

From: s 9(2)(a)
Sent: Sunday, 14 June 2020 1:21 PM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Query P2W Viaducts

Hi Barry,

Is there any opportunity for s 9(2)(a) as a bridge (road and railway bridges) design review engineer working on contract exclusively to the NZTA as bridge regulator?

The exclusive arrangement is to ensure technically competent design reviews (and designs if necessary) without s 9(2)(a) having to “endorse” the designs of others in order for commercial reciprocity.

In the early days (2004 to 2014) of Novare Design this exclusive arrangement worked well with Ontrack and then KiwiRail delivering economic and well-engineered railway bridges free from increasingly common design defects that occur as a result of “deals”.

Please let me know and we can discuss.

Kind regards

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From: s 9(2)(a)
Sent: Thursday, 28 May 2020 5:30 PM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Query P2W Viaducts

Hi Barry,

Following today's discussion I retract both offers of extra information given below. I can be engaged to give expert advice as I have no vested interest in the outcome. Ethically I have done everything to discharge my duty of care. I do not want to see anyone get killed while standing on these clearly under-designed deck panels. This may be construed as my 'opinion' but I would prefer to let the engineering calculations, rationale and any future proper load testing speak to the first failure mode I have raised.

Cheers

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From: s 9(2)(a)
Sent: Monday, 25 May 2020 11:02 AM
To: 'Barry Wright' <Barry.Wright@nzta.govt.nz>
Subject: RE: Query P2W Viaducts

Hi Barry

The first attachment is my report I wrote to NX2 (Acciona) dated 16 March 2020. This version was not sent. I have redacted all references to my employer at the time I wrote the report. This attachment is for the NZTA as regulator.

The 2nd to 4th attachments are 3 pages of calculations dated 31 March 2020 showing a working stress approach that shows the top chord of centre precast panel of the Puhoi Viaduct (only) is significantly overstressed (and likely to buckle and lead to progressive buckling failure) under the initial topping pour (23% overstressed), allowance for ponding due to excessive deflection (36% overstressed) and if there is a misalignment or kink of just 2mm in the 200mm long chord segments (119% overstressed). The final page of the calculations has a sketch of how the existing Okahu deck panels could be re-load tested with a simple set up to prevent the kentledge contributing the bending strength of the panels. These calculations have been sent to Acciona as they requested them.

The panels should be tested to failure and some panels should have the top chords in the mid span region deliberately damaged to varying degrees in order to demonstrate how vulnerable they are to sudden buckling failure during construction. This would be a practical way forward to resolving the first and most critical failure mode outlined in the attached report.

If necessary I can further supply calculations that identify the first 'failure mode' of the Okahu viaduct deck panels during construction.

If necessary I can further supply calculations that identify the second 'failure mode' of the viaduct deck panels during service.

Cheers

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Okahu & Puhoi Viaducts

Deck System Review

16 March 2020

For NZTA

s 9(2)(a)

A large grey rectangular redaction box covers the majority of the page content below the 's 9(2)(a)' label.

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Executive summary

- Upon a request by NX2 (“NX2Group” – PPP special purpose vehicle to deliver the P2Wk project for the NZTA) for [REDACTED] to peer review a temporary works component associated with the deck construction for both the Okahu and Puhoi viaducts two significant deck failure modes have been discovered. The temporary works concerned is critical for both modes of failure.
- By calculation and assessment [REDACTED] have confirmed these deck failure modes are serious. [REDACTED] first notified NX2 of these potentially dangerous failure modes on 6 March 2020 by email. Because of the deck failure modes discovered, [REDACTED] are unable to peer review the Temporary Reinforcement Truss (TRT) as requested by NX2.
- This report formally outlines to NX2 two potential failure modes associated with the viaduct decks. One failure mode is associated with the sudden buckling failure during construction of the TRT’s cast into the precast deck panels. The other failure mode is associated with the fatigue fracturing of the TRT’s embedded into the permanent viaduct deck leading to an accelerated decline in deck durability during its service life.
- The first and most serious failure mode eventuates from a TRT top chord buckling failure. This can potentially occur either in preparation or during the topping pour. Over the two viaducts there are over 17,400m² of precast deck panels that could collapse suddenly and fall nearly 20m to the lowest point of ground level. The sudden nature, high chance and consequence of precast deck failure render the proposed deck system as an unacceptable risk. It is strongly recommended that NX2 prove the resilience beyond doubt and demonstrate adequate margins of safety of the deck system before proceeding further with the viaduct deck construction.
- The second failure mode comes about from the sudden changes in deck stiffness caused by the design intended and highly likely fatigue fractured TRT’s cast into the permanent deck slabs. This failure mode is subordinated to the first only because should the first occur then this will prevent the viaduct decks being constructed as currently intended. Suffice to say should NX2 prove the resilience and safety of the precast system, then it is incumbent upon them to further prove the embedded TRT’s have no detrimental effect on the behaviour and durability of the permanent deck.
- Seven recommendations have been put forward. There is an obligation on NX2 to fully demonstrate to themselves that the two failure modes outlined are either unfounded or can be remedied. It is further suggested that work begin immediately on developing an alternative deck solution and reassessing the existing superstructure and substructure designs.

1 Background

This report has been prepared by [REDACTED] solely for NX2.

The purpose ("Purpose") of this report is to highlight to NX2 the serious concerns held by [REDACTED] with respect to the design of the TRT's and their role in the overall viaduct deck system. The permanent deck and superstructure design have not been considered as part of this report except as they pertain to the TRT's.

This report is confidential and is prepared for [REDACTED] NZTA as Principal and Regulator. The contents of this report may not be used for any other purpose other than in accordance with the stated Purpose, nor may this report be relied on by any other party.

NX2 commissioned [REDACTED] to review the TRT design. [REDACTED] understands there was a request from the NX2 designers ("Beca") that the TRT's should be further reviewed following the required approval by the Temporary Works Engineer ("TWE"). The TWE is Acciona Ingerieria ("A ING"). They are a Spanish subsidiary of Acciona Infrastructures ("A INFRA") – one of the Design and Construction Joint Venture partners ("D&C JV") that make up NX2.

