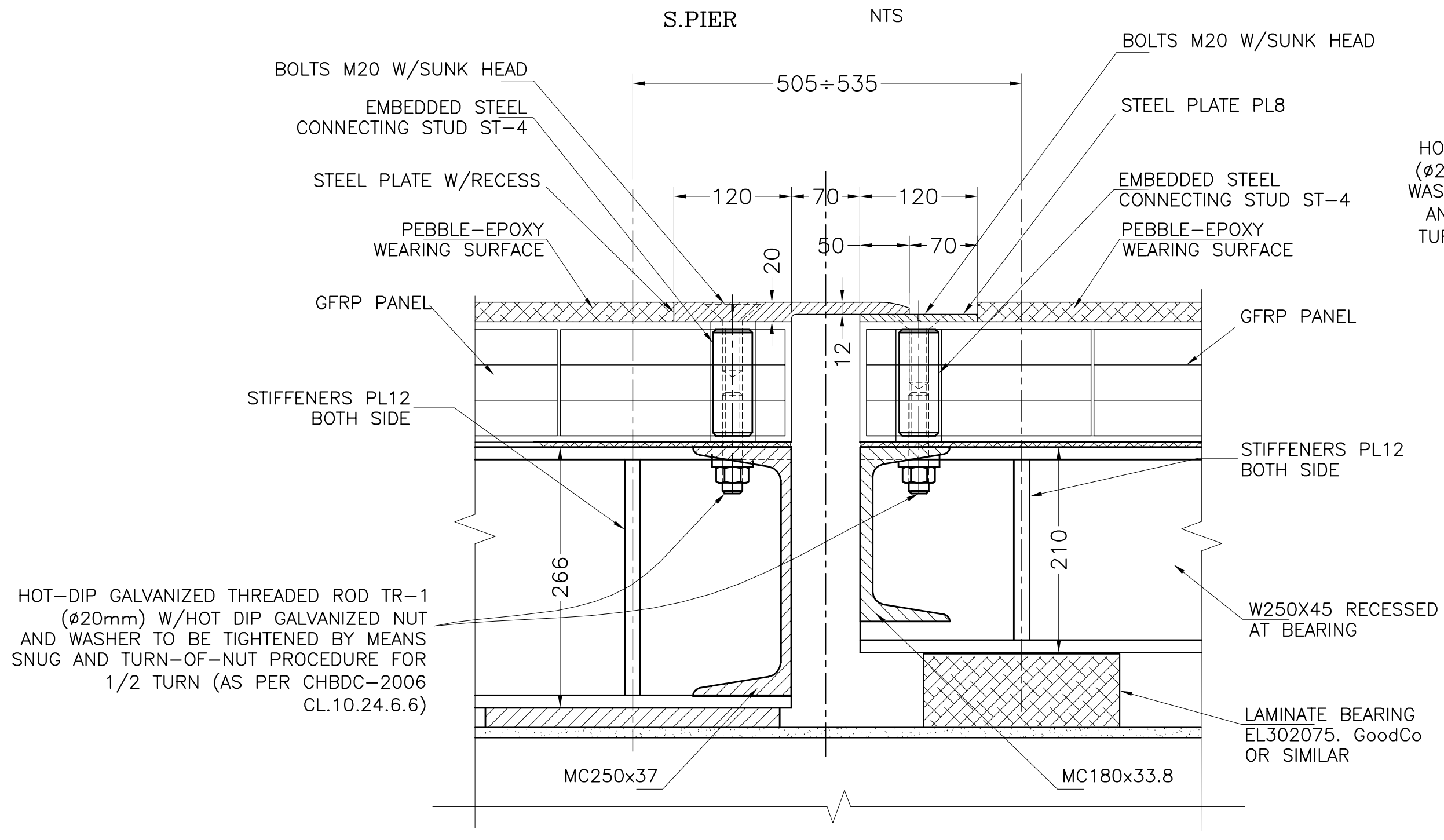
S.PIER NTS



DETAIL 6

EXPANSION JOINT TYP. 2 (SEE DECK PLAN)

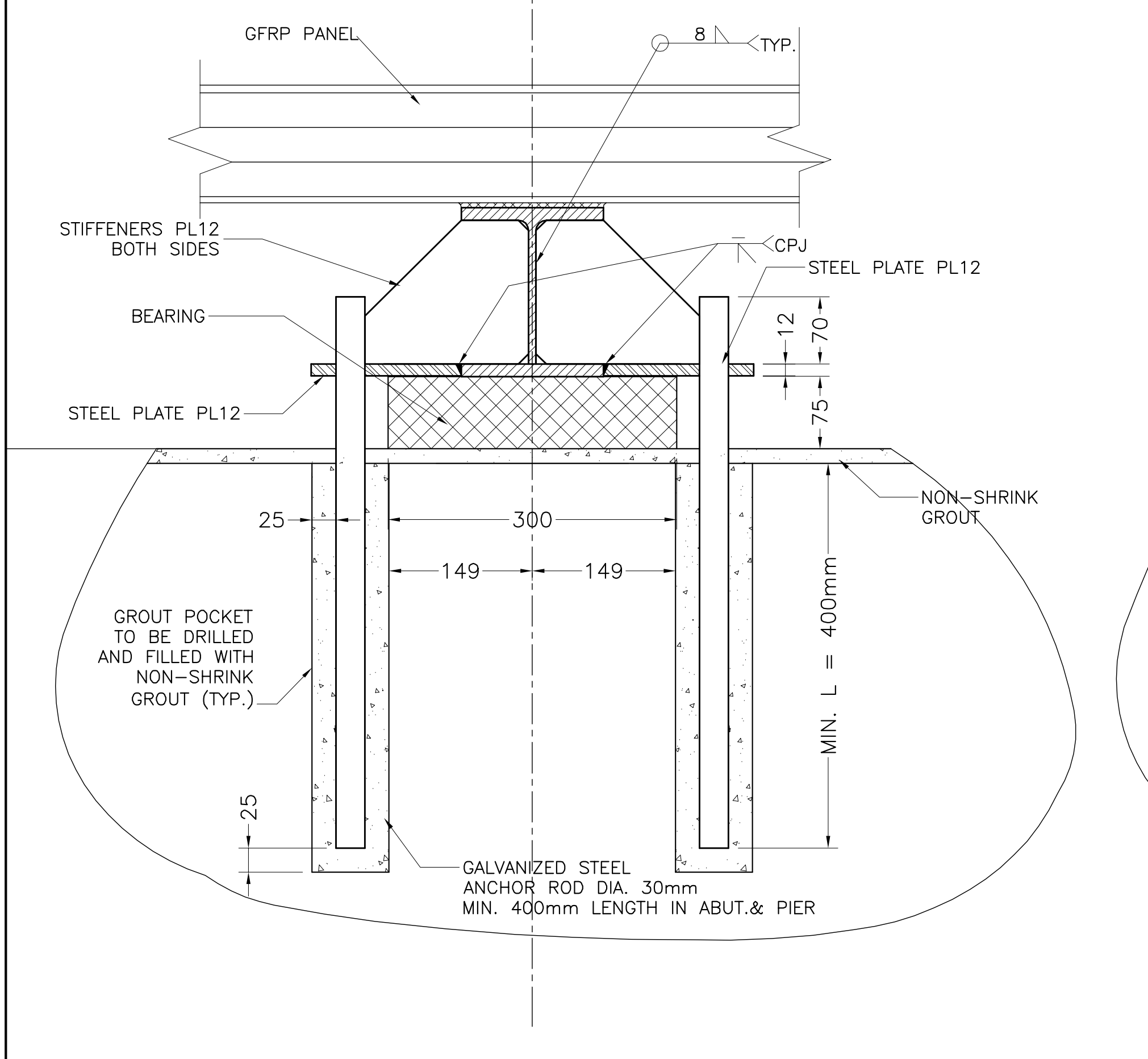
N & S. ABUTM. NTS



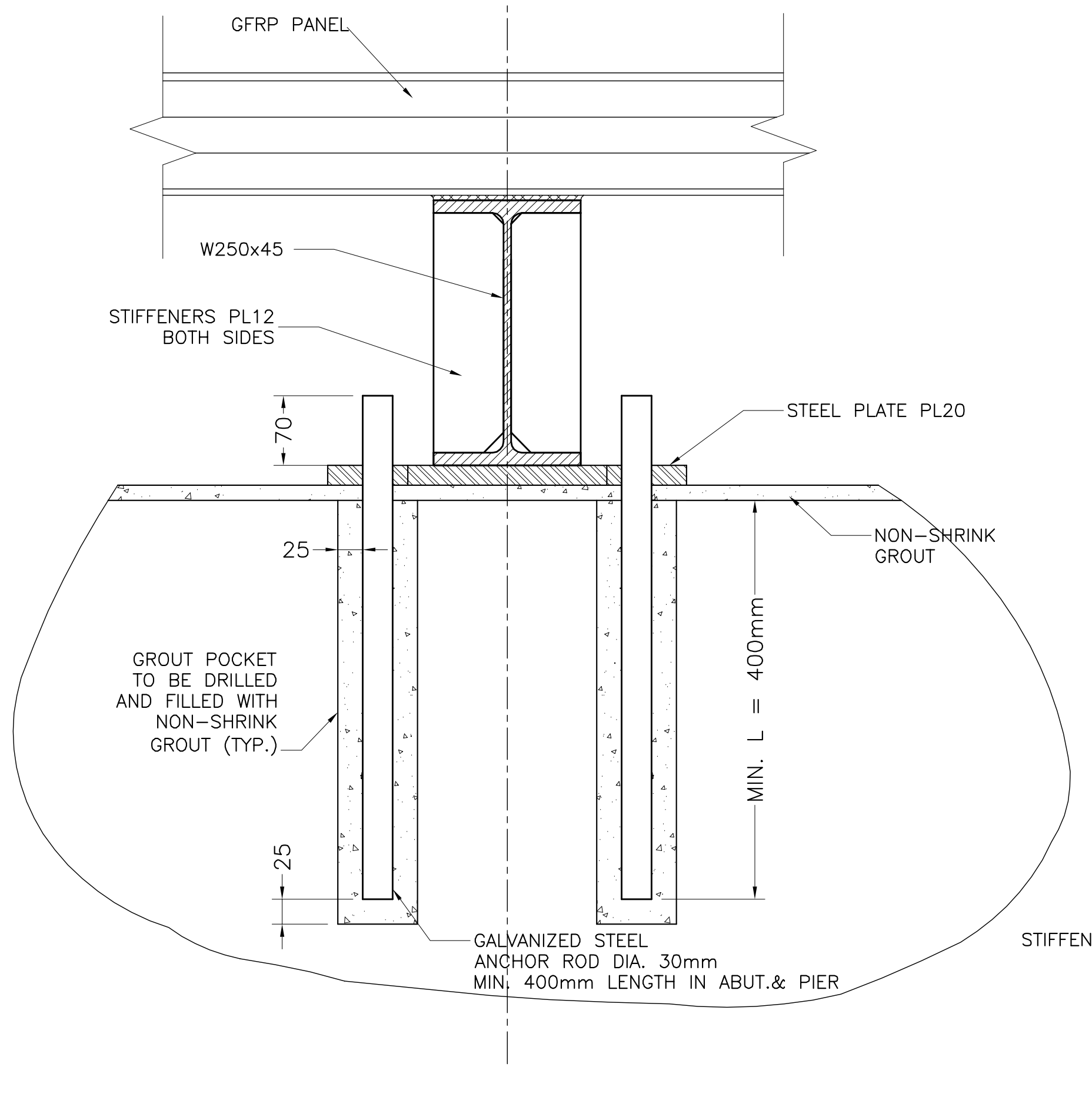
DETAIL 5

EXPANSION JOINT TYP. 1 (SEE DECK PLAN)

N.PIER NTS



SECTION 1



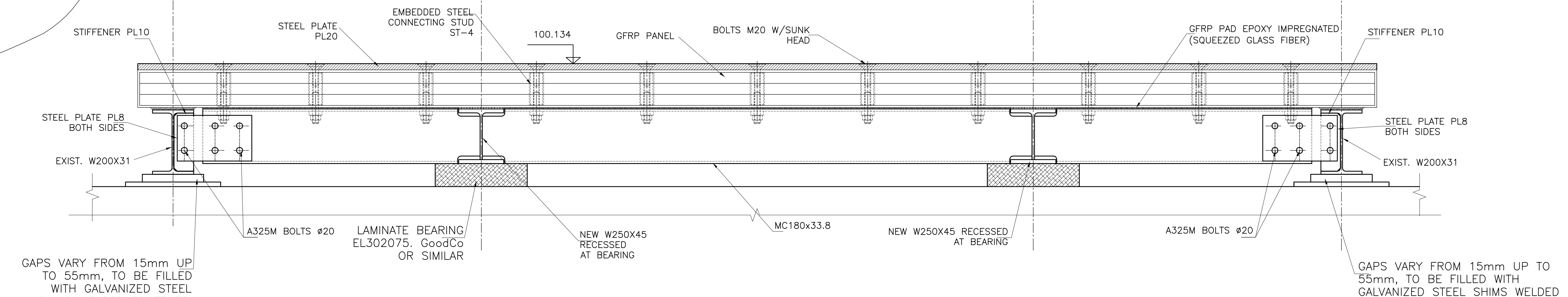
SECTION 2

EXPANSION BEARING DETAILS

NTS

FIXED BEARING DETAILS

NTS



SECTION 3 (TYP.)

NTS

METRIC PRINTING COLOR SETTINGS: 1=0.150, 2=0.250, 3=0.350, 4=0.500, 5=0.350, 6=0.600, 7=0.350, 8=0.150, 9=0.100, 10->255=0.130
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3	10.31.2007	ISSUED FOR CONSTRUCTION	V.R.

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Lyakhovsky M.		29491
NAME	SIGNATURE	BCIN/BCDN

REGISTRATION INFORMATION

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I & F Engineering Corp.	29493
FIRM NAME	BCIN/BCDN

Orientation

N

Stamp

LICENSED PROFESSIONAL ENGINEER
I. FINKELSHTEYN
10.31.2007
PROVINCE OF ONTARIO

Newton Bridge Solutions Ltd.

41 MASSEY ROAD
GUELPH, ONTARIO
N1H 7M6

TEL. 519-341-8944
FAX. 519-822-6159

I & F ENGINEERING CORP
CONSULTING ENGINEERS & MANAGERS

100 DRUMLIN CIRCLE, UNIT 203
CONCORD, ON, CANADA

(905) 669-8445
FAX: (905) 669-4803

Client:

MUNICIPALITY OF HURON SHORES

Project:

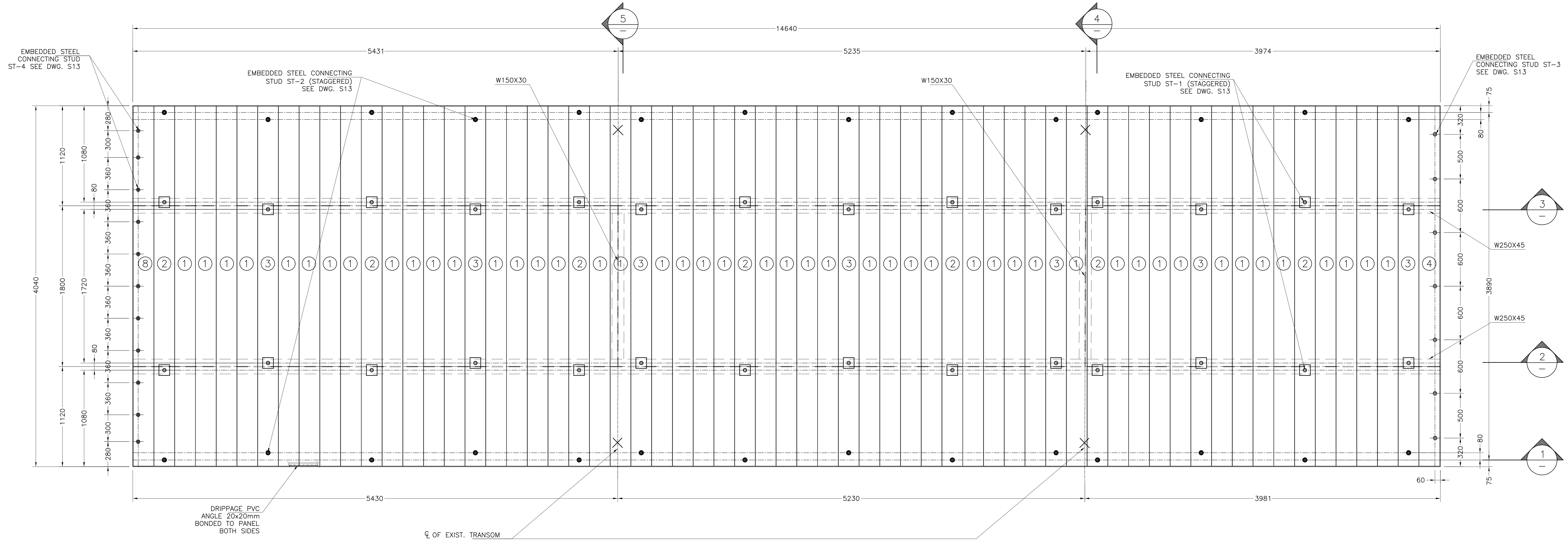
DEAN LAKE BRIDGE
MTO SITE 385-234

Drawing Title:

BEARINGS. DETAILS

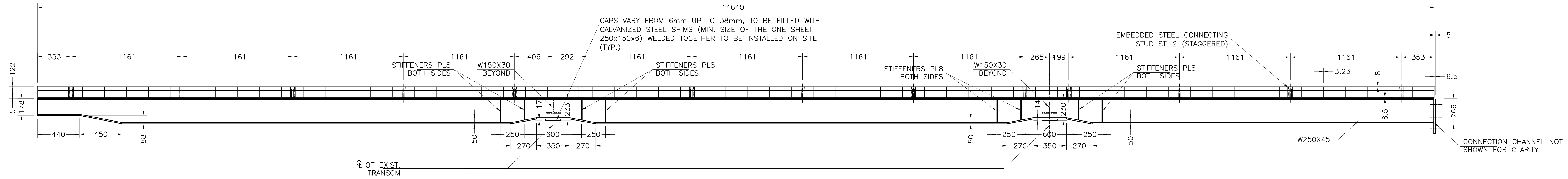
Drawn by: V.R.	Date: 10.10.2007
Checked by: M.L.	Scale: NTS
Project Nr. 07M05	Drawing Nr. S03

CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY
SIZE 1178x841



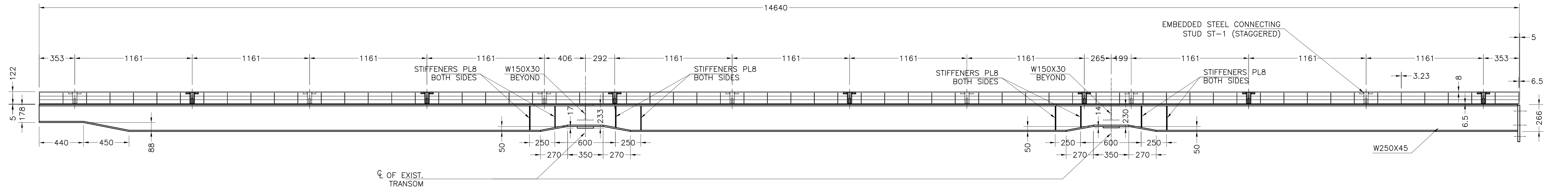
GFRP DECK PANEL-D1a

PLAN SCALE:1:20



SECTION 1

SCALE 1:20



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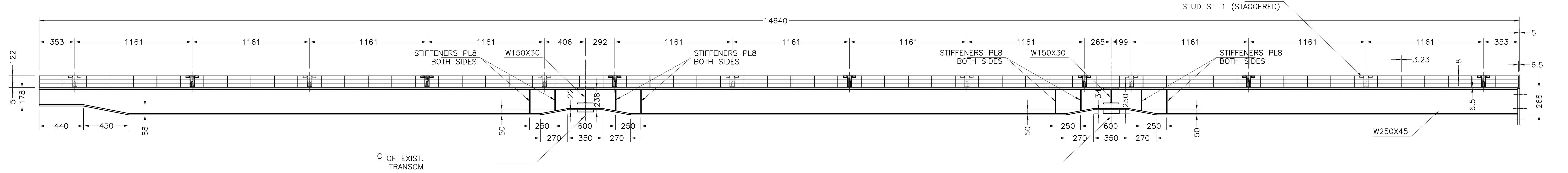
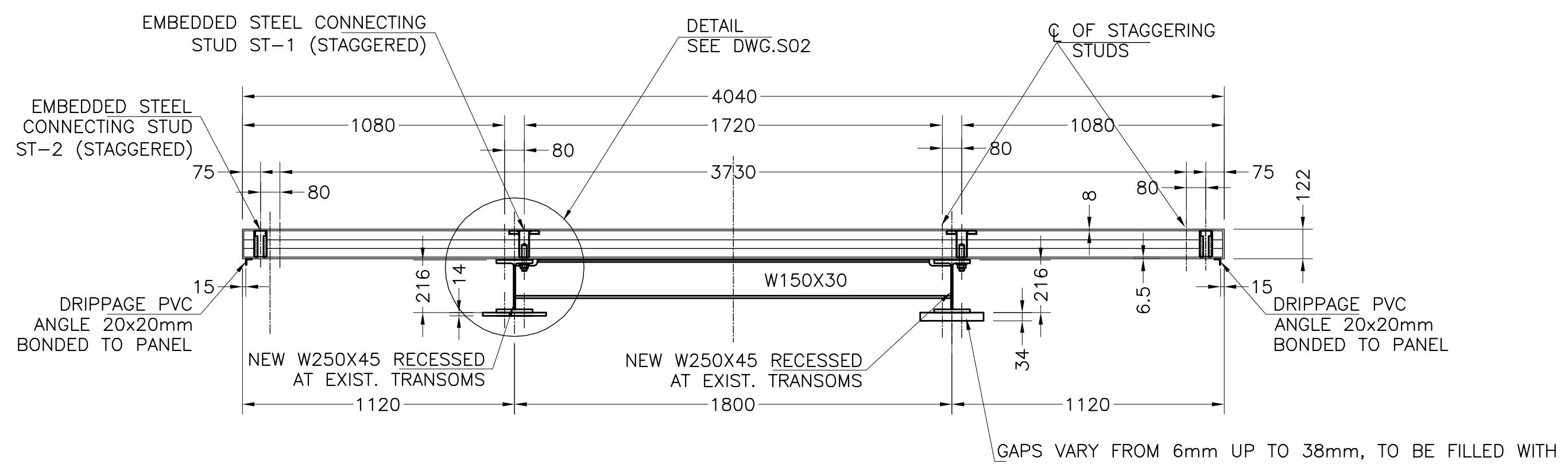
SCALE 1:20



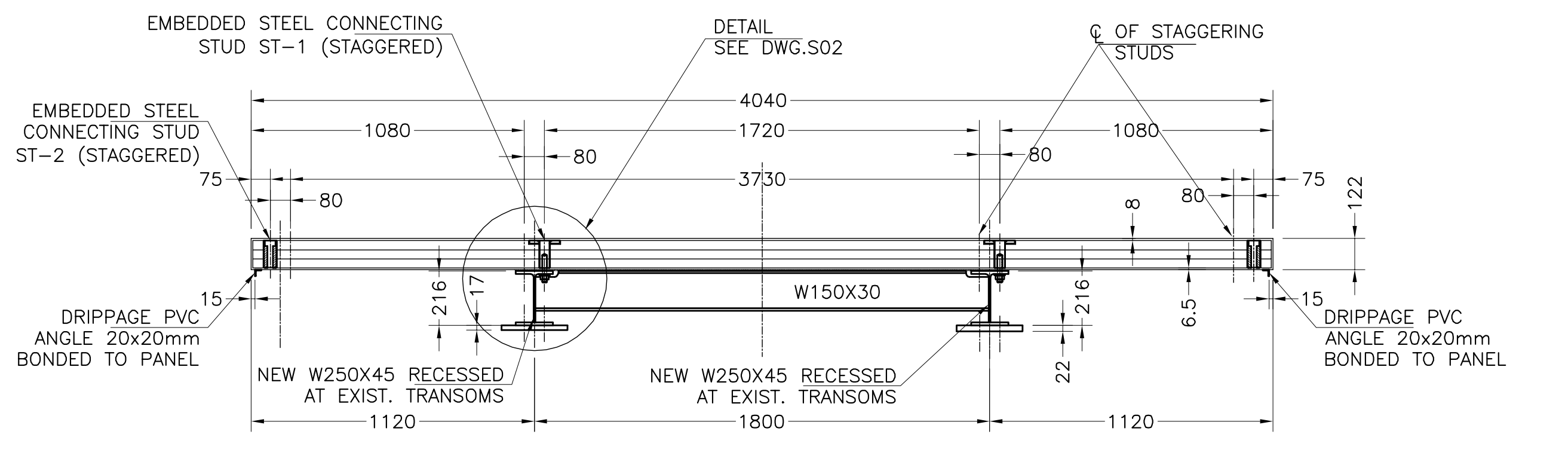
SECTION 3

SCALE 1:20



SECTION 4

SCALE 1:20



SECTION 5

SCALE 1:20

METRIC PRINTING COLOR SETTINGS: 1=0.150, 2=0.250, 3=0.350, 4=0.500, 5=0.350, 6=0.600, 7=0.350, 8=0.150, 9=0.100, 10–>255=0.130
IMPERIAL PRINTING COLOR SETTINGS: 1=0.005, 2=0.010, 3=0.014, 4=0.020, 5=0.014, 6=0.024, 7=0.014, 8=0.005, 9=0.010, 10–>255=0.005

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NAME	SIGNATURE	BCIN/BCDN
Lyakhovsky M.		29491

REGISTRATION INFORMATION

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FIRM NAME	BCIN/BCDN
I & F Engineering Corp.	29493

Orientation

N

Stamp

I. FINKELSHTEYN
10.31.2007

Newton Bridge Solutions Ltd.

