

INNOVATIVE PRECAST CONCRETE DECK SLABS

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ABSTRACT

The cost, traffic accommodation, schedule, construction and other considerations involved in selecting deck replacement material and type for a 550m long two lane suspension bridge in Alberta, are described. Traffic has to be maintained on at least one lane, without interruption. Cast in place concrete, precast concrete, partial precast concrete, "Exodermic" panels and steel plate sandwich (SPS) panels were evaluated.

An innovative design of full depth, precast prestressed concrete deck slab panels for use in new as well as replacement bridge decks, is described. The precast panels are prestressed by pretensioning in the transverse and longitudinal directions. These panels have soffits in the form of elliptical paraboloid and contain prestressing only at the ribs.

The design, fabrication, erection and construction considerations involved in using these panels as deck replacement for this suspension bridge, are described.

1. INTRODUCTION

Alberta Infrastructure and Transportation has engaged Associated Engineering to design and supervise construction of deck replacement and other work on the Dunvegan Bridge, BF 73949, Sec. 2:68, over the Peace River on Highway 2. This paper describes the feasible alternatives for the deck replacement and compares them on the basis of various functional and cost parameters.

The alternatives evaluated are:

1. Partial depth precast concrete panels
2. Full depth precast concrete panels
3. Cast-in-place concrete
4. Sandwich Plate System (SPS)
5. Exodermic deck panels, either precast or cast-in-place
6. Orthotropic steel deck
7. FRP deck

With the exception of fully cast-in-place concrete, for all alternatives it is assumed that the deck replacement will be panel by panel, that is, as a panel of concrete is removed, a new panel will be placed. The following traffic accommodation strategies will be considered with the decreasing order of preference:

1. One lane open to traffic with occasional closure of the bridge for up to 45 min.
2. Closure of bridge at night, say from about 10 p.m. to 6 a.m.
3. Closure of bridge for all traffic, except for emergency vehicles.

The user cost will be considered for all of the alternatives (it will add about 100 km on alternative route). More than 15% of vehicles are trucks (log haul etc.). Average traffic AADT is 2530 and ASDT is 2610.

Among the seven systems to be evaluated, the FRP deck is not considered here because it does not have the fire resistance and the unit weight is too dissimilar to the existing deck. The orthotropic deck is not considered further, for similar reasons.

2. DECK REPLACEMENT – GENERAL CONSIDERATIONS

The existing bridge deck plan at suspended and approach spans are shown in Figure 1. The floor stringers in the suspended spans have adequate capacity as a non-composite section to carry the required loads. However, it is desirable to have a degree of composite action to reduce the deflection under live loads and to hold the deck down under dynamic loads. Existing concrete deck contains 3C6 (3 inch high channel sections weighing 6lb/ft x 8 inch long) channel shear connectors spaced at 4ft 8 ½

inches (that is 3 per 18'-9" span). No additional shear connectors are necessary, however, shear connectors can be added as necessary to obtain fully composite action.

The stringers/girders at the approach spans are 916mm (36 inches) deep and are composite in the positive moment regions of the interior girders of the cantilevered spans. The approach span girders are structurally arranged as balanced cantilevers. These girders do not have adequate load capacity for CL-800 loading, and, therefore, should be made continuous by moment splices at hinge locations and made composite in all positive moment regions of all girders.

It is expected that the demolition of the existing deck slab could be performed at the rate of about 105 to 320m²/shift for an average of 210 to 640m²/shift (i.e. 4 stringer spans of full bridge width or 8 stringer spans of half width). The total bridge deck area is 6750m² (5070m² of suspended span and 1780m² of approach span). Therefore, the demolition will take about 34 shifts to 66 shifts.

In order to reduce the construction loading and traffic on the bridge, a gantry system could be set up which would travel on rails supported by the stiffening trusses. The demolished concrete and reinforcement could be carried to the ends of the bridge, where they could be stockpiled for recycling.

Originally the bridge deck was cast in 75ft sections (i.e. 4 panels/stringer spans). The deck replacement construction schedule will be controlled by the demolition, regardless of the type of replacement deck system.

The deck replacement system should have dead load similar to the original design deck loading (6 ½ inches or 165mm thick deck slabs plus 50mm overlay). The stringers, floor trusses, cables, hangers, etc. have adequate live load capacity, but the stiffening trusses do not. The lane load portion of CL800 has 9kN/m uniform load per lane, which is equivalent to about 80mm of concrete or ACP. However, while the reduction of dead load (i.e. lowering of the weight of the deck slab) will increase the live load capacity of the approach spans, it may not be beneficial at the suspended span. With the reduction in the uniform dead load, the deflection of the suspension cables will be greater due to live load, especially due to concentrated live loads covering parts of the span. For example, if one half of the middle suspended span is loaded by traffic load, the extreme deflections will occur at the quarter points of the span, downward in the loaded half and upwards in the unloaded half. The distortion of the cable is caused by the variation in the funicular curve; therefore, a reduced live load to dead load ratio will reduce the change of shape, thus reduce stresses in the stiffening truss.

3. DECK REPLACEMENT ALTERNATIVES

Five deck replacement alternatives are being considered and are described below. The feasibility of each alternative, including previous experience with the deck system, maintenance and repair requirements, the advantages and disadvantages of the design, cost estimate, compatibility with de-icing and fire resistance systems and impact on traffic accommodation during construction are discussed in Section 4.

Alternative 1 – Partial Depth Precast Concrete Panels

The new deck will consist of partial depth precast concrete panels with a cast-in-place concrete topping and a 2-course ACP wearing surface on an asphaltic waterproofing system. The new deck will be connected to the existing stringers by headed studs which will be bolt connected to the stringers. The studs will be grouped together and located through holes in the precast panels and be cast into the cast-in-place topping. Temporary spring clips with shim washers will be used for initial placement of the panels. In addition, levelling bolts passing through inserts in the precast panels and resting on the middle of the stringers directly above the web will be used for levelling the panels. The panels will be supported by a mortar bed on the top flange of the stringers.