A ING carried out detailed design and a supplementary annex for the design of the TRT's cast into precast deck panels as 'strong backs'. The design report is dated Feb. 2018 and the Annex (for shear capacity improvements of the TRT's) is dated Feb. 2020. The Annex is further supplemented by a description of the weld procedure specifications ("WPS") but this is not dated as such. The actual WPS is dated 29 Sep. 2017 and the reinforcement bar mechanical testing is dated 11 Oct. 2018.

The construction drawings numbered P2Wk-DRG-SE-19-0107 Rev. 1 and P2Wk-DRG-SE-18-0107 Rev. 1, for the Okahu and Puhoi viaducts respectively, and both dated 27 Sep. 2018, called for the TWE to 'satisfy themselves' that the TRT's are adequate for the temporary state when the 185mm (for Okahu viaduct) or 205mm (for Puhoi viaduct) thick topping concrete is poured onto the precast deck panels.

The precast panels are intended to span transversely across the 4 girder lines of each viaduct. The girders are numbered 1 to 4 from one side to the other. The typical 'outer' precast panels are 130mm thick and supported on girders 1 and 2, and 3 and 4, and include a cantilevered portion of deck beyond girder 1 and 4. The typical 'centre' panel is also 130mm thick simply supported between girder 2 and 3.

All precast panels are stiffened with TRT's at 300mm centres across the typically 2.7m panel width. Each TRT has a single HD20 rebar top chord and two HD20 rebar bottom chords. The vertical centroid distance between top and bottom chords is 154mm for the Okahu viaduct and 174mm for the Puhoi viaduct. The chords are held apart by diagonal HD10 bent 'zig-zag' diagonals which are welded (4mm fillet to the outside and inside bend) to the chords at 200mm centres. Typical TRT's are shown in Photo 1.



Photo 1: Typical Temporary Reinforcement Trusses prior to the bottom chords being cast into the 130mm thick precast deck panel.

█ understands only the precast panels for the Okahu viaduct have been cast to date. These have been cast at Wilsons Precast, Heritage Way, Otara, Auckland. Photos 2 and 3 below show some of these panels and these photos were taken on 12 Mar. 2020. █ are unsure how many of these precast panels have been cast to date.



Photo 2: Typical centre precast deck panels stacked and ready to be transported to the Okahu site.



Photo 3: Typical outer precast deck panels stacked and ready to be transported to the Okahu site.

█ understands the precast panels cast to date have been made to the A ING construction issue drawings (No. 1 to 7) and without date stamp.

As clearly stated in the construction drawings numbered P2Wk-DRG-SE-19-0107 Rev. 1 and P2Wk-DRG-SE-18-0107 Rev. 1, the TRT's are to remain within the permanent deck and to become 'redundant' during service loading. Due to their welded fabrication they are expected to suffer from fatigue fractures through the chords and diagonals.

2 Concerns with the Precast Deck System

█ wish to outline two significant concerns associated with the TRT's as integral components with the total viaduct deck system. Namely the prospect of sudden failure during deck construction resulting in the collapse of precast deck panels, and secondly the durability failure of the permanent deck during service life caused by the effects of the fatigue fractured TRT's.

The first is of highest priority as it is a potential safety issue.

These concerns prevent █ from approving the TRT design as requested by NX2.

These concerns should be addressed by the NX2 before erecting the viaduct deck system.

These concerns are more completely described as failure modes in Sections 3.1 and 3.2 below.

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3 Failure Modes of the Precast Deck

3.1 Sudden Top Chord Buckling Failure during Construction

- Let us consider the centre precast panel spanning 6.25m between girders 2 and 3 top flange outstands of the Okahu viaduct and 6.95m span for the Puhoi Viaduct. The model for analysis is a simply supported span. Each 2.7m wide panel has 9No. TRT's at 300mm centres providing the flexural support ('strong backs') to carry the panel self-weight; construction loading during the topping pour preparation; and the wet concrete topping pour.
- These loads are resisted by the TRT's in flexure which is maximum at midspan and by shear which is maximum at the main girder supports. The TRT top chords are in compression and bottom chords are in tension. There is additional permanent reinforcement within the 130mm thick precast slabs that also contribute to the tension action of the moment resisting couple within the span. The tension capacity of the precast slab has been ignored because it yields suddenly whereas the reinforcement is expected to elongate and yield in a ductile manner prior to failure.
- When the TRT top chord compression stresses are sufficiently high, then the HD20 top chord will buckle suddenly. That is a sudden loss of the moment resisting couple that was provided by the statically determinate TRT's. The TRT's have no redundant load paths. The top chord buckling capacity is governed largely by its slenderness ratio. The slenderness ratio is the top chord effective length over its radius of gyration; which for a 20mm diameter bar is 5mm.
- Selecting the top chords effective length requires both guidance from codes and engineering judgement from the designer or reviewer.
 - The completely unconservative selection of 'fixed – fixed' end restraints will give an unconservative top chord buckling capacity. The assumed end fixity will also attract additional bending stresses within the top chord due to possible misalignments between restraints or damage (cranks or kinks) within the chords. The damage may come about due to the precast panel construction process or preparation of the topping pour when the top permanent reinforcing is placed. Allowance for some degree of top chord misalignment or damage is considered prudent when using a fixed – fixed model. ■ calculations show the top chords are vulnerable to sudden buckling with just a 2mm misalignment between restraints 200mm apart or a 2mm kink within the top chord. The combination of axial and bending stresses reduces the unconservativeness associated with the fixed-fixed model.
 - A reasonable selection of 'pinned-pinned' end restraints more accurately accounts for the fact that the top chord acts as a series of 200mm long chords. It is slightly conservative in that it assumes the restraints have negligible torsional restraint. ■ calculations show all panels for both the Okahu and Puhoi viaducts will be loaded beyond typical safety margins used to prevent top chord during the topping pour. As with the fixed – fixed model the top chords are vulnerable to damage which introduce P-delta effects and dramatically reduce buckling capacity compared to an ideal straight chord.
 - ■ considered the unlikely situation of a missing restraint and therefore an effective length of 400mm. This indicated the TRT top chord would buckle for all precast panels of both the Okahu and Puhoi viaducts during the topping pour.
- There are about 6,500 TRT top chord compression regions in the Okahu and Puhoi viaducts with a combined area of precast panels exceeding 17,400m². This represents a high probability of sudden top chord buckling and leading to progressive collapse of any 2.7m wide panel. Progressive collapse occurs as any buckled top chord quickly sheds its lost axial capacity to the TRT top chords immediately adjacent thus overloading them leading to their buckling.

