

From: s 9(2)(a)
 Sent: Wednesday, 11 March 2020 4:30 PM
 To: Barry Wright <Barry.Wright@nzta.govt.nz>
 Subject: RE: Welded rebar in bridge decks - confidential

Hi Barry,

I have been investigating deck solutions for the Okahu and Puhoi Viaducts.

I have received no evidence to date from NX2 to suggest the top chord rebars (temporary reinforcement trusses) can practically be prevented from buckling under expected loading associated with preparation and pouring the topping. Even if the trusses were remade with a much more robust top chord to reduce the chances of damage and buckling its still a fairly risky situation because buckling is a sudden failure – no warning. It also does not eliminate the risk of fatigue fracture, change of deck stiffness and points high moment - curvature coinciding likely points of truss fracture within the deck.

I have considered the following alternative deck solutions in the interests of expediting a solution. I have listed these in the table below and the likely reasons for eliminating from further investigation (a short list).

| Deck Type | Potential Reasons for Elimination |
|--|--|
| Cast in-situ full depth deck | Complex temporary formwork off bottom girder flanges and on sand jacks for release – awkward. Staged construction – trade off between extent of formwork and speed of construction. |
| Full depth precast panels | Require pockets over shear studs limiting cross section to cantilever over outer girders (1 & 4) which implies prestressing – requires a stressing bed. Cast in-situ stitch over intermediate girders (2 & 3) -if lifting weight to be reduced. Post tensioning in bridge longitudinal direction to achieve continuity over piers – awkward. |
| Part partial depth & part full depth precast | Unable to have continuity steel over piers – could beef up girders or look to use post tensioning – awkward. (Partial depth precast is too flexible even with minimal topping to allow continuity steel over piers). |
| Orthotropic steel deck | Requires additional of transverse aligned floor beams between girders and longitudinal aligned U shaped stiffeners – too much site work working at height. |
| Open Grid Steel Deck | Too noisy – New to NZ |
| Half Filled Grid Deck | New to NZ – limited span will require floor beams and stringers - awkward |
| Fibre Reinforced Polymer (FRP) | New to NZ – requires investigation |
| Sandwich Plate System (SPS) | New to NZ – requires investigation |
| Timber | Durability and limited span will require floor beams and stringers, requires investigation. |

In confidence we have what I believe to be workable solution to an awkward situation but I will wait to see what happens tomorrow before putting it forward for further consideration. As it stands we have only been requested to peer review the existing temporary reinforcement truss system.

Cheers

s 9(2)(a)



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From: s 9(2)(a)
Sent: Tuesday, 10 March 2020 11:04 AM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Welded rebar in bridge decks - confidential

Hi Barry,

Thank you for getting back to me.

See my responses below in red.

Cheers

s 9(2)(a)

Out of Scope

Out of Scope

Thank you for maintaining confidentiality. I understand from sources on site that there is a general awareness that there may be issues with the decks.

Out of Scope

Yes we have raised the two potential deck failure mechanisms – one during construction and one during service life. Acciona have requested a meeting with me on Thursday 12 March. I am not sure of the purpose of the meeting? E.g. to defend the current deck design to me as would be peer reviewer for the temporary reinforcement truss or request s 9(2)(a) to come up with another deck solution or something else? I suspect they want the former.

Out of Scope

Understood. NX2 may be able to produce evidence and satisfy the NZTA that my concerns are not valid.

Out of Scope

Depends on the second point. If NX2 want a new deck solution (conditional on the existing superstructure) then a prerequisite will be a thorough review of the girders (superstructure) and a less sophisticated review of the substructure. They will also need to meet their commercial obligations to s 9(2)(a) and the various subcontractors required for the rework. If Acciona want to proceed as the current deck system then it may be imprudent for s 9(2)(a) to be drawn into other 'design deficiency' issues of another consultant(s).

Out of Scope

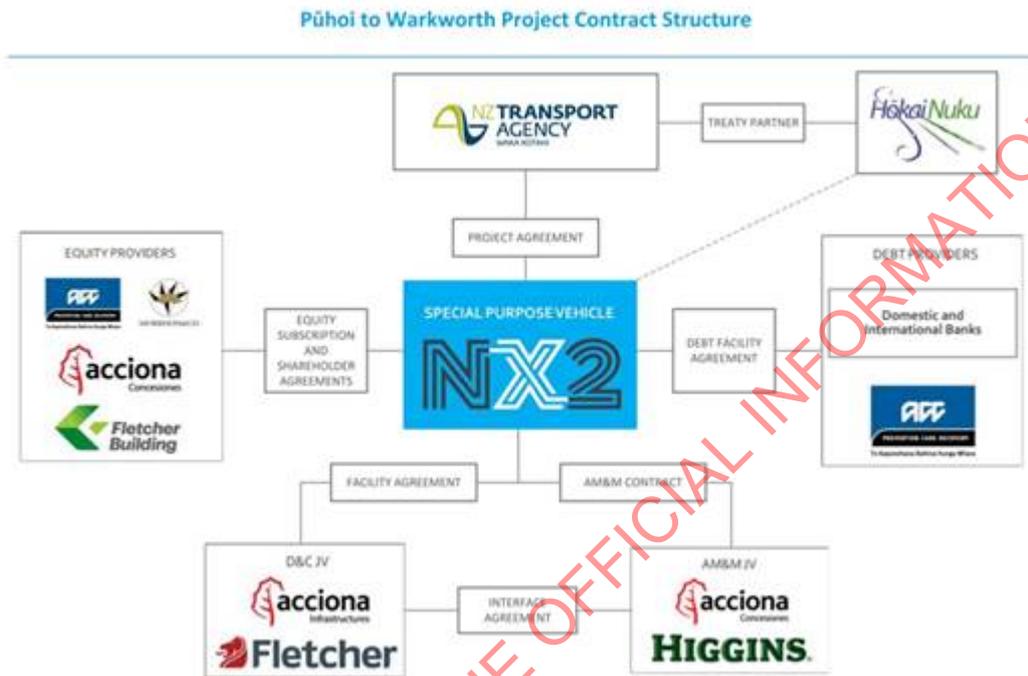
. A new deck should cost about the same as the proposed deck although the cost of the welded rebar trusses is sunk – they are just scrap value in my view. The costs of modifying the existing girders in-situ and largely erected will need to be weighed up against the cost of new girders. There are also time costs and reputation costs to consider.

Out of Scope

We see what happens on Thursday. I am not going up there to get in the middle of a scrap between Acciona, Fletchers, Beca, Aecom, Morrison&Co, and the ACC; if it goes this way then the NZTA will need to step in I believe. I can go up to site to better understand the situation (and confirm my ideas) and if need be develop a solution to the issues I have raised.

Out of Scope

We have this org chart from the NZTA website shown below.



Assuming my misgivings about the viaducts deck design are correct then the following factors will come into play:

- Major cost blow out (viaduct deck area x ¼ typical bridge cost = \$20M; this is likely to be lower bound) and who pays is a moot point?????
- Major delays (according to the latest project newsletter (Feb 2020) then all structures are to be complete Oct 2020 while road surfacing is to be complete Sep 2021. Suggest as a starting point we work back from the latter date and allow for deck solution optioneering (scheme report); deck redesign including DPS; superstructure (and substructure) check; cost estimates; programme consideration; decision on design to either modify in-situ or remake girders; decision on tendering or nominated subcontractors; material order; site preparation (if existing girders to be modified); construction/fabrication; erection; barriers etc.
- Ramifications and reputations associated with the above - ??????????

§ 9(2)(a) will want to be protected from any fallout, engaged to develop practical solutions, concentrate on the engineering design and get paid promptly.

All the above suggests the NZTA need to be involved. § 9(2)(a) can provide ongoing bridge engineering support direct to the NZTA.

If my misgivings are proven to be unfounded then § 9(2)(a) would still like to provide ongoing bridge engineering services to the NZTA. We can discuss opportunities after these issues are dealt with one way or another.

From: s 9(2)(a)
Sent: Monday, 9 March 2020 10:10 AM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Welded rebar in bridge decks - confidential

Morning Barry,

Given our misgivings regards the proposed viaduct deck system that we were asked to peer review a critical part of, that is the temporary reinforcement trusses; we are now awaiting evidence from NX2 that could demonstrate our concerns are unfounded. From our conversation last Thursday NX2 comments similar 'we have done this before' and 'it seems every engineer in NZ have looked at these viaducts' were made.