Two alternative types of partial depth panels have been considered and are as follows:

Alternative 1a – Precast Concrete Flat Panel

The panel thickness shall be approximately 90mm and the cast-in-place portion approximately 85mm, for a total depth of 175mm as required by CSA S6-00. Each panel shall be 5715mm in length, which is the span of the stringers; and shall be half the width of the deck (4677mm) (see Figure 2).

The precast portion of the deck is pre-tensioned and the cast-in-place topping is reinforced with one layer of reinforcing in the longitudinal and transverse directions.

Alternative 1b – Partial Depth Precast Shell Panels

The concept is similar to alternative 1a, except that the shell soffit panel will give greater stiffness to the structure than will the flat panel.

The precast shell panels shall be 5715mm long and half the width of the deck (4677mm). Each panel is formed of four (4) elliptical paraboloids joined together with transverse and longitudinal stiffening ribs (see Figure 3). The elliptical paraboloids have a minimum thickness of 50mm and are thicker at the rib locations.

The elliptical paraboloids work by arching action, therefore there is no need for reinforcement in either the precast shell panels. Prestressed reinforcement is provided in the ribs.

Alternative 2 – Full Depth Precast Concrete Panels

The new deck will consist of full depth precast concrete panels with a 2-course ACP wearing surface on an asphaltic waterproofing system. The panels will extend to the full width of the bridge deck, with no longitudinal joints.

The new deck will be connected to the existing stringers by headed studs which will be bolt connected to the stringers. The studs will be located in holes in the precast panels which will be filled with epoxy grout. Erection, levelling and panel support are similar to that for the partial depth precast panels.

Three alternative types of full depth panels have been considered and are as follows:

Alternative 2a – Solid Slab Deck Panel

The panel thickness shall be approximately 190mm as required by CSA S6-00. Each panel is the full width of the deck in accordance with CSA S6-00. Each panel will be 2857.5 mm long, which is half the stringer span (see Figure 4).

The precast panels shall be reinforced with two (2) layers of reinforcement in the longitudinal and transverse directions. The most preferable and acceptable system now requires transverse pretensioning and longitudinal post-tensioning of at least 1.7MPa.

Alternative 2b – Waffle Slab Deck Panel

The concept is similar to alternative 2a, except that the waffle slab will have a reduced weight when compared to the solid flat slab. The waffle slab consists of transverse and longitudinal ribs spaced at approximately 600mm on centre forming a grid pattern. The ribs have a depth of 250mm while the portion between the ribs has a depth of only 75mm (see Figure 5).

The waffle slabs are transversely pre-tensioned and longitudinally post-tensioned with strands within each rib, and the concrete will be fibre reinforced.

Alternative 2c – Shell Soffit Deck Panel

The concept is similar to alternative 1b, except that in this case the shell panel is full depth. The precast shell panels shall be 2857.5 mm long and the full width of the deck. Each panel is formed of four (4) elliptical paraboloids joined together with transverse and longitudinal stiffening ribs (see Figure 6). The elliptical paraboloids have a minimum thickness of 100mm and are thicker at the rib locations, varying from 175mm to 250mm at the edges.

The elliptical paraboloids work by arching action, therefore theoretically there is no need for reinforcement, except prestressing at the ribs in both directions, and fibre reinforcing in the concrete.

Alternative 3 – Fully Cast-In-Place Concrete Deck

The new deck will be a fully cast-in-place concrete deck with a 2-course ACP wearing surface on an asphaltic waterproofing system. The new deck will be connected to the existing stringers by existing C-channel shear connectors and new additional shear connectors.

Two alternatives have been considered and are as follows:

Alternative 3a – Cast-in-Place Deck with Formwork Removed

The deck will be approximately 175 mm in depth. The deck will be formed as is typically done for removable formwork (see Figure 7, Section B).

The deck will be reinforced with two (2) layers of reinforcement in the longitudinal and transverse directions.

Alternative 3b – Cast-in-Place Deck with Stay-in-Place Metal Forms

The deck will be approximately 175 mm in depth. The deck will be formed with a stay in place metal decking (see Figure 7, Section C). The deck will be reinforced with two (2) layers of reinforcement in the longitudinal and transverse directions. The details will follow now established practice of the Texas Department of Transportation.

Alternative 4 – Sandwich Panel System

Replace the existing concrete deck with the Sandwich Panel System (SPS) and a 2-course ACP wearing surface on an asphaltic waterproofing system.

The Sandwich Panel System (SPS) is a patented system by Intelligent Engineering (Canada) Limited (IE) and was developed in conjunction with industry partner Elastogran GmbH, a member of the BASF Group. The system consists of two steel plates bonded to a compact polyurethane elastomer core (see “Ultimate Strength of an SPS Bridge – The Shenley Bridge, Quebec” by S.J. Kennedy et al., 2004 Annual Conference of the Transport Association of Canada).

SPS has been used extensively in the construction of new and in the rehabilitation of ships over the past 6 years in many countries including Canada, U.S.A., Germany and the United Kingdom. Intelligent Engineering has approvals from the major ship classification societies and regulatory authorities for the use of SPS in new and rehabilitation of ships. It is desirable to use SPS compared to orthotropic steel decks, since the amount welding and labour required is reduced which reduces costs and the number of fatigue sensitive details.

