From: Barry Wright
Sent: Tuesday, 19 May 2020 9:03 AM
To: s 9(2)(a) [redacted]
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi s 9(2)(a) [redacted]

I have talked again with the project rep.

He is finding it difficult to raise the issue and retain confidentiality.

We would like to be open, say that we understand there may be an issue if difference of opinion and get 3rd part to review or something like that.

That of course would have potential implications for you.

Any thoughts??

Cheers
Barry

From: s 9(2)(a) [redacted]
Sent: Thursday, 30 April 2020 9:03 AM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi Barry,

In relation to problem 2 stated below here are the relevant clauses from NZS3101 and the Bridge Manual addressing welded reinforcement in bridge decks. They were prepared for internal communication (within quotes). Also attached are extracts from the viaducts 'design report' and For Construction drawings outlining the Designers need for welded reinforcement within the viaduct decks.

"The following are some pertinent clauses from NZS3101 that is called up by the NZTA Bridge Manual for the design of concrete bridge decks, and the Bridge Manual clauses that you should be aware of. It would seem both the NX2 Designer and I agree that the heavily welded (temporary) reinforcement truss will fracture in fatigue. Please note reference to 'temporary' in my view is a misnomer as it will be cast within the permanent deck, hence I prefer 'welded' because that is what it is.

The problem is the on-going effects on the deck slab behaviour due to a number of discrete changes in deck stiffness resulting from the fracture of the welded reinforcement truss. I have explained how I think the deck failure mode will be arrived at and I have implied this is likely to come about in much less than 100 years design life giving rise to both safety and durability issues. Given the stage and commercial organisation of the project stating the bald engineering will be unpalatable, but my view is s 9(2)(a) [redacted] cannot be drawn into the design of a potentially flawed bridge deck system that is fraught with danger. We can assist with the design of an alternative deck system with a 100 year design life if NX2 are unable to get sign-off of the welded reinforcement truss or prove this system is adequate for 100 years.

It will become clear from below that heavily welded reinforcement is not permitted in bridge decks. It is also becoming clear the NZTA rely on the NX2 for all design, peer review and construction. There is a public expectation that bridges should be unfailingly safe and durable.

Please let me know the outcome.

12.8.2.3 *Reinforcement*

For slabs meeting the above conditions, the deck reinforcement shall comprise:

(a) Layers of reinforcement in two directions at right angles in the top and bottom of the slab, placed as close to the outside surfaces as possible, as permitted by cover requirements;
(b) The reinforcing steel shall have a yield strength greater than or equal to 420 MPa;
(c) The minimum amount of reinforcement shall be 570 mm^2/m of steel in each direction in the bottom layer, 380 mm^2/m of steel in each direction in the top layer;
(d) All reinforcement shall be straight bars except that hooks may be provided where required;
(e) The maximum spacing of the reinforcement may be 300 mm;
(f) The bars shall be spliced by lapping or by butt welding, or by mechanical connections satisfying 8.7.5.2 only;
(g) For skew angles, θ, greater than 25°, the specified reinforcement in both directions shall be doubled in the end regions of the deck. The span end regions are as defined in Figure 12.2.

C12.8.2.3 *Reinforcement*

Prototype tests have indicated that 0.2 % reinforcement in each direction in both the top and bottom layers, placed at the minimum required cover, satisfies strength requirements. However, the conservative value of 0.3 % of the gross area, which corresponds to 570 mm^2/m in a 190 mm thick slab, is specified for better crack control in the positive moment area. Field measurements show very low stresses in the negative moment steel; this is reflected by the 380 mm^2/m requirement, which is about 0.2 % reinforcement steel.

Lap welded splices are not permitted due to fatigue considerations. Tested and pre-approved mechanical splices may be permitted when lapping of reinforcement is not possible or desirable, as often occurs in staged construction or widenings.

Beam and slab bridges with a skew exceeding 25° have shown a tendency to develop torsional cracks due to differential deflections in the end zone, and therefore the provision of additional reinforcement is required in the end zones to counter this.