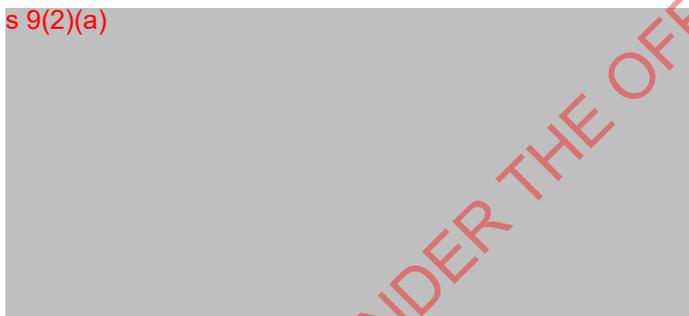
Nevertheless the failure mechanisms outlined are based on a combination of theory and experience. In our mind they are valid concerns.

Given the state of the viaduct construction (some girders erected and others scheduled to be erected shortly) and the tight programme for completion this has prompted us to consider 'alternative' solutions to the deck issue should this be required. As a consequence we have had to examine the steel girder sections because of the composite action between the deck and the girder.

I note that the web slenderness in the sagging regions of the girders is beyond code limits. That is the web is too thin according to both AS5100.6 (required – see clause 5.9.1) and AASHTO LRFD (well proven; - see article 6.10.2.1.1). The purpose of these provisions is to prevent web buckling even in these lower shear regions.

I am happy to meet and discuss any of these matters confidentially. It seems to me these two significant viaducts (both 4 lanes and over 300m long) warrant some serious and urgent design investigation and perhaps reconsideration.

s 9(2)(a)



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From: s 9(2)(a)
Sent: Friday, 6 March 2020 1:46 PM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Welded rebar in bridge decks - confidential

Hi Barry,

Here is my response to NX2Group after yesterday's meeting within quotes.

"Thank you for your time and explanation yesterday.

As promised our anticipated failure mechanism of the proposed viaduct decks during service life is outlined below under point 2. But first we would like to explain the anticipated failure mechanism of the proposed viaduct decks during construction under point 1. This is of more immediate concern.

We encourage the NX2Group to gather any evidence it can to prevent either of these failure mechanisms from occurring and present this evidence to the NZTA as Principal and Regulator. We would like to be furnished with this evidence so that we can work with the NX2Group to solve these deck safety and durability issues promptly.

We are aware of your tight programme, the two thirds of the temporary reinforcement trusses on site and general commitment of NX2 to this proposed deck system. Nevertheless we feel obligated to outline our concerns regarding the proposed viaduct deck system.

1. Anticipated failure mechanism of the proposed viaduct deck system during construction.

- a. Considering the two viaducts we estimate over 38km of top chords are in compression and will have their maximum axial compressive loading when the concrete topping (205mm thick) is being poured. Any damage resulting in kinks or misalignment of HD20 top chord will dramatically reduce the buckling capacity (i.e. a sudden failure) due to P-delta effects.
- b. We would suggest with the best intentions in the world, it is highly likely some DH20 top chords will be damaged by construction loading preparing for the topping pour.
- c. A failure of one top chord in sudden buckling will shed additional compression load onto the adjacent top chords.
- d. This may lead to a progressive failure of a particular 2.7m wide precast panel.
- e. This is irrespective of the theoretical buckling capacity of a perfectly straight HD20 top chord (between points of restraint) as Acconia Ingeneria have based their calculations on. Given the extent of top chords it is impractical to expect pure axial compression; there will be bending moments and moment magnification leading to sudden buckling failure.

2. Anticipated failure mechanism of the proposed viaduct deck system during service loading.

- a. The deck slab spanning transversely across the 4 girders and cantilevering outwards is very stiff with both the required deck reinforcement and the additional stiff temporary reinforcing truss. From the drawings truss chords are at the same depth as the deck slab transverse bars. We assume no reliance has been placed on the temporary reinforcing truss in the design of the ordinary slab reinforcement. It might be that the decks have nearly twice as much transverse reinforcement as a full depth deck slab. That is say half from the truss and half from the ordinary reinforcement. Hence the deck slab is very stiff.
- b. After an unknown period (much less than 100 years) of service loading, particularly from heavy truck axles impacting on every truss, the temporary reinforcing truss is expected to crack due to fatigue loading at quite low cyclic stresses. This is the design intent as per the drawings. The truss is likely to fracture in regions of highest strain. That is over the girders (hogging moment) and between the girders (sagging moment) but fatigue crack propagation could be at any weld or stress riser along the length of the trusses. The location of fatigue cracking is practicably unpredictable. Given the extent of welding then the degree of residual stresses within the trusses may be significant. That is the bottom chords could be under significant tension due

- both residual stresses and deck construction loading before vehicle loading places additional demands on these chords.
- c. Once the trusses fracture through the rebar chords then the stiffness of the deck slab at that point is much less relative to the remainder of the deck slab.
 - d. Further traffic loading will concentrate deck rotations at these locations of low relative stiffness and higher strain. The deck slab will no longer deflect like it is monolithic with constant stiffness; but will behave like a series of stiff straight sections connected between with less stiff hinges.
 - e. At the points of lower stiffness; higher deck rotations; and subject to high cyclic load concentrations there will be considerable 'working' of the fractured temporary reinforcing truss within the deck slab. Concrete cracking and corrosion pockets at the ends of the fractured bars and within the slab will develop.
 - f. It is likely there will end up with larger than normal longitudinal cracks above the girders and along the underside between the girders. As water accumulates within the deck slab then corrosion of the shear studs could occur and loss of composite action result over a longer period of time; some corrosion pockets are likely to be directly over the girders.

We understand from our conversation yesterday that BECA are the Designer and Aecom are the Peer Reviewer. They put forward and approved the proposed deck system. This obligated the Temporary Works Engineer (Acciona Ingeneria) to sign-off the temporary reinforcement truss, which has been done. The Designer now wants a NZ independent peer review of the temporary reinforcement truss. NX2Group have asked s 9(2)(a) to do this. We have not been able to analyse the temporary reinforcement truss in pure isolation because it is integral with the total viaduct deck system. We are concerned about the above two failure mechanisms occurring and have suggested these are either proven incorrect by supporting evidence, or NX2Group notify the NZTA forthwith of these failure mechanisms.

We are ready and willing to design an alternative unfailing safe and durable viaduct deck system for the P2Wr project and work with all concerned parties to achieve this on a best for project basis."

s 9(2)(a)

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From: s 9(2)(a)
Sent: Friday, 6 March 2020 1:42 PM
To: 'Barry Wright' <Barry.Wright@nzta.govt.nz>
Subject: RE: Welded rebar in bridge decks - confidential

Hi Barry,

According to NX2Group or perhaps they were speaking as Acciona, the Designer is BECA & T&T; the Peer Reviewer is Aecom; the NZTA technical reviewers are both GHD and Jacobs. The Temporary Works Engineer as it currently stands is Acciona Ingeneria – a Spanish subsidiary of Acciona. On the drawings there are some familiar names as designer and design verifier. Attached are the relevant pages from the DPS, the Okahu Viaduct construction issue drawing set; and the Puhoi Viaduct construction issue drawing set – with my clouding to indicate references to the proposed viaduct deck system.

Acciona did mention that GHD and Jacobs raised no issues regards the deck system on behalf of NZTA.

If these roles are not precisely as I have indicated then its because s 9(2)(a) have not been told correctly. We did specifically ask who is doing what?

We had been drawn into this because Acciona had asked us for some geotechnical advice and I am not sure if BECA or T&T are aware of this; they subsequently asked us to peer review the welded (temporary) reinforcement trusses.

Whilst Acciona gave their permission for us to discuss the issues with BECA and Aecom, I believe NX2 Group should either produce evidence to prove the failure mechanisms raised are incorrect, or refer the 'oversights' to the NZTA directly to seek a resolution.

I am keen s 9(2)(a) do not get embroiled in peer reviewing the rebar trusses in isolation, only to find at a later date we approved the 'lynch pin' for a number of potential failure mechanisms. Clearly we want to keep on good terms with s 9(2)(a), Acciona.

We can contribute to an unfailingly safe and durable deck system using the existing girders.

s 9(2)(a)



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- ▶ This solution is aesthetically cleaner, with a visually pleasing deck overhang due to a longer cantilever (3 m) and this solution has no crossbeams between the two centre girders
- ▶ This design was developed and successfully used in Europe and is well documented in engineering papers. This team brings the international experience to promote a bridge form not typically used in New Zealand to enhance the visual appearance of the two large viaducts.

This innovative solution achieves the Project Key Outcomes for “Safety”, “High Quality Asset” and “Improving the Transport Agencies Reputation”.