Although SPS has been used extensively in the ship building industry, up to this point in time it has only been used twice in bridge construction. Once in new construction of the Shenley Bridge in Quebec and

once on an existing rail bridge in Berlin, Germany. The Shenley Bridge is a 22m long single span bridge constructed of 3 steel girders with an SPS deck. The Berlin Rail bridge is a 9m long trial section spanning between two steel girders and replacing the existing steel deck which has fatigue problems. There are major deck replacement projects underway in the United States, which are now in the design phase, and SPS will be used for the rehabilitation of a bridge deck over Massawippi River on Highway 108 in Lennoxville, Quebec. The 43.5m by 12m bridge will use 18 SPS panels. Intelligent Engineering, the developers of the system, have conducted a large number of tests including fire resistance tests which all have positive results. Also, IE have developed their own barrier system which has been crash tested. Therefore, there is an abundance of design information on SPS and the Canadian Bridge Code will be including design provisions for this system.

Alternative 5 – Exodermic™ Bridge Deck System

This alternative includes the Exodermic™ bridge deck and a 2-course ACP wearing surface on an asphaltic waterproofing system. The Exodermic™ bridge deck is a patented system by the D.S. Brown Company (see www.exodermic.com). The Exodermic™ bridge deck is comprised of a reinforced concrete slab on top of, and composite with, an unfilled galvanized steel grid.

The Exodermic™ bridge deck was first used in 1984 in the deck replacement and widening of the Driscoll Bridge, New Jersey. In mid-1990's, the design of the Exodermic™ bridge deck was revised and has been used in many large deck replacement projects in the United States. The system has the flexibility in that the concrete portion of the deck can either be precast or cast-in-place.

4. COMPARISON OF DECK REPLACEMENT ALTERNATIVES

Attached Table 1 shows comparison of all the alternatives for various aspects on a scale of 1 to 5, 5 being the most desirable.

**Table 1
Comparison of Deck Replacement Alternatives**

ALTERNATIVE	1a	1b	2a	2b	2c	3a	3b	4	5
Unit weight, kPa Including 80mm ACP W.S.	6.0	5.6	6.3	5.6	5.6	6.0	6.3	3.8	4.5
Unit Weight, kPa Not including W.S.	4.2	3.8	4.5	3.8	3.8	4.2	4.5	2.0	2.7
Score	5	5	5	5	5	5	5	4	4
Adaptability to variation in site conditions	Med	Med	Low	Med	Med	High	High	High	High
Score	3	4	1	2	3	5	5	5	4
Schedule	Avg	Avg	Avg	Fast	Fast	Slow	Avg	Fast	Avg
Score	3	3	4	4	5	1	2	5	3
Previous Known Performance	3	3	4	3	3	4	2	3	3
Durability	3	4	4	5	5	4	3	3	3
Fire Performance	4	4	5	4	4	4	3	3	2
Economy	4	4	3	4	4	4	5	2	3
Traffic Accommodation	5	5	3	4	4	3	4	5	4
Total	30	32	29	31	33	30	29	30	26

Unit Weight: As described in the “general considerations” above, lower unit weight is not necessarily more desirable. All of the alternatives with full depth concrete have approximately the same dead load. The sandwich Panel System (SPS) is only 53% of the weight of concrete and is more flexible both for live and wind loads.

Adaptability to variation in site conditions: Past problems attributable to partial depth precast panels and full depth precast panels mainly relate to construction details and workmanship at site. In this respect, the panels with shell soffit is better than flat panels because of their greater stiffness. The partial depth precast panels are better than full depth panels because most of the out of tolerance could be compensated by the C.I.P. concrete. Proper matching of contacts and connections are most difficult with the full depth precast panels. Completely C.I.P. concrete and SPS panel systems are the most adaptable to varying site conditions; C.I.P. due to the ability to vary the thickness and other dimensions, SPS due to its flexibility.

Schedule: Based on traffic accommodation with a one lane closure and occasional night time closures in summer, SPS is the fastest. The full depth precast panels will be as fast if a longitudinal joint is introduced at centerline of the bridge (i.e., half width panels instead of full width panels). Partial depth precast panels and exodermic panels are faster than C.I.P. because of the elimination of most formwork and less reinforcing work at site.

Previous Known Performance: C.I.P. concrete on temporary formwork has the longest performance history. However, the results are mixed. The C.I.P. deck on steel girders usually has more shrinkage cracks (especially if they are composite). Problems have been reported by the Transportation Department of the States of Michigan and Florida with the use of partial depth precast panels, but Texas has successfully used them and they are currently used on many bridge deck rehabilitation projects. The reported problems with the partial depth panels are attributable to improper bedding and construction details. There were problems attributable to full depth precast panels as well, mainly relating to the transverse joints, contacts and connections of the various elements. However, most of these problems have been identified and the technology has evolved over the last 30 years so that a number of transportation authorities such as the New England Regional States have standardized their use. The SPS panels do not have a significant history, although it is slated to be used in a major bridge rehabilitation project in Florida, but none on suspension bridges so far.

The Exodermic Deck System has been used for almost 20 years. They are lighter and faster than C.I.P. decks but not crack free (compared to transversely pretensioned and longitudinally post tensioned prestressed concrete panel). The system has the advantage that it could be fully precast/prefabricated or have the C.I.P. placed in the field.

Durability: Properly constructed prestressed concrete deck (prestressed transversely and longitudinally) is the most durable. Therefore, durability of all the concrete systems could be improved by prestressing (the score given in the Table is based on prestressing in both directions for alternatives 2b and 2c only). The presence of reinforcement in the top layer reduces durability (because of the propensity for shrinkage cracking at the top and the effect of chlorides on the steel). Stay-in-place galvanized metal forms, although approved for use and being widely used in a number of states such as Michigan and Texas, have not had good performance history in Alberta. The SPS system relies on the protective membrane and wearing surface for durability of the top steel layer. The Exodermic Deck System has the durability similar to that of partial depth panels.