8.7.5.2 *Performance requirements for mechanical connections*

Mechanical connections shall:

(a) satisfy the requirements of 8.6.11 for mechanical anchors;

(b) when tested in tension or compression, as appropriate, to the application, exhibit a change in length at a stress of $0.7f_y$ in the bar, measured over the length of the coupler, of less than twice that of an equal length of unspliced bar;

(c) satisfy the requirements of 2.5.2.2 when used in situations where fatigue may develop.

8.7.5.3 *Use of welded splices and mechanical connections*

Welded splices in tension or compression shall meet the requirements of 8.7.4.1 (a) or (b).

Mechanical connections in tension or compression shall meet the requirements of 8.7.5.2.

C8.7.5.2 *Performance requirements for classification as a "high-strength" mechanical connection*

A stiffness criterion is imposed on mechanical splices of C8.7.5.2(b) to ensure that large premature cracks are not produced by excessive extensions in splicing devices. Accordingly the displacement of the spliced bars relative to each other and measured in a test over the length of the connector, should not exceed twice the elongation of the same size of unspliced bar over the same measured distance when subjected to 0.7 f_y.

C8.7.5.3 *Use of welded splices and mechanical connections*

See commentary on 8.7.4.1(c). This clause describes the situation where welded splices or mechanical connections with capacity less than the actual breaking strength of the bar may be used. It provides a relaxation in the splice requirements where the splices or connections are staggered and an excess reinforcement area is available. The criterion of twice the computed tensile stress is used to cover sections containing partial tensile splices with various percentages of the total reinforcement continuous.

8.7.4.1 *Classification of welded splices*

Welded splices shall be classified as follows:

(a) A "full strength" welded splice is one in which the bars are butt welded to develop in tension the breaking strength of the bar;

(b) A "high strength" welded splice is one in which the bars are lap welded or butt welded to develop the lower characteristic yield strength of the bar or better.

C8.7.4 *Welded splices and mechanical connections*

Designers should avoid the need to weld reinforcing steel if possible as follows:

(a) Where butt jointing is required there is a good range of coupling devices available. Lapping, particularly of smaller bars, may also be an option;

(b) Tack welding of stirrups or ties to main bars may result in a reduction in capacity of the main bar, either through metallurgical changes, or the generation of notches due to undercut if the procedures of AS/NZS 1554:Part 3 are not followed;

(c) Where welds are required to provide lightning protection, care should be taken to choose a route through non-critical members.

Welds complying with 8.7.4.1(a) can withstand the most severe strain or stress cycles. Hence they are acceptable in all locations, in particular, for splicing main longitudinal reinforcement in plastic hinge regions and in beam-column joints. Weld quality should comply with the requirements of AS/NZS 1554: Part 3, Section 9 for "Direct Butt Splices".

The categories of splices in 8.7.4.1(b) will be adequate for large bars in main members outside plastic hinge regions and for welded splices in stirrups, ties, spirals or hoops. The limit of the breaking strength of the bar will ensure that the strength of the connection will be greater than the maximum design force in the bar. Weld quality should comply with the requirements of AS/NZS 1554:Part 3, Section 9 for "Other splices".

2.5.1 *General*

Requirements such as those for fatigue, removal or loss of support, together with other performance requirements shall be considered in the design of the structure in accordance with established engineering principles.

2.5.2 *Fatigue (serviceability limit state)*

2.5.2.1 *General*

The effects of fatigue shall be considered where the imposed loads and forces on a structure are repetitive in nature.

2.5.2.2 *Permissible stress range*

At sections where frequent stress fluctuations occur, the stress range in reinforcing bars, excluding stirrups and ties, caused by the repetitive loading at the serviceability limit state, shall be equal to or less than the appropriate limit given in either (a) or (b) below: A2

(a) The stress range shall be equal to or less than the value given in the Table below, where D is the diameter of the bend measured to the inside of the bar and d_b is the diameter of the bar.

Stress range, MPa	**150**	**135**	**120**	**90**	**50**
D/d_b	>25	20	15	10	5

Interpolation may be used for intermediate values of D/d_b.

(b) Appropriate values are found from a special study in which the influence of the following factors is considered:

- (i) The shape of deformations and bar marks;
- (ii) The composition and diameter of the reinforcement;
- (iii) The method of manufacture;
- (iv) The diameter of bends in the reinforcement;
- (v) The influence of embedment of the bar in cracked concrete;
- (vi) The histogram of stress variation over the expected life of the structure.