41 MASSEY ROAD
GUELPH, ONTARIO
N1H 7M6

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CONSULTING ENGINEERS & MANAGERS
100 DRUMLIN CIRCLE, UNIT 203
CONCORD, ON, CANADA
(905) 669-8445
FAX: (905) 669-4803

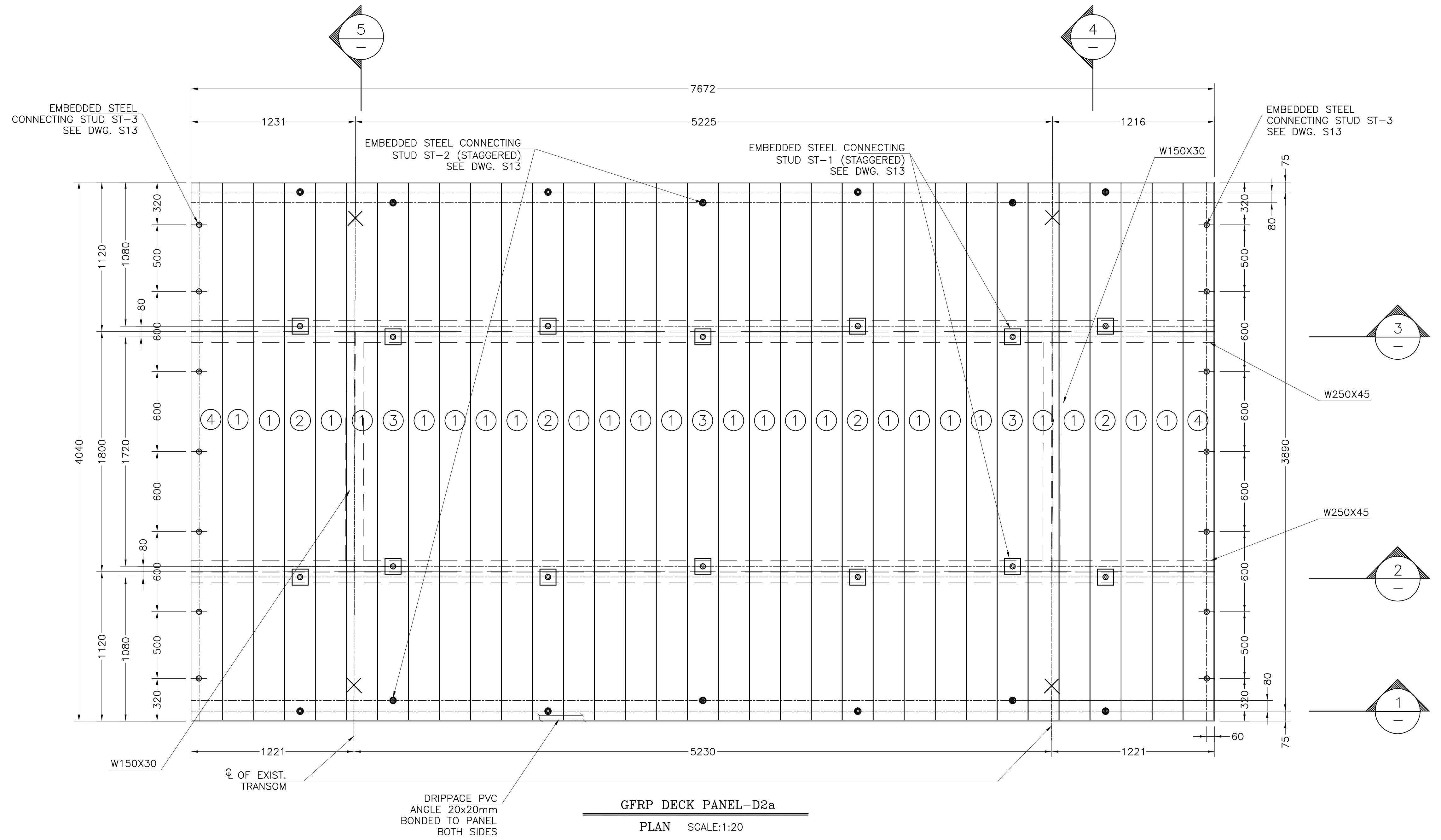
Client:
MUNICIPALITY OF HURON SHORES

Project:
DEAN LAKE BRIDGE
MTO SITE 385-234

Drawing Title:
DECK PANEL D1a

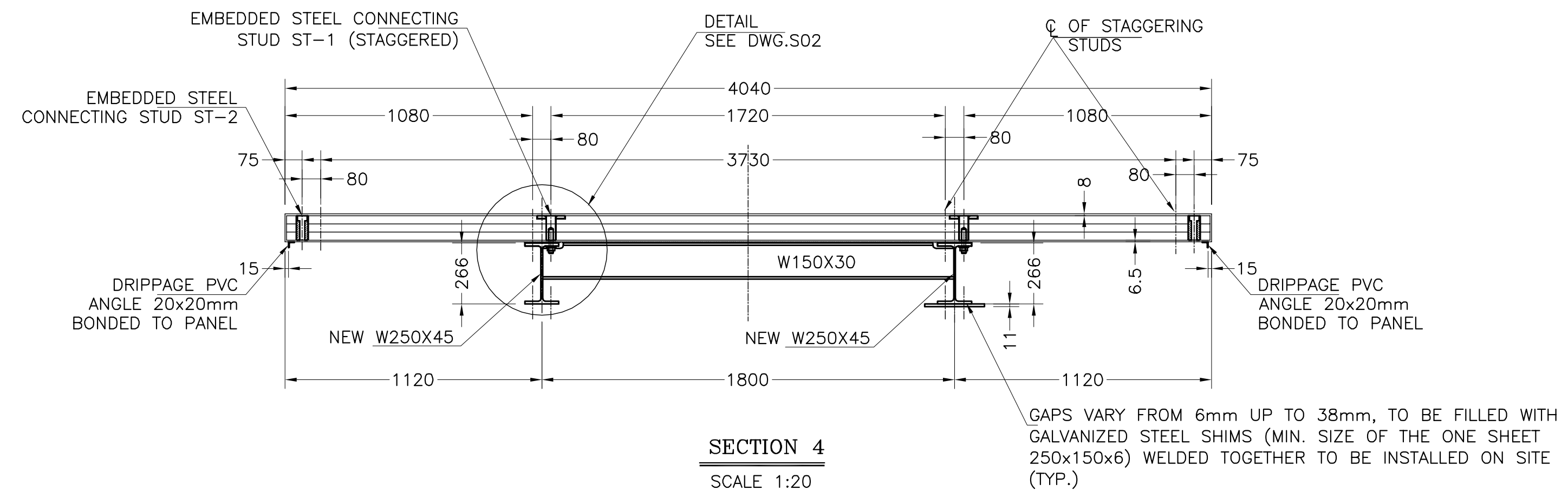
Drawn by: V.R.	Date: 10.10.2007
Checked by: M.L.	Scale: 1:20
Project Nr. 07M05	Drawing Nr. S04

CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY
SIZE 1178x841



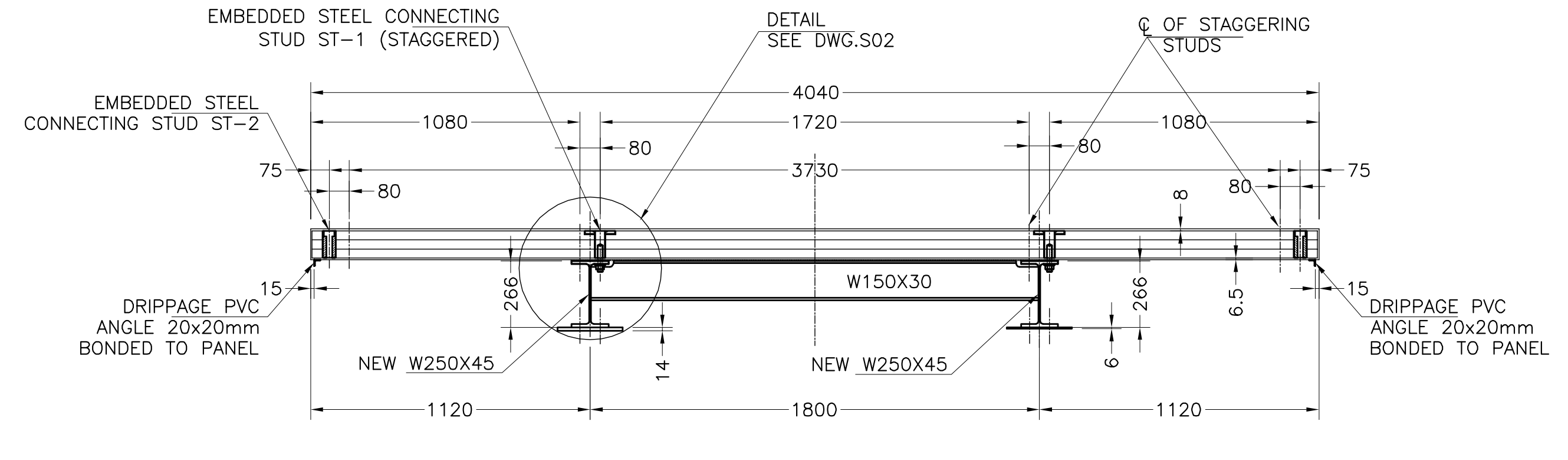
GFRP DECK PANEL-D2a

PLAN SCALE:1:20



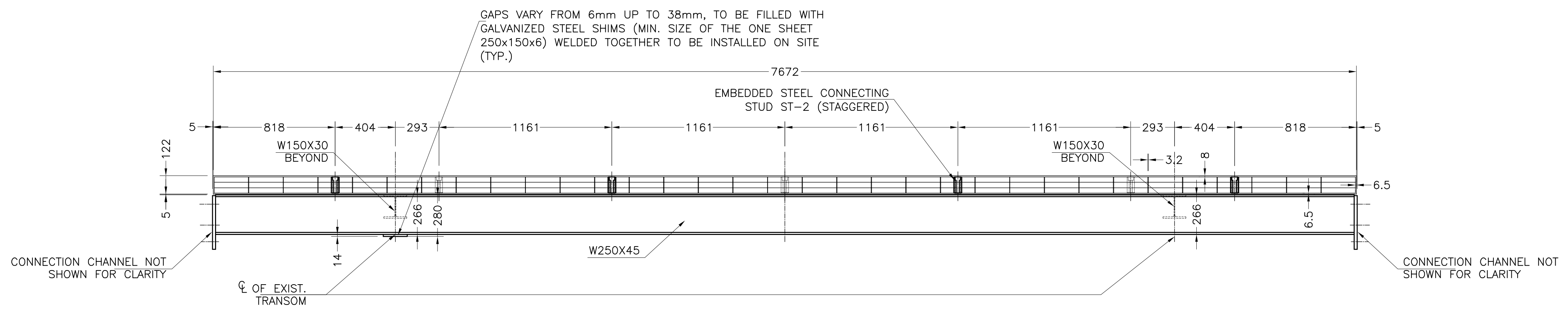
SECTION 4

SCALE 1:20



SECTION 5

SCALE 1:20



SECTION 1

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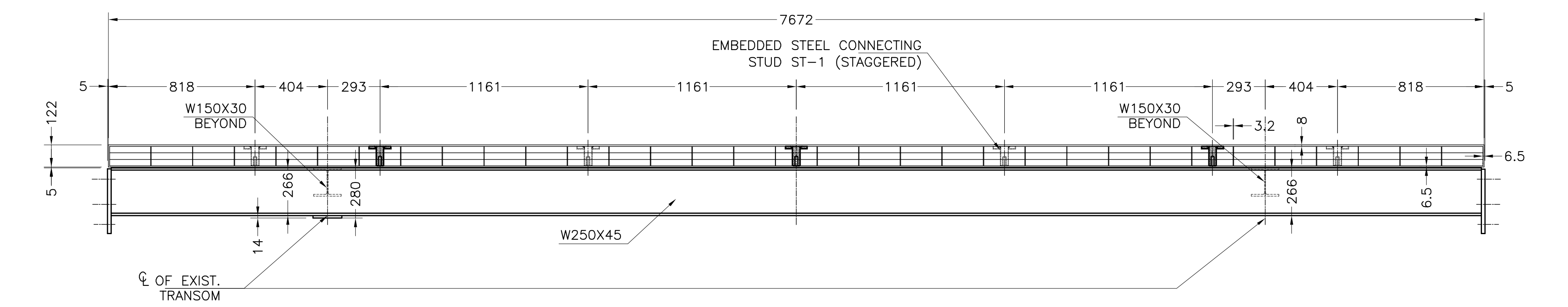
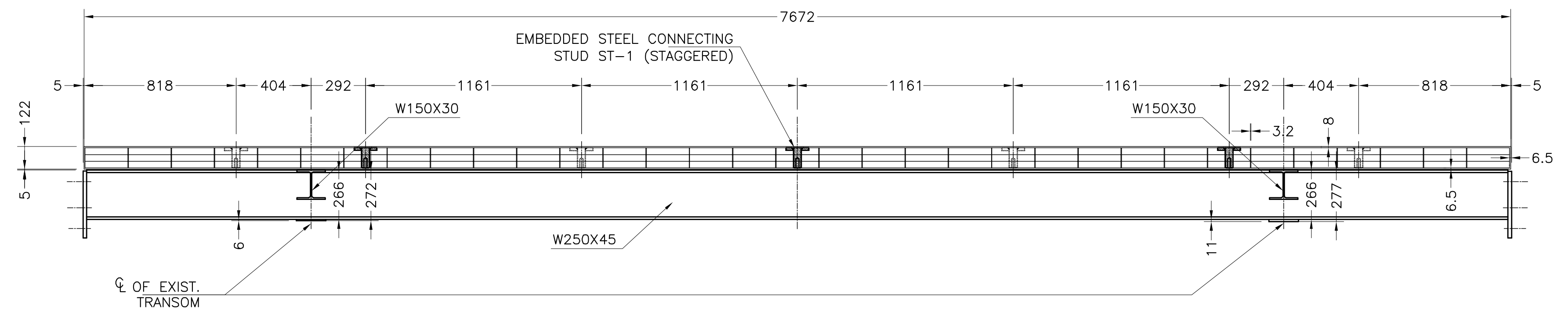


SECTION 2

SCALE 1:20



SECTION 3

SCALE 1:20

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Lyakhovsky M.		29491

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Orientation

Stamp

I. FINKELSHTEYN
10.31.2007

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FAX: (905) 669-4803

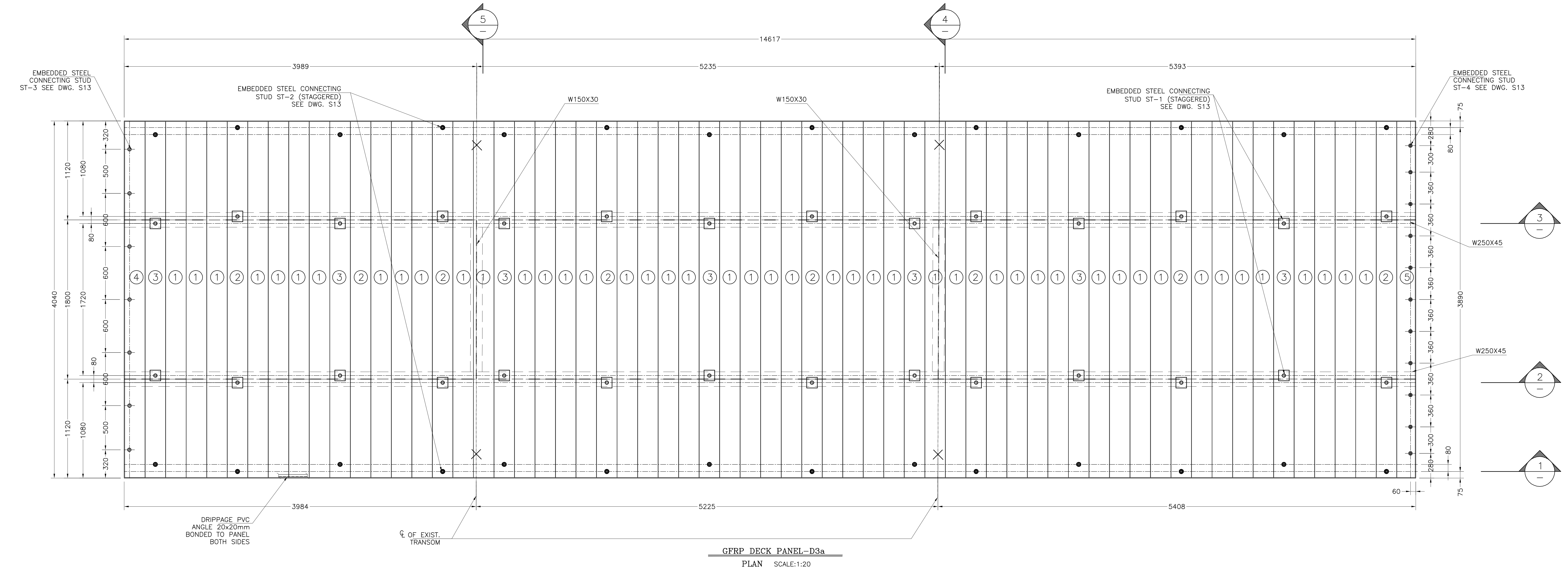
Client:
MUNICIPALITY OF HURON SHORES

Project:
DEAN LAKE BRIDGE
MTO SITE 385-234

Drawing Title:
DECK PANEL D2a

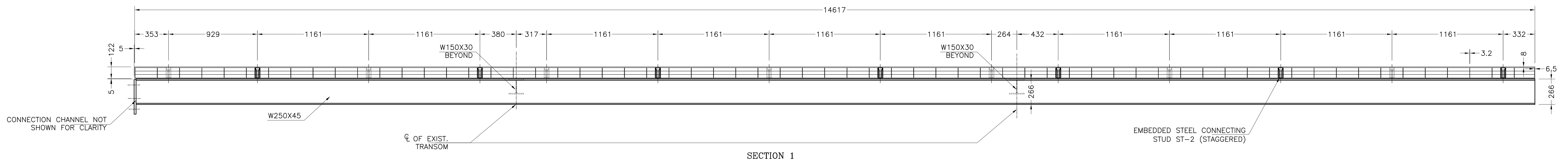
Drawn by: V.R.	Date: 10.10.2007
Checked by: M.L.	Scale: 1:20
Project Nr. 07M05	Drawing Nr. S05

CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY
SIZE 1178x841



GFRP DECK PANEL-D3a

PLAN SCALE:1:20



SECTION 1

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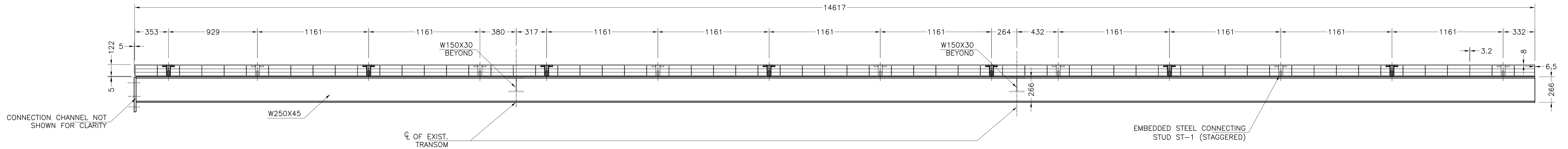
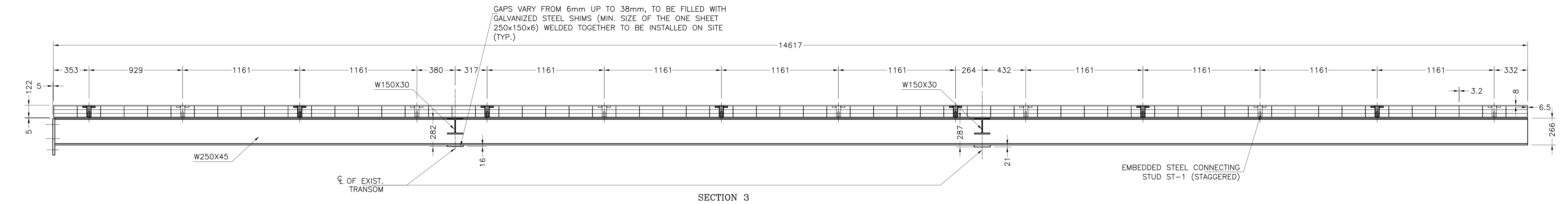


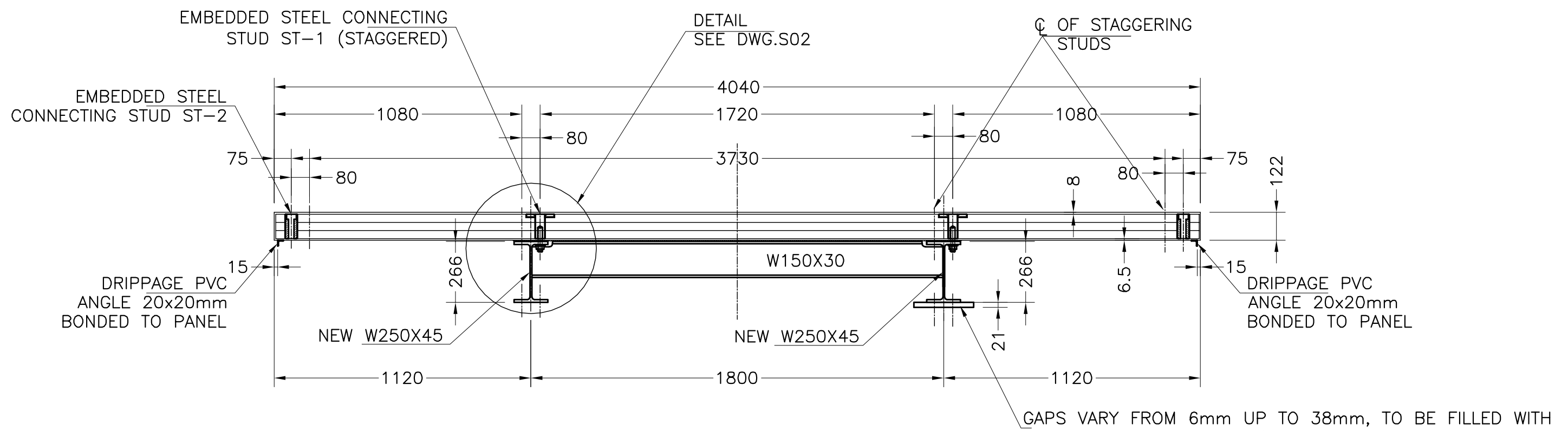
SECTION 2

SCALE 1:20



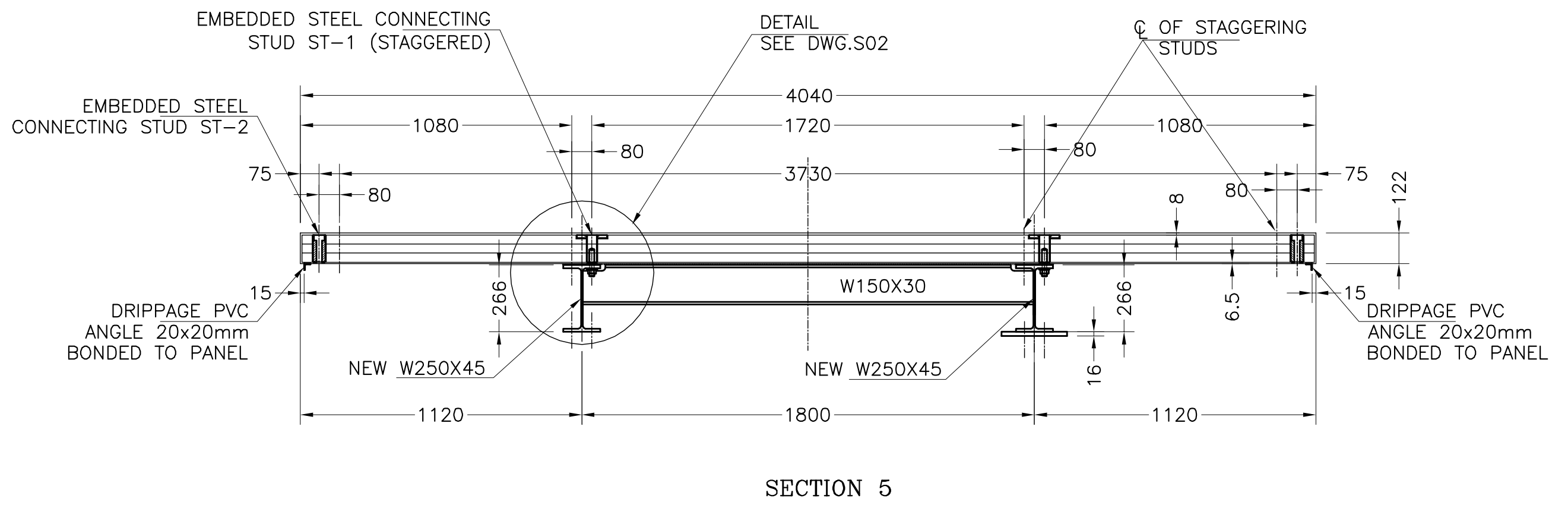
SECTION 3

SCALE 1:20



SECTION 4

SCALE 1:20



SECTION 5

SCALE 1:20

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I & F Engineering Corp.	29493
FIRM NAME	BCIN/BCDN