Fire Performance: Performance of bridge deck under hydro carbon fires is a significant consideration for this bridge. Because of its increased thickness and thermal characteristics, the full depth precast flat panel will be the most resistant. Alternatives 2b and 2c have fibre reinforcement in the concrete. The type of fibres should result in good fire performance and, therefore, steel fibres are not preferred, polypropylene fibres at 0.1 to 0.15% by volume should be used. Fire performance of SPS panels under load is not known, although the conductivity of the elastomer is very low and, therefore, is likely to be at least better than average.

Economy: The conceptual level cost estimates for various systems are as follows.

	1a	1b	2a	2b	2c	3a	3b	4	5
Total for deck systems only, \$M	\$3.0	\$2.7	\$3.5	\$3.0	\$3.0	\$3.0	\$2.5	\$8.0	\$3.5
Cost/m ²	\$445	\$400	\$518	\$445	\$445	\$445	\$370	\$1185	\$518
Total including demolition, \$M	\$4.75	\$4.75	\$5.2	\$4.7	\$4.7	\$4.8	\$4.3	\$9.65	\$5.25

The costs in the first row do not include demolition, time charges, preparation of steel girder surfaces, shear connectors, or any member strengthening. Demolition cost will be approximately \$1M, including cleaning of steel surfaces. The balanced cantilever at approach spans have to be made continuous and shear connectors should be added, at an approximate cost of \$0.5M. Time charges (lane rental) at a rate of 3000/day will vary from about 0.15M for alternate 4 to \$0.3M for alternative 3a.

Traffic Accommodation: Considering that it will take about 66 shifts with night time closure to demolish concrete and erect new full width deck, compared to about 38 with one lane closure, there is no advantage to be gained with full width panels. The cost difference between full width and half width precast panels is negligible and the half width panels have the advantage of eliminating the transverse joint at mid span of the stringers. The saving in total schedule of about 20 days with full bridge closure is not adequate to offset the user costs to the public. Therefore, all the systems should be evaluated based on one lane open, with fewer than about 10 night time closures for partial depth precast option and C.I.P. deck option. With the SPS and Exodermic deck options, the demolition and erection sequence should be adjusted so as to minimize the unequal loading between the upstream and downstream cables. With partial depth precast panels, precast panels placed at two panels thick and moved as the concrete placement progresses, in the suspended spans.

5. EVALUATION AND SELECTION

As seen in Table 1, Alternative 2c, full depth precast panel with shell soffit scores the highest when all the parameters are given equal weight followed closely by Alternatives 1a, 1b, 2b, 3a and 4. On the approach spans, partial depth panels or C.I.P. concrete system or SPS are more preferable because of their ability to accommodate varying cross slopes and plan curvature.

Based on initial cost, the SPS system has been eliminated from further consideration. Based on all the considerations listed above, full depth half width shell soffit precast panels have been selected for the replacement of deck in the suspended span. Because of the superelevation and spiral curves at the approach span, cast-in-place concrete (with formwork removed) has been selected for the portion of the bridge.

6. DESIGN DETAILS OF SHELL SOFFIT PRECAST PANELS

Figures 8 and 9 show the details of the precast panels as designed. The deck replacement contract will be tendered in 2007/2008. The width of these panels are only half of the bridge width in order to replace one lane at a time. The soffit of the cells in each panel has spherical paraboloid profile. Each panel is pretensioned at ribs in the longitudinal and transverse directions.

Prestressing in the longitudinal direction is important to obtain continuity and load distribution by arching action, as determined by the finite element analyses (see Figure 10 a & b). The panels were modeled using SAP2000 Code, using shell elements.

Proper connections of precast panels to the steel stringers and to each other are imperative for successful fabrication, erection and performance of the bridge deck. The through bolt connections illustrated in Figures 8 and 9 involve the least amount of pockets and other stress raisers, yet provide flexibility and adequate tolerance. Panel to panel connections are made at the joints with through bolts, which are post tensioned after all the adjustments are made, but prior to tensioning the bolts connecting the panels to the stringers.



Photo 1



Photo 2



Photo 3

Figure 1 - View from North Approach (Photo 1),
View from South-West Bank (Photo 2), Typical Sections at Deck Underside (Photo 3)