2.5.2.3 *Highway bridge fatigue loads*

For highway bridges, the vehicle loading specified by the New Zealand Transport Agency's Bridge Manual shall be used as a basis for assessing the fatigue stress range.

C2.5.2 *Fatigue (serviceability limit state)*

C2.5.2.1 *General*

Members in some structures, for example deck slabs of bridges, may be subject to large fluctuations of stress under repeated cycles of live loading.

C2.5.2.2 *Permissible stress range*

The limitations on the range of stress of 150 MPa under live load, irrespective of the grade of reinforcing used, are based on AASHTO standards [2.6] and were considered necessary to avoid the possibility of premature fatigue failure in the reinforcing bars. The range of stress of 150 MPa is allowed for straight reinforcing steel. The effect of the 150 MPa range is usually to limit crack widths to approximately 0.25 mm.

This stress range is further reduced in the CEB-FIP Code where the stress occurs in a bar bend (as a function of d_b) and where corrosion can be expected [2.17] and further general information on fatigue may be obtained from References 2.18 and from "Comite Euro-internationale du Beton, "Fatigue of Concrete Structures", Bulletin D' Information No. 188, June 1988.

The allowed relaxation of the requirements of this clause, if a special study is made, is in recognition of views expressed [2.19] that the specified requirements are conservative. The requirements of a special study may be deemed to be satisfied if the following revised AASHTO procedures [2.6] are followed:

Concrete

The stress range, f_{cr} , between the maximum compressive stress (f_{cmax}) and the minimum compressive stress (f_{cmin}) in the concrete at the serviceability limit state, at points of contraflexure and at sections where stress reversals occur, shall not exceed $0.5f'_c$ where:

f_{cr} $= f_{cmax} - f_{cmin}$

f_{cmin} is the minimum compressive stress level in the concrete due to dead load, creep, shrinkage, temperature, etc. (MPa)

f_{cmax} $= f_{cmin}$ plus the additional compressive stress due to live load plus impact (MPa)

Reinforcement

The stress range, f_{sr}, between the maximum tension stress (f_{smax}) and the minimum stress (f_{smin}) in straight reinforcement at serviceability limit state, shall not exceed:

f_{sr} $= f_{smax} - f_{smin} = [\ 145 - 0.33\ f_{smin} + 55\ (r\ /h_d\ )\ ]$

f_{smin} is the algebraic minimum stress level due to dead load, creep, shrinkage, temperature etc. (MPa) (tension positive, compression negative)

f_{smax} $= f_{smin}$ plus the additional tension stress due to live load plus impact (MPa)

$r\ /\ h_d$ is the ratio of base radius to height of rolled-on transverse deformation; when the actual value is not known use 0.3.

Bends in primary reinforcement and welding shall be avoided in regions of high stress range. The suitability of mechanical connections for splices should be checked where repetitive stress fluctuations occur.

Fatigue shall be checked for normal serviceability limit state live loads only. Overloads are specifically excluded from the requirements of this clause.

From the NZTA Bridge Manual:

f. Mechanical coupling and anchorage of reinforcing bars

Mechanical couplers for the jointing of reinforcing steel and mechanical anchorages for the anchoring of reinforcement shall satisfy the requirements of NZS 3101[1] clauses 8.7.5 and 8.6.11 except as modified herein.

Couplers and mechanical anchors for the jointing or anchorage of reinforcing steel shall possess an ultimate tensile strength exceeding that of the maximum upper bound ultimate tensile strength of the reinforcing bar size and grade to be joined or anchored. (This requirement shall be taken as replacing NZS 3101[1] subclauses 8.7.5.2(a) and 8.6.11.2.)

The mode of failure of the coupled or anchored bar shall be by ductile yielding of the bar, with the bar developing its ultimate tensile strength at a location outside of the coupler or anchorage and away from any zone of the bar affected by working (eg by cold forging). This mode of failure shall be ensured when tested with reinforcement of yield strength within ±10% of the upper characteristic yield strength as defined by AS/NZS 4671 *Steel reinforcing materials*[14]. Where the coupler or mechanical anchor and ends of the bars are threaded as the means of achieving the coupling between components, there shall be no thread stripping or evidence of significant distortion of the threads at the failure load of the bar.