6.10.3.3 Okahu and Pūhoi composite deck solution

The most common solution for new composite steel girders in New Zealand uses the ladder bridge form which has the cross girders spanning transversely between the main longitudinal girders. The deck spans longitudinally between the cross girders (Figure 2). These cross beam slab supports reduce local bending. In such situations the distance between crossbeams is typically to 3m or 4m and shear connectors are provided to make the cross beam composite with the deck.

The steel girder bridge predominates among the composite solutions employed in Europe, and has the cross girders generally located around the mid depth of the main beams (Figure 1). The deck slab spans transversely and is supported by the main girders. In such situations the distance between cross beams is between 6 - 10 mand they do not have shear connectors.

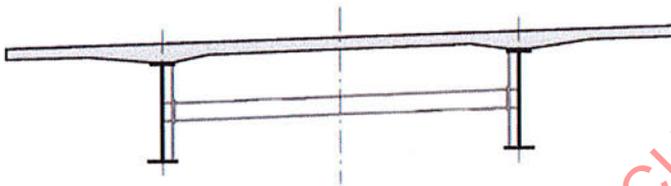


Figure 1: Cross girder at mid depth (Europe detail shown).

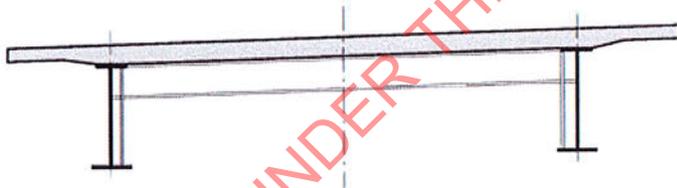


Figure 2: Cross girder also supporting the slab (Europe detail shown).

The deck usually comprises of a precast bottom section which is used for permanent formwork for the insitu top half of the deck. This form of construction is used for both ladder and steel girder solutions.

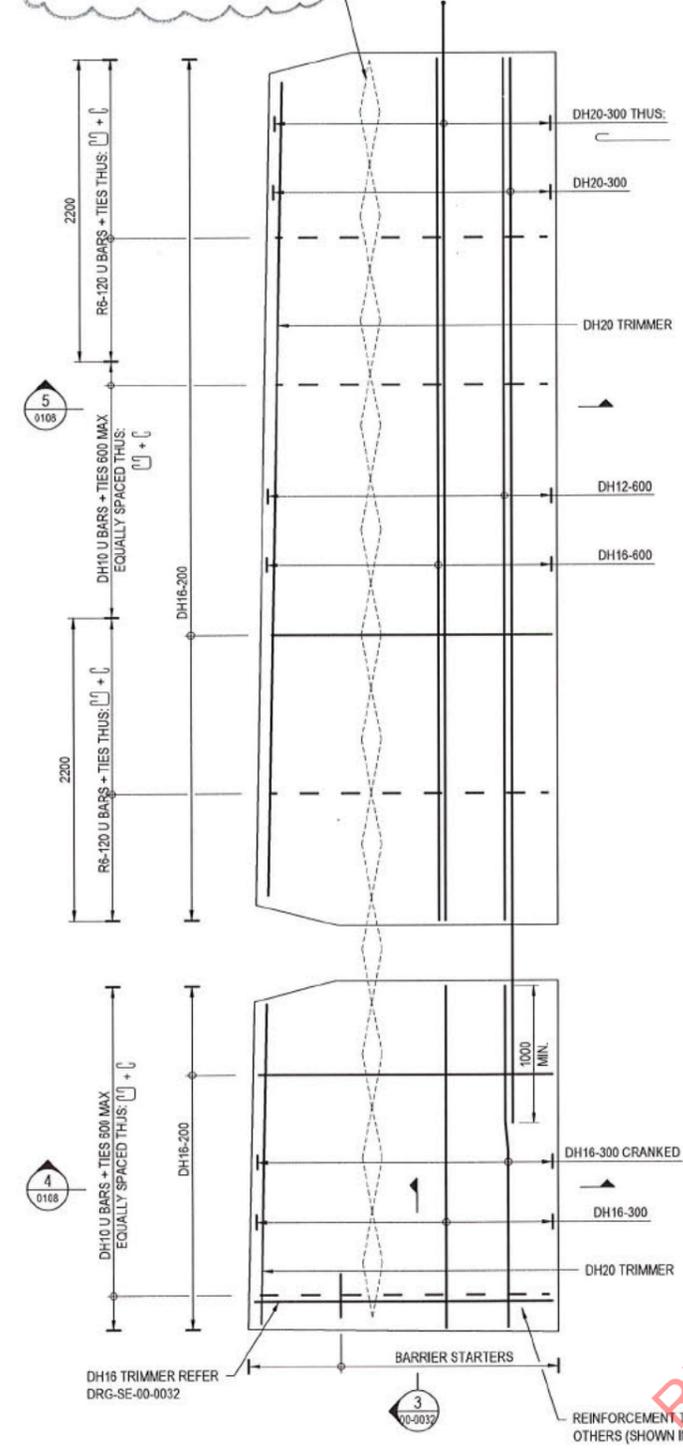
The advantage of this steel girder deck solution is that the precast deck section spans transversely between two main girders and cantilevers outward over the outer girder. The cantilever is supported by a steel truss system integral with the precast slab. This means the precast deck requires no permanent deck support for the cantilever. The ladder bridge typically uses steel outriggers to temporarily support the deck cantilever. This affects aesthetics since there are now external stiffeners visible on the exterior girder for the ladder solution.

The steel girder and deck design means that there are no steel transoms required between the central two girders. The innovative deck design has been used overseas and is considered a significant safety improvement during construction (requires less working at height time).

NOTES:

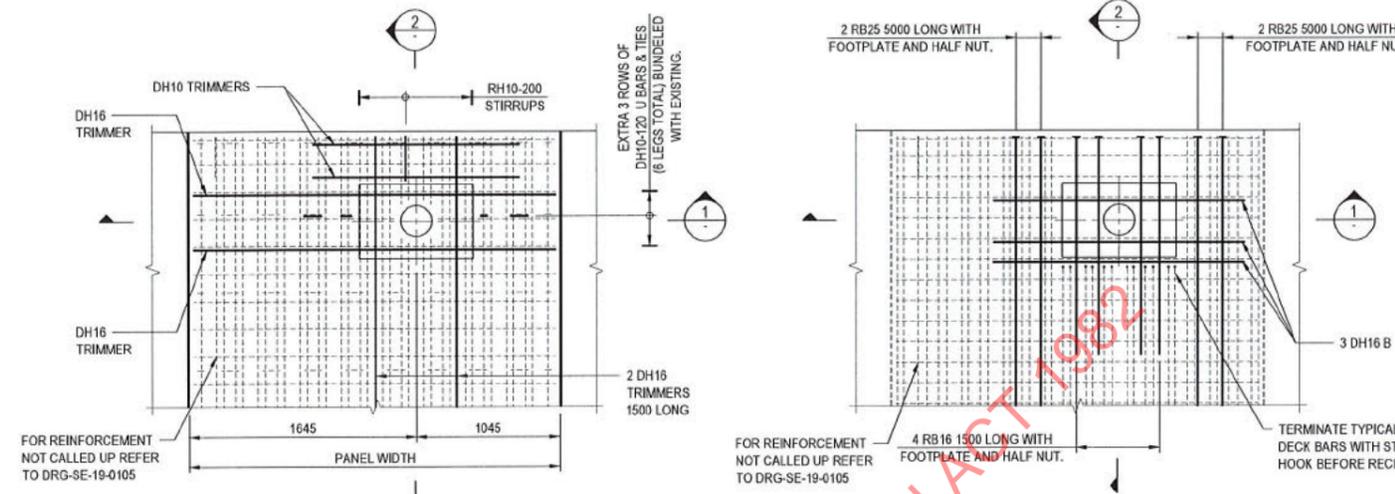
1. GENERAL
 - 1.1 REFER DRG-SE-19-0002 FOR GENERAL NOTES.
 - 1.2 FOR INFORMATION RELATING TO SERVICE PENETRATIONS AND SPACING, REFER TO DRG-SE-19-0135.
 - 1.3 INSITU SLAB SHALL BE POURED TO ACHIEVE A TOTAL DECK SLAB THICKNESS OF 315mm RATHER THAN TO A GEOMETRIC LEVEL.
- REINFORCED CONCRETE
 - 2.1 FOR REINFORCED CONCRETE GENERAL NOTES, REFER DRG-SE-00-0010 & 0011.
 - 2.2 MINIMUM 28 DAY CONCRETE STRENGTH (f_c) SHALL BE 40 MPa.
 - 2.3 MINIMUM COVER TO REINFORCEMENT FOR BOTH PRECAST AND INSITU DECK SHALL BE: 50mm, REFER TO DRG-SE-00-0010 FOR TOLERANCES.
 - 2.4 REINFORCING STEEL SHALL BE GRADE 500 TO AS/NZS 4671 REFER SPECIFICATION (680-SPEC-001-NX2).
 - 2.5 CURING TO BE UNDERTAKEN AS PER THE REQUIREMENTS OF NZS 3109 & PROJECT SPECIFICATIONS. MINIMUM CURING DURATION - 7 DAYS UNLESS AGREED OTHERWISE WITH DESIGNER.
 - 2.6 CONCRETE SURFACE FOR UNDERSIDE OF PRECAST SLAB SHALL BE F4 AS PER NZS 3114.
 - 2.7 CONCRETE FINISH TO TOP OF PRECAST PANELS SHALL BE TYPE B CONSTRUCTION JOINT TO NZS 3109.
 - 2.8 CONCRETE FINISH TO TOP OF 315mm THICK DECK SHALL BE U5 FINISH APART FROM BARRIER AREA WHERE A TYPE B CONSTRUCTION JOINT TO NZS 3109 IS REQUIRED.
 - 2.9 ALL PRECAST PANEL PERMANENT DECK REINFORCEMENT AND TEMPORARY TRUSSES SHALL BE CO-ORDINATED BY THE CONSTRUCTOR TO SUIT THE LOCATION OF THE SHEAR STUDS ON STEEL GIRDERS.