Orientation

N

Stamp

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PROVINCE OF ONTARIO

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Client:

MUNICIPALITY OF HURON SHORES

Project:

DEAN LAKE BRIDGE
MTO SITE 385-234

Drawing Title:

DECK PANEL D3a

Drawn by: V.R.	Date: 10.10.2007
Checked by: M.L.	Scale: 1:20
Project Nr. 07M05	Drawing Nr. S06

CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY
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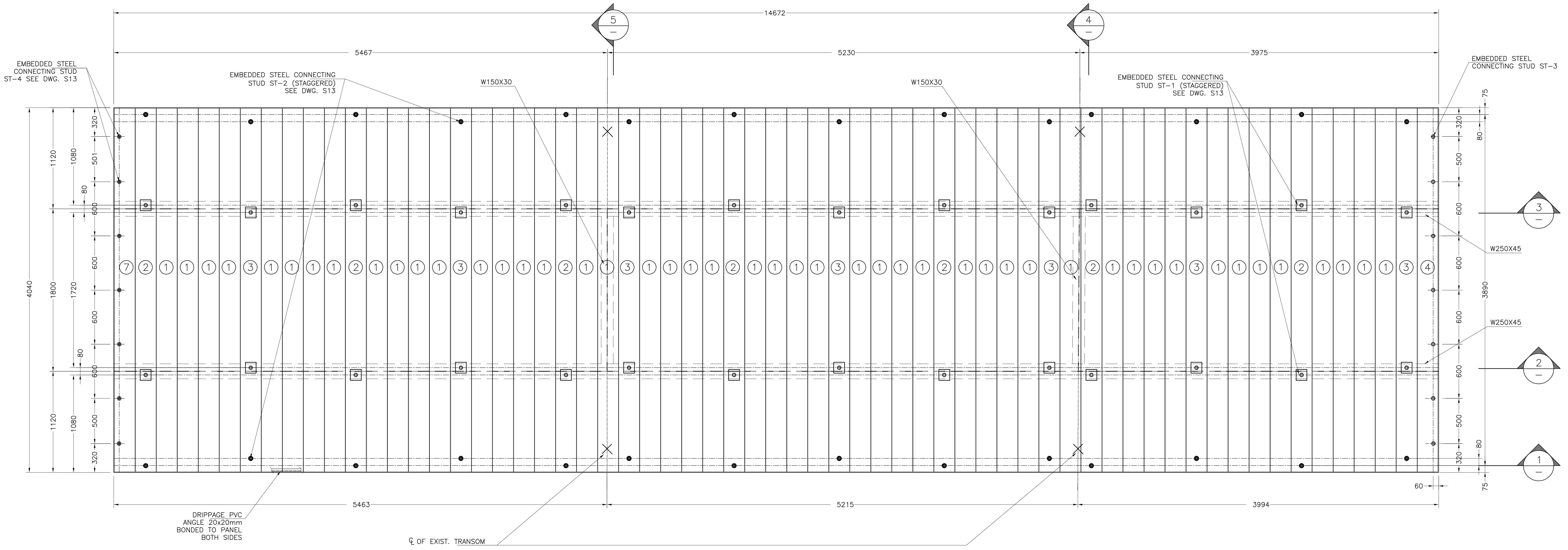
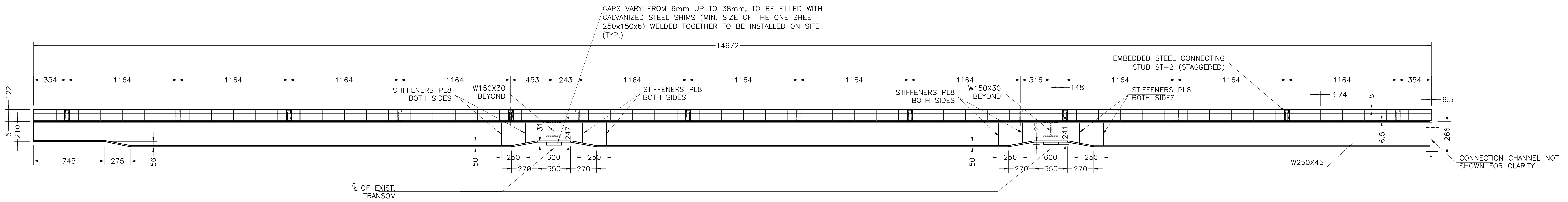


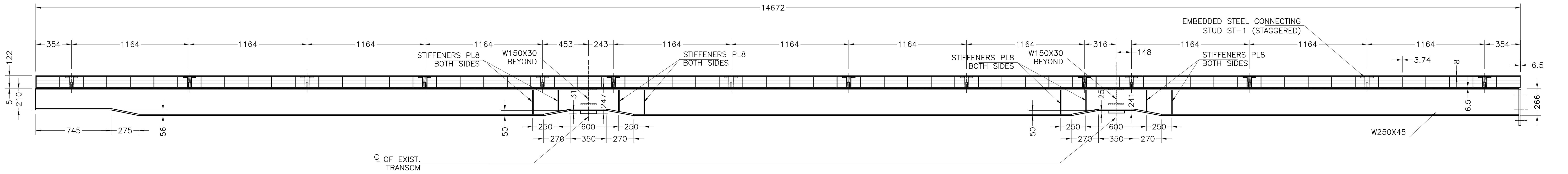
GFRP DECK PANEL–D1b

PLAN SCALE:1:20



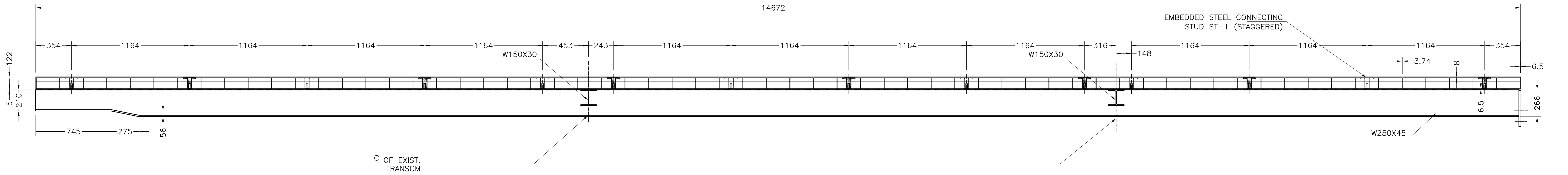
SECTION 1

SCALE 1:20



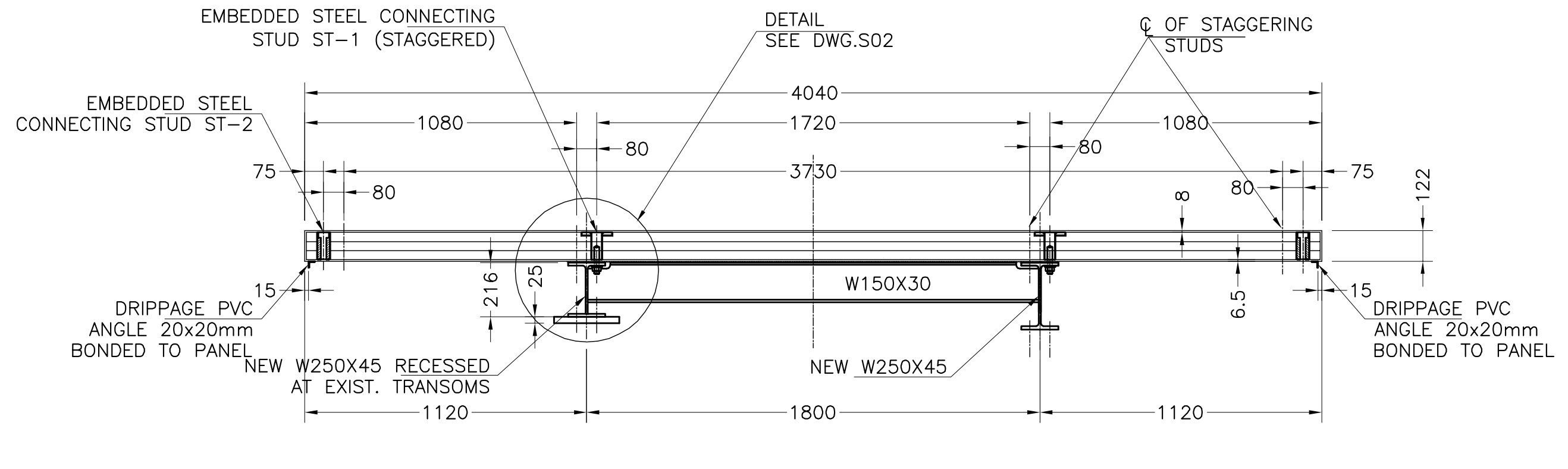
SECTION 2

SCALE 1:20



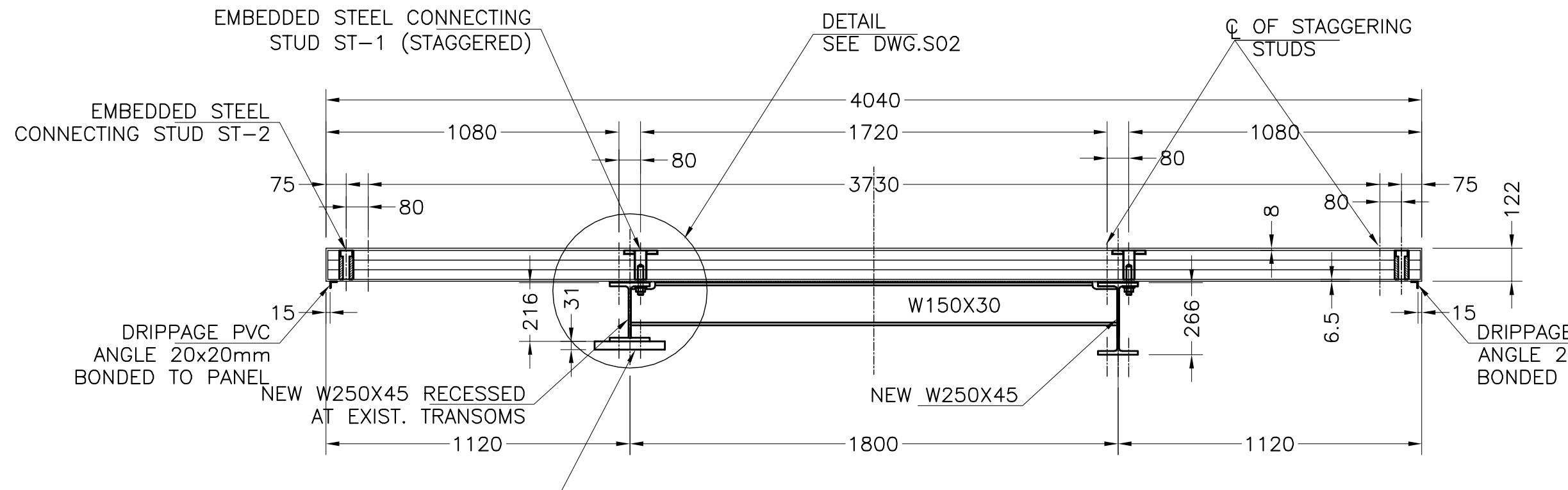
SECTION 3

SCALE 1:20



SECTION 4

SCALE 1:20



GAPS VARY FROM 6mm UP TO 38mm, TO BE FILLED WITH GALVANIZED STEEL SHIMS (MIN. SIZE OF THE ONE SHEET 250x150x6) WELDED TOGETHER TO BE INSTALLED ON SITE (TYP.)

SECTION 5

SCALE 1:20

METRIC PRINTING COLOR SETTINGS: 1=0.150, 2=0.250, 3=0.350, 4=0.500, 5=0.350, 6=0.600, 7=0.350, 8=0.150, 9=0.100, 10->255=0.130
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NAME	SIGNATURE	BCIN/BCDN
Lyakhovsky M.		29491

REGISTRATION INFORMATION

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FIRM NAME	BCIN/BCDN
I & F Engineering Corp.	29493

Orientation

N

Stamp

LICENSED PROFESSIONAL ENGINEER
I. FINKELSHTEYN
10.31.2007
PROVINCE OF ONTARIO

Newton Bridge Solutions Ltd.

41 MASSEY ROAD
GUELPH, ONTARIO
N1H 7M6

TEL. 519-341-8944
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CONSULTING ENGINEERS & MANAGERS

100 DRUMLIN CIRCLE, UNIT 203
CONCORD, ON, CANADA

(905) 669-8445
FAX: (905) 669-4803

Client:

MUNICIPALITY OF HURON SHORES

Project:

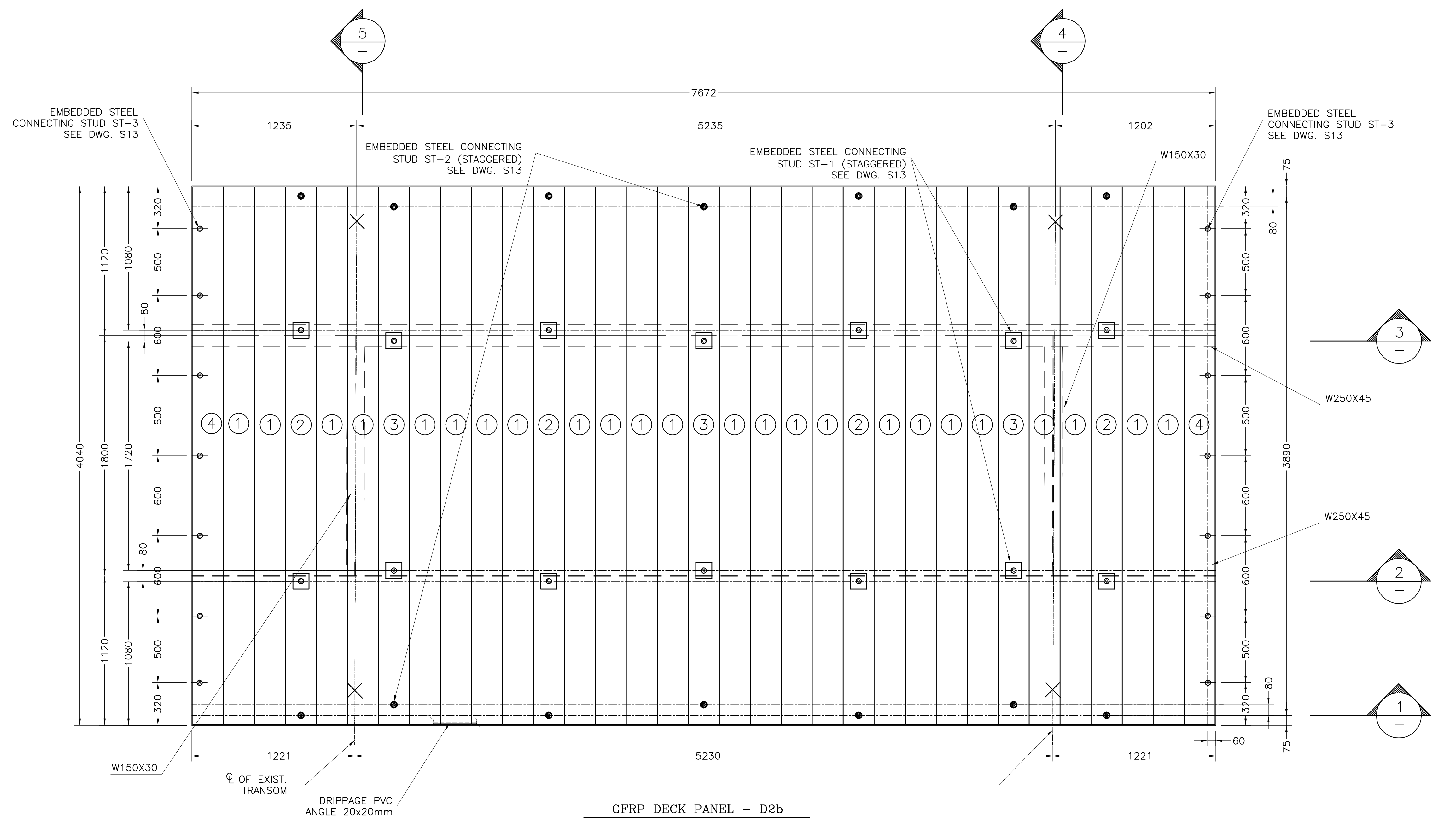
DEAN LAKE BRIDGE
MTO SITE 385-234

Drawing Title:

DECK PANEL D1b

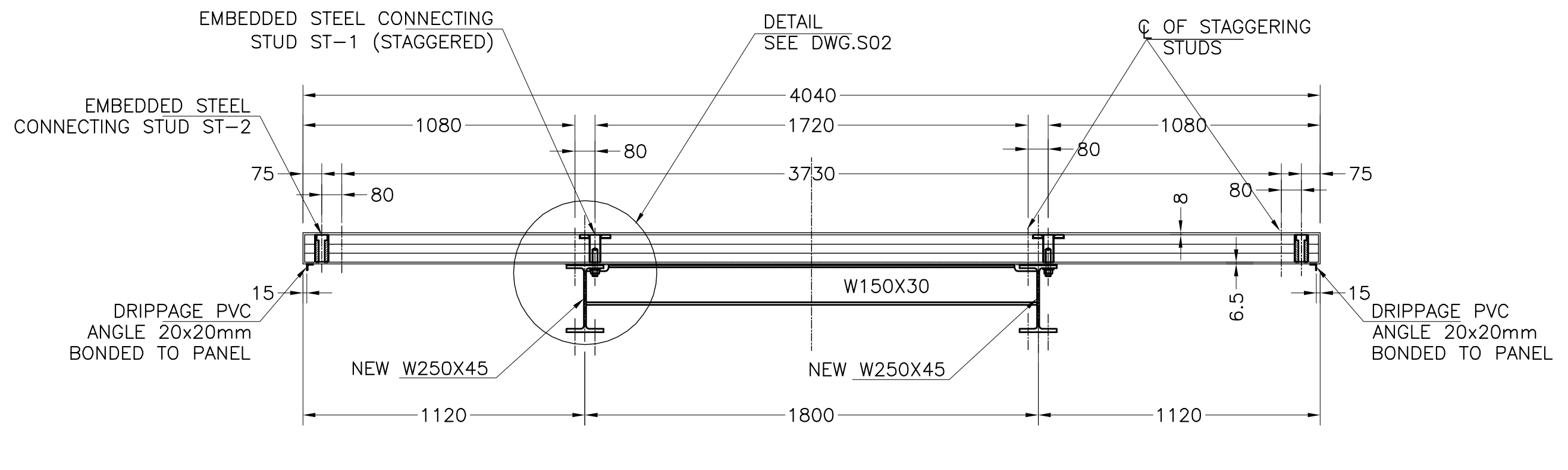
Drawn by: V.R.	Date: 10.10.2007
Checked by: M.L.	Scale: 1:20
Project Nr. 07M05	Drawing Nr. S07

CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY
SIZE 1178x841

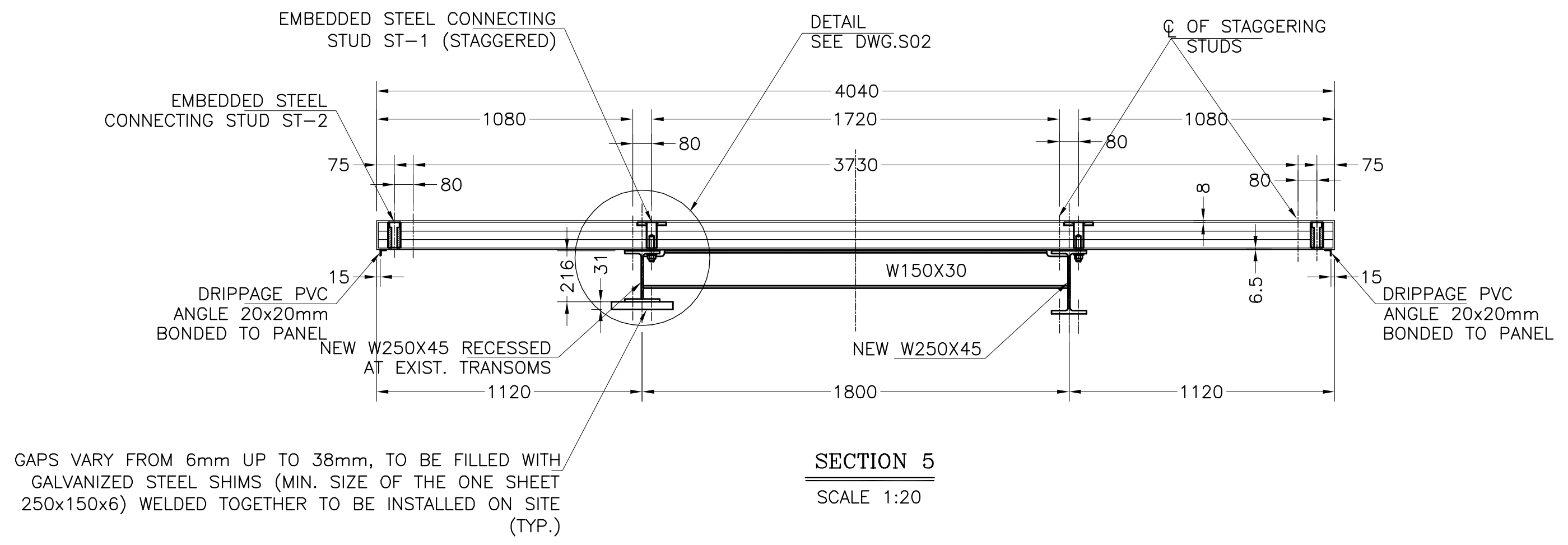


GFRP DECK PANEL – D2b

PLAN SCALE:1:20



SECTION 4
SCALE 1:20



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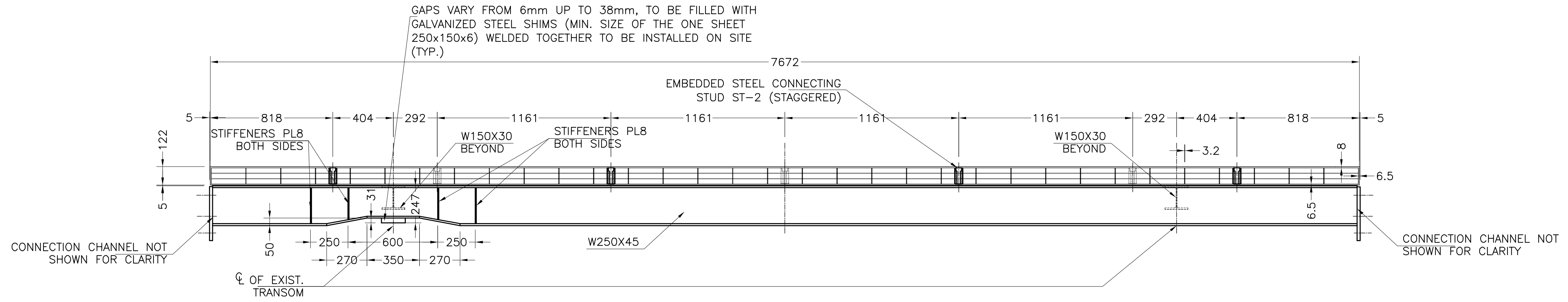
SCALE 1:20