NZS 3101[1] subclauses 8.7.5.2(b) and (c), and subclause 8.9.1.3 (in respect to mechanical couplers and anchorages) shall be deleted and replaced with:

Mechanical couplers and anchorages shall satisfy the cyclic load performance requirements specified by ISO 15835-1 *Steels for the reinforcement of concrete - Reinforcement couplers for mechanical splices of bars* part 1 Requirements[15] and ISO 15835-2 *Steels for the reinforcement of concrete - Reinforcement couplers for mechanical splices of bars* part 2 Test methods[16] as follows:

i. When tested in accordance with ISO 15835-2[16] clause 5.6.2, for alternating tension and compression test of large strains in the mechanical splice, the residual elongations after 4 cycles, u_4, shall be less than 0.3mm, and u_8 shall be less than 0.6mm.

ii. Where high cycle fatigue is a consideration, the mechanical connection shall satisfy the requirements of ISO 15835-1[15] –properties under high cycle fatigue loading. The testing shall comply with ISO 15835-2[16] clause 5.5.

Couplers and mechanical anchors for the jointing or anchorage of reinforcing steel shall be proven by an appropriate test acceptable to the road controlling authority to possess resistance to brittle fracture. Where couplers and anchorages are of sufficient size to enable Charpy V notch test specimens to be cut from them, Charpy V notch testing shall be undertaken. Where this test method is applied, a Charpy V-notch impact resistance equal to or greater than 27 joules shall be achieved when tested at 0°C in accordance with AS 1544.2 *Methods for impact tests on metals* part 2 Charpy V-notch[17] and assessed for acceptance as specified by AS/NZS 3678 *Structural steel – Hot-rolled plates, floorplates and slabs*[18] table 10.

Cast iron couplers or anchorages shall not be used.

I suspect the HumeSlab and other precast deck systems I have seen around the world comply to something like ISO 15835 – 1.

Where, in the design of a structure or new works to a structure, reinforcement is designed to be joined by mechanical coupling, the reinforcement to be used shall be either grade 300E or grade 500E complying with AS/NZS 4671[14], for which the maximum upper bound ultimate tensile strengths may be taken as:

- Grade 300E: 570MPa
- Grade 500E: 840MPa

Reinforcing steel of grades 250N, 500L and 500N shall not be used where mechanical coupling is required. Where the ends of grade 500E bars are to be threaded as a means of achieving the coupling, only microalloyed bars, and not quenched and tempered bars, shall be used.

Where, in the modification or strengthening of an existing structure, coupling to embedded reinforcement of unknown maximum ultimate tensile strength is proposed, the reinforcement shall either be tested to establish its ultimate tensile strength or a conservative over estimation made of its ultimate tensile strength as the basis for selection and design of the couplers in order to ensure that the performance requirements specified above are satisfied.

Where the means of coupling is through use of parallel threaded couplers with the ends of the bars to be joined enlarged in diameter by cold forging prior to threading, the cold forging process will locally alter the mechanical properties of the ends of the bars. The potential for brittle fracture in the reinforcing bar shall be avoided. Quality assurance and control procedures shall be employed to ensure that the brittle fracture resistance and ultimate tensile strength of the cold forged sections of the bars satisfy the requirements above and that failure of the bar is by ductile yielding and at its ultimate tensile strength is at a location away from the coupling and zones of cold forging.

j. Design for fatigue

In the application of NZS 3101[1] clause 2.5.2.2, the stress range due to repetitive loading to be considered in flexural reinforcing bars shall be that due to live loading corresponding to table 3.1 load combination 1A, but without pedestrian (FP) loading.

In the application of NZS 3101[1] clause 19.3.3.6.2, the stress range due to frequently repetitive live loading shall be that due to live loading corresponding to table 3.1 load combination 1A, but without pedestrian (FP) loading. The stress range due to infrequent live loading shall be taken to be that due to live loading, overload, wind loading and temperature effects corresponding to all other load combinations of table 3.1, including load combination 1A with pedestrian loading.