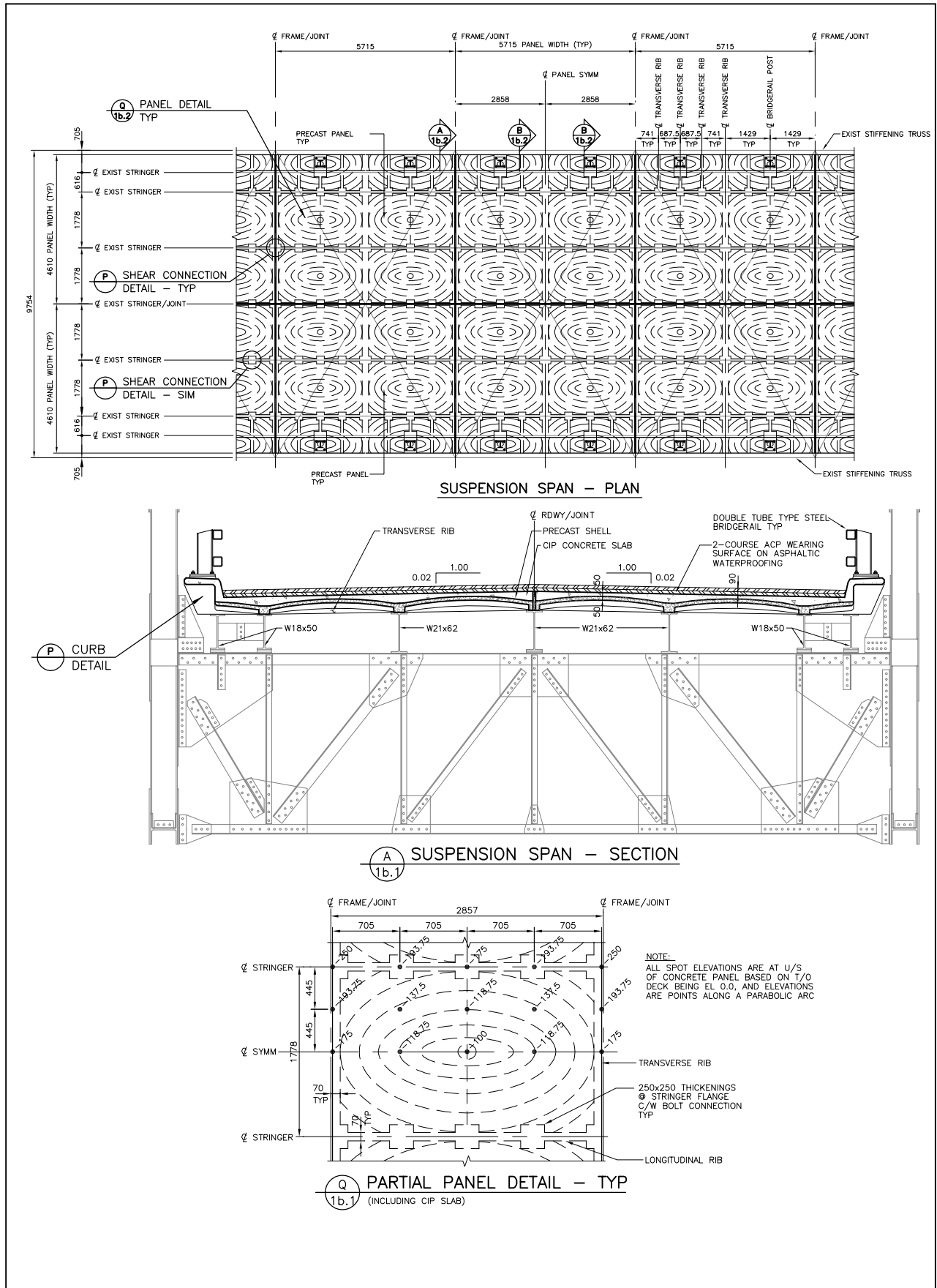


Figure 3

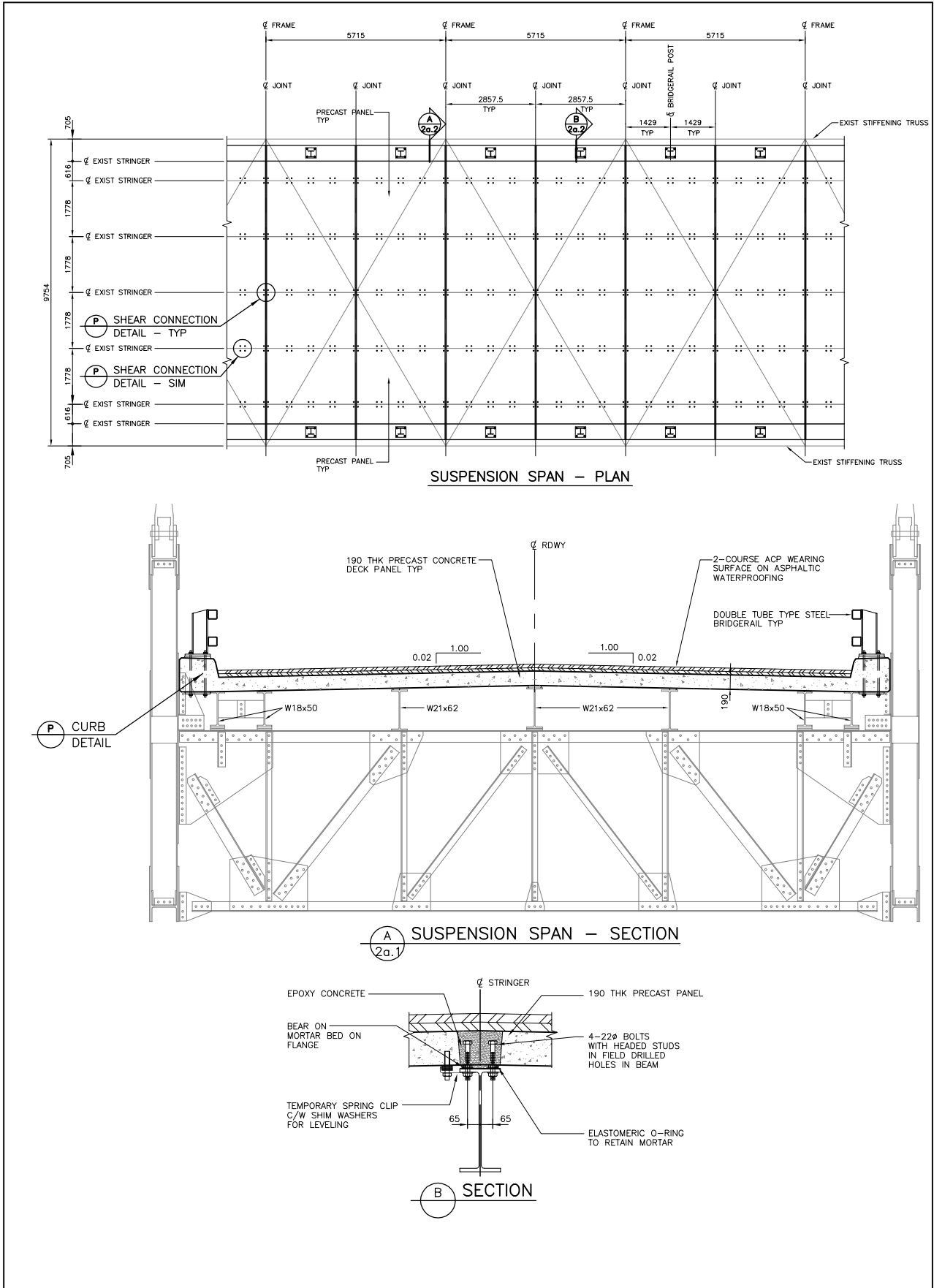