SECTION 1
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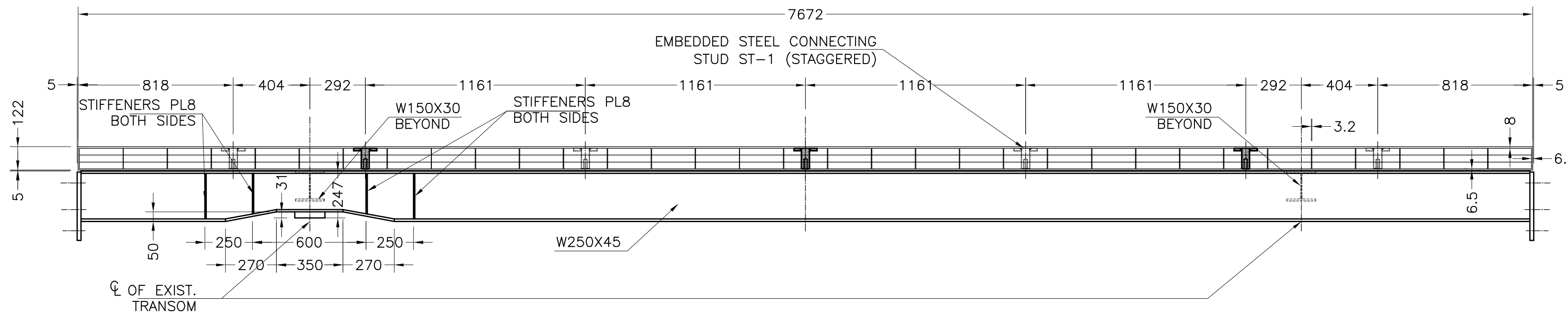
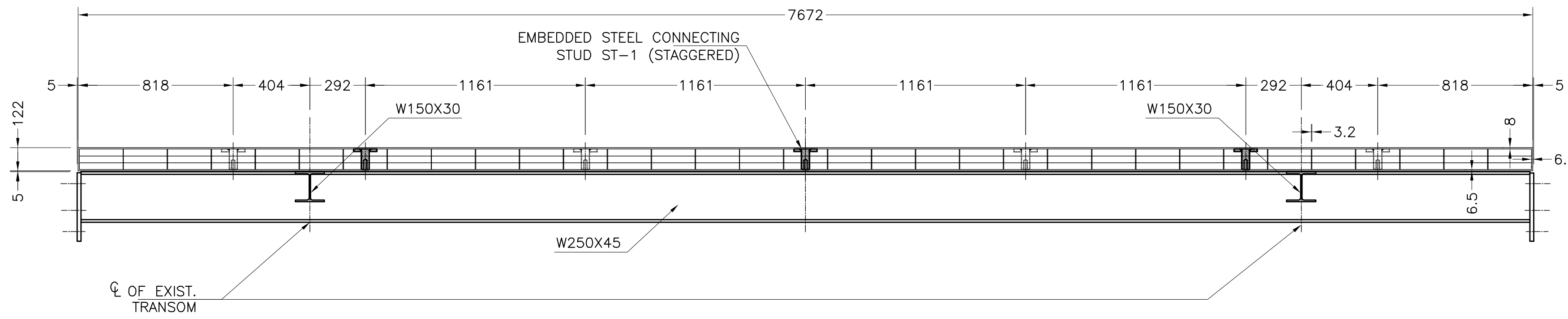


SECTION 2
SCALE 1:20



SECTION 3
SCALE 1:20

METRIC PRINTING COLOR SETTINGS: 1=0.150, 2=0.250, 3=0.350, 4=0.500, 5=0.350, 6=0.600, 7=0.350, 8=0.150, 9=0.100, 10->255=0.130
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MUNICIPALITY OF HURON SHORES

Project:

DEAN LAKE BRIDGE
MTO SITE 385-234

Drawing Title:

DECK PANEL D2b

Drawn by: V.R.	Date: 10.10.2007
Checked by: M.L.	Scale: 1:20
Project Nr. 07M05	Drawing Nr. S08

CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY

SIZE 1178x841

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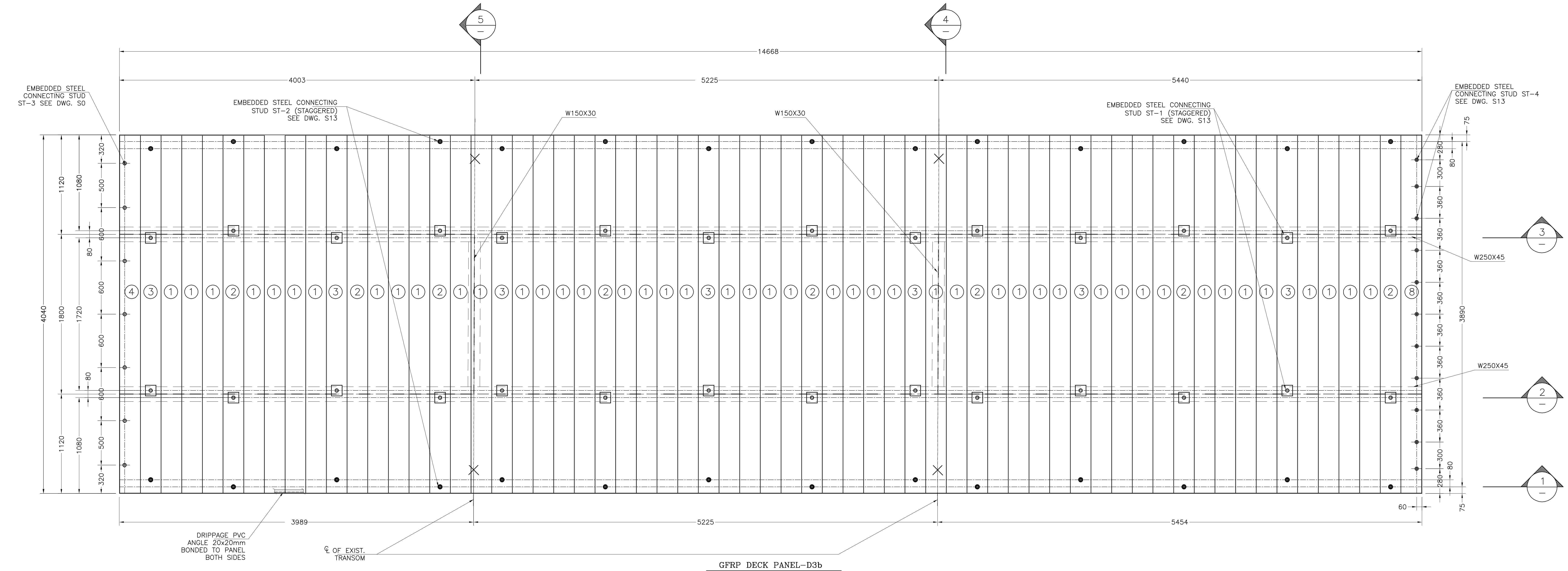
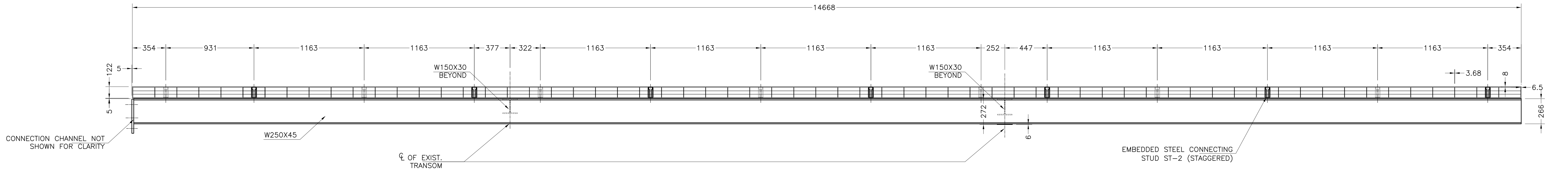


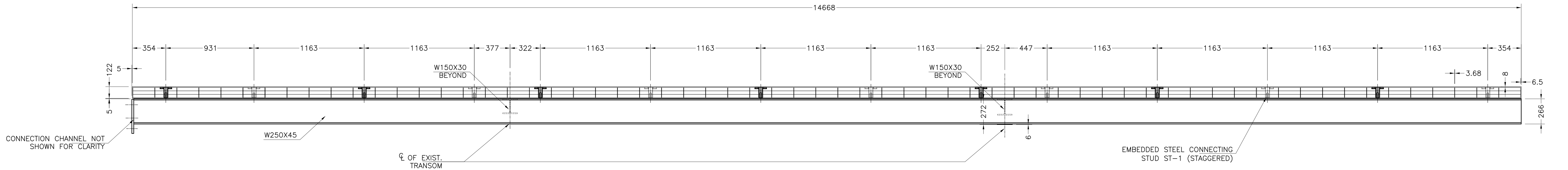
GFRP DECK PANEL-D3b

PLAN SCALE:1:20



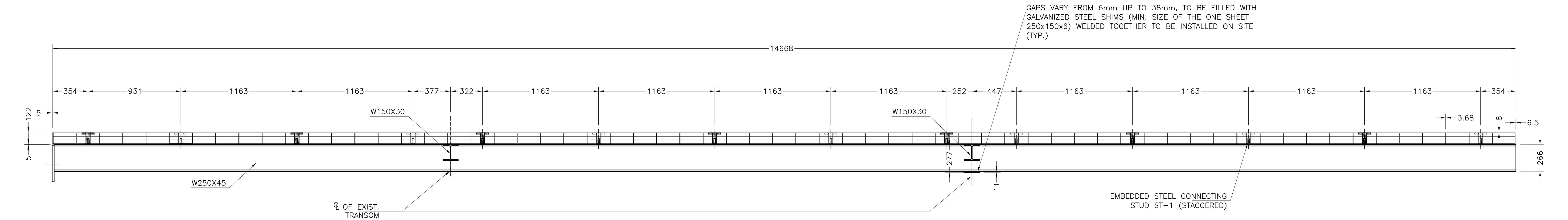
SECTION 1

SCALE 1:20



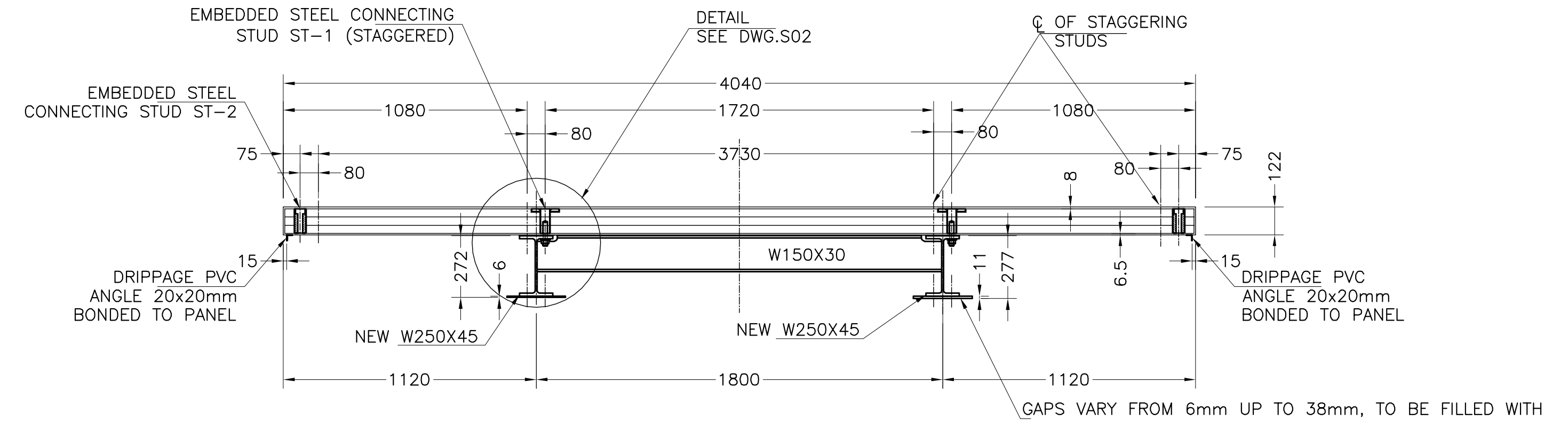
SECTION 2

SCALE 1:20



SECTION 3

SCALE 1:20



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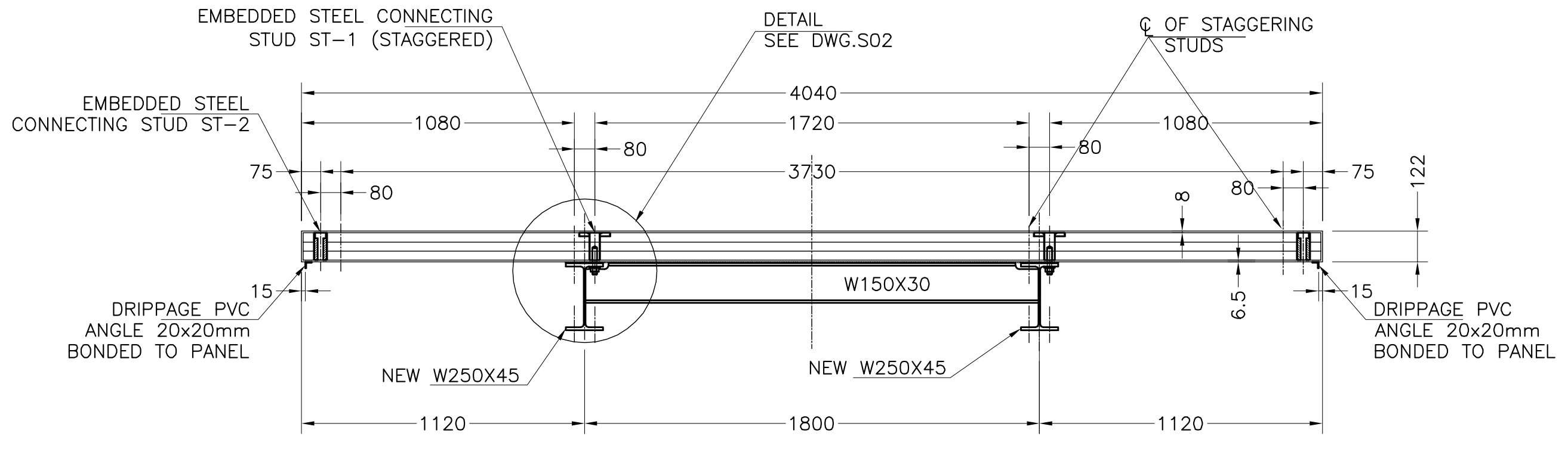
SECTION 4

SCALE 1:20



SECTION 5

SCALE 1:20

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Orientation

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Stamp

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Client:

MUNICIPALITY OF HURON SHORES

Project:

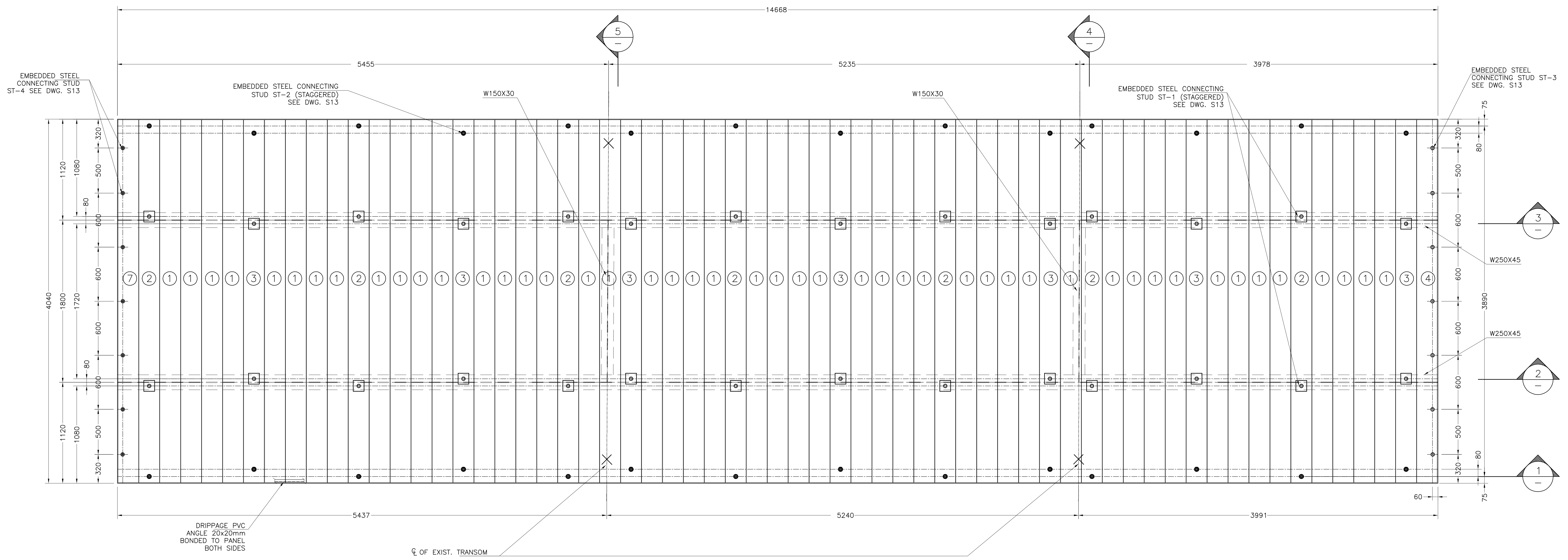
DEAN LAKE BRIDGE
MTO SITE 385-234

Drawing Title:

DECK PANEL D3b

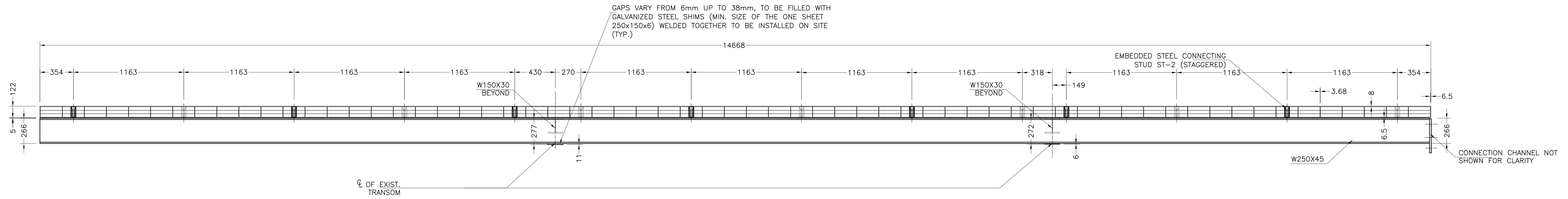
Drawn by:	Date:
V.R.	10.10.2007
Checked by:	**Scale:**
M.L.	1:20
Project Nr.	**Drawing Nr.**
07M05	S09

CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY
SIZE 1178x841



GFRP DECK PANEL-D1c

PLAN SCALE:1:20



SECTION 1

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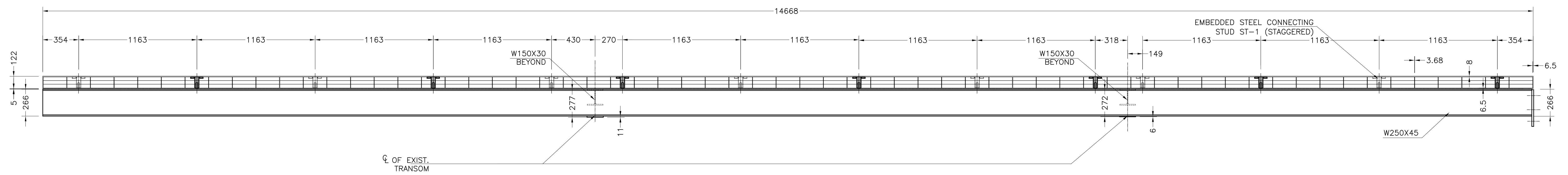
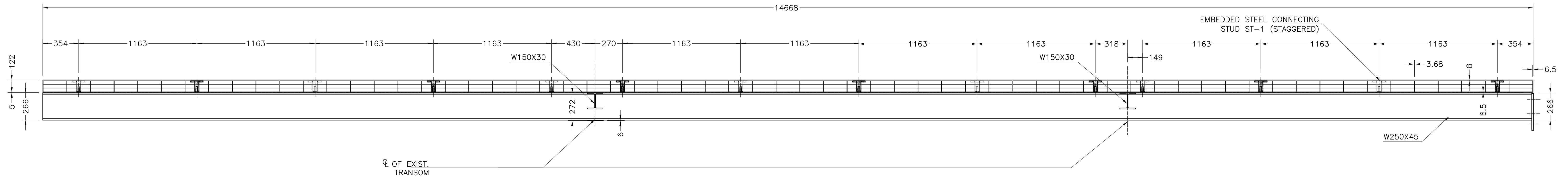


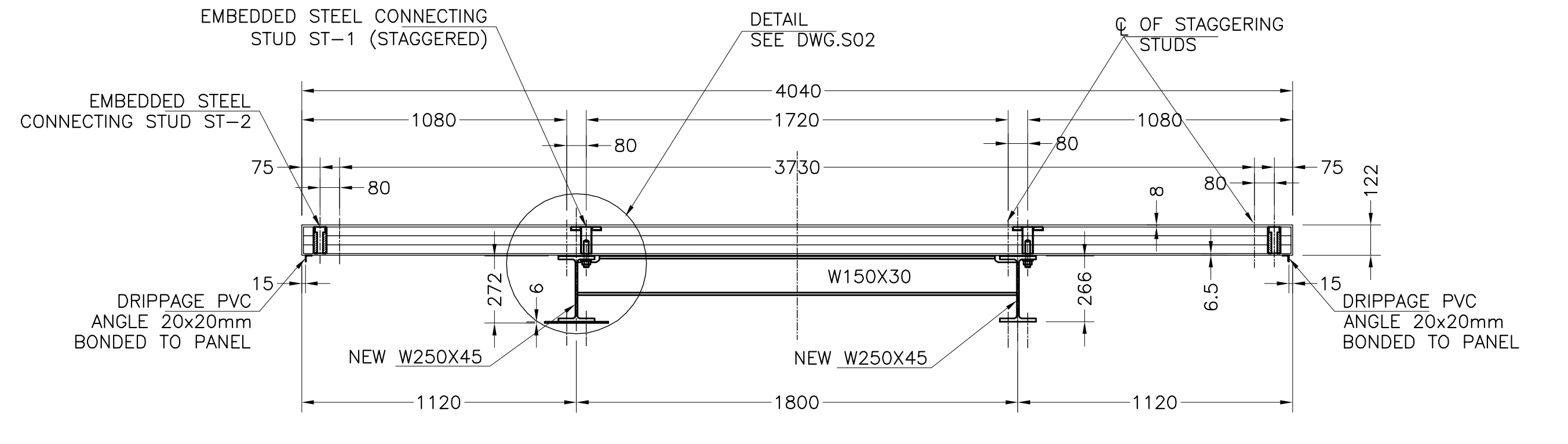
SECTION 2

SCALE 1:20



SECTION 3

SCALE 1:20



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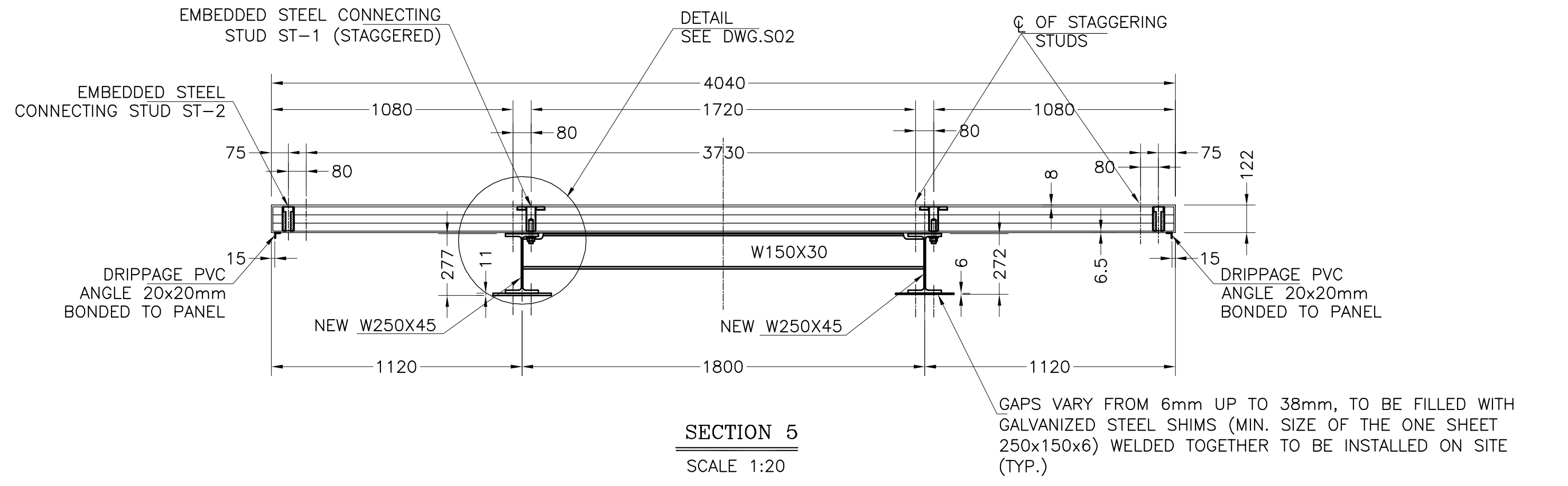
SCALE 1:20