REINFORCEMENT TRUSSES BY OTHERS. SIZE AND SETOUT TO BE DETERMINED BY C.J.V. (SINGLE TRUSS SHOWN INDICATIVELY)



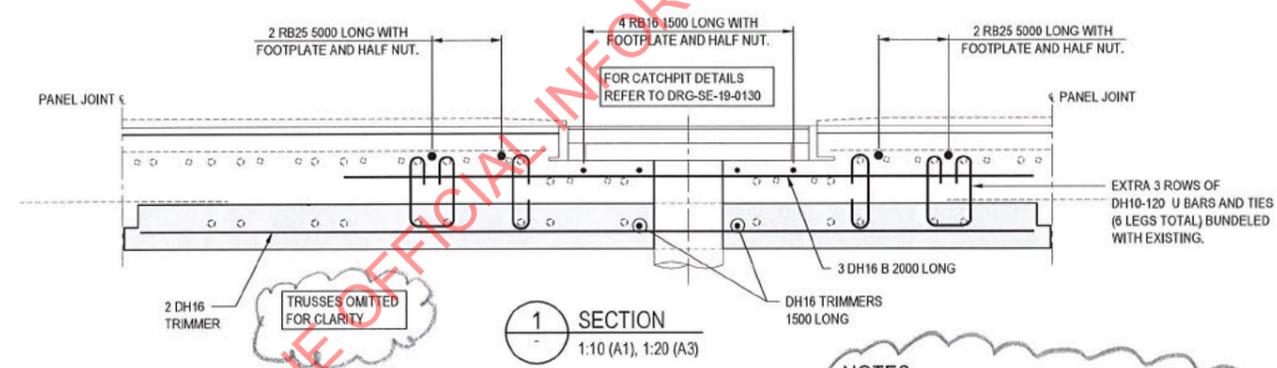
C PANEL TYPE S REINFORCEMENT DETAILS
1:25(A1), 1:50(A3)

NOTE: REFER TO SECTIONS FOR U BAR AND TIE LAYOUT

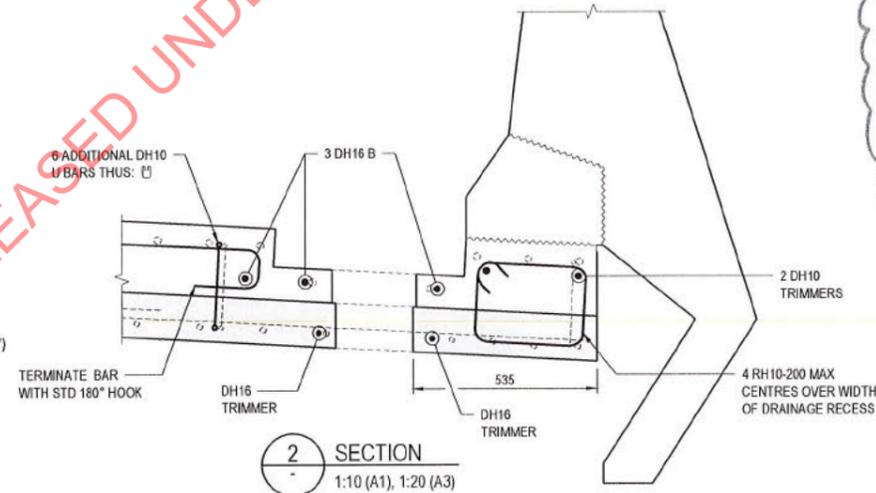


C PLAN- PRECAST PANEL AT CATCHPIT
ADDITIONAL REINFORCEMENT
1:25 (A1), 1:50 (A3)

D PLAN- INSITU DECK AT CATCHPIT
ADDITIONAL REINFORCEMENT
1:25 (A1), 1:50 (A3)



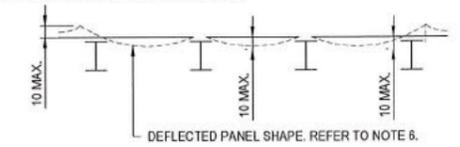
1 SECTION
1:10 (A1), 1:20 (A3)



2 SECTION
1:10 (A1), 1:20 (A3)

NOTES:

- TEMPORARY TRUSS DESIGN.
1. DESIGN OF THE PRECAST PLANKS AND THE TEMPORARY REINFORCING TRUSS (INCLUDING TRUSS LOCATION) IS THE CONSTRUCTORS RESPONSIBILITY. AS DEFINED BY BA 30/90 AND AS 3010. THE CONSTRUCTOR SHALL ALLOW FOR ALL CONSTRUCTION LOADS PLUS THE DEAD LOAD, INCLUDING THE EFFECTS OF CONCENTRATED LOADING BOTH ON THE PANEL AND LOCALLY ON THE TOP TRUSS MEMBER.
 2. THE CONSTRUCTOR SHALL SATISFY THEMSELVES THAT THE PRECAST SLAB AND STEEL TRUSS TEMPORARY WORKS HAS SUFFICIENT STRENGTH AND STABILITY DURING ALL STAGES OF LIFTING AND ERECTION. NOTE THAT THE TEMPORARY WORKS IS NOT PART OF THE DESIGN PROCESS AND IS ALSO NOT COVERED BY THE PEER REVIEW OF THE DESIGN. IT IS THEREFORE REQUIRED THAT THE TEMPORARY WORKS ENGINEER SATISFY THEMSELVES THAT THE PRECAST SLAB AND STEEL TRUSS TEMPORARY WORKS ARE ADEQUATE FOR THE TEMPORARY STATE, UNTIL THE DECK IS MADE COMPOSITE AND CAN SUPPORT ITS SELF-WEIGHT.
 3. THE TEMPORARY REINFORCING TRUSS IS CONSIDERED REDUNDANT FOR THE PERMANENT WORKS DUE TO FATIGUE ISSUES CAUSED FROM WELDING OF THE TRUSS BARS.
 4. THE CONSTRUCTOR SHALL PROVIDE ADEQUATE LATERAL SUPPORT TO PREVENT THE PRECAST PANEL FROM DISPLACING OFF THE STEEL GIRDER FLANGE.
 5. THE CONSTRUCTOR SHALL COORDINATE THE TEMPORARY REINFORCING TRUSS WITH THE PERMANENT GIRDER SHEAR STUDS, PERMANENT DECK SLAB REINFORCING, SERVICE INSERTS, DRAINAGE CATCHPITS AND ANY OTHER STRUCTURAL ELEMENT (PERMANENT OR TEMPORARY) TO AVOID CLASHES ON SITE.
 6. THE TEMPORARY TRUSSES SHALL BE CAMBERED AS REQUIRED TO LIMIT THE TOTAL SLAB DEAD LOAD DEFLECTION TO 10mm MAXIMUM DIFFERENCE BETWEEN THEORETICAL PRECAST SLAB DEFLECTION AND THE THEORETICAL STRAIGHT LINE BETWEEN TOP FLANGE EDGES OF ADJACENT GIRDERS. PANEL DEFLECTIONS EXCEEDING THIS DEFLECTION LIMITATION SHALL BE DISCUSSED AND ACTION AGREED WITH THE DESIGNER.