Figure 4

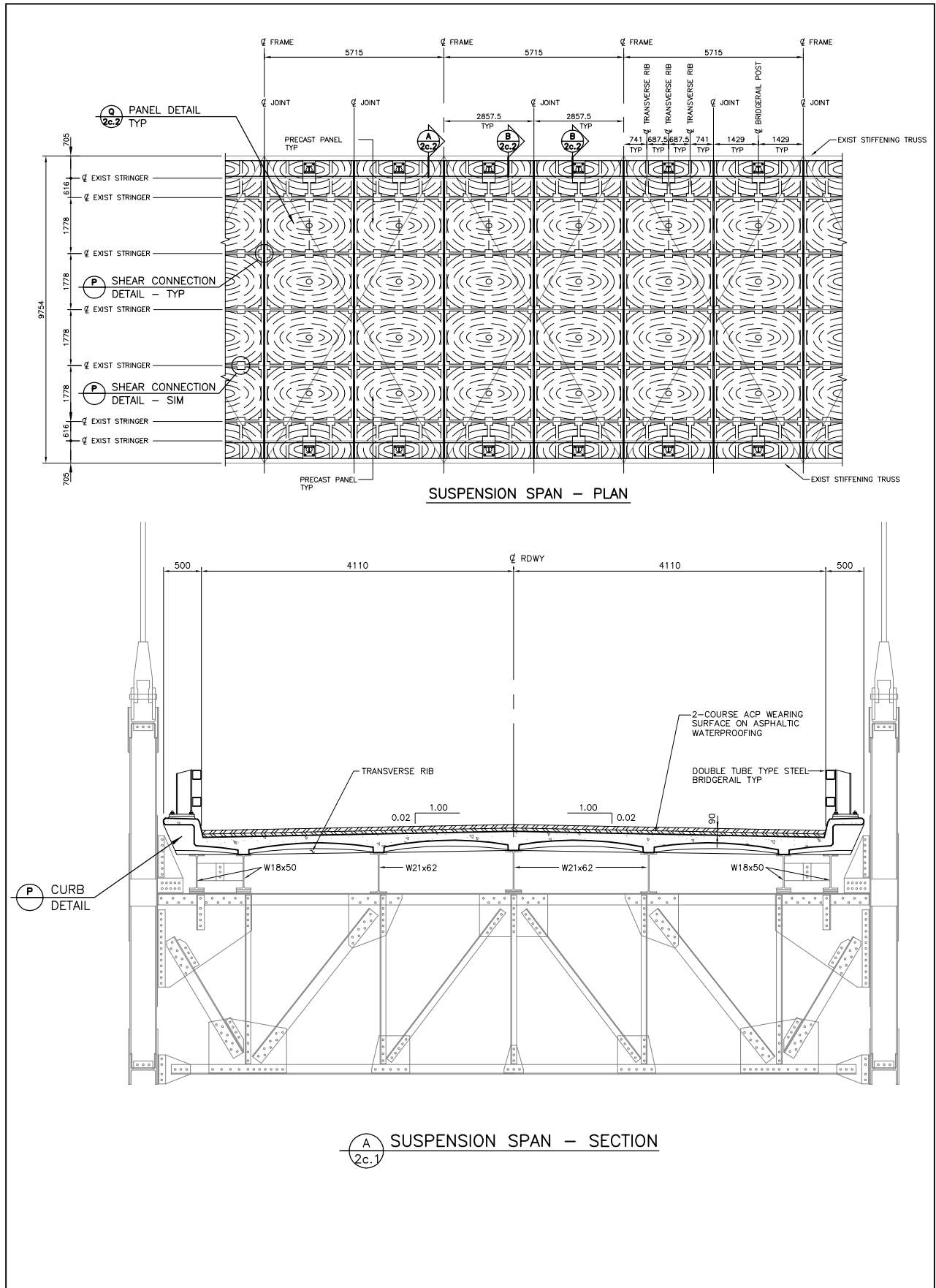


Figure 6

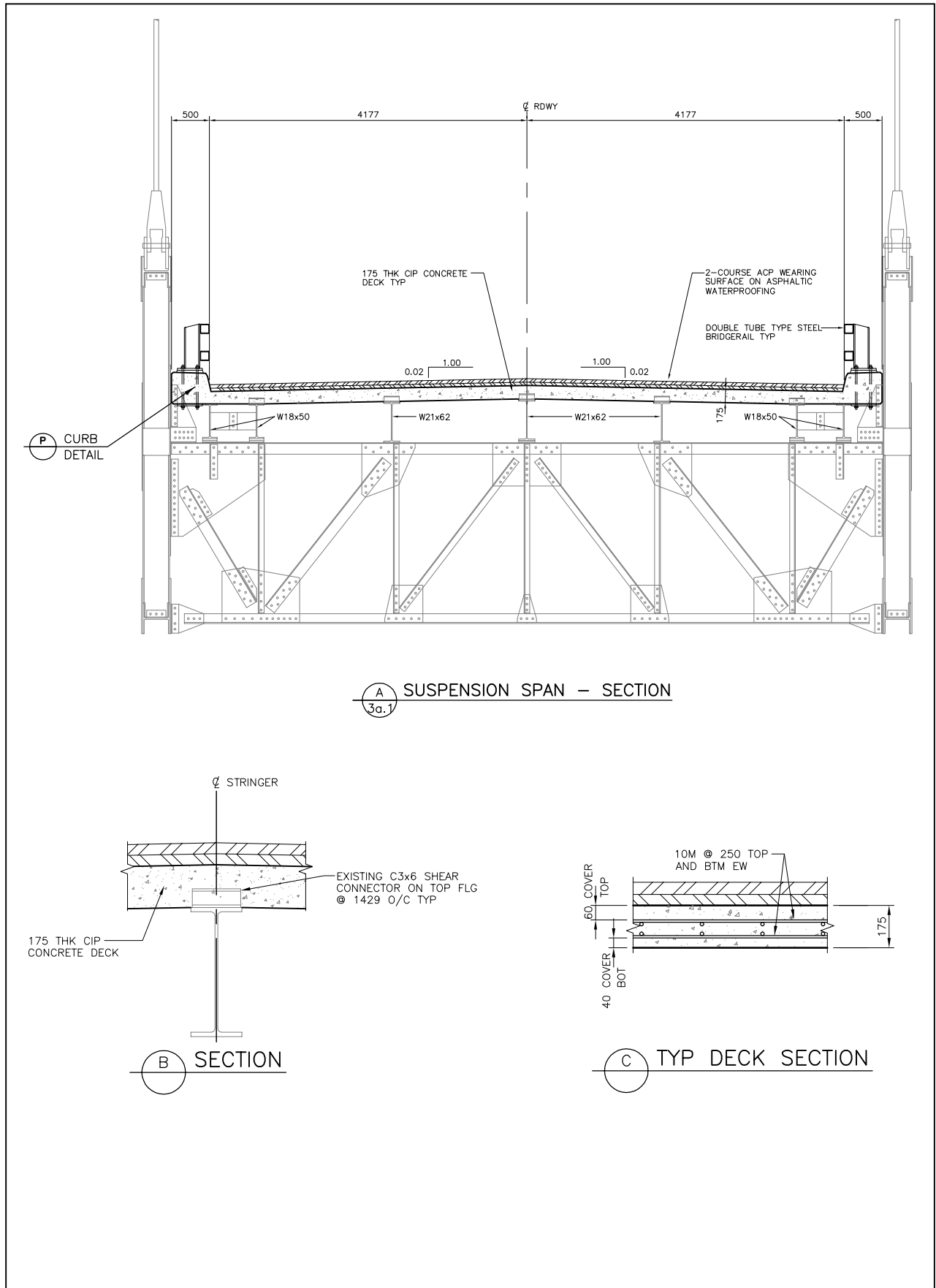