- The maximum falling height of both viaducts is about 20m. This represents unacceptable life-threatening consequences of failure.
- The high probability of failure coupled with the potentially life-threatening consequence of failure represents an unacceptable risk which must be fully addressed before further construction of the design deck system proceeds.
- Additionally, the precast panels appear not to have been cambered and are flat. This can be seen in Photos 2 and 3 above. [REDACTED] calculate by simple elastic theory that the Okahu centre panel have midspan deflections of over 30mm occurring during the topping pour. This will cause 'ponding' of the topping concrete and exacerbate both the deflections and the compression of the TRT top chords. Any mounding of the topping concrete during the pour will further exacerbate deflections and top chord compressions although temporarily.
- No solutions to prevent this failure mode have been presented at this stage.
- [REDACTED] have received load testing results for a centre and an outer precast panel of the Okahu viaduct. The load tests were required by drawing P2Wk-DRG-SE-19-0107 Rev. 1. [REDACTED] have reviewed a package consisting load test planning documents, emailed results, and photographs for the load testing carried out on the 23rd and 27th Aug. 2019. Generally, the measured deflections were slightly less than half that expected by elastic theory. This suggests the deck panels were effectively twice as stiff as would be expected using typical material properties. This further suggests that the formwork panels (from plywood) placed direct onto the TRT top chords are clamped to the top chords by the kentledge (precast segment stacking cradles of 4.0tonne weight and filled Traffix Water-Wall barriers each of 0.5tonne weight) during load testing formed a rudimentary but effective composite top flange to stiffen the precast panel; and thereby result in abnormally low deflections. It is suggested if further load testing is carried out on the precast deck panels, then the kentledge is completely separated from the TRT top chords, and it is placed such that no possible 'running' or longitudinal shear connection exists between the TRT's or precast panel and the kentledge. The precast panels should also be tested to failure so that the failure mode may be recorded. Some precast panels should be load tested with deliberately damaged (kinked) or misaligned TRT top chords to demonstrate a likely scenario. The outer panels should be loaded first between the supports, and only then on the cantilever as this loading reduces compression in the TRT top chords spanning between the supports.



Photo 4: This shows load testing of an Okahu centre panel.

- NX2 has supplied [REDACTED] with a briefing paper generally outlining bridge deck systems similar to that described in the construction drawings. They include the Omnia Bridge deck system and 5 other bridge examples plus load testing outlining of a 6th bridge. The Omnia bridge system is used up to 3.7m spans with the planks most often in the longitudinal orientation; this reduces the number of potential fatigue cycles. The planks are only a single truss wide to prevent the risk of progressive collapse. Omnia alert contractor's attention to prevent damage to top chords by lifting only at points of top chord restraint; this is to reduce the risk of top chord buckling. The other bridge deck examples shown have spans of an estimated 4m plus a 2m relieving cantilever on each

end; 2.45m simply supported; 2.0m continuously supported; 3.0m and 2.0m plus 1.5m relieving cantilever; and 2.2m simply supported. The load tested panels are most greatly loaded on the relieving cantilever; this reduces the adjacent maximum simply supported moment by at least half the cantilever moment. Generally, the examples shown have much smaller spans to that proposed for the Okahu and Puhoi viaduct precast panels. The top chord compression is in proportion to the square of the span divided by the depth of the TRT. A pure simply supported precast deck panel of a particular span length will have larger TRT top chord compressive stresses than when compared to the same span but with the TRT also supporting a cantilever.

3.2 Durability Failure during Service Loading

- Let us consider the 315mm or 335mm thick permanent reinforced concrete (RC) decks with the TRT's cast within. According to construction drawings numbered P2Wk-DRG-SE-19-0107 Rev. 1 and P2Wk-DRG-SE-18-0107 Rev. 1, Beca consider the TRT's are to become redundant due to fatigue considerations. [REDACTED] agree with this and estimate fatigue fracturing of the TRT's will be within 1 year of opening to traffic. This is based on an assumed fatigue detail classification (which considers the weld embrittlement, eccentricity of between the chord and diagonal creating an abrupt stress riser) and an estimated number of load cycles. The chord stresses due to the cyclic live loads can be compared to the allowable fatigue stress range calculated from the assumed fatigue detail classification and estimated load cycles. The actual stress range within the chords is greater than the allowable fatigue stress range after just one year of traffic.
- Fatigue fracturing of the TRT's is most likely to occur first at points of high bending moment; that is in the top chords over the girders and in the bottom chords between the girders where stress ranges are greatest. The initial stiffness of these relatively thin decks is governed by the unfractured TRT's and the permanent reinforcement. See Photo 5 below. At points of TRT chord fracture there will be points of sudden change in deck stiffness.
- At the deck regions immediately adjacent a sudden change of stiffness due the TRT fatigue fracture, there will be much sharper curvatures in the deck compared to the deck sections further away from the change in stiffness. By linear elastic beam theory, the inverse of radius of curvature is equal to the inverse of the area of effective reinforcement, for any applied constant moment and material elastic modulus. That is the bending in the deck slab will be tighter at points of reduced effective reinforcement.
- The permanent deck is therefore expected to behave like a series of stiffer links connected by less stiffer connections at points where bending moment is highest. The higher strains associated with the sharper deck curvature will cause cracking in the deck cover concrete. That is to top cover concrete above the girders and the bottom cover concrete between the girders.



Photo 5: This shows one of the less typical Okahu deck panels in the process of being tied. It shows the extent of reinforcement within the deck panels made-up with the TRT's (shown rusted) and the permanent top reinforcement (shown grey).