WITNESS POINT

2. THE CONSTRUCTOR SHALL SATISFY THEMSELVES THAT THE TRUSS DESIGN IS ADEQUATE WITH 100MM SEATING SUPPORT ON THE GIRDERS. THE SUPPORT CONDITION ASSESSMENT MADE BY THE CONSTRUCTOR SHALL INCLUDE AN ALLOWANCE FOR: GIRDER VERTICAL POSITION TOLERANCE, SLOPING TOP FLANGE TOLERANCE, AND GIRDER PRECAMBER.
3. THE TEMPORARY REINFORCING TRUSS IS CONSIDERED REDUNDANT FOR THE PERMANENT WORKS DUE TO FATIGUE ISSUES CAUSED FROM WELDING OF THE TRUSS BARS.
4. THE CONSTRUCTOR SHALL PROVIDE ADEQUATE LATERAL SUPPORT TO PREVENT THE PRECAST PANEL FROM DISPLACING OFF THE STEEL GIRDER FLANGE.
5. THE CONSTRUCTOR SHALL COORDINATE THE TEMPORARY REINFORCING TRUSS WITH THE PERMANENT GIRDER SHEAR STUDS, PERMANENT DECK SLAB REINFORCING, SERVICE INSERTS, DRAINAGE CATCHPITS AND ANY OTHER STRUCTURAL ELEMENT (PERMANENT OR TEMPORARY) TO AVOID CLASHES ON SITE.

7. FOUR FULL WIDTH STANDARD PANELS (TWO CANTILEVER TYPES AND TWO SIMPLY SUPPORTED SLABS) SHALL BE LOAD TESTED. THE TEST SLABS SHALL HAVE THEIR DEFLECTIONS MEASURED AND COMPARED AGAINST THEORETICAL DEFLECTIONS PREVIOUSLY CALCULATED BY C.J.V. TESTING OF DEFLECTIONS SHALL BE MEASURED 4 HOURS AFTER EQUIVALENT DEAD LOAD OF INSITU DECK IS APPLIED. THE SLABS SHALL THEN BE TESTED TO ULS DEMANDS, DEFLECTION MEASURED AND THEN TESTED TO THEIR DESTRUCTION TO CONFIRM FAILURE MECHANISM AND LOAD CAPACITY.
8. DECK HAS BEEN DESIGNED ASSUMING THE FOLLOWING LOCKED-IN STRESSES EXIST IN THE BOTTOM REINFORCEMENT
SLS LOAD CASE : 89MPa
ULS LOAD CASE : 120MPa
ONCE THE TEMPORARY TRUSS SYSTEM DESIGN HAS BEEN FINALISED BY THE TEMPORARY WORKS ENGINEER THE VALUE ABOVE SHALL BE CONFIRMED AND CHECKED WITH THE DESIGNER.

HOLD POINT FOR CONSTRUCTION

| | | | | | |
|-----|------------------|----|-----|------|----------|
| No. | Revision | By | Chk | Appd | Date |
| 1 | FOR CONSTRUCTION | | | | 24.09.18 |

| | | | | | |
|---------------------|--------------|--------|----------|---------------------------|--------|
| Original Scale (A1) | Design | s 9(2) | 16.11.17 | Approved For Construction | s 9(2) |
| AS SHOWN | Drawn | (a) | 06.11.17 | | |
| Reduced Scale (A3) | Dwg Verifier | | 27.09.18 | | |
| HALF SIZE | Dwg Check | | 27.09.18 | | |

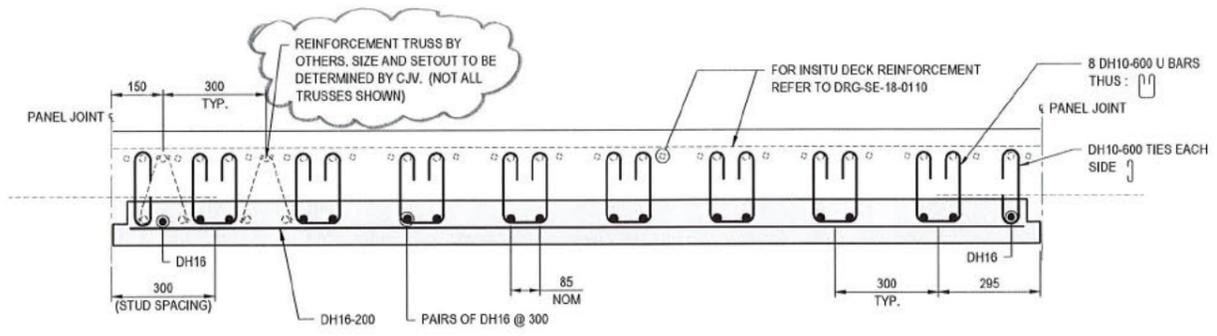


PŪHOI TO WARKWORTH MOTORWAY



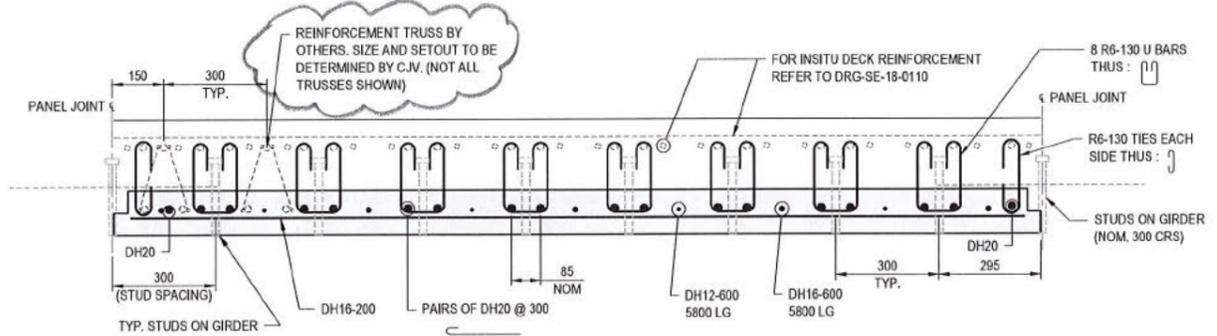
| | | | |
|---------|---|-------------|---------------------|
| Subject | 12-11 OKAHU VIADUCT | Discipline | STRUCTURAL |
| Title | PRECAST DECK PLANKS REINFORCEMENT DETAILS - SHEET 3 | Drawing No. | P2Wk-DRG-SE-19-0107 |
| | | Rev | 1 |

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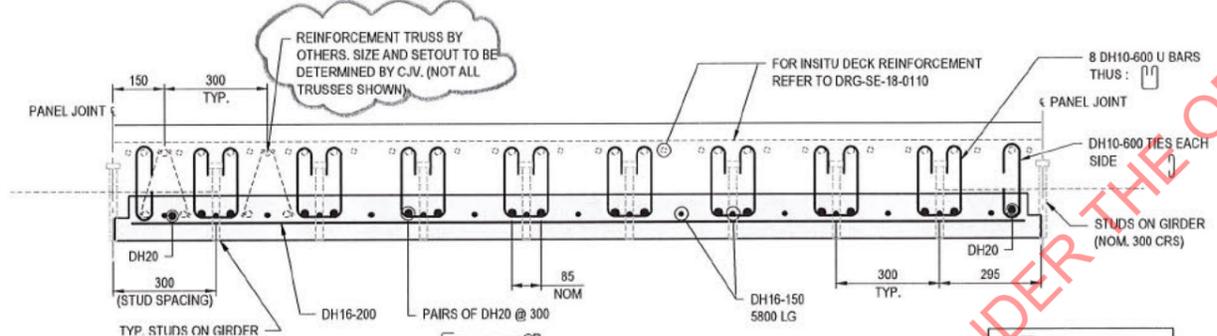
1 PANEL TYPES A, A1, F AND H REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

NOTE:
PANEL TYPE A DRAWN,
PANELS A1, F AND H SIMILAR



2 PANEL TYPES A, A1, F AND H REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

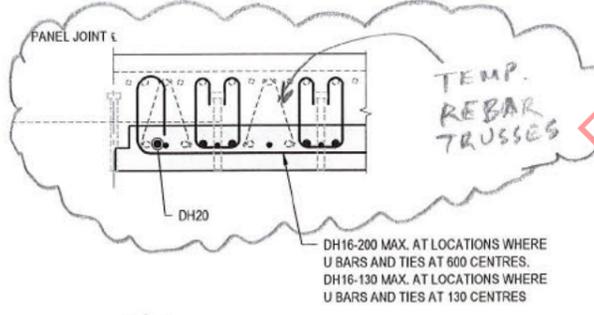
NOTE:
PANEL TYPE A DRAWN,
PANELS A1, F AND H SIMILAR