SECTION 5

SCALE 1:20

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Client:

MUNICIPALITY OF HURON SHORES

Project:

DEAN LAKE BRIDGE
MTO SITE 385-234

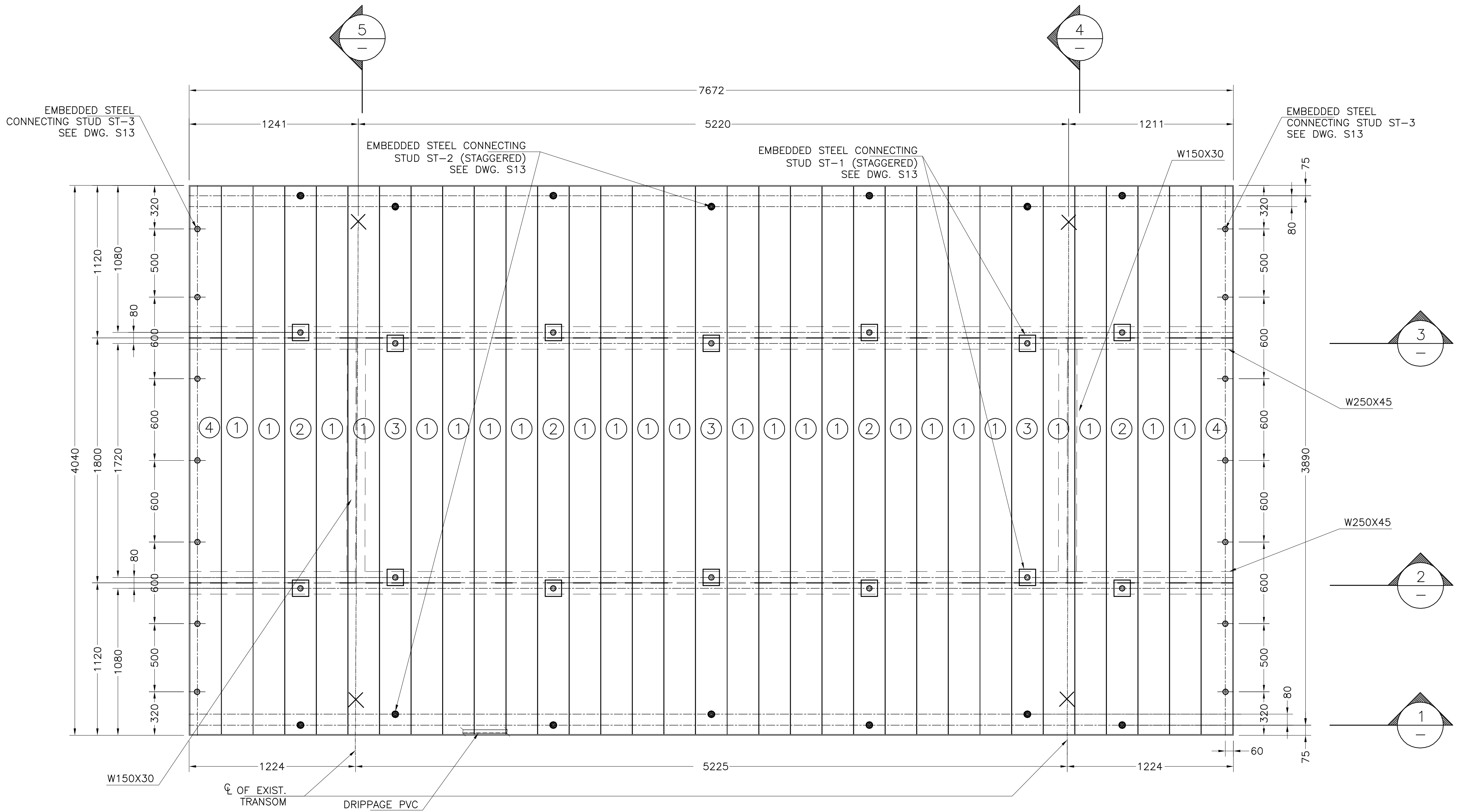
Drawing Title:

DECK PANEL D1c

Drawn by: V.R.	Date: 10.10.2007
Checked by: M.L.	Scale: 1:20
Project Nr. 07M05	Drawing Nr. S10

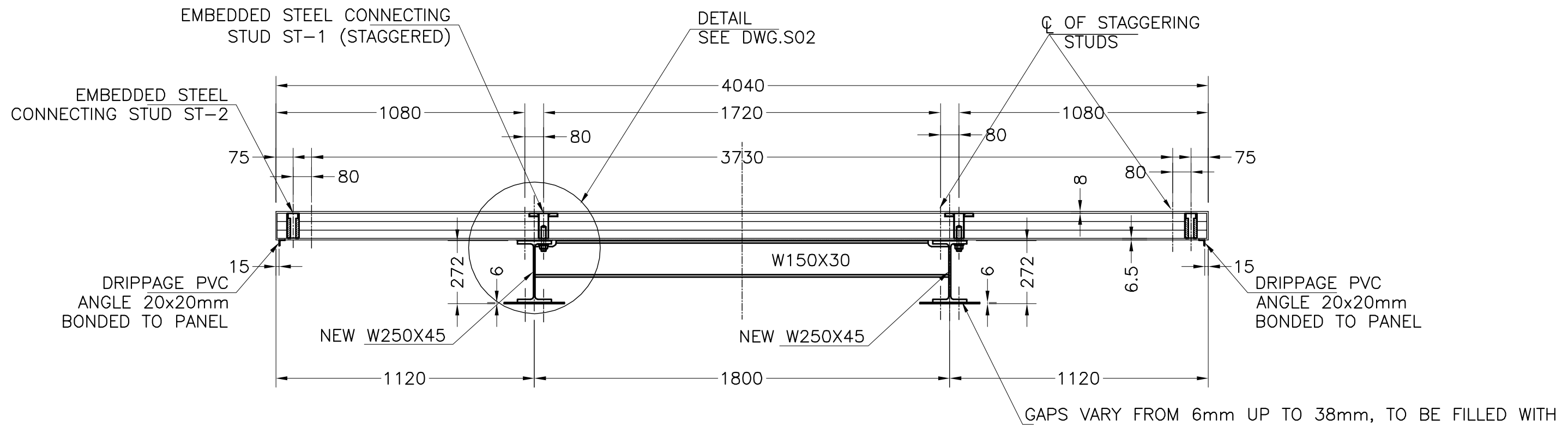
CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY

SIZE 1178x841



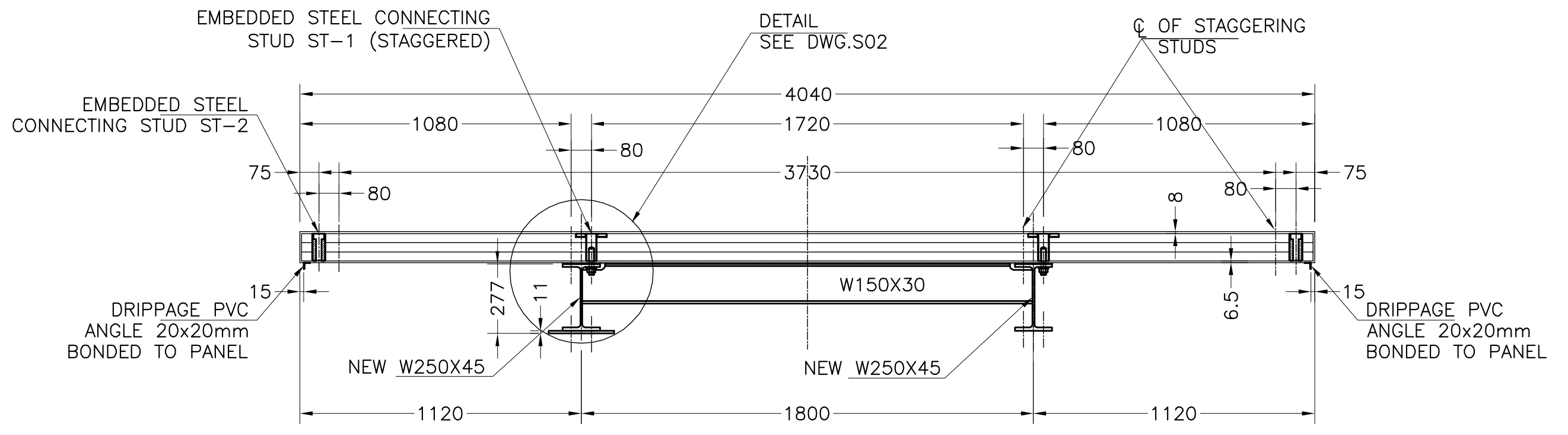
GFRP DECK PANEL – D2c

PLAN SCALE:1:20



SECTION 4

SCALE 1:20



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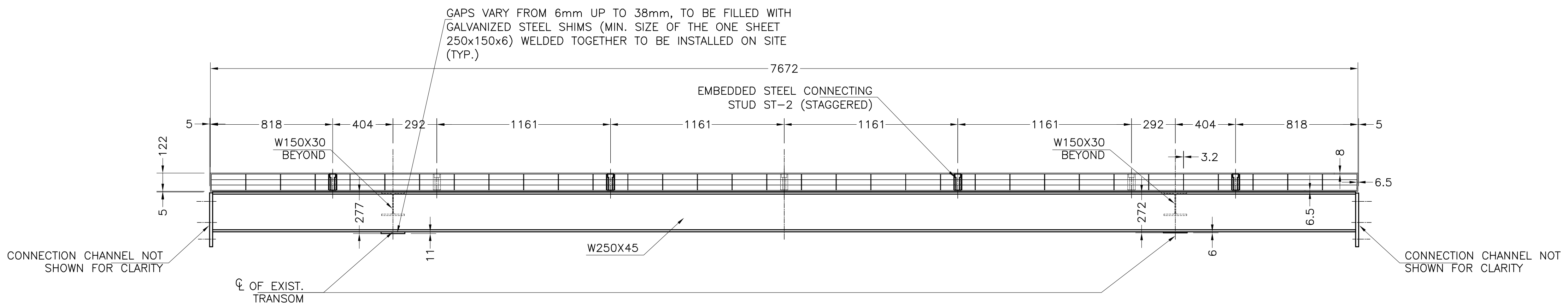

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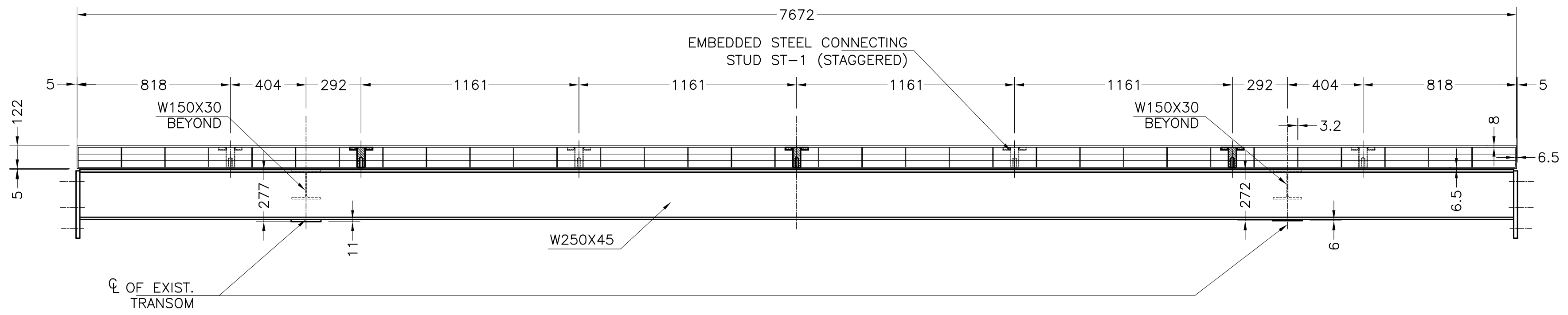
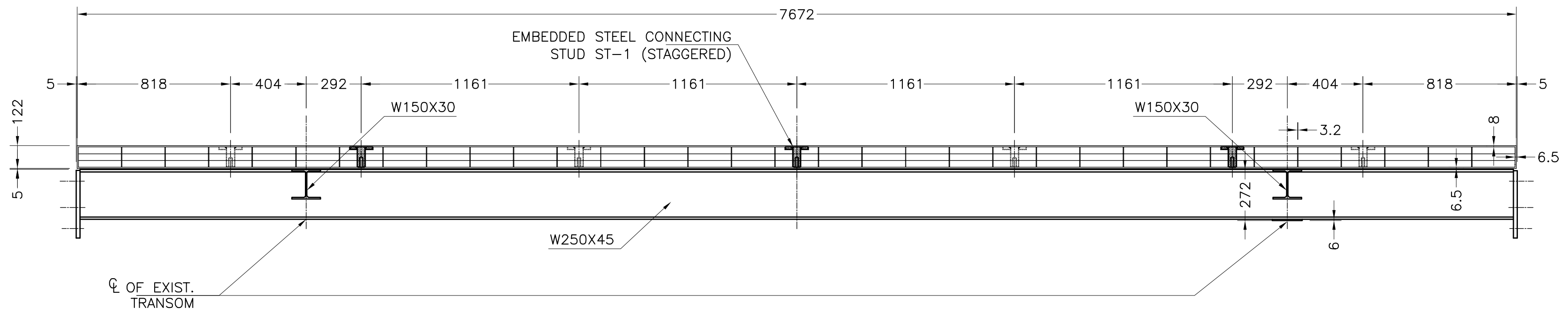


SECTION 2

SCALE 1:20



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SCALE 1:20

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MTO SITE 385-234

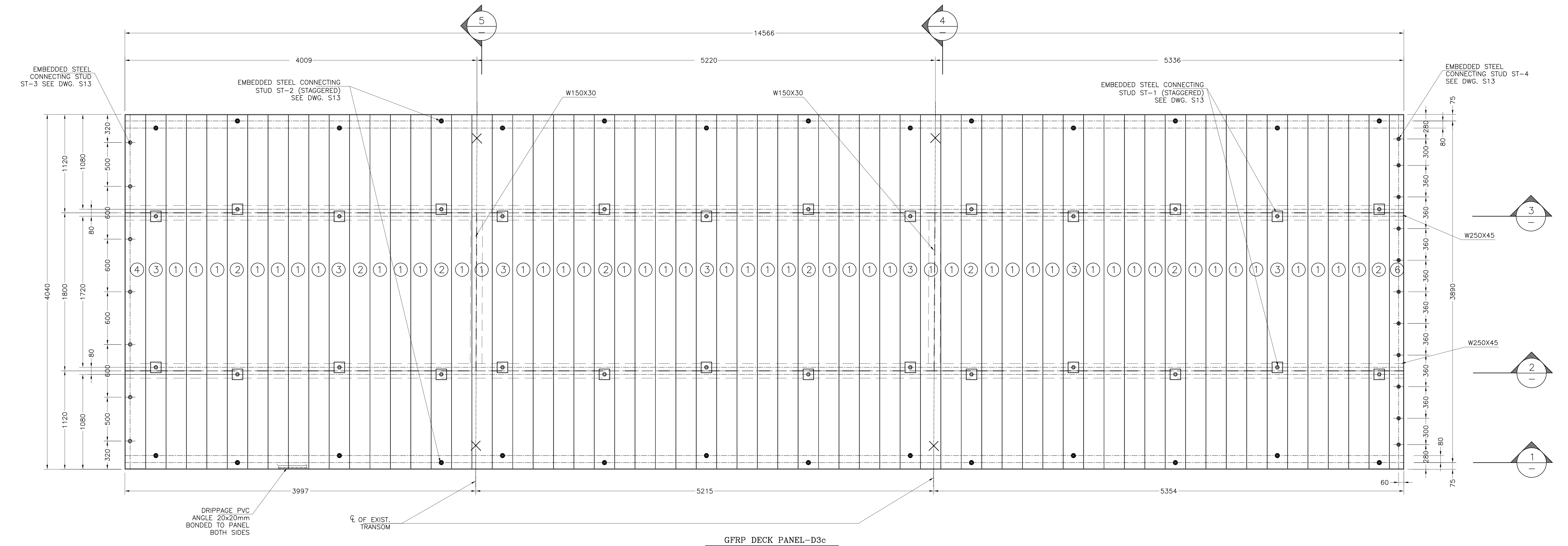
Drawing Title:

DECK PANEL D2c

Drawn by: V.R.	Date: 10.10.2007
Checked by: M.L.	Scale: 1:20
Project Nr. 07M05	Drawing Nr. S11

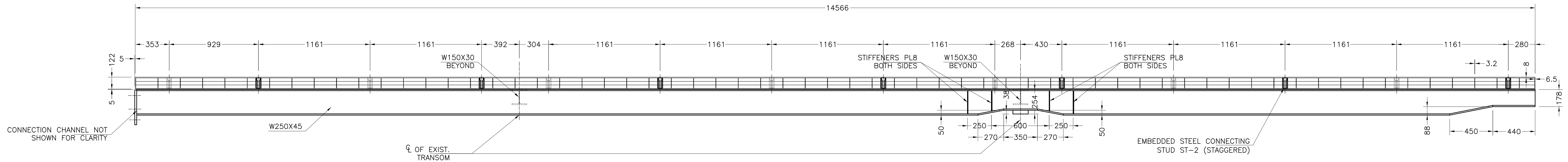
CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY
SIZE 1178x841

METRIC PRINTING COLOR SETTINGS: 1=0.150, 2=0.250, 3=0.350, 4=0.500, 5=0.350, 6=0.600, 7=0.350, 8=0.150, 9=0.100, 10–>255=0.130
IMPERIAL PRINTING COLOR SETTINGS: 1=0.005, 2=0.010, 3=0.014, 4=0.020, 5=0.014, 6=0.024, 7=0.014, 8=0.005, 9=0.010, 10–>255=0.005



GFRP DECK PANEL-D3c

PLAN SCALE:1:20



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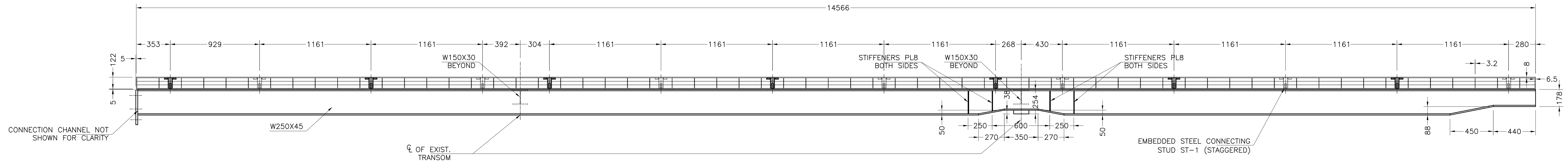

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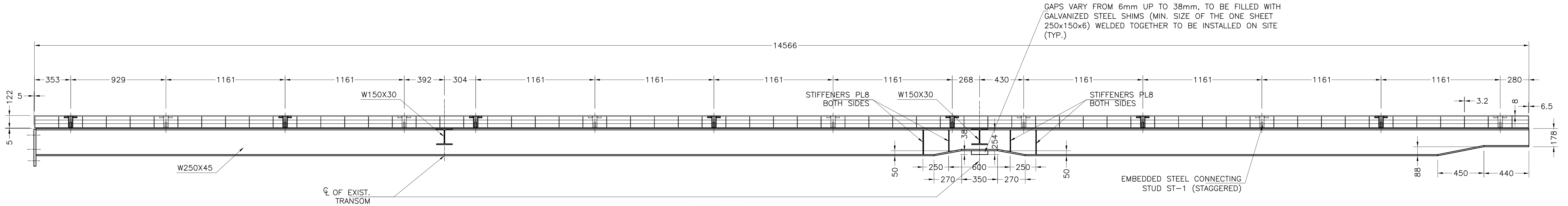

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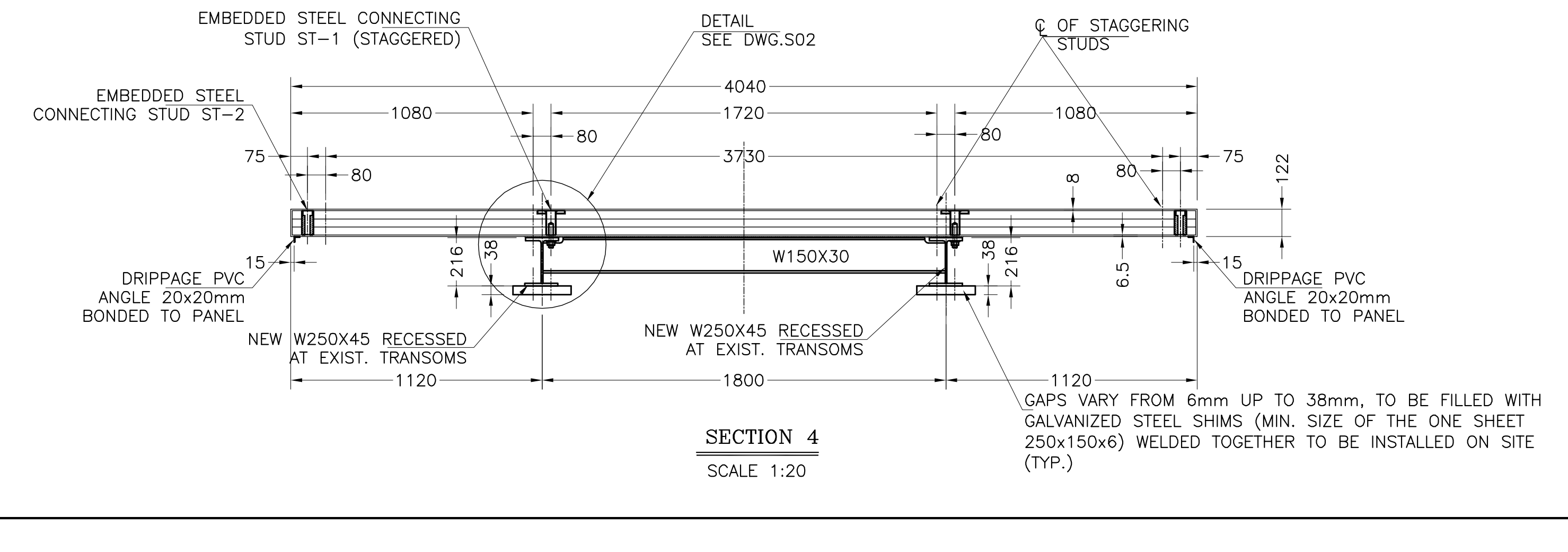

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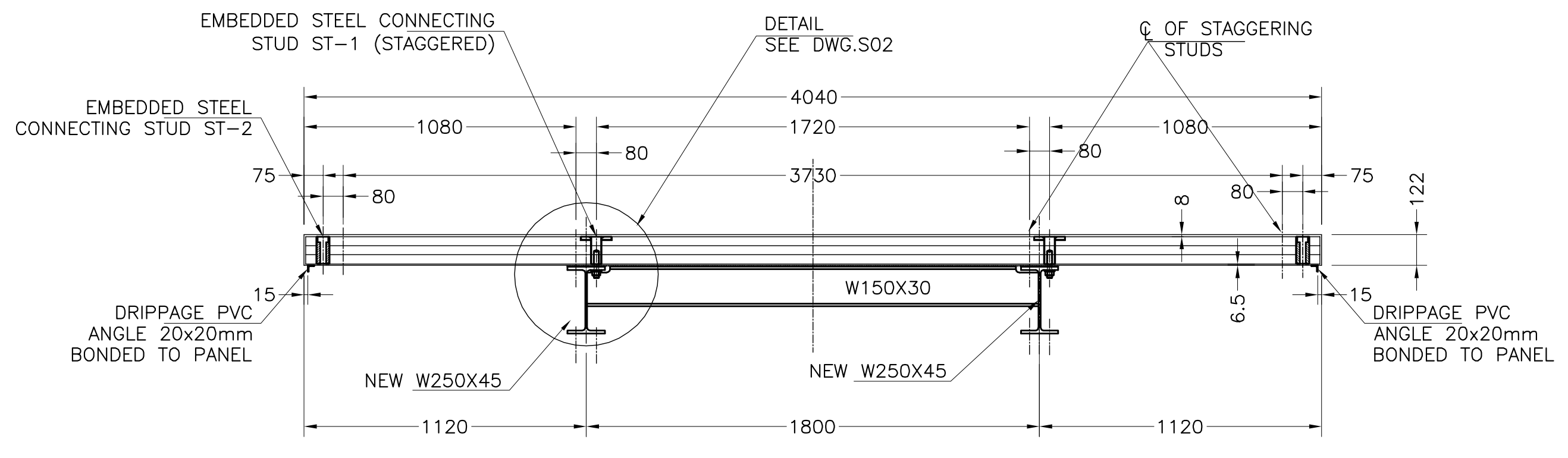

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IMPERIAL PRINTING COLOR SETTINGS: 1=0.005, 2=0.010, 3=0.014, 4=0.020, 5=0.014, 6=0.024, 7=0.014, 8=0.005, 9=0.010, 10->255=0.005

NO.	DATE:	REVISION	CH'D
1	10.10.2007	ISSUED FOR REVIEW	V.R.
2	10.24.2007	ISSUED FOR FINAL COORDINATION	V.R.
3	10.31.2007	ISSUED FOR CONSTRUCTION	V.R.