- Further cyclic traffic loading will also result in the fractured TRT chords 'working' within the deck creating cavities for moisture to collect. This could cause premature corrosion of all the reinforcement within the deck close to these locations.
- It is difficult to estimate the reliable life of the deck, but it is not expected to behave as a typical RC bridge deck slab with only gradual changes of stiffness as transverse reinforcement is carefully curtailed to suit moment demands.
- No solutions to prevent this failure mode have been presented at this stage.
- No evidence of past performance of this type of deck with embedded and fractured TRT's have been presented at this stage.
- To reiterate [REDACTED] have made no analysis of the permanent deck.

4 Recommendations

█ strongly recommends the following actions for NX2 to undertake in regard of the Okahu and Puhoi Viaduct deck system:

1. Act urgently and decisively in re-evaluating the deck system as shown on the construction drawings P2Wk-DRG-SE-19-0107 Rev. 1 and P2Wk-DRG-SE-18-0107 Rev. 1, and the two potential failure modes outlined in this report. See Sections 3.1 and 3.2.
2. Request the A ING as TWE to re-calculate the TRT top chord buckling capacity and margins for safety during the work associated with preparing for the topping pour and pouring the topping. It is suggested the tension capacity of the 130 thk concrete precast be ignored in this re-calculation. The nature, probability and consequences of a sudden precast deck failure demand healthy margins of safety. It is recommended they account for some top chord damage in determining buckling capacity to practically reflect reality.
3. Conduct further load testing of the already cast Okahu viaduct precast deck panels. Ensure the panels are accurately supported to mimic loading of the topping pour as accurately as possible. Ensure kentledge offers no lateral restraint to the TRT top chords or provides any top flange effect to authentically mimic the wet concrete topping loading. Test panels with undamaged TRT top chords and those with deliberately damaged or misaligned TRT top chords. Test to failure and note the failure mode. Place kentledge only between the lines of support first; not on the cantilever portion of the outer panels. Record loading and deflections accurately. It is suggested that a professional testing agency or the University of Auckland Civil Engineering school be engaged to assist NX2 develop a thorough load testing regime for the precast deck panels.
4. Continue to gather evidence in support of the proposed deck system with embedded TRT's. The evidence should be based on similar spans (6m to 7m) and for similar sized TRT's.
5. Gather evidence of the designer's design philosophy and calculations and peer reviewer's approval pertaining to the deck system currently under construction.
6. Investigate options for an alternative deck system. This should be carried out in parallel with all recommendations above. An alternative deck system investigation should include a detailed re-evaluation the superstructure currently being erected, an assessment of the existing substructures, and confirmation of the proposed deck dimensions for both viaducts.
7. Following point 6 above the development of a new construction programme and cost estimate associated with a recommended alternative deck system.

5 Conclusions

█ were engaged by NX2 to peer review and approve the TRT's after they had some repair work carried out on them. The viaduct designers requested an independent New Zealand based engineer to approve them. Until then A ING of Spain, as the TWE, had approved them.

█ undertook a first principles analysis of the TRT's to be approved and their functional roles within the total viaduct deck system.

From investigation of the viaduct construction drawings the dual roles of the TRT's are:

- Act as 'strong backs' to allow the thin precast panels to span over and cantilever beyond the 4 girder lines without temporary works;
- To fracture in fatigue and become redundant within the thicker permanent viaduct deck system.

█ discovered that the TRT's perform a limiting or critical role in the deck system during construction and in service.

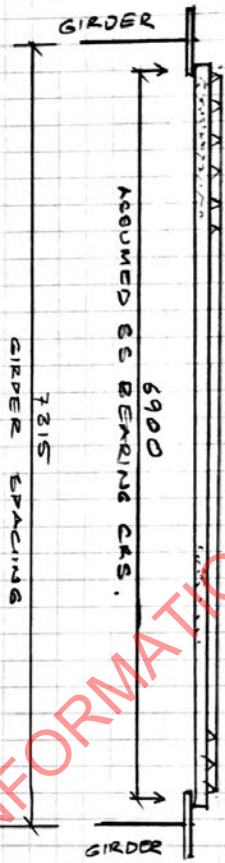
█ concludes the TRT's are critical to two potential failure modes; one sudden during construction and the other resulting in the accelerated decline of deck durability.

█ further concludes that NX2 should fully address these two potential deck failure modes. Section 4 of this report suggests 7 recommendations for NX2 to undertake prior to the proposed deck system being erected onto the girders.

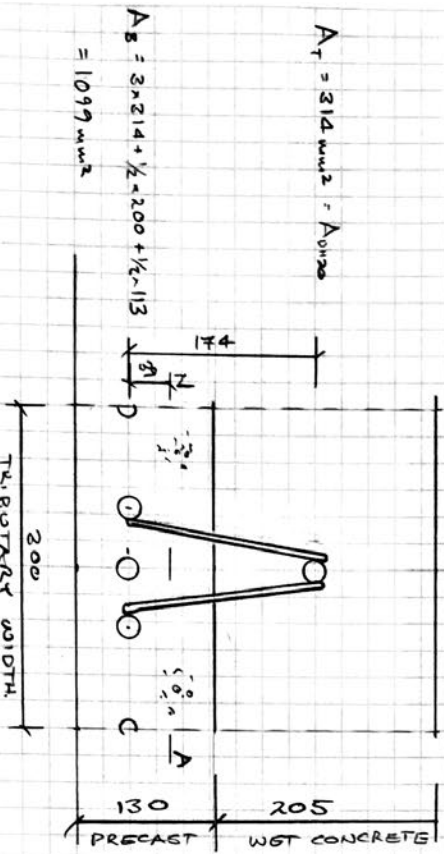
█ are unable to approve the TRT's given their critical contribution to the two possible failure modes outlined in Section 3 of this report.

PUBLIC VIADUCT - CENTRE PANEL
 SIMPLY SUPPORTED MODEL

BOTH PRECAST PANEL WITH 174 DEEP TRUSSES @ 200 C/S.