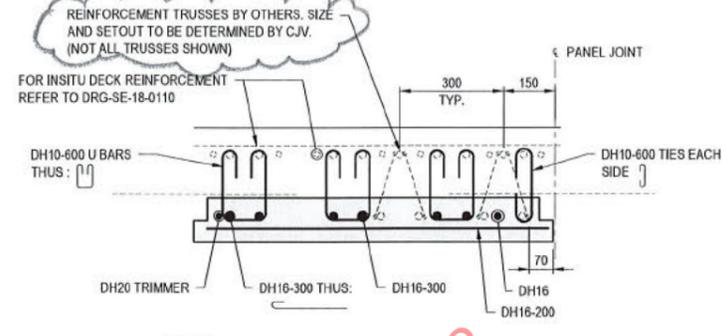
3 PANEL TYPES B, B1, G AND J REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

NOTE:
PANEL TYPE B DRAWN,
PANELS B1, G AND J SIMILAR

NOTE:
U BARS MAY BE REPLACED
WITH CLIPS OF SAME
DIAMETER AND SPACING
i.e. =>

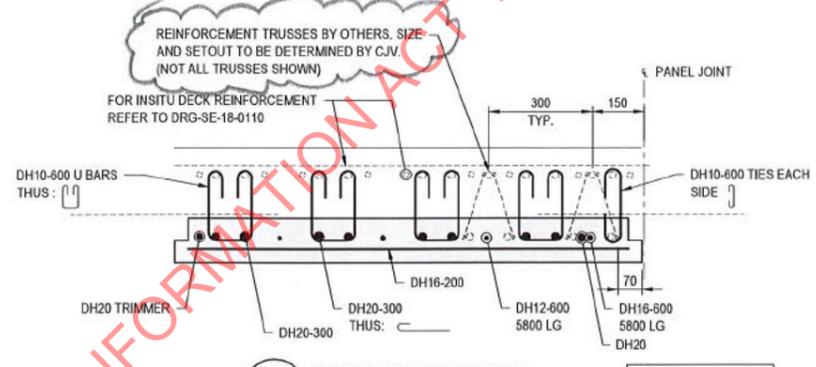


A PROPOSED CJV ALTERNATIVE OPTION TO REPLACE END TIE WITH DH16 HOOK BAR
1:10(A1), 1:20(A3)



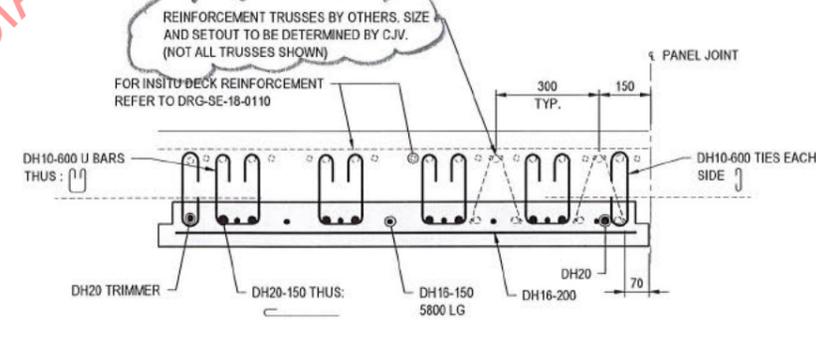
4 PANEL TYPE C AND E REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

NOTE:
PANEL TYPE C DRAWN,
PANEL E SIMILAR



5 PANEL TYPE C AND E REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

NOTE:
PANEL TYPE C DRAWN,
PANEL E SIMILAR



6 PANEL TYPE D REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

7. FOUR FULL WIDTH STANDARD PANELS (TWO CANTILEVER TYPES AND TWO SIMPLY SUPPORTED SLABS) SHALL BE LOAD TESTED. THE TEST SLABS SHALL HAVE THEIR DEFLECTIONS MEASURED AND COMPARED AGAINST THEORETICAL DEFLECTIONS PREVIOUSLY CALCULATED BY THE C.J.V. TESTING OF DEFLECTIONS SHALL BE MEASURED 4 HOURS AFTER EQUIVALENT DEAD LOAD OF INSITU DECK IS APPLIED. THE SLABS SHALL THEN BE TESTED TO ULS DEMANDS, DEFLECTION MEASURED AND THEN TESTED TO THEIR DESTRUCTION TO CONFIRM FAILURE MECHANISM AND LOAD CAPACITY.

WITNESS POINT

8. DECK HAS BEEN DESIGNED ASSUMING THE FOLLOWING LOCKED-IN STRESSES EXIST IN THE BOTTOM REINFORCEMENT
SLS LOAD CASE: 89MPa
ULS LOAD CASE: 120MPa
ONCE THE TEMPORARY TRUSS SYSTEM DESIGN HAS BEEN FINALISED BY THE TEMPORARY WORKS ENGINEER THE VALUE ABOVE SHOULD BE CONFIRMED AND CHECKED WITH THE DESIGN ENGINEER.

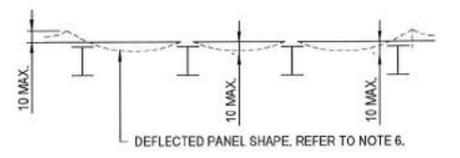
HOLD POINT

NOTES:

- GENERAL**
 - REFER DRG-SE-18-0002 FOR GENERAL NOTES.
 - FOR INFORMATION RELATING TO SERVICE PENETRATIONS AND SPACING, REFER TO DRG-SE-18-0135.
 - INSITU SLAB SHALL BE POURED TO ACHIEVE A TOTAL DECK SLAB THICKNESS OF 335mm RATHER THAN TO A GEOMETRIC LEVEL.
- REINFORCED CONCRETE**
 - FOR REINFORCED CONCRETE GENERAL NOTES, REFER DRG-SE-00-0010 & 0011.
 - MINIMUM 28 DAY CONCRETE STRENGTH (f_c) SHALL BE 40 MPa.
 - MINIMUM COVER TO REINFORCEMENT FOR BOTH PRECAST AND INSITU DECK SHALL BE: 50mm, REFER TO DRG-SE-00-0010 FOR TOLERANCES.
 - REINFORCING STEEL SHALL BE GRADE 500 TO AS/NZS 4671 REFER SPECIFICATION (680-SPEC-001-XX2).
 - CURING TO BE UNDERTAKEN AS PER THE REQUIREMENTS OF NZS 3109 & PROJECT SPECIFICATIONS. MINIMUM CURING DURATION - 7DAYS UNLESS AGREED OTHERWISE WITH DESIGNER.
 - CONCRETE SURFACE FOR UNDERSIDE OF PRECAST SLAB SHALL BE F4 AS PER NZS 3114.
 - CONCRETE FINISH TO TOP OF PRECAST PANELS SHALL BE TYPE B CONSTRUCTION JOINT TO NZS 3109.
 - CONCRETE FINISH TO TOP OF 335mm THICK DECK SHALL BE U5 FINISH APART FROM BARRIER AREA WHERE A TYPE B CONSTRUCTION JOINT TO NZS 3109 IS REQUIRE.
 - ALL PRECAST PANEL PERMANENT DECK REINFORCEMENT AND TEMPORARY TRUSSES SHALL BE CO-ORDINATED BY THE CONSTRUCTOR TO SUIT THE LOCATION OF THE SHEAR STUDS ON STEEL GIRDERS.