● ISSUED FOR CONSTRUCTION
○ ISSUED FOR PRICING
○ ISSUED FOR BUILDING PERMIT
○ ISSUED FOR PRELIM. REVIEW
○ ISSUED FOR GENERAL REVIEW
○ ISSUED FOR PROGRESS REVIEW
○ ISSUED FOR FINAL COORDINATION

The undersigned has reviewed and takes responsibility for this design, and has the qualifications and meets the requirements set out in the Ontario Building Code to be a designer.

QUALIFICATION INFORMATION

Required unless design is exempt under Division C-3.2.5.1 of the 2006 Ontario Building Code (or 2.17.5.1 of the 1997 Ontario Building Code)

NAME	SIGNATURE	BCIN/BCDN
Lyakhovsky M.		29491

REGISTRATION INFORMATION

Required unless design is exempt under Division C-3.2.4.1 of the 2006 Ontario Building Code (or 2.17.4.1 of the 1997 Ontario Building Code)

FIRM NAME	BCIN/BCDN
I & F Engineering Corp.	29493

Orientation

N

Stamp

LICENSED PROFESSIONAL ENGINEER
I. FINKELSHTEYN
10.31.2007
PROVINCE OF ONTARIO

Newton Bridge Solutions Ltd.

41 MASSEY ROAD
GUELPH, ONTARIO
N1H 7M6

TEL. 519-341-8944
FAX. 519-822-6159

I & F ENGINEERING CORP
CONSULTING ENGINEERS & MANAGERS

100 DRUMLIN CIRCLE, UNIT 203
CONCORD, ON, CANADA

(905) 669-8445
FAX: (905) 669-4803

Client:

MUNICIPALITY OF HURON SHORES

Project:

DEAN LAKE BRIDGE
MTO SITE 385-234

Drawing Title:

DECK PANEL D3c

Drawn by: V.R.	Date: 10.10.2007
Checked by: M.L.	Scale: 1:20
Project Nr. 07M05	Drawing Nr. S12

CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY
SIZE 1178x841

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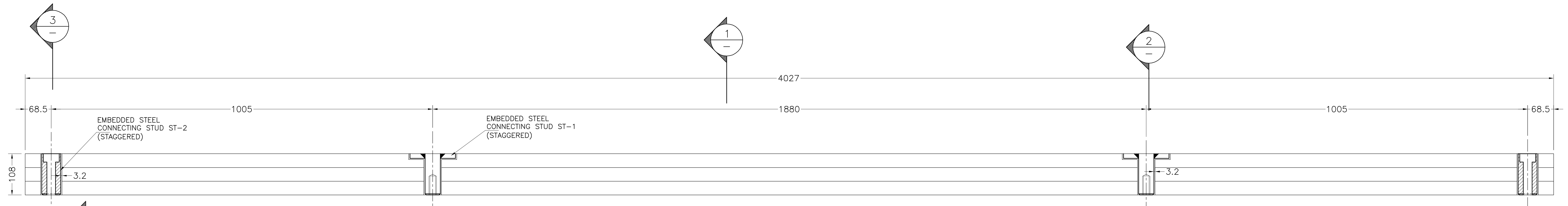
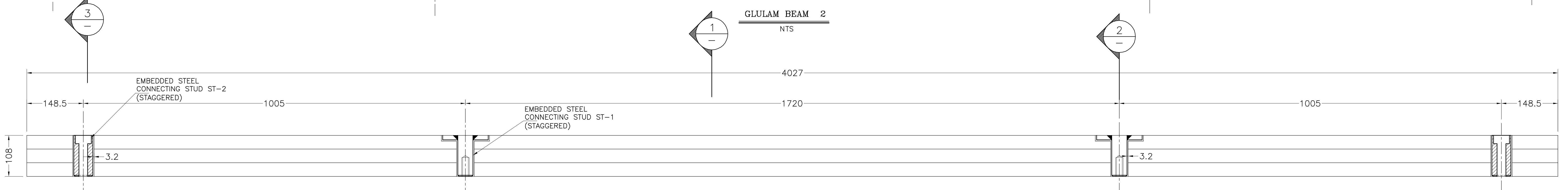


GLULAM BEAM 2

NTS



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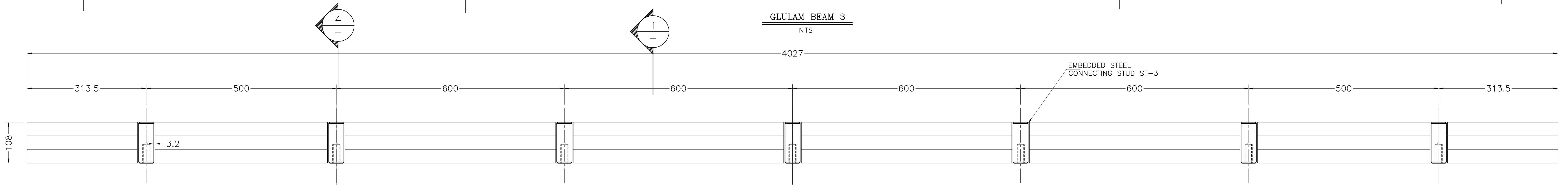

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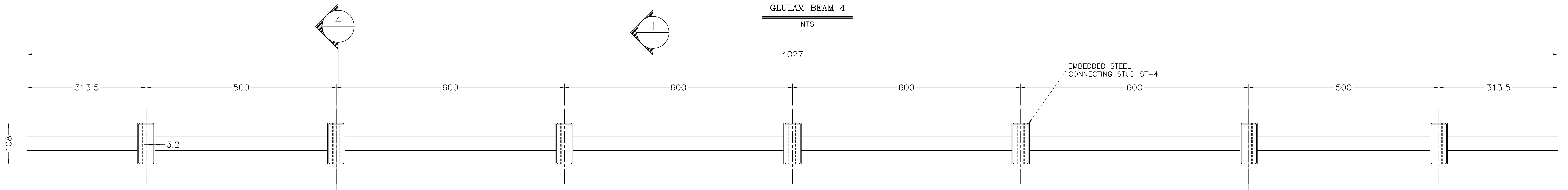

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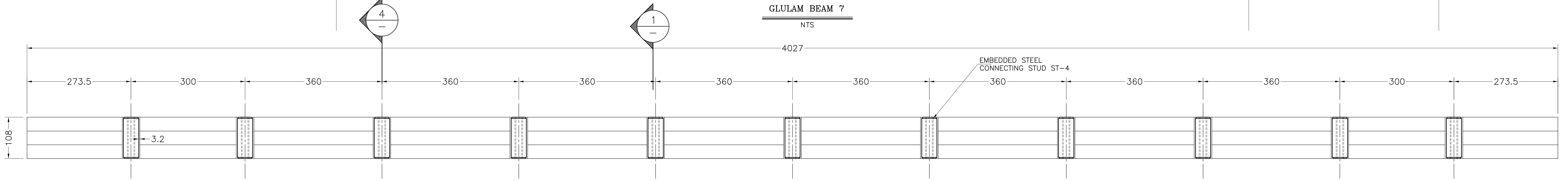
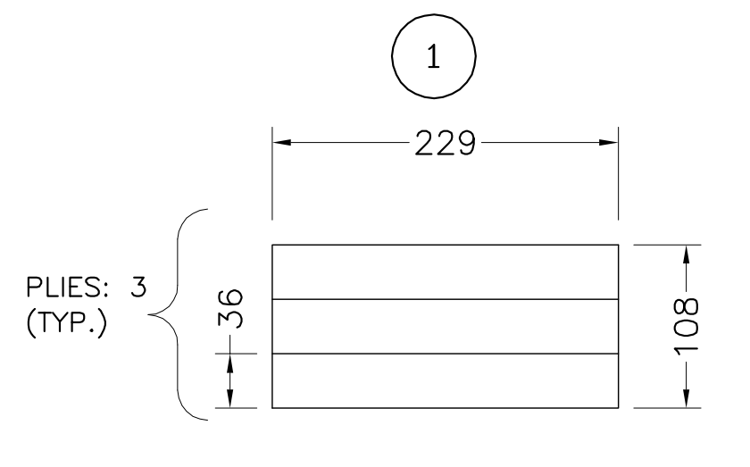


GLULAM BEAM 5, 6, 8

NTS



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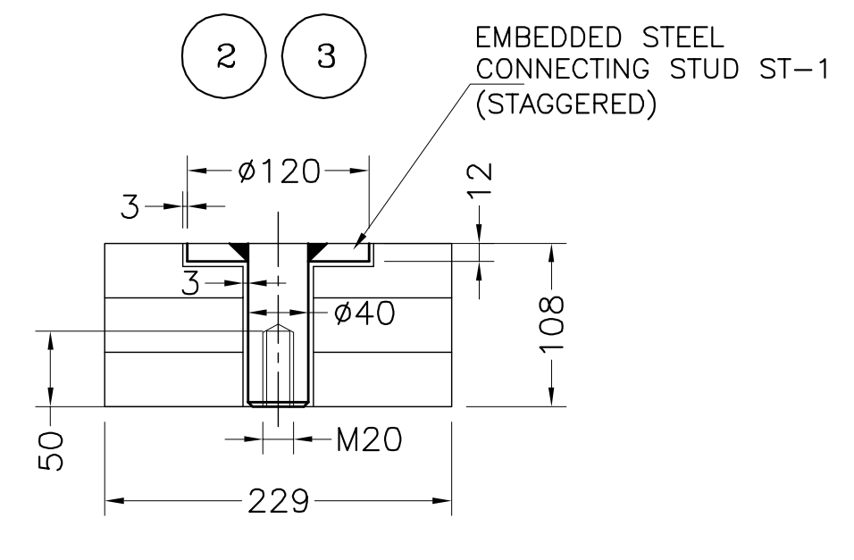

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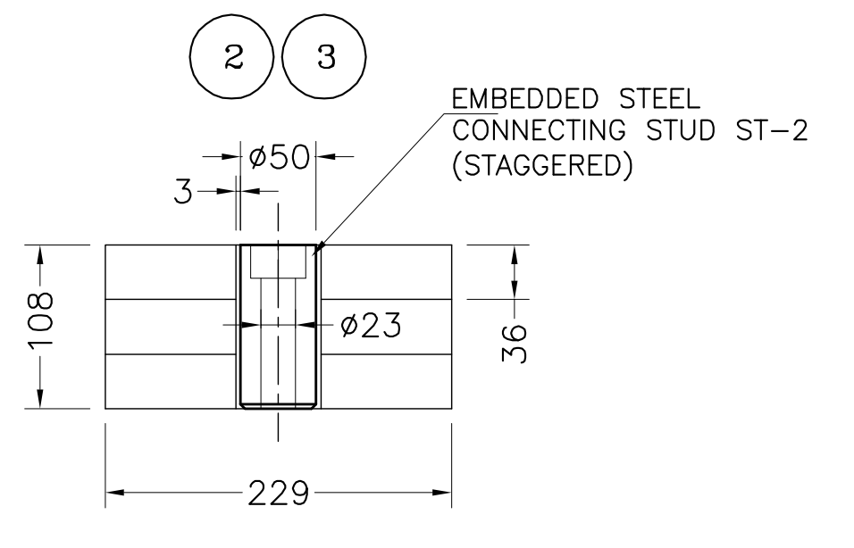

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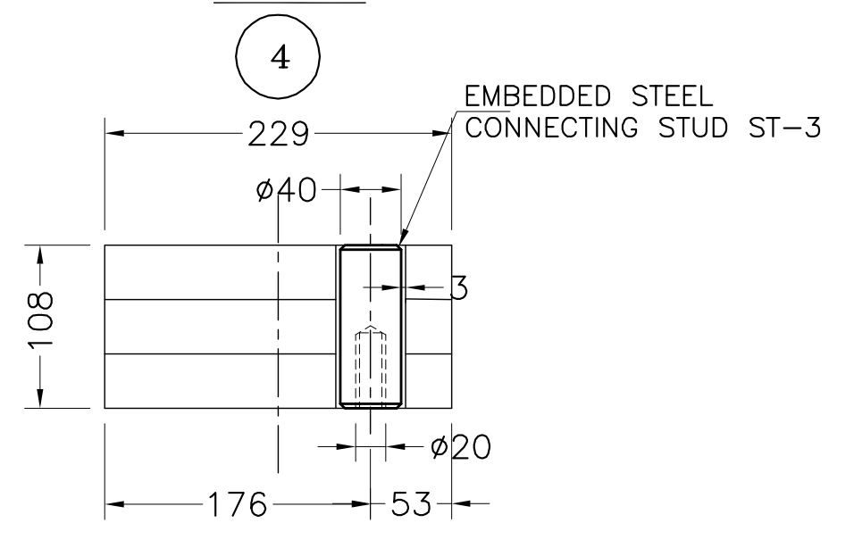

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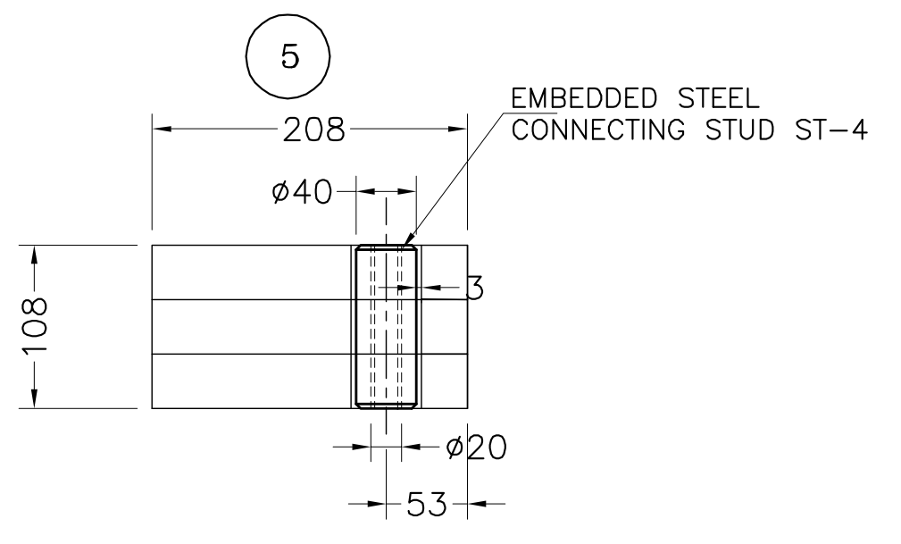

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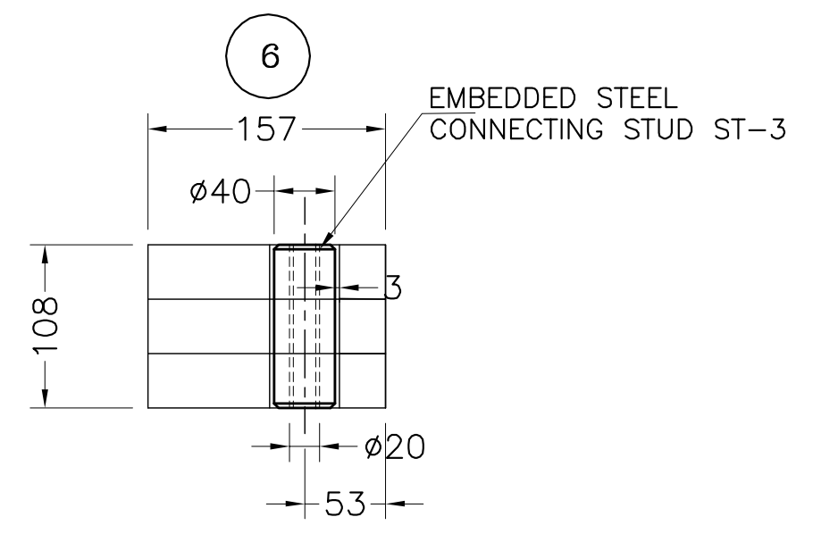

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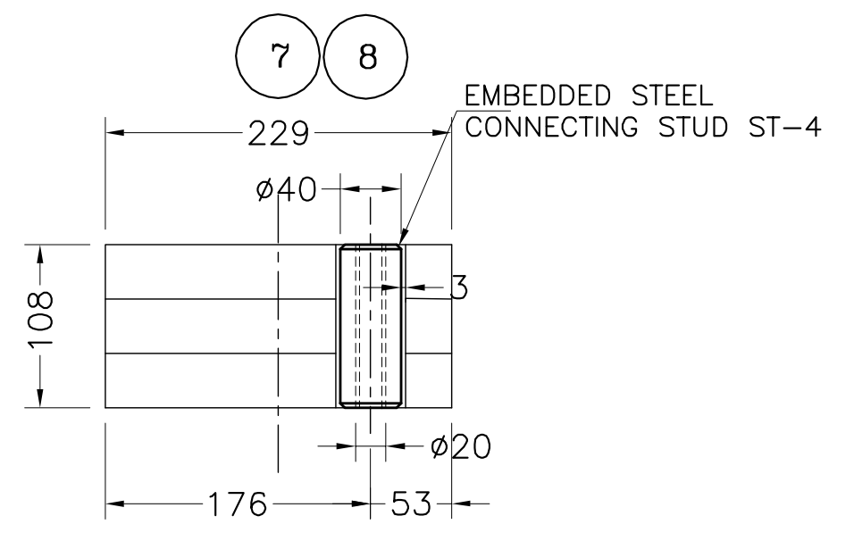

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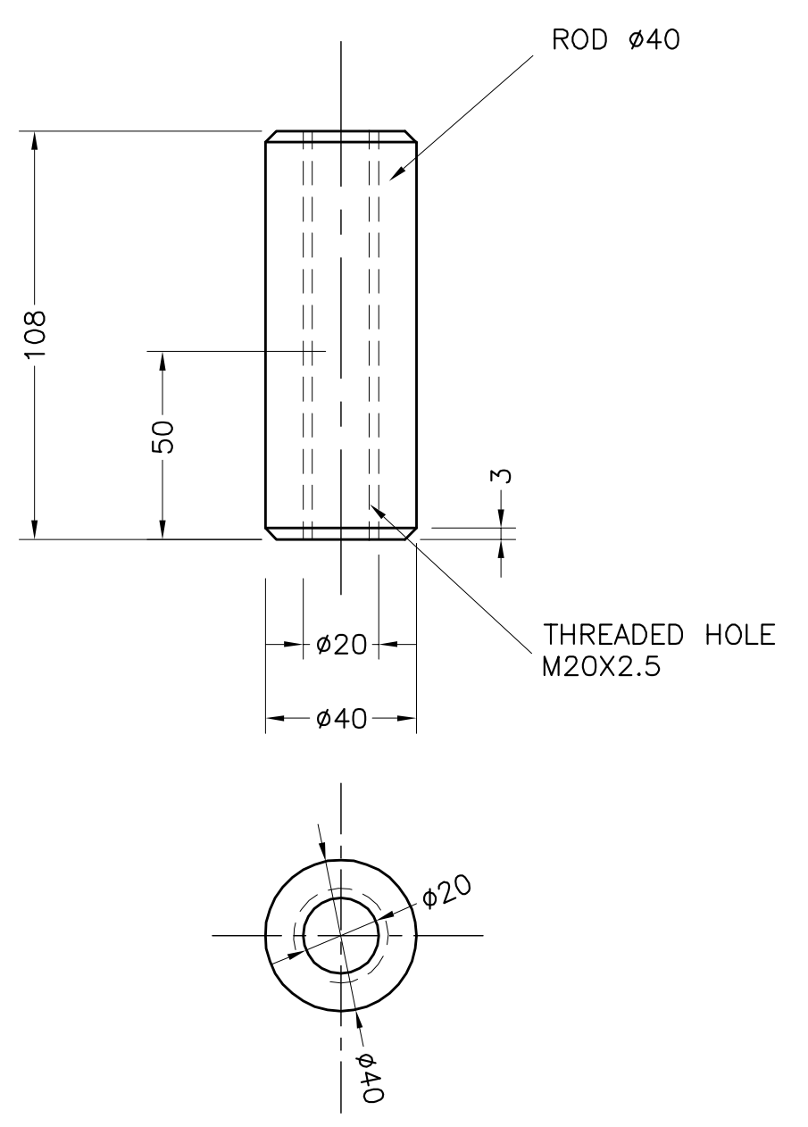

STEEL CONNECTING STUD ST-4

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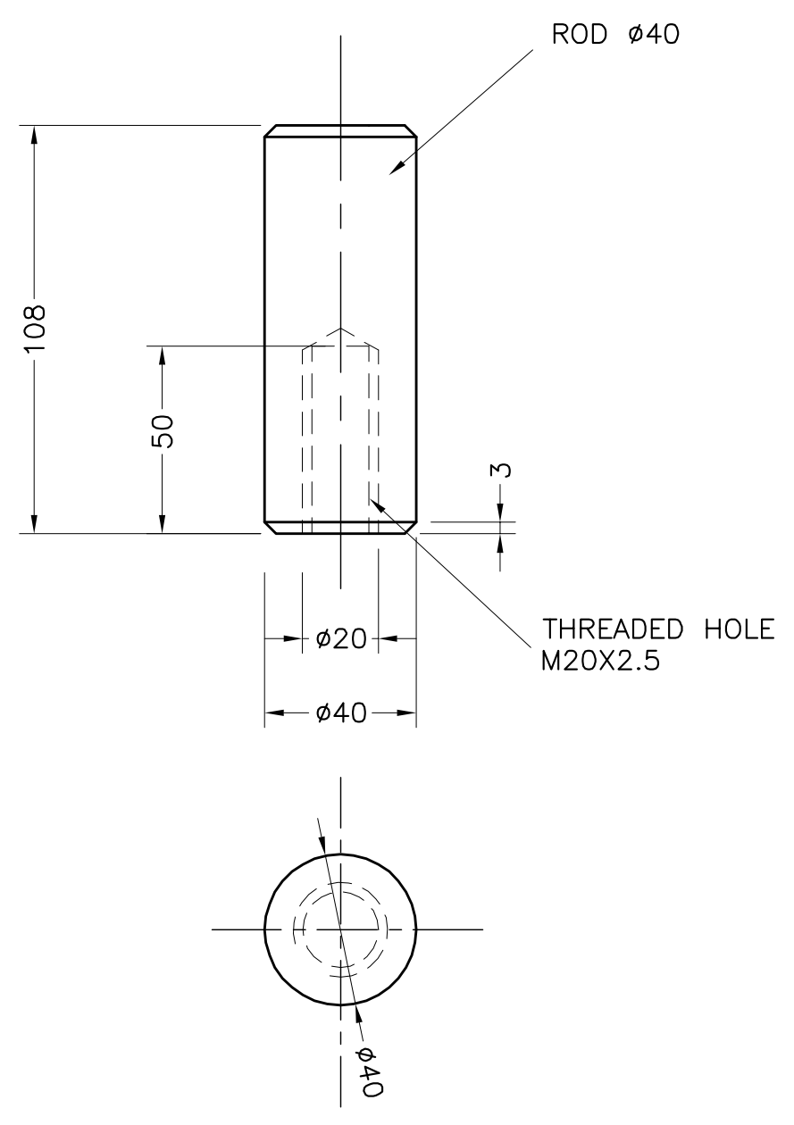

STEEL CONNECTING STUD ST-3

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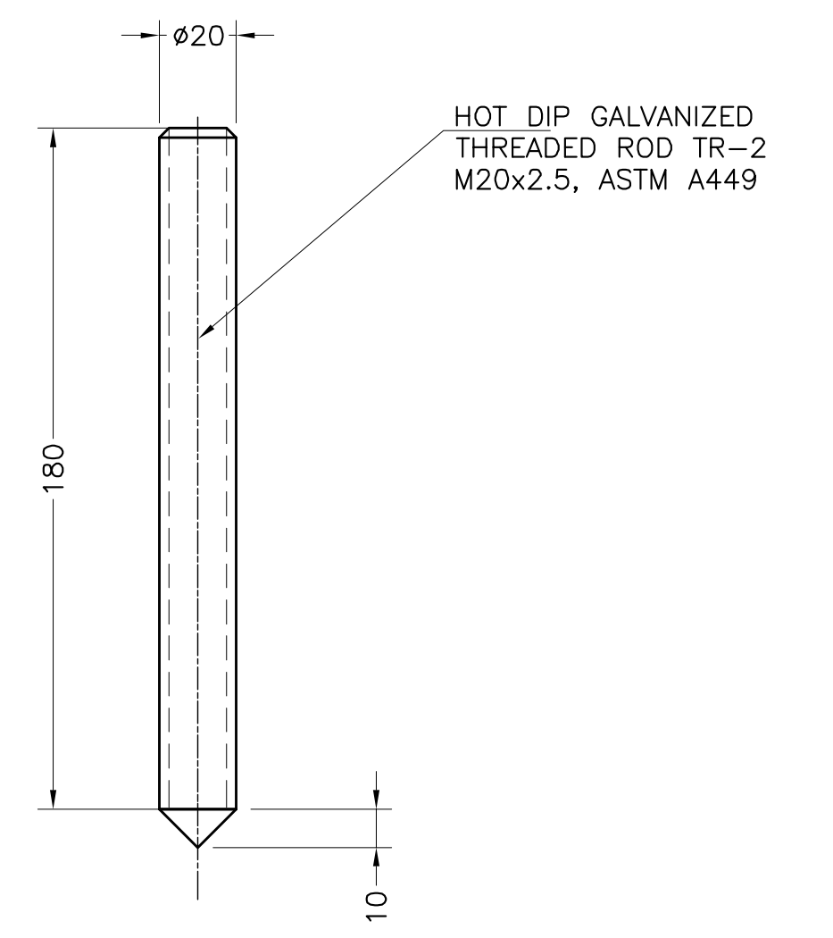