CROSS SECTION



$$A_T = 314 \text{ mm}^2 = A_{\text{truss}}$$

$$A_B = 3 \times 214 + \frac{1}{2} \times 200 + \frac{1}{2} \times 113 = 1099 \text{ mm}^2$$

WORKING LOAD

$$\text{WIDTH} = 0.8 \text{ m}$$

$$\text{DEPTH} = 0.235 \text{ m}$$

$$\text{LOAD} = 25 \text{ kN/m}^2$$

$$\text{WORKING UDL} = 0.8 \times 0.235 \times 25 = 2.5 \text{ kN/m}$$

$$\text{MIDSPAN M} = 6.7^2 \times 2.5 / 8 = 14.9 \text{ kNm}$$

SECTION PROPERTIES

$$\text{AREA} = A_T + A_B = 314 + 1099 = 1413 \text{ mm}^2$$

FIRST MOMENT OF AREA - ABOUT BOTTOM CHORD

$$EAY = 214 \times 174 = 54,636 \text{ mm}^3$$

HEIGHT OF NEUTRAL AXIS

$$\bar{y} = 54,636 / 1,413 = 39 \text{ mm}$$

MOIETY PROPERTIES

$$I_{\text{top}} = \pi \frac{20^4}{64} = 7854 \text{ mm}^4$$

$$I = (\frac{7854}{314})^{1/2} = 5 \text{ mm}$$

$$Z = 7854 / 20 = 785 \text{ mm}^3$$

SECOND MOMENT OF AREA

$$I = 7.85 \times 10^3 \text{ mm}^4 = 0.01 \times 10^6 \text{ mm}^4$$

$$+ 214(174 - 39)^2 = 5.72 \times 10^6 \text{ mm}^4$$

$$+ 1099(39)^2 = 1.67 \times 10^6 \text{ mm}^4$$

$$+ 3 \times 7.85 \times 10^3 = 0.02 \times 10^6 \text{ mm}^4$$

$$\text{Total } I = 7.42 \times 10^5 \text{ mm}^4$$

ELASTIC MODULI

$$Z_{\text{TOP}} = 7.42 \times 10^5 / (174 - 39) = 55.0 \times 10^3 \text{ mm}^3$$

$$Z_{\text{BOT}} = 7.42 \times 10^5 / 39 = 190.0 \times 10^3 \text{ mm}^3$$

STRESSES

TOP CHORD (COMPRESSION)

$$\sigma_{Pa} = 14.9 \times 10^6 \text{ N/mm}^2 / 55.0 \times 10^3 \text{ mm}^3 = 271 \text{ MR} \approx 0.54 f_y$$

WHERE $f_y = 500 \text{ MPa}$.

AXIAL LOAD IN TOP CHORD

$$P_{comp} = 271 \text{ N/mm}^2 \times 314 \text{ mm}^2 = 85 \text{ KN}$$

$$\text{CHECK } P_{comp} = 14.9 \times 10^6 / 174 = 86 \text{ KN} - \text{OK}$$

ACTION TO CHECK TOP CHORD COMPRESSION BY AT LEAST TWO INDEPENDENT METHODS.

DETERMINE ALLOWABLE STRESS TO PREVENT TOP CHORD BUCKLING.

ASSUME PIN-PIN END RESTRAINTS $\therefore l_e = 200 \text{ mm}$.

$$\text{SLENDERNESS} = l_e / r = 200 / 5 = 40 < 100$$

USE S.I. VERSION OF AREA (CLIP 15) > 12

$$F_{all} = 0.6 f_y - (635 \frac{F_y}{E})^{3/2} (\frac{l_e}{r})^2$$

allows for residual stress & accidental eccentricities.

$$= 0.6 \times 500 - \frac{(635 \times 500)^{3/2}}{200 \times 10^3} \times 40^2$$

$$= 300 \text{ MPa} - 80 \text{ MPa} \approx \text{LOAD FACTOR} = \frac{1}{0.6} = 1.67$$

$$= 220 \text{ MPa} \approx 0.44 f_y \approx \text{STRENGTH RED} = 1.5 / 0.9$$

$$f_a / F_{all} = 271 / 220 = 1.23 \text{ 23\% OVERSTRESSED}$$

CHECK DEFLECTION (MIDSPAN)

$$\delta = S W L^4 / 384 EI$$

$$= 5 \times 2.5 \times 6900^4 / 384 \times 200 \times 10^3 \times 7.42 \times 10^6 = 50 \text{ mm}$$

ADDITIONAL MOMENT DUE TO PONDING OF WET CONCRETE

ASSUME TRIANGULAR LOAD W

$$W = 6.9 \text{ m} \times 1/2 (0.05 \times 0.3 \times 25) = 1.3 \text{ KN}$$

ADDITIONAL MOMENT:

$$M_{ponding} = 1.3 \text{ KN} \times 6.9 \text{ m} / 6 = 1.5 \text{ KNm}$$

TOTAL MOMENT = 14.9 KNm + 1.6 KNm = 16.4 KNm

$$P_{comp} \approx 16.4 \times 10^3 / 174 = 94 \text{ KN of } 86 \text{ KN}$$

INCREASED AXIAL STRESS

$$f_a = 94 \times 10^3 / 314 = 299 \text{ MR} \approx 0.6 f_y$$

$$f_a / F_{all} = 299 / 220 = 1.36 \text{ 26\% OVERSTRESSED}$$

ADD A 2mm OR 1100 MISALIGNMENT $\Delta = 2 \text{ mm}$.

ADDITIONAL BENDING MOMENT

$$M_B = P_{comp} \Delta / 2 = 94 \times 0.001 = 0.094 \text{ KNm}$$



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BENDING STRESS IN TOP CHORD

$$f_b = M_b / Z = 0.094 \times 10^6 \text{ Nmm} / 785 \text{ mm}^3 = 120 \text{ MR} = 0.24 F_y$$

FOR COMBINED AXIAL & BENDING MEMB (CHAP 15)

$$\frac{f_a}{F_{a11}} + \frac{f_b}{F_b \left[1 - \frac{f_a}{0.514 \pi^2 E} \left(\frac{L_e}{r} \right)^2 \right]}$$