NZS3101 Chapter 19 pertains to prestressed design which is not applicable for the viaduct decks.

4.3.1 General

Design for the steel componentry of bridge substructures, and any seismic load resisting componentry expected to behave inelastically, shall comply with NZS 3404 *Steel structures standard*[25]. Design for the steel componentry of bridge superstructures, including seismic load resisting components expected to behave elastically, shall be in accordance with AS 5100.6 *Bridge design* part 6 Steel and composite construction[26]. This applies also to the design of steel componentry of major culverts, stock underpasses and pedestrian/cycle subways.

Until such time as requirements for brittle fracture appropriate to New Zealand are incorporated into AS 5100.6[26], design for brittle fracture shall comply with NZS 3404[25]. In addition to plates and rolled sections, consideration shall also be given to the brittle fracture of steel elements complying with standards other than those listed by NZS 3404[25] (eg fixings, high strength bars).

The design of concrete deck slabs for composite bridges for the actions of live load on the concrete deck shall be in accordance with NZS 3101[1], except that the design of shear connection between the concrete deck slab and steel girders and the design for longitudinal shear occurring within the deck slab and paps shall comply with AS 5100.6[26]. The requirements of AS 5100.6[26] section 6.1, as they relate to the design of the concrete deck slab, where they require a greater quantity of reinforcement than required by NZS 3101[1], shall also be complied with.

The NZTA research report 525 *Steel-concrete composite bridge design guide*[27] provides guidance on the design of steel girder bridge superstructures to AS 5100.6[26].

With the above I have tried to be complete and not selective. If I have omitted something relevant then it is by accident, not by intent."

By my calculations based on an assumed number of 6t axles and AASHTO detail classification E the Temporary Reinforcement Trusses will fail within the first year.

Are you able to let me know the timeframe and outcome of the NZTA investigation?

s 9(2)(a)

[redacted]

- 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.

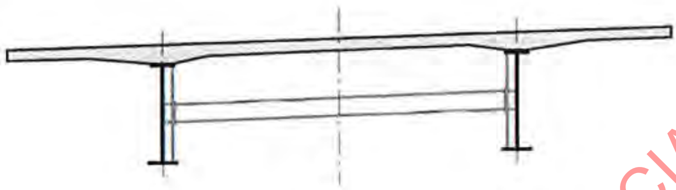


Figure 1: Cross girder at mid depth (Europe detail shown).