NOTES:

- TEMPORARY TRUSS DESIGN.
- DESIGN OF THE PRECAST PANELS AND THE TEMPORARY REINFORCING TRUSS (INCLUDING TRUSS LOCATION) IS THE CONSTRUCTOR'S RESPONSIBILITY. AS DEFINED BY BA 36/99 AND A33610, THE CONSTRUCTOR SHALL ALLOW FOR ALL CONSTRUCTION LOADS PLUS THE DEAD LOAD IN THE TRUSS DESIGN, INCLUDING THE EFFECTS OF CONCENTRATED LOADING BOTH ON THE PANEL AND LOCALLY ON THE TOP TRUSS MEMBER. THE CONSTRUCTOR SHALL SATISFY THEMSELVES THAT THE PRECAST SLAB AND STEEL TRUSS TEMPORARY WORKS HAVE SUFFICIENT STRENGTH AND STABILITY DURING ALL STAGES OF LIFTING AND ERECTION. NOTE THAT THE TEMPORARY WORKS IS NOT PART OF THE DESIGN PROCESS AND IS ALSO NOT COVERED BY THE PEER REVIEW OF THE DESIGN. IT IS THEREFORE REQUIRED THAT THE TEMPORARY WORK ENGINEER SATISFY THEMSELVES THAT THE PRECAST SLAB AND THE STEEL TRUSS SYSTEM ARE ADEQUATE FOR THE TEMPORARY STATE UNTILL THE DECK IS MADE COMPOSITE AND CAN SUPPORT ITS SELF WEIGHT.
 - THE CONSTRUCTOR SHALL SATISFY THEMSELVES THAT THE TRUSS DESIGN IS ADEQUATE WITH 100MM SEATING SUPPORT ON THE GIRDERS. THE SUPPORT CONDITION ASSESSMENT MADE BY THE CONSTRUCTOR SHALL INCLUDE AN ALLOWANCE FOR GIRDER VERTICAL POSITION TOLERANCE, SLOPING TOP FLANGE TOLERANCE, AND GIRDER PRECAMBER.
 - THE TEMPORARY REINFORCING TRUSS IS CONSIDERED REDUNDANT FOR THE PERMANENT WORKS DUE TO FATIGUE ISSUES CAUSED FROM WELDING OF THE TRUSS BARS.
 - THE CONSTRUCTOR SHALL PROVIDE ADEQUATE LATERAL SUPPORT TO PREVENT THE PRECAST PANEL FROM DISPLACING OFF THE STEEL GIRDER FLANGE.
 - THE CONSTRUCTOR SHALL COORDINATE THE TEMPORARY REINFORCING TRUSS WITH THE PERMANENT GIRDER SHEAR STUDS, PERMANENT DECK SLAB REINFORCING, SERVICE INSERTS, DRAINAGE CATCHPITS AND ANY OTHER STRUCTURAL ELEMENT (PERMANENT OR TEMPORARY) TO AVOID CLASHES ON SITE.
 - THE TEMPORARY TRUSSES SHALL BE CAMBERED AS REQUIRED TO LIMIT THE TOTAL SLAB DEAD LOAD DEFLECTION TO 10MM MAXIMUM DIFFERENCE BETWEEN THEORETICAL PRECAST SLAB DEFLECTION AND THE THEORETICAL STRAIGHT LINE BETWEEN TOP FLANGE EDGES OF ADJACENT GIRDERS. PANEL DEFLECTIONS EXCEEDING THIS DEFLECTION LIMITATION SHALL BE DISCUSSED AND ACTION AGREED WITH THE DESIGNER



FOR CONSTRUCTION

| | | | | | |
|-----|------------------|----|-----|------|----------|
| No. | Revision | By | CHK | Appd | Date |
| 1 | FOR CONSTRUCTION | | | | 24.09.18 |

| | | | |
|---------------------|----------------|----------|---------------------------|
| Original Scale (A1) | Design | 16.10.17 | Approved For Construction |
| AS SHOWN | Drawn | 09.11.17 | |
| Reduced Scale (A3) | Design Checker | 27.09.18 | |
| HALF SIZE | Design Checker | 27.09.18 | |



PŪHOI TO WARKWORTH MOTORWAY



| | |
|---------|---|
| Subject | 12-12 PŪHOI VIADUCT |
| Title | PRECAST DECK PLANKS REINFORCEMENT DETAILS - SHEET 3 |

| | |
|-------------|---------------------|
| Discipline | STRUCTURAL |
| Drawing No. | P2Wk-DRG-SE-18-0107 |
| Rev | 1 |

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From: s 9(2)(a)
Sent: Friday, 6 March 2020 10:58 AM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Welded rebar in bridge decks - confidential

Hi Barry,

s 9(2)(a) had a meeting with Acciona yesterday.

Please see my in-house advice (in quotes) for the further confidential notification to NZTA of further safety issues associated with the proposed deck system for the Puhoi and Okahu Viaducts.

“Not raised yesterday was how the Contractor (Fletcher/Acciona) intend to protect the top chords of the welded reinforcement trusses from damage anytime between now (2/3's of welded reinforcement trusses are on site now) and when the topping is poured?

Considering the two viaducts I estimate over 38km of top chords are in compression and will have their maximum axial compressive loading when the concrete topping (205mm) is being poured. Any damage resulting in kinks or misalignment of HD20 chord will dramatically reduce the buckling capacity (i.e. a sudden failure) due to P-delta effects.

Acciona claim they have used this method of deck construction before, but I would suggest with the best intentions in the world, it is highly likely some DH20 top chords will be damaged by construction loading preparing for the topping pour. A failure of one top chord in buckling will shed additional compression load onto the adjacent top chords. This may lead to a progressive failure of a particular 2.7m wide precast panel. This is irrespective of the theoretical buckling capacity of a perfectly straight HD20 top chord as Acciona Ingeneria have based their calculations on. Given the extent of top chords it is impractical to expect pure axial compression; there will be bending moments and moment magnification leading to sudden failure.

In my view the deck system as designed has an unacceptably high risk of failure during construction (as above); and an unacceptability high risk of premature durability failure in the service load condition (see my previously outlined description of failure mode). This is despite of the claims that 'it's been done before' and 'almost every engineer in NZ' has reviewed it.

I estimate Acciona have over 200t of trusses onsite already at say \$1M cost; so they have an investment to be protected plus any other associated costs with rework. BECA and Aecom have reputations to protect. Jacobs and GHD will be defensive about not picking up these issues on behalf of the NZTA.

If the NX2Group cant gather enough evidence to assure the NZTA (Principal and Regulator) that the two failure mechanisms I have outlined are not valid, then they should notify the NZTA immediately of the two failure mechanisms.

Ethically s 9(2)(a) has a duty of care to alert any safety issues; and not review the work of others without first notifying them.”

Ethically raising these issues is the correct thing (they are valid based on my engineering know-how) to do but in the NZ bridge engineering network it can also be extremely detrimental due to protection of reputation. Therefore the NZTA should continue to satisfy themselves as both Principal and Regulator to assure the proposed viaduct deck system is safe and durable. In my response to Acciona I will encourage them to notify NZTA of our concerns but understand the commercial motivations may make this difficult for them.

Kind regards

s 9(2)(a)

s 9(2)(a)



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From: s 9(2)(a)
Sent: Wednesday, 4 March 2020 8:50 AM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Welded rebar in bridge decks - confidential

Hi Barry,

I have now received the DPS and the Construction issue drawings for both the Puhoi and Okahu Viaducts from the Contractor.

The DPS states at the bottom of page 4 "The innovative deck design has been used overseas and is considered a significant safety improvement during construction (requires less working at height time)."

If the NZTA investigates these overseas bridges I would be interested in their findings.

I hold to my thoughts below on the deck system.

Cheers

s 9(2)(a)

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From: s 9(2)(a)
Sent: Tuesday, 3 March 2020 10:34 AM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Welded rebar in bridge decks - confidential

Hi Barry,

Last night I have received drawings issued for detailed design for the Puhoi Viaduct from an undisclosed source; not from the Contractor.

The following pertinent points can be made from drawing P2Wk – DRG -SE-18-0107 Rev A dated Aug 2017.

1. The designers intent was that temporary reinforcing truss (i.e. That is the heavily welded 500 grade rebar trusses from Spain?) were only required for strength during construction, but would remain within the permanent deck slab.
2. The designer intends the temporary reinforcing truss to be considered redundant for the permanent works due to fatigue issues caused from welding of the truss bars. E.g. my concern.
3. The designer intent was the temporary truss system design to be finalised by the Temporary Works Engineer and checked with the Design Engineer.