CRITICAL BUCKLING WITH $F_{a11} = 1.95$

$$F_b = 0.55 \cdot F_y$$

$$= 0.55 \times 500 = 275 \text{ MR}$$

$$\frac{P_a}{0.514 \pi^2 E} \left(\frac{L_e}{r} \right)^2 = \frac{299 \times 40^2}{0.514 \cdot \pi^2 \cdot 200 \times 10^3} = 0.47$$

$$\frac{f_a}{F_{a11}} + \frac{f_b}{275(1-0.47)} = \frac{299}{220} + \frac{120}{145} = 1.36 + 0.83 = 2.19$$

119% OVERSTRESSED

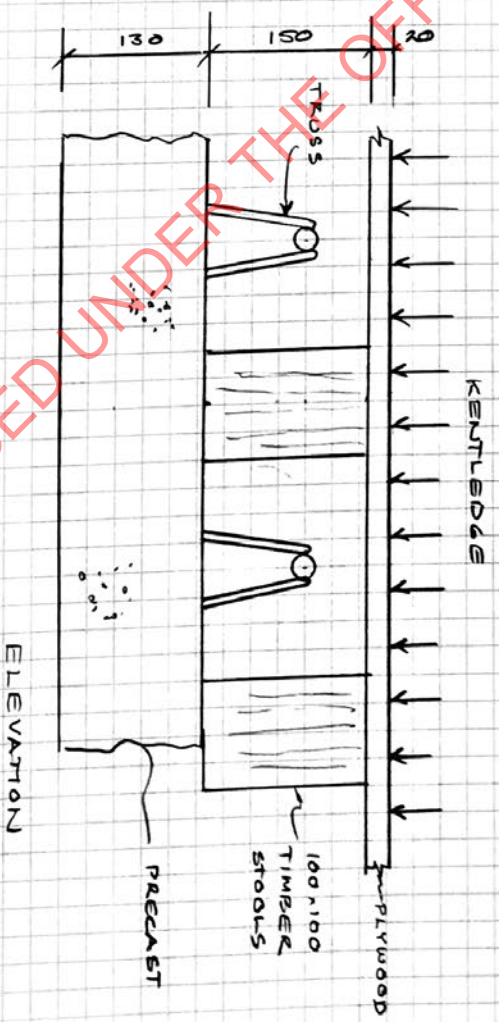
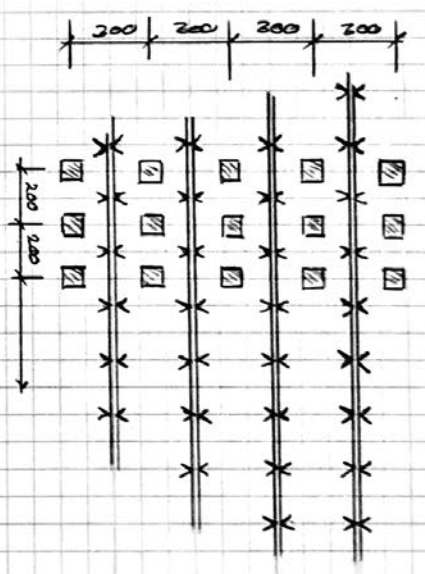
SUMMARY

INITIAL POUR	1.23	> 1.0	NO GOOD
POUNDING	1.36	> 1.0	NO GOOD
2mm MISALIGNMENT	2.19	> 1.0	NO GOOD

COLLAPSE IMMINENT - TOP CHORDS BUCKLE

RECOMMEND ACTIONS INDEPENDENTLY VERIFY

LOAD TEST PLATFORM



From: s 9(2)(a)
Sent: Friday, 22 May 2020 7:37 PM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: Re: Query P2W Viaducts

Hi Barry,

To my mind it would have been prudent if the independent reviewer had compared the truss top chord buckling capacity to the expected loads during the topping pour - a quick calculation; and also to comment on the transverse alignment of the trusses and therefore the high number of fatigue load cycles imposed on the trusses and the change in slab stiffness - by simple beam theory.

The propriety systems mentioned could be used with some success and much care in low traffic volume bridges and certainly as a flooring system in buildings which they were developed for. They are not for 4 lane viaducts with a combined total of over 16,000m² of deck area with 100year design life in my opinion.

The report offered to the NZTA makes comment on the deficiencies of the load test conducted on the Okahu Viaduct panels and why they gave false results and the calculations offered suggested changes to any future load testing set up to prevent the kentledge becoming a rudimentary but effective 'top flange' - a dropped bundle of rebar and the wet topping concrete will offer no such effective stiffening to the trussed precast slabs.

It would be my strong recommendation for the NZTA to contract s 9(2)(a) as a bridge design expert to review the For Construction drawings and the Shop Drawings of the Viaduct Precast Slabs and trusses. I am certain the Precast Slabs are significantly under designed and prone to fatigue failure leading to unintended deck behaviour and premature cracking. I can't think how the existing deck panels can be salvaged but if anyone can it will be s 9(2)(a)

Kind regards

s 9(2)(a)

On Fri, 22 May 2020 at 5:59 PM, s 9(2)(a) wrote:
Hi Barry,

I stand by my concerns and the documents offered to the NZTA that confirm and outline those concerns.

Kind regards

s 9(2)(a)

From: s 9(2)(a)
Sent: Tuesday, 19 May 2020 10:49 AM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Principal Structures Engineer - NTZA

Hi Barry,

I have been asked to leave s 9(2)(a), which I have.

I discovered (early March 2020) the viaduct deck slab defects. The deck concept was bought to Okahu and Puhoi Viaducts by Acciona of Spain. They have allowed for a very economic viaduct superstructures (about 185kg/m² compared to an average for those spans of 250kg/m²) but were always fatigue prone and this known by the designers (see attached).

Acciona are a 'platinum' client s 9(2)(a) and have not appreciated these engineering shortcomings coming to light.

These shortcomings have got through the following points where the engineers involved failed to act.

1. BECA detailed design
2. AECOM peer review
3. GHD design review on behalf of the NZTA – Principal and Regulator
4. Jacobs design review on behalf of the NZTA
5. Acciona temporary reinforcement truss (TRT) design
6. Acciona review of TRT following repairs
7. BECA design verification prior to issue of For Construction drawings dated 27 August 2018
8. Viaduct deck panel load testing
9. s 9(2)(a) review – discovery of deck panel design flaws (grossly under-designed) and notification to Acciona – March 2020
10. CASE International review of the TRT's
11. Acciona are now at the point where some of the deck panels are or about to be transferred from Otara to Okahu where they will be placed onto the girders and become dangerous.