THREADED ROD TR-2

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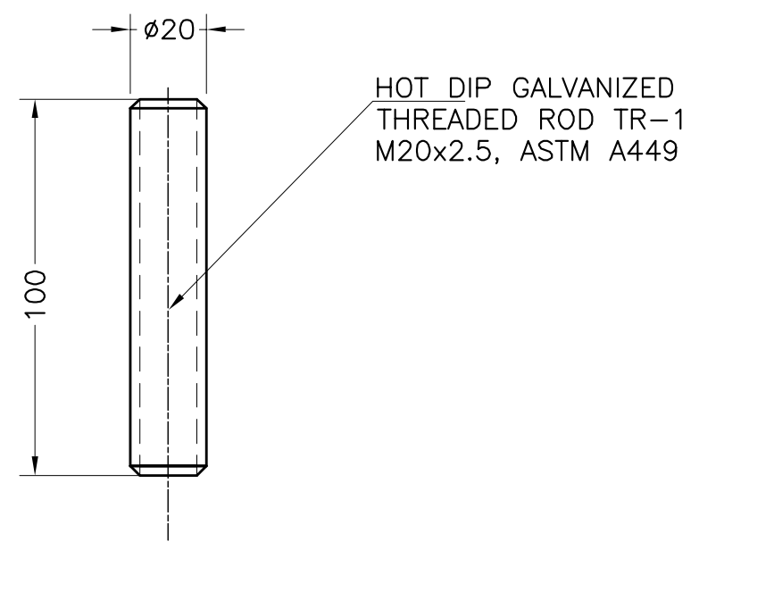

THREADED ROD TR-1

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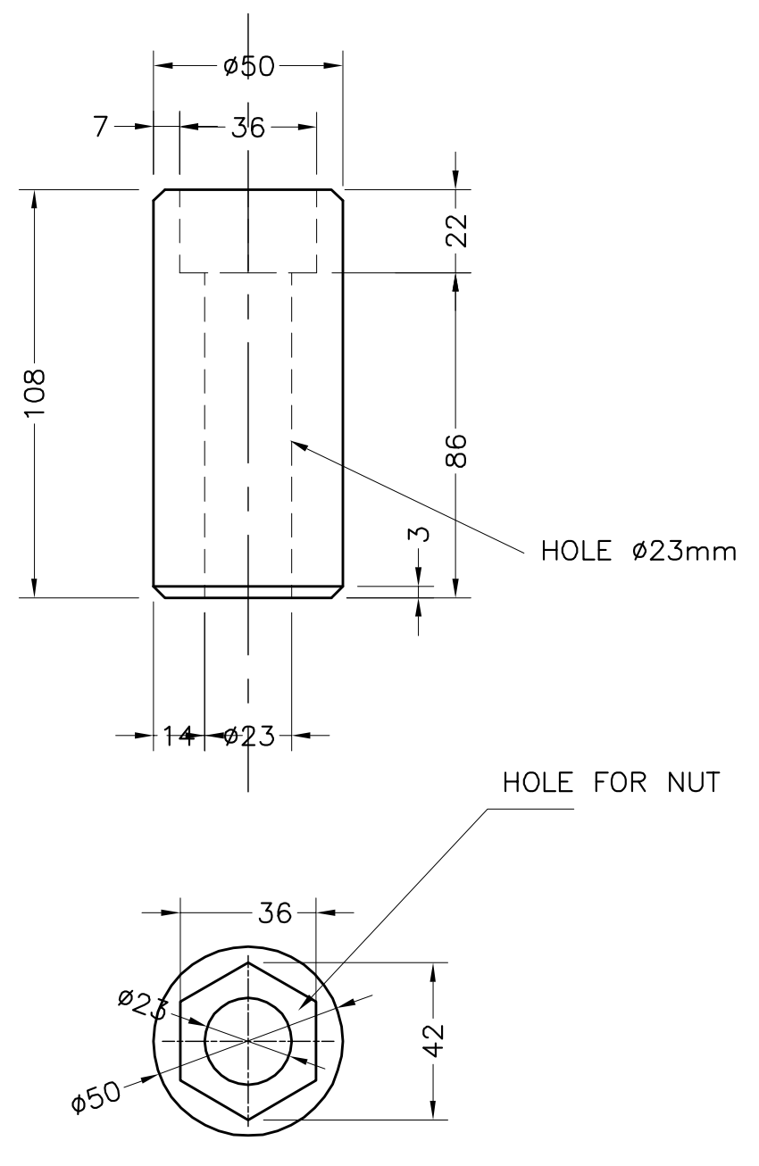

STEEL CONNECTING STUD ST-2

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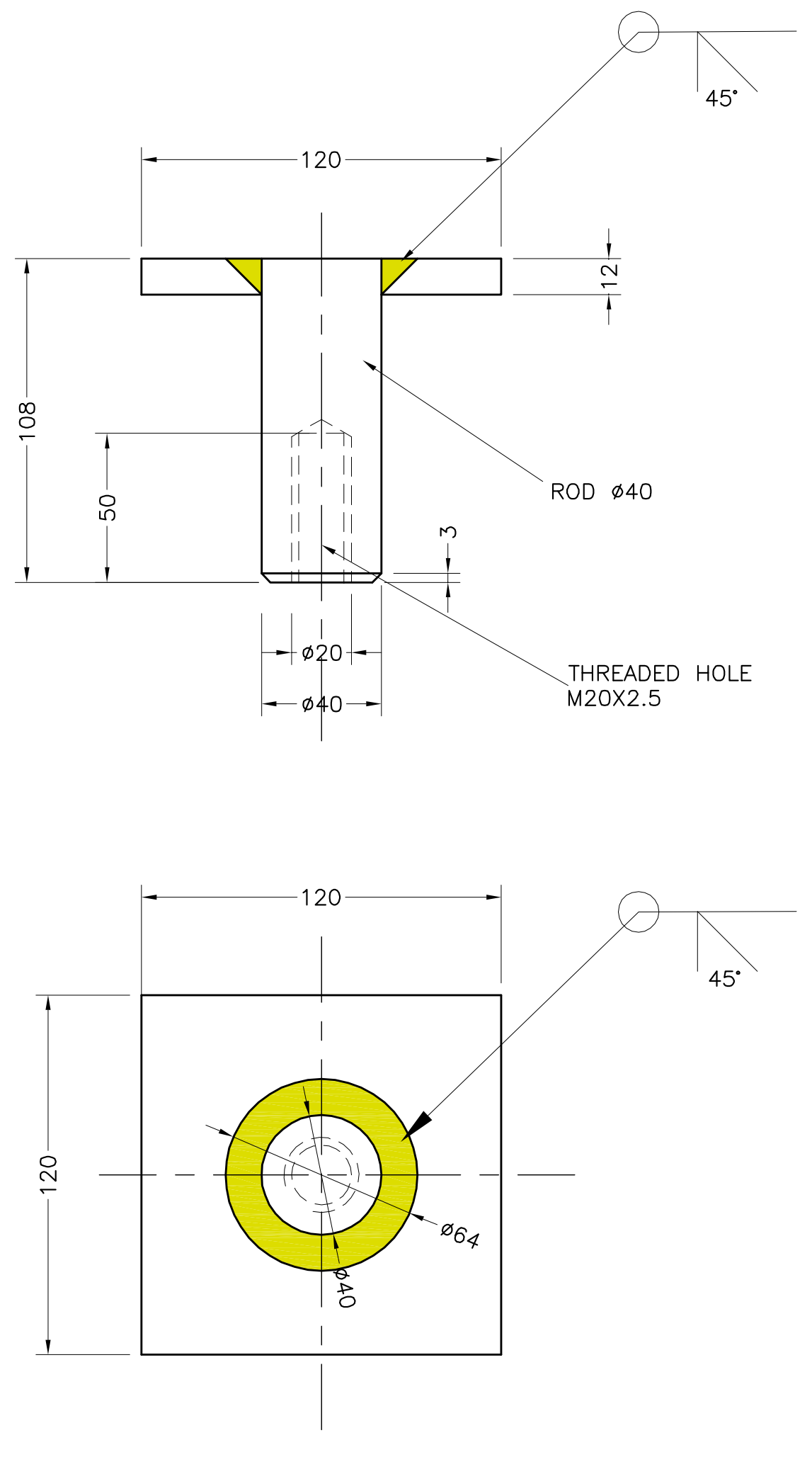

STEEL CONNECTING STUD ST-1

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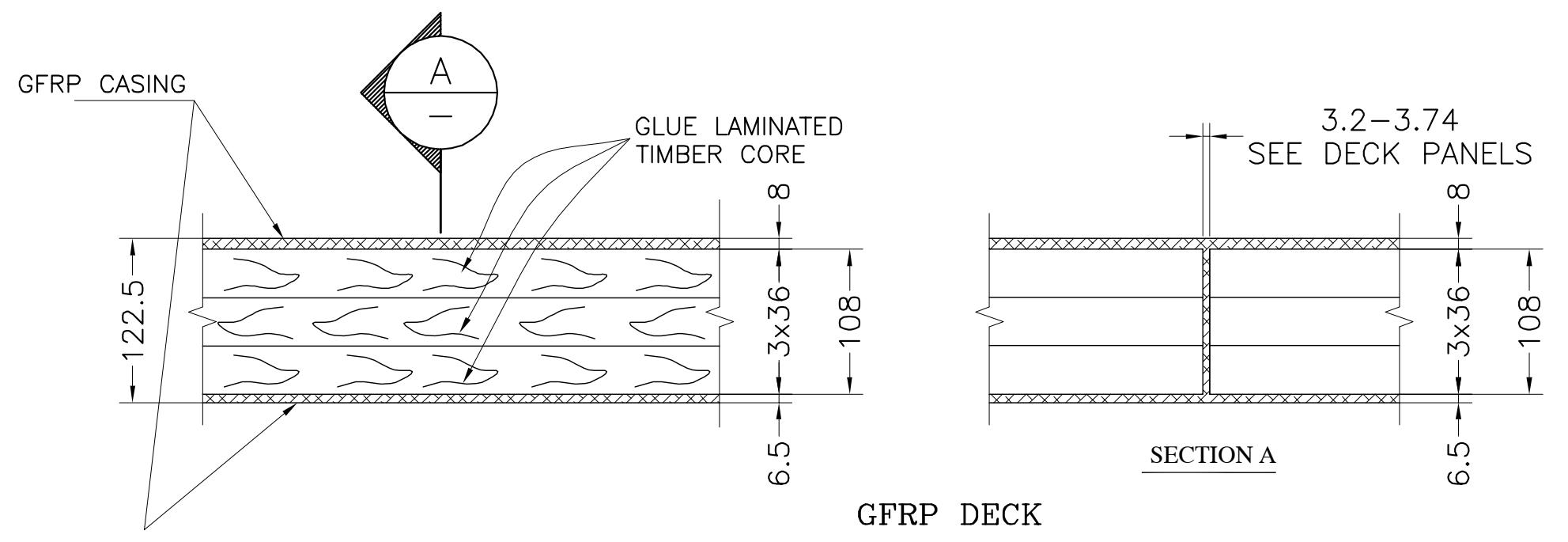

GFRP DECK
TYPICAL SECTION

NTS

THIS DRAWING, AS AN INSTRUMENT OF SERVICE, IS PROVIDED BY AND IS THE PROPERTY OF I & F ENGINEERING CORP.
THE CONTRACTOR MUST VERIFY AND ACCEPT RESPONSIBILITY FOR ALL DIMENSIONS AND CONDITIONS ON SITE AND MUST NOTIFY I & F ENGINEERING CORP OF ANY VARIATIONS FROM THE SUPPLIED INFORMATION.
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CONSTRUCTION MUST CONFORM TO ALL APPLICABLE CODES AND REQUIREMENTS OF AUTHORITIES HAVING JURISDICTION.
BEFORE PROCEEDING WITH EACH NEW PHASE OF THE CONSTRUCTION PROCEDURE, THE CONTRACTOR SHALL OBTAIN APPROVAL FROM THE INSPECTOR AND/OR CONSULTANT.
THE CONTRACTOR WORKING FROM DRAWINGS NOT SPECIFICALLY MARKED "FOR CONSTRUCTION" MUST ASSUME FULL RESPONSIBILITY AND BEAR COSTS FOR ANY CORRECTIONS OR DAMAGES RESULTING FROM HIS WORK.

NO.	DATE:	REVISION	CH'D
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NAME	SIGNATURE	BCIN/BCDN
Lyakhovsky M.		29491

REGISTRATION INFORMATION

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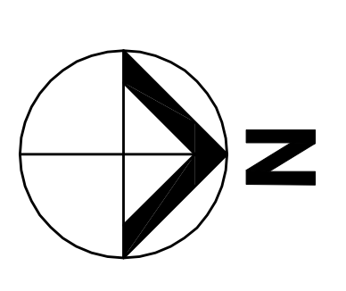
FIRM NAME	BCIN/BCDN
I & F Engineering Corp.	29493

Orientation



Stamp

LICENSED PROFESSIONAL ENGINEER
I. FINKELSHTEYN
10.31.2007
PROVINCE OF ONTARIO

Newton Bridge Solutions Ltd.

41 MASSEY ROAD
GUELPH, ONTARIO
N1H 7M6

TEL. 519-341-8944
FAX. 519-822-6159

I & F ENGINEERING CORP

CONSULTING ENGINEERS & MANAGERS

100 DRUMLIN CIRCLE, UNIT 203
CONCORD, ON, CANADA

(905) 669-8445
FAX: (905) 669-4803

Client:

MUNICIPALITY OF HURON SHORES

Project:

DEAN LAKE BRIDGE
MTO SITE 385-234

Drawing Title:

DECK DETAILS

Drawn by: V.R.	Date: 10.10.2007
Checked by: M.L.	Scale: NTS
Project Nr. 07M05	Drawing Nr. S13

CAD FILE \DWG\
LAST UPDATED: YEAR/MONTH/DAY

SIZE 1178x841

APPENDIX B: Steel Testing Results

ACUREN

Acuren Group Inc.
2180 Speers Road
Oakville, ON, Canada L6L 2X8
www.acuren.com

Phone: 905.825.8595
Toll Free: 877.299.2857

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CERT #3977.01 & 3977.04

Client	
TULLOCH ENGINEERING INC. 200 Main Street Thessalon, Ontario P0R 1L0	# Laboratory Report

Attention	*Client's Order Number*	*Date*	*Report Number*
Matt Kirby	Pay by Credit Card	Mar. 4, 2024	128-23-TUL001-J110160 Rev. 00

Client's Material /Product Description	*Date Sample Received*	*Material / Product Specification*
12 steel strip samples with paint on, each 2-1/4" wide x 10" long x thickness 3 pcs labeled E4, E5 and E6 3 pcs labeled SE1, SE2 and SE3 3 pcs labeled SW1, SW2, W3 3 pcs labeled W4, W5 and W6	Dec. 15, 2023	-----

1. Chemical Analysis

(By: OES, ASTM E415-21) (% by Weight)

Element	Result	Result	Result
	E6	SE2	W3
Fe	98.90	98.90	98.90
C	0.27	0.27	0.24
Si	0.14	0.14	0.14
Mn	0.44	0.44	0.44
P	0.054	0.054	0.052
S	0.053	0.051	0.050
Cr	0.01	0.01	0.01
Mo	< 0.0020	< 0.0020	< 0.0020
Ni	0.01	0.01	0.01
Al	0.006	0.006	0.003
Co	0.01	0.01	0.01
Cu	0.06	0.06	0.06
Nb	0.003	0.003	0.003
Ti	< 0.0010	< 0.0010	< 0.0010
V	< 0.00050	< 0.00050	< 0.00050
CE (1)	0.34	0.35	0.32

Note 1: The carbon equivalent value calculated per ASTM A6/A6M-23 as follows:
CE= C + Mn/6 + (Cr + Mo + V)/5 + (Ni + Cu)/15

2. Chemical Analysis*

(ASTM E1019-18, ASTM E1097-12(Reapproved 2017) (Modified), ASTM E1479-16) (% by Weight)

Element	W5	E5
Fe	98.7	98.7
C	0.046(2)	<0.01
Si	0.17	0.20
Mn	0.20	0.15
P	0.21(3)	0.18(3)
S	0.039(4)	0.034
Cr	<0.01	0.01
Mo	<0.01	<0.01
Ni	0.03	0.05
Al	-	-
Co	-	-
Cu	-	-
Nb	-	-
Ti	-	-
V	-	-
Mg	-	-
CE (1)	0.08	0.03

Note 1: The carbon equivalent value calculated per ASTM A6/A6M-23 as follows:
CE= C + Mn/6 + (Cr + Mo + V)/5 + (Ni + Cu)/15

Note 2: The Carbon result of W5 sample is approximate and is the average of 5 results (0.050 wt.%, 0.036 wt.%, 0.036 wt.%, 0.048 wt.%, 0.058 wt.%).

Note 3: The Phosphorus values of W5 and E5 samples are approximate due to lying outside analytical range.

Note 4: The Sulphur result of W5 sample is approximate and is the average of 5 results (0.037 wt.%, 0.033 wt.%, 0.033 wt.%, 0.042 wt.%, 0.048 wt.%).

3. Tensile Test

(ASTM A370-23)

- One tensile specimen taken in lengthwise direction from each sample

Tensile Property	Result		
	E4	E5	E6
Dimensions of Specimen Reduced Section (in)	0.506(w) x 0.296(t)	0.506(w) x 0.310(t)	0.503(w) x 0.266(t)
Ultimate Tensile Strength (psi)	56 000	46 000	72 500
Yield Strength (Offset = 0.2%) (psi)	37 400	33 400	45 200
Elongation (in 2”-Manual Method) (%)	19	10	29

Tensile Property	Result		
	SE1	SE2	SE3
Dimensions of Specimen Reduced Section (in)	0.506(w) x 0.293(t)	0.506(w) x 0.267(t)	0.506(w) x 0.261(t)
Ultimate Tensile Strength (psi)	49 400	70 500	72 500
Yield Strength (Offset = 0.2%) (psi)	38 500	45 300	47 900
Elongation (in 2”-Manual Method) (%)	14	28	26

Tensile Property	Result		
	SW1	SW2	W3
Dimensions of Specimen Reduced Section (in)	0.500(w) x 0.302(t)	0.502(w) x 0.264(t)	0.502(w) x 0.274(t)
Ultimate Tensile Strength (psi)	54 000	69 000	70 500
Yield Strength (Offset = 0.2%) (psi)	37 200	44 900	43 800
Elongation (in 2”-Manual Method) (%)	26	30	33

Tensile Property	Result		
	W4	W5	W6
Dimensions of Specimen Reduced Section (in)	0.505(w) x 0.278(t)	0.507(w) x 0.300(t)	0.507(w) x 0.302(t)
Ultimate Tensile Strength (psi)	70 000	44 300	50 500
Yield Strength (Offset = 0.2%) (psi)	44 200	35 400	36 600
Elongation (in 2”-Manual Method) (%)	30	9.0	20

Weldability statement: Note that carbon equivalence is only a qualitative assessment of potential welding problems and should never be solely relied on to ensure weld integrity.
All tested materials are weldable, however in addition to consideration of carbon equivalent values, and in order to ensure weld integrity, the proper use of welding specifications, coupled with the knowledge of actual construction conditions must also be used.

Feng Yan

Feng Yan, Metallurgist,
Test Specialist, Mechanical Testing

Seyed Mousavi

Seyed Mousavi, M.Eng
Chemical Lab

Client acknowledges receipt and custody of the report or other work (“Deliverable”). Client agrees that it is responsible for assuring that acceptance standards, specifications and criteria in the Deliverable and Statement of Work (“SOW”) are correct. Client acknowledges that Acuren is providing the Deliverable according to the SOW, and not any other standards. Client acknowledges that it is responsible for the failure of any items inspected to meet standards, and for remediation. Client has 15 business days following the date Acuren provides the Deliverable to inspect it, identify deficiencies in writing, and provide written rejection, or else the Deliverable will be deemed accepted. The Deliverable and other services provided by Acuren are governed by a Master Services Agreement (“MSA”). If the parties have not entered into an MSA, then the Deliverable and services are governed by the SOW and the “Acuren Standard Service Terms” (www.acuren.com/serviceterms) in effect when the services were ordered

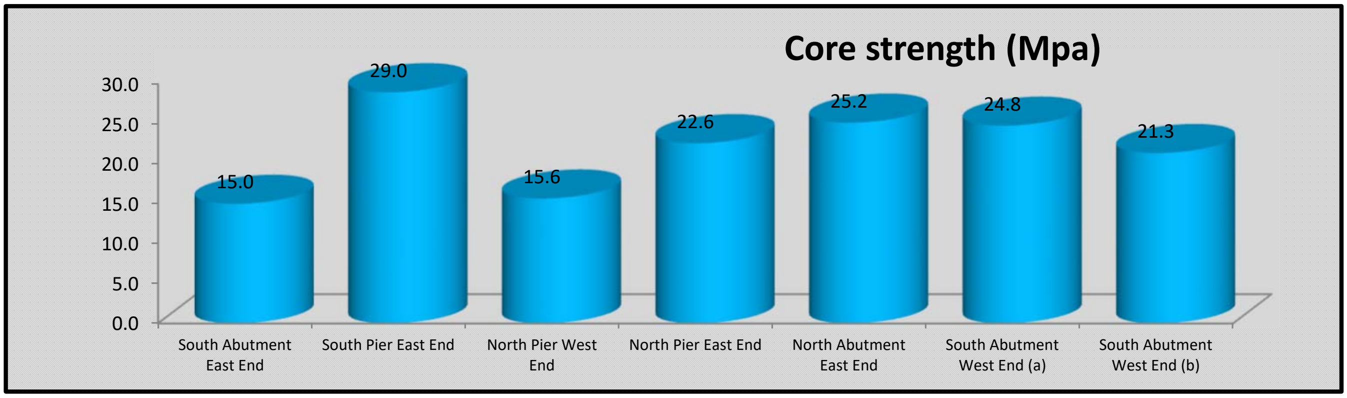
The Client Representative who receives this report is responsible for verifying that any acceptance standards listed in the report are correct, and promptly notifying Acuren of any issues with this report and/or the work summarized herein. The owner is responsible for notifying Acuren in writing if they would like their samples returned or placed into storage (at their cost) otherwise, all samples/specimens associated with this report will be disposed of 60 days after the report date.
NOTES:
A) Any tests subcontracted to an approved subcontractor are highlighted above (*)
B) Results in this report relate only to the item(s) tested as provided by the client unless otherwise indicated.
C) This report shall not be reproduced except in full without the prior written approval of Acuren Group Inc.

APPENDIX C: Concrete Compressive Strength Results

Concrete Core Compressive Strength Report

PROJECT: Municipality of Huron Shores 2023 Biennial Bridge Inspections - Phase 201 Dean Lake Bridge Steel Repairs
CONTRACT: 23-0862
DATE SAMPLED: November 29, 2023
RUN BY: H. Logan
DATE TESTED: December 11, 2023
SOURCE: Concrete Cores

Sample ID	Distance from top of run (cm)	Height (mm)	Diameter (mm)	L/D Ratio	Correction Factor	Peak Load (lbs)	Compressive Stength (Mpa)
South Abutment East End	N/A	94.20	93.60	1.0	0.872	23160	15.0
South Pier East End	N/A	98.20	93.60	1.0	0.885	44800	29.0
North Pier West End	N/A	93.60	93.60	1.0	0.870	24190	15.6
North Pier East End	N/A	96.90	93.60	1.0	0.881	34920	22.6
North Abutment East End	N/A	96.30	93.60	1.0	0.879	38910	25.2
South Abutment West End (a)	N/A	126.10	93.60	1.3	0.944	38320	24.8
South Abutment West End (b)	N/A	100.30	93.60	1.1	0.892	33010	21.3



REMARKS: South Pier West End and North Abutment West End samples could not be tested due to a lack of material that meets the 1:1 minimum L/D ratio requirement

CLIENT: Municipality of Huron Shores

APPENDIX D: Sampling Location Drawing

Municipality of Huron Shores

Dean Lake Bridge

Steel Coupon Samples and Concrete Core Locations

DEAN LAKE ROAD,
IRON BRIDGE, ON.