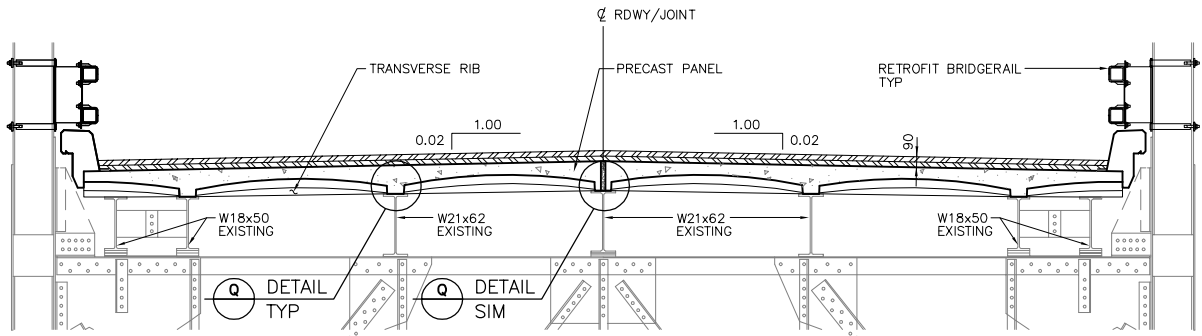
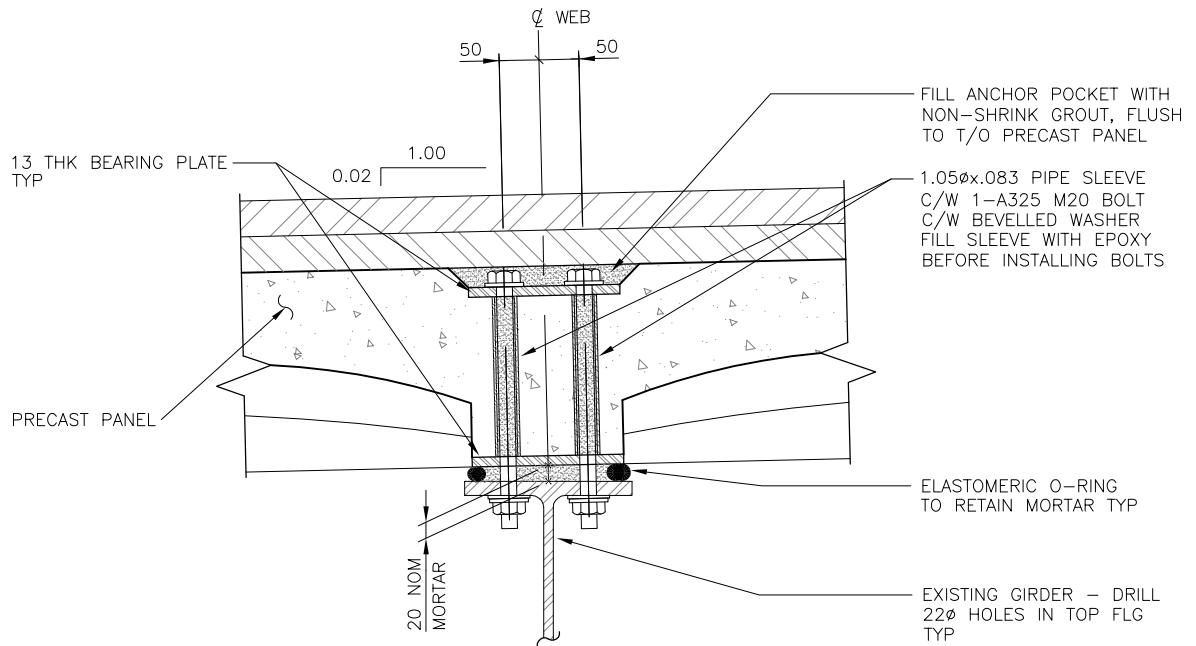