So lots of organisations involved and arguably they should have known better given these are probably the biggest bridges currently under construction in NZ currently. The deck panels only become dangerous once they are on the girders. Workers on or underneath them are vulnerable to their collapse under expected loading.

I can redact references to s 9(2)(a) of my first draft report, that was not sent to Acciona, and send it to NZTA if they wish? I can also send my calculations that 'identify the two failure modes' if NZTA wish? A watered-down latter version of my report was sent and it was 'from' me but this was under an imposition rather than my prudent engineering judgement.

s 9(2)(a) also sent a later letter to Acciona confirming the under design of the precast deck truss and outlining where Acciona may have gone wrong in their design of the temporary reinforcement truss. I had suggested to s 9(2)(a) that this letter may stiffen Acciona's resolve to proceed with the under-designed deck slabs, and further suggested s 9(2)(a) were better to persuade Acciona of the folly to proceed as intended in a low key and trusted advisor manner.

Over to the NZTA as to the degree and depth of investigation it carries out. I know it's a trade-off between a possible manslaughter charge or a \$20M plus rework. So for both political and economic reasons it will require some significant decision making.

I know s 9(2)(a) is a close colleague of mine, but he would be my recommendation to the NZTA for a 3rd party investigator. s 9(2) private email is s 9(2)(a) will find it difficult that fellow NZ bridge engineers can 'deliberately' allow welded 500 grade rebar in bridge deck slabs – as did I when I first raised this matter with you.

I suspect the NZTA doesn't want to pre-empt any investigation but I know there is a workable solution, but it doesn't involve the Acciona deck panels already made (because of fatigue prone detailing) and that is the time and cost issues.

Depending on the outcome of any investigation I can work with the NZTA to develop a safe and durable viaduct deck solution if they wish?

Kind regards

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.

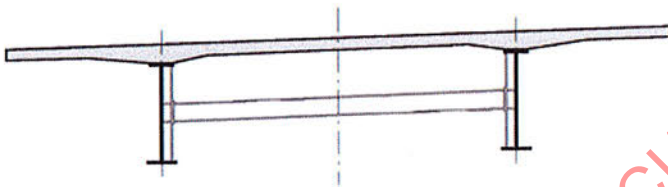


Figure 1: Cross girder at mid depth (Europe detail shown).

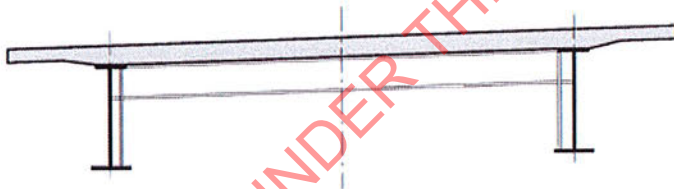
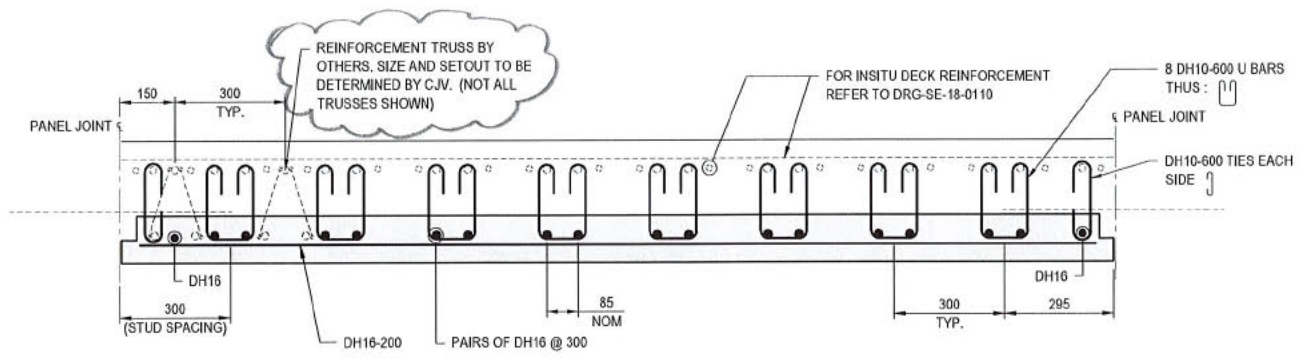


Figure 2: Cross girder also supporting the slab (Europe detail shown).

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.

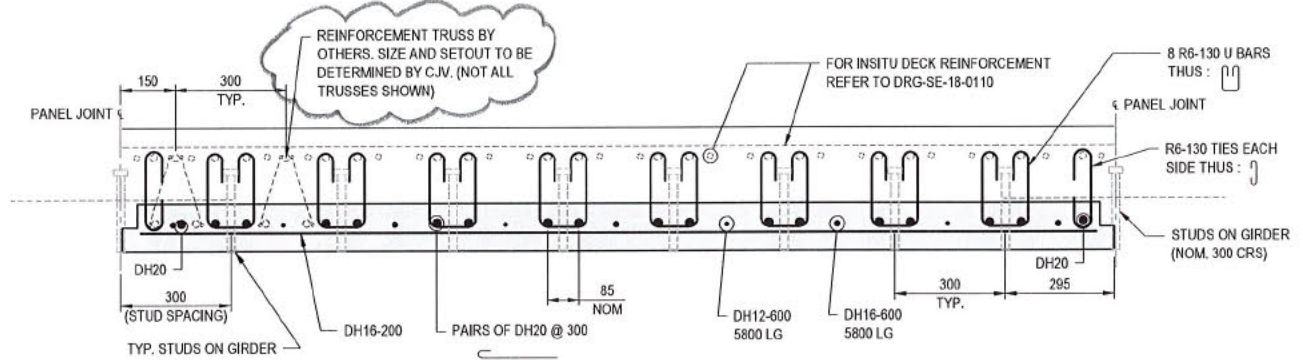
The advantage of this steel girder deck solution is that the precast deck section spans transversely between two main girders and cantilevers outward over the outer girder. The cantilever is supported by a steel truss system integral with the precast slab. This means the precast deck requires no permanent 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).