My thoughts:

4. Given what is on the drawings has got to construction I would assume the Peer Reviewer has scrutinised and approved this.
5. The likely failure mechanism of this deck system should be checked by fatigue trials or previous examples.
6. My hypothesis of the probable failure mechanism is as follows:
 - a. The deck slab spanning transversely across the 4 girders and cantilevering outwards is very stiff with both the required deck reinforcement and the additional stiff temporary reinforcing truss. The truss chords are at the same depth as the deck slab transverse bars. I assume no reliance has been placed on the temporary reinforcing truss in the design of the ordinary slab reinforcement.
 - b. After a period of service loading the temporary reinforcing truss begins to crack due to fatigue loading at quite low cyclic stresses. The truss is likely to snap in regions of highest strain. That is over the girders (hogging moment) and between the girders (sagging moment) but breaks could be at any weld or stress riser along the length of the trusses. The location of fatigue cracking is practicably unpredictable.
 - c. Once the trusses break then the stiffness of the deck slab is much less relative to the remainder of the deck slab.
 - d. Further traffic loading will concentrate deck rotations at these locations of low relative stiffness and higher strain. The deck slab will not deflect like it is monolithic with constant stiffness.
 - e. At the points of lower stiffness; higher deck rotations; and subject to high cyclic load concentrations there will be considerable 'working' of the broken temporary reinforcing truss within the deck slab. Corrosion pockets at the ends of the broken bars and within the slab will develop.
 - f. I suspect there will end up with larger than normal longitudinal cracks above the girders and along the underside between the girders. Corrosion of the shear studs and loss of composite action could happen over a longer period of time as the corrosion pockets are likely to be directly over the girders.
7. The deck system is certainly novel for New Zealand. Expecting bars to break due to fatigue within a permanent bridge deck requires a great deal of investigation prior to implementation. I think it imprudent as per point 6 above.
8. I suspect the Designer has now realised what was originally thought to be a good idea allowing for fewer widely - spaced girders, now has durability and safety concerns; hence getting the Contractor to have another Consultant to sign-off on this deck system. My advice to my s 9(2)(a) colleagues (who are not bridge experts) dealing with the Contractor is that we need to fully understand the

design intent and process for the deck and the use of the temporary reinforcement trusses is highly doubtful; therefore no sign-off is likely – which is what the Contractor was seeking. I suspect the Contractor will find someone else. That is the NZTA should not expect s 9(2)(a) to be commissioned to sign-off the temporary reinforcing truss design; I certainly wont be signing it off.

9. I understand the programme is now 11 weeks behind schedule.
10. A deck solution is not likely to involve reuse of the existing temporary reinforcing trusses which will have cost the Contractor plenty hence the need to maintain confidentiality.
11. Given the commercial imperatives at play then the NZTA needs to either gain confidence in the proposed deck system or develop a new deck system and check the superstructure and substructure.
12. The span of 7.3m between girder centres and programme will be a significant factors in a practical deck solution.

Would the NZTA like s 9(2)(a) to help investigate a deck solution for both of these viaducts? We are now on s 9(2)(a) if you wish to discuss further.

Kind Regards

s 9(2)(a)

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From: s 9(2)(a)
Sent: Monday, 2 March 2020 2:45 PM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Welded rebar in bridge decks - confidential

Hi Barry,

Confidential

Thank you for the NZTA's support of my questioning.

At this stage I'm not entirely clear on who the designer, or peer reviewer are or what the design process was hence my request for confidentiality. Clearly there are significant commercial imperatives at play as the welded rebars are on site (from Spain I think) ready to be cast into the decks and the project is already well behind programme. We were asked by Acciona Ingenieria to approve the welded rebar detail because the designer wouldn't sign off.

My bold assumption (and hope) is that the NZTA would be insuring they end up with 100-year design life bridges that are safe and durable.

The bridges concerned are the Okahu and Puhoi Viaducts on the PPP P2WK.

I'm also aware of the current litigious actions on the designers for the Transmission Gully project; the other PPP going on at the moment. Hence I reiterate the need for confidentiality.

I can share what information I have with the NZTA for them to make their own inquires but I wish my notification of the NZTA to remain strictly confidential.

Given my experience raising the absence of transverse web stiffeners (in regions of high shear) on the interior girders of the Atiamuri bridge (because AS5100.6 is the only bridge code I know that doesn't require them!) I was therefore tentative to raise this as an issue with the NZTA. I also raised the extensive use of permanent ASTM A490 bolts on 7 newish (2016) railway bridges with KiwiRail and this has been extremely detrimental to my once flourishing career. These high strength bolts are prone to Hydrogen Embrittlement (HE) and generally prohibited from permanent bridge works in North America where the bolts are made. The NZTA as Rail Regulator took KiwiRail's assurance that the bolts are OK.

Over to the NZTA.

Kind regards

s 9(2)(a)

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From: s 9(2)(a)
Sent: Monday, 2 March 2020 1:28 PM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Cc: s 9(2)(a)
Subject: RE: Welded rebar in bridge decks

Hi Barry and s 9(2)(a)

The rebar concerned is not from NZ. Its yet to be established if the bars meet the NZS 4671 specification – a prerequisite for NZS 1554.3 Section 4 testing. The WPS provided is not in English but section 4 would require much more information.

The bars concerned are marked as both 500E and 500N.

Of more fundamental concern is the that bars are welded about every 200mm along their length; they are aligned transversely across the bridge (both top and bottom within the deck slab) and therefore subjected to 'feeling' every axle load. The combination of bar embrittlement (from welding whether approved or not) and high frequency fatigue loading is our primary concern. Notwithstanding this is the fact that such a potentially fraught detail can get into detailed design, pass peer review and is now about to be constructed is also serious given the safety concerns and probability of brittle failure under load of any one of hundreds of critical bars.

The practical and safe answer is that such deck reinforcement was not advisable to begin with because of weld embrittlement and fatigue loading; but to get to imminent construction it seems prudent that there should have been some level of NZTA approval required hence my question.

I have requested s 9(2)(a) speak to the designer and peer reviewer and review the DPS to fully understand the design rationale for the deck slab design.

Cheers

s 9(2)(a)

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From: s 9(2)(a)

Sent: Monday, 2 March 2020 11:55 AM

To: Barry Wright <Barry.Wright@nzta.govt.nz>

Subject: Welded rebar in bridge decks

Hi Barry,

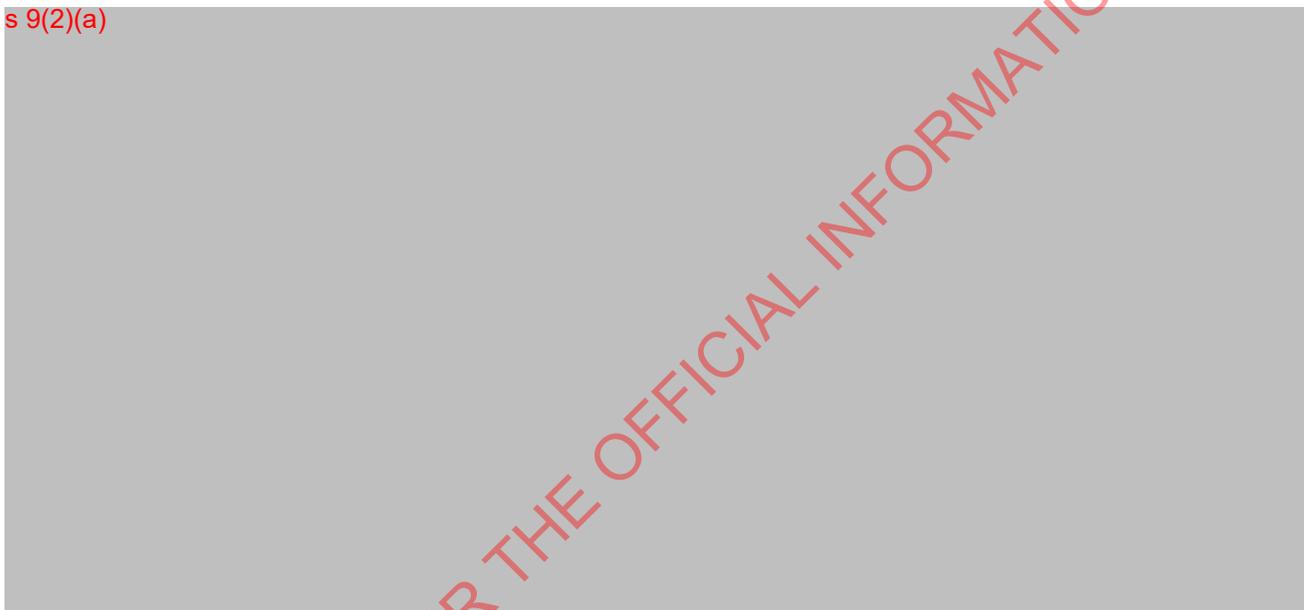
Its come across my desk an urgent request to approve welded (500 Grade) rebars for a concrete bridge deck for a highway bridge currently under construction in NZ.

This practice is certainly not recommended interpreting the Bridge Manual and NZS3101 even if NZS 1554.3 allows it to be carried out.

I wanted to know if the NZTA has given any such approvals to use welded rebars in any bridge decks currently under construction?

Kind regards

s 9(2)(a)



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