Figure 7



TYPICAL SUSPENSION SPAN – SECTION



Q PANEL INSTALL – DETAIL

Figure 9

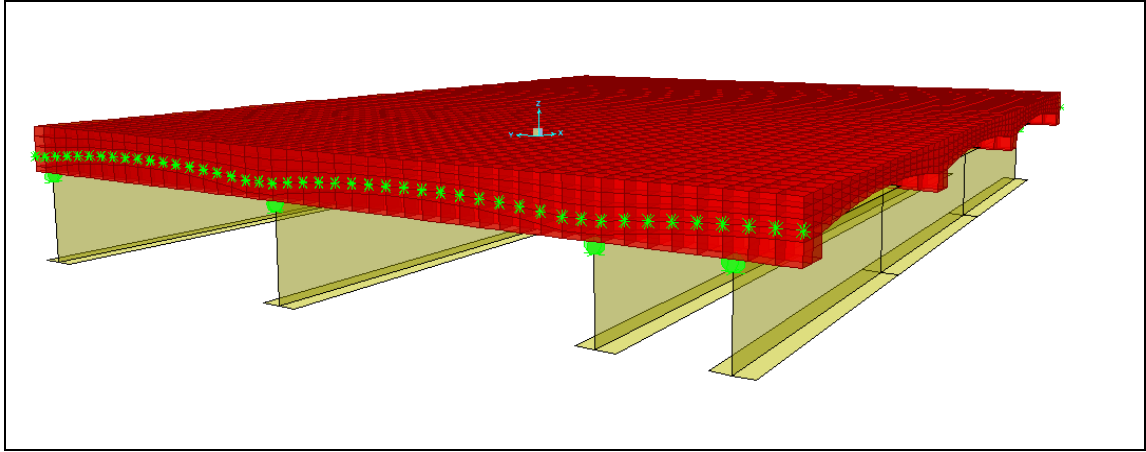


Figure 10 (a)

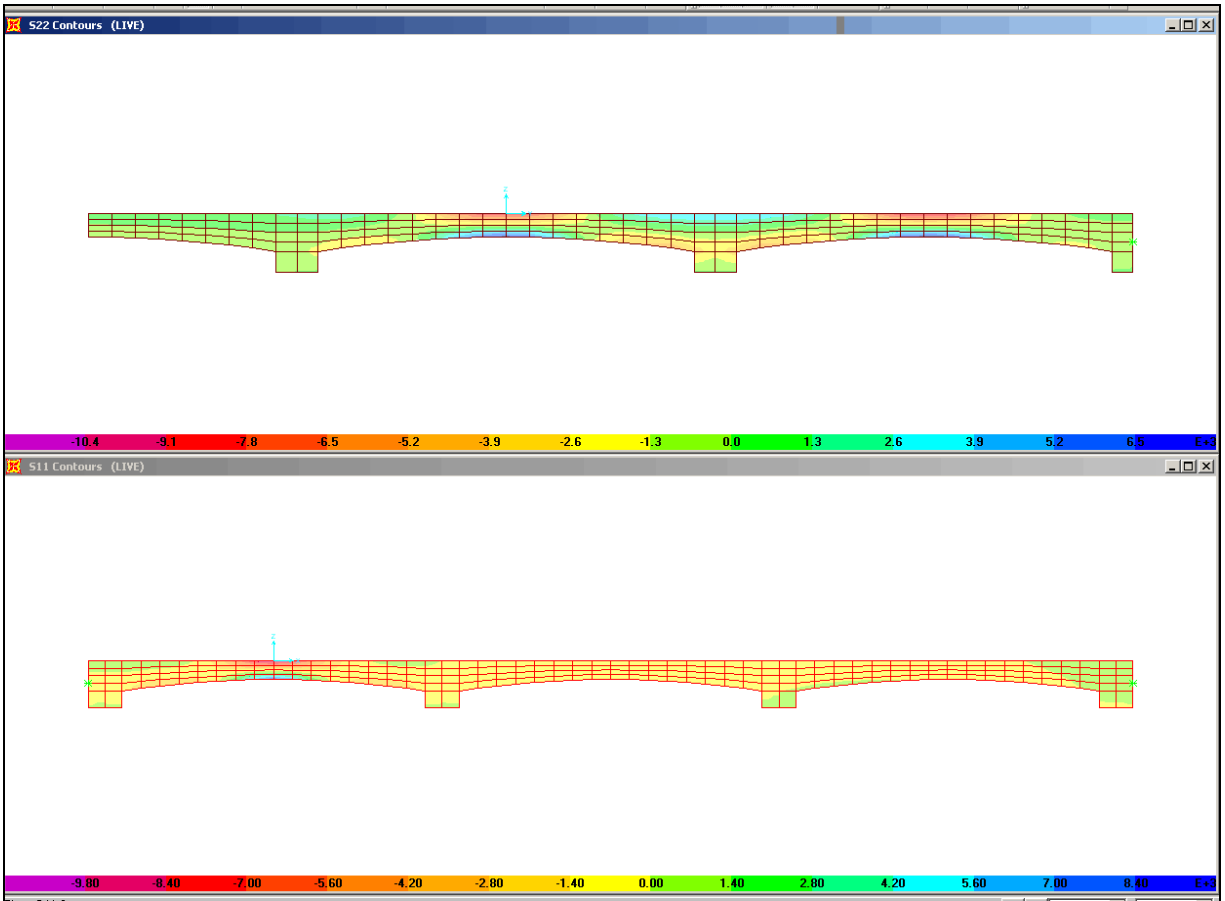


Figure 10 (b)