Municipality of Huron Shores
7 Bridge Street
P.O. Box 460
Iron Bridge, ON
P0R 1H0
ph (705) 843-2033

PRIME CONSULTANT
TULLOCH ENGINEERING INC.
MATT KIRBY, PEng.
71 BLACK ROAD, UNIT
#8 SAULT STE. MARIE,
ON P6B 0A3
ph (705) 949-1457
matt.kirby@tulloch.ca

PROJECT No. 230862-201

PROJECT DRAWING LIST

STRUCTURAL

DRAWING No.	DESCRIPTION	REV.
S1	SITE PLAN	0
S2	PLAN AND ELEVATION	0

TULLOCH
ISSUED FOR
REPORT
APRIL 12, 2024

SITE PLAN

SCALE: 1/128" = 1'-0"

PROJECT:

Muncipality of Huron Shores

Dean Lake Bridge - Iron Bridge, ON.

Steel Coupon Samples and Coring Locations

DRAWING:

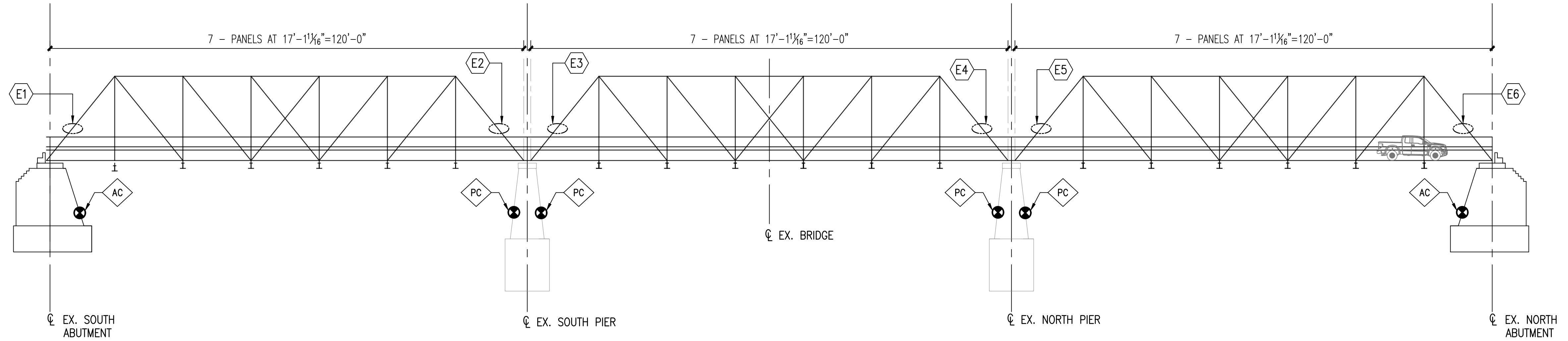
SITE PLAN

No.	DATE	ISSUES / REVISIONS	BY
0	APR. 12, 2024	ISSUED FOR REPORT	BWJ

DRAWN BY: BWJ	CHECKED BY: SR	PROJECT No.: 230862-201
DESIGNED BY: MK	APPROVED BY: MK	DRAWING No.: S1
SCALE: As Noted	DATE: DEC. 1, 2023	REVISION No.: 0

ENGINEER'S SEAL

TULLOCH

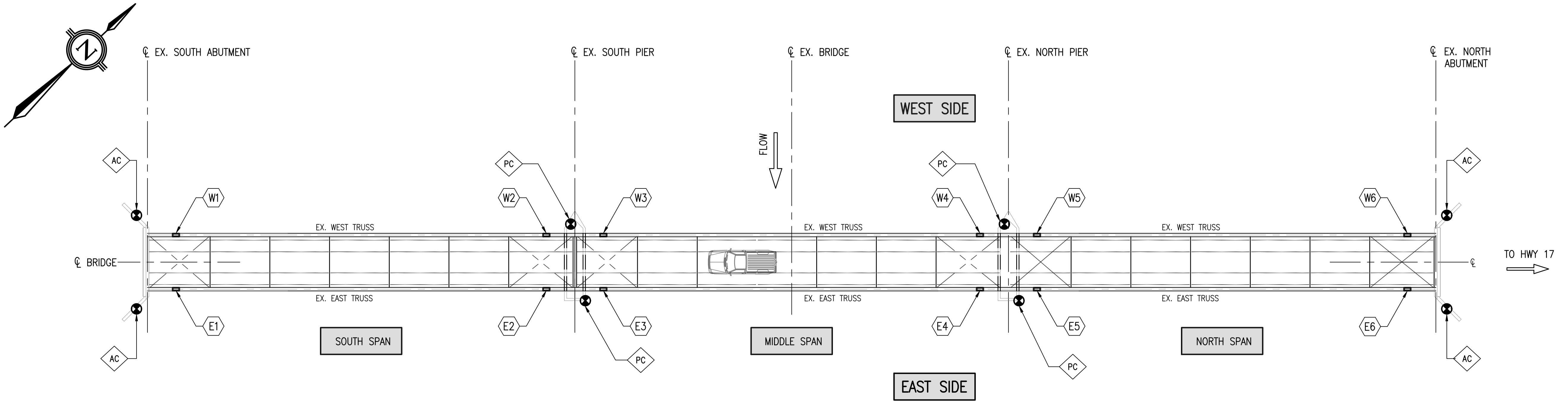


EAST ELEVATION

SCALE: 1/32" = 1'–0"

EAST TRUSS (AS SHOWN)

WEST TRUSS (OPPOSITE HAND)



PLAN

SCALE: 1/32" = 1'–0"

LEGEND

⬡ STEEL BATTEN PLATE REMOVAL FOR TESTING.
NUMBERS W1 THROUGH W6 FROM SOUTH TO NORTH.
SAMPLES TAKEN FROM EACH TRUSS FOR A TOTAL OF 12 SAMPLES.
W1 = WEST TRUSS SAMPLE 1
E3 = EAST TRUSS SAMPLE 3

AC ABUTMENT CORES
TAKEN ON THE CORNERS OF THE WINGWALLS AT THE ABUTMENT FACE.

PC PIER CORES
TAKEN ON THE SOUTH SIDE AT THE WEST ENDS OF THE TRIANGULAR NOSING.
THE EAST END OF THE PIERS WERE CORED ON THE NORTH SIDE

No.	DATE	BY	ISSUES / REVISIONS
0	APR. 12, 2024	BWJ	ISSUED FOR REPORT

DRAWN BY: BWJ
CHECKED BY: SR
PROJECT No. : 230862-201
DESIGNED BY: MK
APPROVED BY: MK
DRAWING No. S2
REVISION No. 0
SCALE: As Noted
DATE: DEC. 1, 2023

ENGINEER'S SEAL

PROJECT:
Muncipality of Huron Shores
Dean Lake Bridge- Iron Bridge, ON
Steel Coupon Samples and Coring Locations

DRAWING:
PLAN AND ELEVATION

TULLOCH

APPENDIX E: Bridge Surveying Information from M.F. Tulloch Surveying (comparing 1989, 1996 & 2004)

TULLOCH

MF TULLOCH INC.
200 Main St.
PO Box 579
Thessalon, ON
P0R 1L0

T. 705 842.3372
F. 705 842.2658
800 797.2997

www.tulloch.on.ca.
tulloch@tulloch.on.ca.

04-083
September 28, 2004

The Municipality of Huron Shores
P.O. Box 460
Iron Bridge, ON P0R 1H0

Attention: Tom Dumont, Road Superintendent

Dear Tom:

Re: Monitoring Program Dean Lake Bridge

Further to your request we have completed a check of the monitoring points on the Dean Lake Bridge. The survey was completed on August 19th and August 24th 2004. The control monuments that were used from the last report completed in 1996 were used to tie in the monitoring points. Each point was tied in from both ends of the bridge and the results averaged. You will note that when reviewing results that two point numbers 6001 and 7007 were missing and two others 7004 and 7008 were paved over. Therefore there are no comparisons to be made for these points. I have attached a spreadsheet which summarizes the coordinates and elevations for the points in August 2004 as well as from 1996 and 1989. Comparison is made from August 2004 to June 1989 and from August 2004 to July 1996. Results are shown in metres.

Please note that the measurements cannot be expected to be exact to the millimetre due to a number of factors including environmental, equipment tolerances and care taking during the fieldwork. The measurements could also be affected by possible movement of the control monuments over the years due to the effects of environmental influences such as frost and physical disturbances.

I trust that this information as presented will be sufficient for your purposes. Please call if you need clarification on our survey and calculation methods.

Yours very truly,

Marshall Thompson

Marshall D. Thompson, P.Eng.
TULLOCH ENGINEERING & SURVEYING

MDT/ss

Encl.

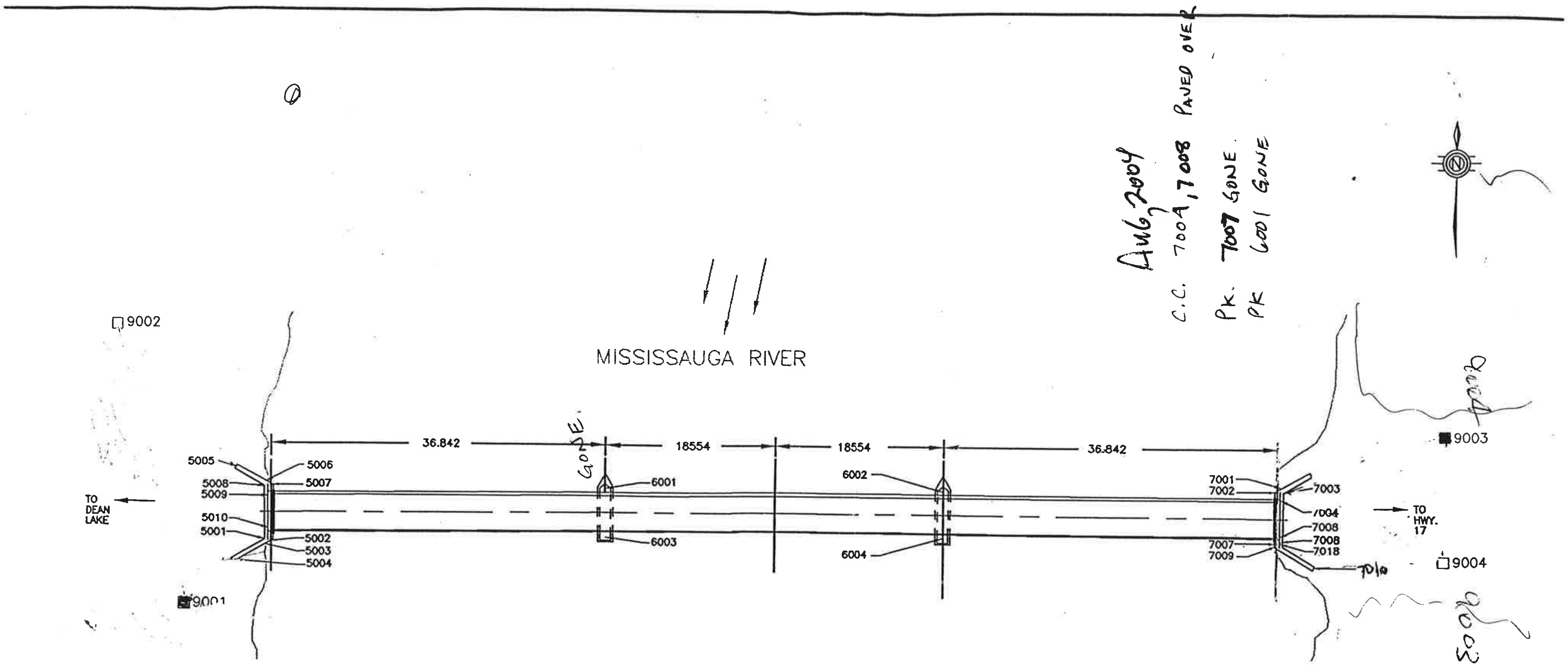
04-083 T.Dumont 092804

DEAN LAKE BRIDGE MONITORING PROGRAM
PROJECT No. 04-083 08/27/04

Point No.	AVERAGE SHOTS - August 2004			Difference (August 04 - July 96)				Difference (August 04 - June 89)			
	North	East	Elevation	North	East	Total	Elevation	North	East	Total	Elevation
9001	5000.000	5000.000	100.000	Control Point				Control Point			
9002	4977.714	4970.451	100.477	Control Point				Control Point			
9003	5182.330	5000.000	100.498	Control Point				Control Point			
9004	5181.359	4990.127	100.546	Control Point				Control Point			
5001	5016.089	4994.529	100.673	-0.006	0.007	0.009	0.014	0.004	-0.001	0.004	0.018
5002	5017.026	4995.006	99.897	0.002	0.015	0.015	0.021	-0.015	-0.021	0.026	-0.002
5003	5016.521	4995.336	99.918	-0.004	0.004	0.005	0.017	-0.020	-0.034	0.039	-0.008
5004	5014.268	4996.585	99.914	0.000	0.014	0.014	0.021	0.005	0.006	0.007	0.020
5005	5014.183	4986.084	99.879	0.007	0.002	0.008	0.004	0.004	0.004	0.005	0.027
5006	5016.667	4987.377	99.960	0.009	-0.003	0.009	0.007	0.008	0.003	0.008	0.031
5007	5017.181	4987.699	99.941	0.009	0.000	0.009	0.008	-0.006	-0.028	0.029	0.002
5008	5016.218	4988.112	100.676	-0.009	-0.002	0.009	0.006	0.001	0.004	0.004	0.028
5009	5016.204	4989.450	100.670	-0.001	-0.001	0.001	0.003	-0.002	0.006	0.006	0.026
5010	5016.153	4993.245	100.659	0.000	-0.002	0.002	0.007	0.004	0.002	0.004	0.019
6001	MISSING										
6002	5090.452	4990.176	100.283	0.008	0.010	0.013	-0.006	-0.001	0.007	0.007	-0.001
6003	5053.214	4994.770	100.200	0.002	0.008	0.008	0.018	0.011	0.015	0.018	0.017
6004	5090.303	4995.755	100.289	-0.005	0.021	0.021	0.019	0.002	0.037	0.037	0.027
7001	5127.102	4989.974	99.861	0.006	0.010	0.011	0.004	-0.006	0.010	0.012	0.001
7002	5126.580	4990.298	99.857	-0.002	0.014	0.014	0.005	-0.006	0.006	0.008	0.006
7003	5127.746	4990.650	100.690	0.003	0.015	0.015	0.001	-0.002	0.015	0.015	-0.003
7004	PAVED OVER										
7007	MISSING										
7008	PAVED OVER										
7009	5127.009	4997.272	100.174	0.003	0.034	0.034	0.025	0.015	0.037	0.040	0.030
7010	5129.453	4999.199	99.881	0.000	0.033	0.033	0.025	0.006	0.039	0.039	0.029
7018	5126.525	4996.920	100.169	0.007	0.025	0.026	0.023	0.014	0.032	0.034	0.031

DEAN LAKE BRIDGE MONITORING PROGRAM
PROJECT No. 96071 08/27/04

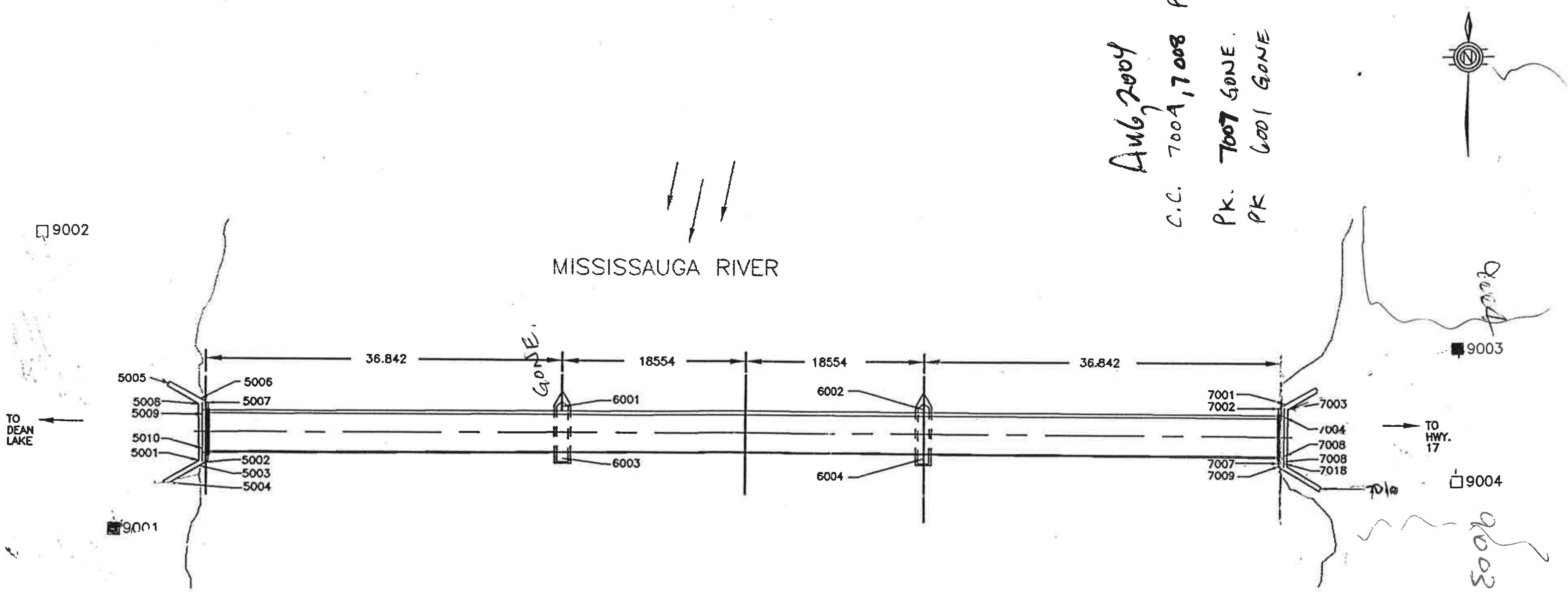
Point No.	AVERAGE SHOTS - July 1996			Difference (July 96 - June 89)				ADJUSTED SHOTS - June 1989		
	North	East	Elevation	North	East	Total	Elevation	North	East	Elevation
9001	5000.000	5000.000	100.000	Control Point				5000.000	5000.000	
9002	4977.714	4970.451	100.477	Control Point				4977.714	4970.451	
9003	5182.330	5000.000	100.498	Control Point				5182.330	5000.000	
9004	5181.359	4990.127	100.546	Control Point				5181.359	4990.127	
5001	5016.095	4994.522	100.659	0.010	-0.008	0.013	0.004	5016.085	4994.530	100.655
5002	5017.024	4994.991	99.877	-0.017	-0.036	0.040	-0.023	5017.041	4995.027	99.899
5003	5016.525	4995.333	99.901	-0.016	-0.037	0.041	-0.025	5016.541	4995.370	99.926
5004	5014.268	4996.571	99.893	0.005	-0.008	0.010	-0.001	5014.263	4996.579	99.894
5005	5014.176	4986.082	99.876	-0.003	0.002	0.004	0.023	5014.179	4986.080	99.852
5006	5016.658	4987.380	99.953	-0.001	0.006	0.006	0.024	5016.659	4987.374	99.929
5007	5017.172	4987.700	99.933	-0.015	-0.028	0.031	-0.007	5017.187	4987.727	99.939
5008	5016.226	4988.114	100.670	0.009	0.006	0.011	0.022	5016.217	4988.108	100.648
5009	5016.205	4989.451	100.668	-0.001	0.007	0.007	0.023	5016.206	4989.444	100.644
5010	5016.153	4993.247	100.653	0.004	0.004	0.006	0.013	5016.149	4993.243	100.640
6001	5053.436	4989.344	100.185	-0.011	-0.012	0.016	0.001	5053.447	4989.356	100.183
6002	5090.445	4990.167	100.289	-0.009	-0.003	0.009	0.005	5090.453	4990.169	100.284
6003	5053.213	4994.762	100.182	0.009	0.007	0.012	-0.001	5053.203	4994.755	100.183
6004	5090.308	4995.734	100.270	0.006	0.016	0.017	0.008	5090.301	4995.718	100.262
7001	5127.096	4989.964	99.857	-0.012	0.000	0.012	-0.003	5127.108	4989.964	99.860
7002	5126.582	4990.284	99.853	-0.004	-0.008	0.009	0.001	5126.586	4990.292	99.851
7003	5127.743	4990.635	100.689	-0.005	0.000	0.005	-0.004	5127.748	4990.635	100.693
7004	5127.483	4991.853	100.669	-0.012	-0.016	0.020	-0.008	5127.495	4991.869	100.677
7007	5127.405	4995.664	100.634	-0.007	-0.007	0.010	-0.024	5127.412	4995.671	100.658
7008	5126.321	4996.949	99.877	0.012	0.010	0.016	0.009	5126.308	4996.939	99.867
7009	5127.006	4997.238	100.149	0.012	0.003	0.012	0.005	5126.994	4997.235	100.144
7010	5129.453	4999.166	99.856	0.005	0.006	0.008	0.004	5129.447	4999.160	99.852
7018	5126.518	4996.895	100.146	0.007	0.007	0.010	0.007	5126.511	4996.888	100.138



Aug, 2004
C.C. 700A, 7008 PAVED OVER
PK. 7007 GONE.
PK 6001 GONE

PLAN

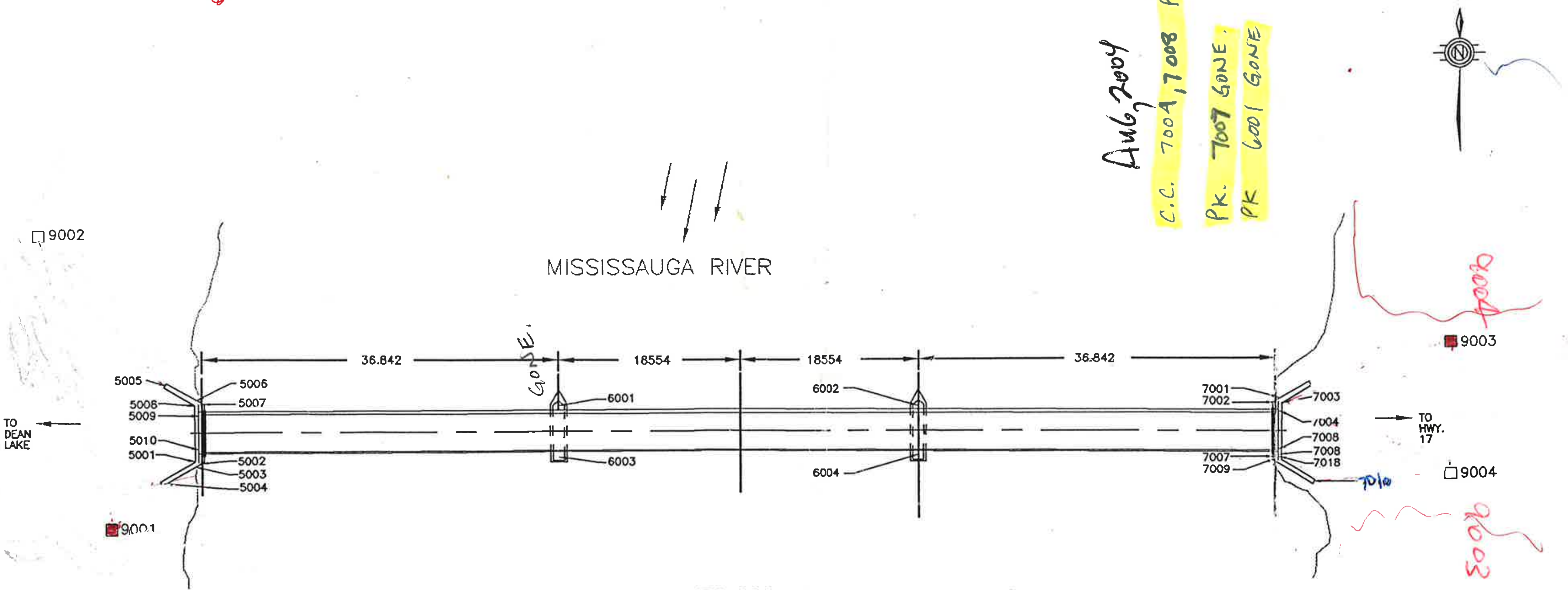
1:500



Aug, 2004
C.C. 700A, 7008 PAVED OVER
PK. 7007 GONE.
PK 6001 GONE

PLAN

1:500



Aug, 2004

C.C. 7004, 7008 PAVED OVER

PK. 7007 GONE.

PK 6001 GONE

PLAN

1:500