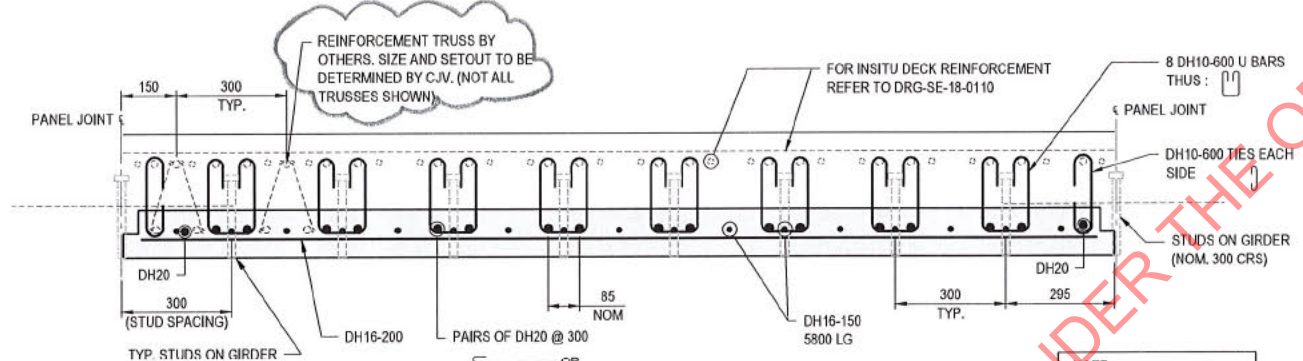
1 PANEL TYPES A, A1, F AND H REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

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)

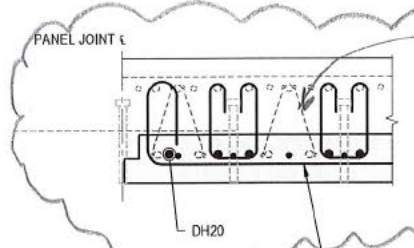
NOTE:
PANEL TYPE A DRAWN,
PANELS A1, F AND H SIMILAR



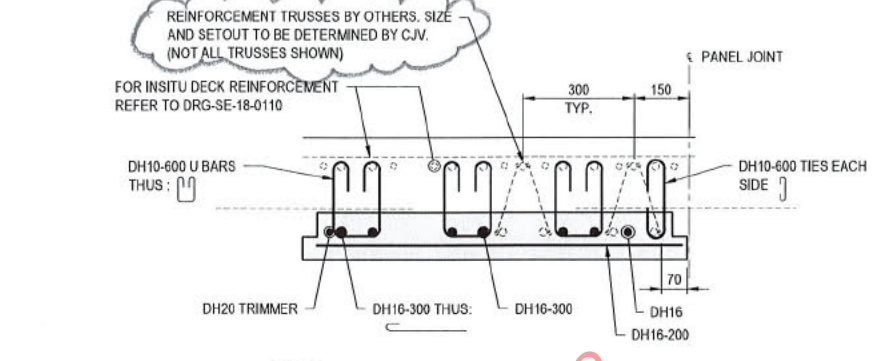
3 PANEL TYPES B, B1, G AND J REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

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. =>

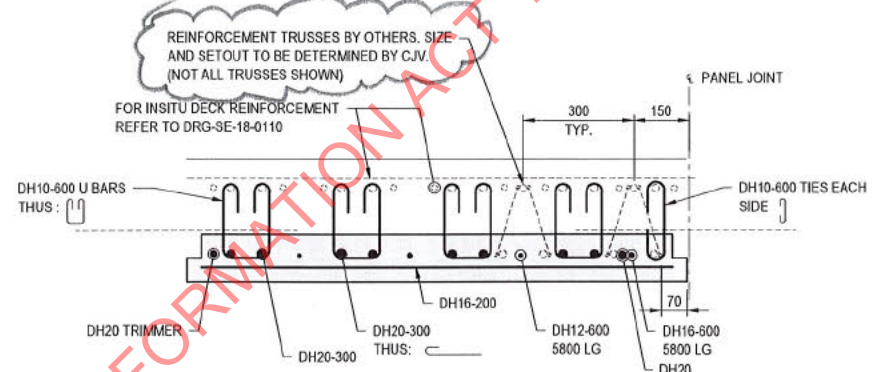


A PROPOSED CJV ALTERNATIVE OPTION TO REPLACE END TIE WITH DH16 HOOK BAR
1:10(A1), 1:20(A3)



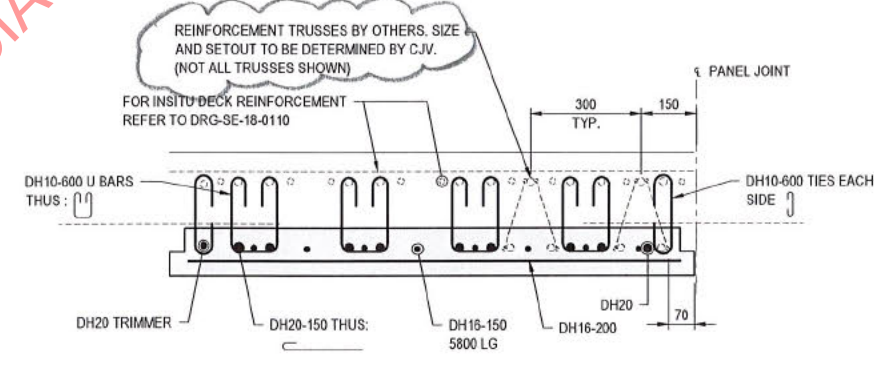
4 PANEL TYPE C AND E REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

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 CJV. 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.

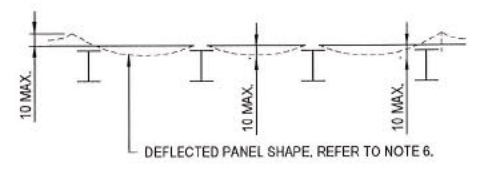
HOLD POINT

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	Date 01.10.18



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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