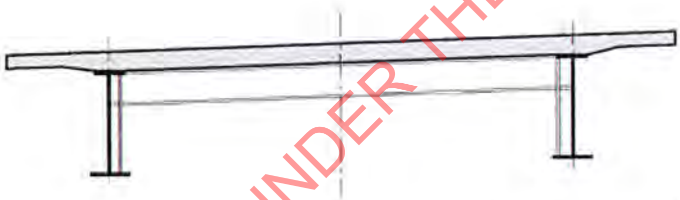
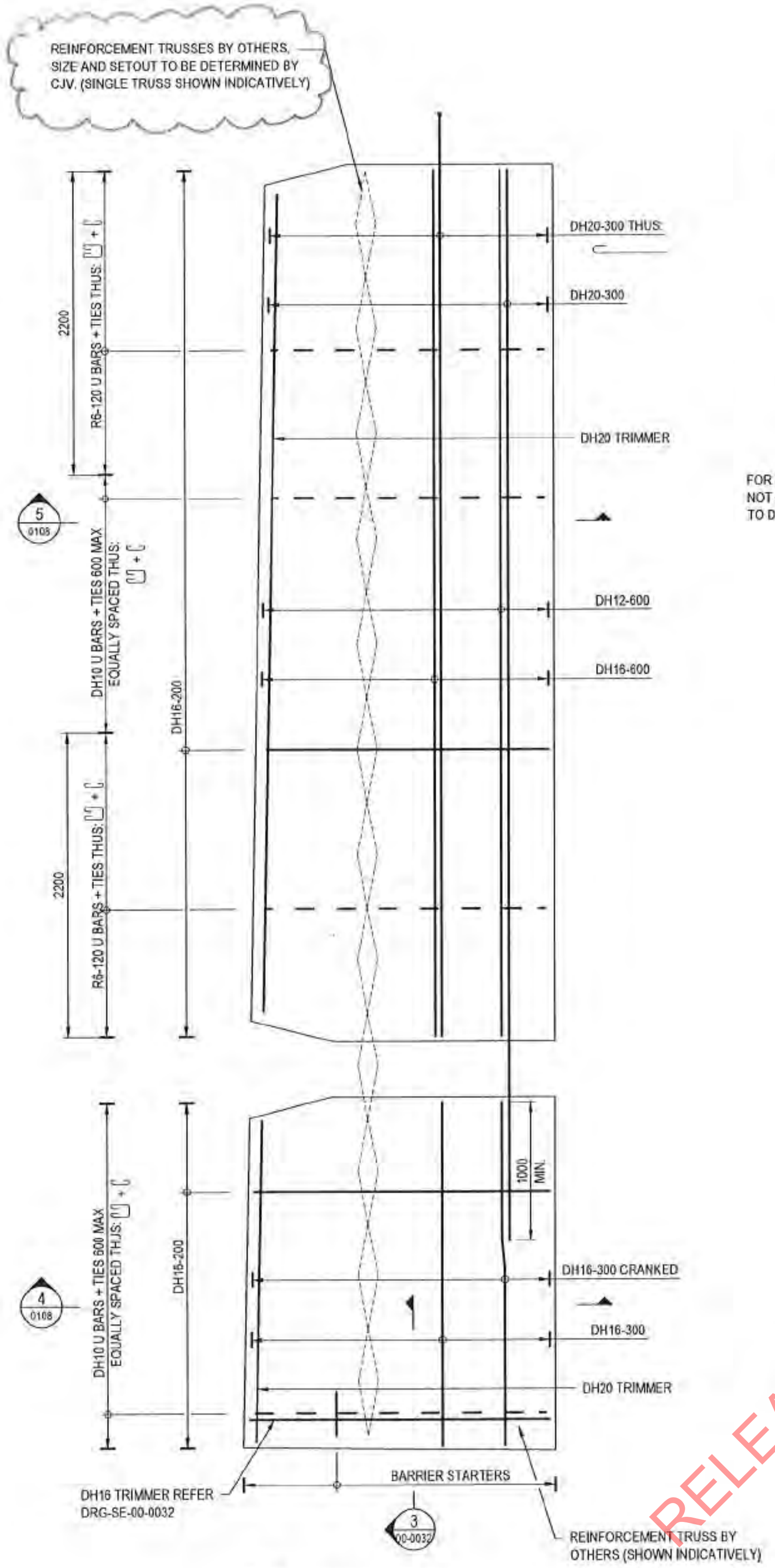


Figure 2: Cross girder also supporting the slab (Europe detail shown).

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.

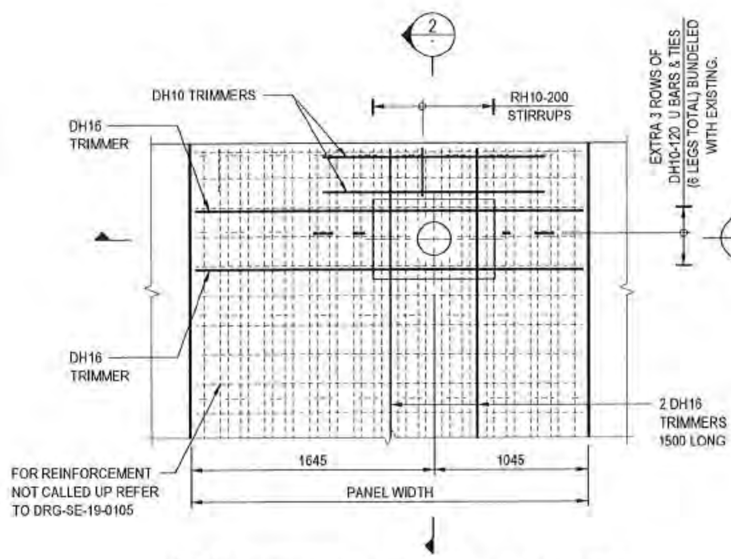
The advantage of this steel girder deck solution is that the precast deck section spans transversely between two main girders and cantilevers outward over the outer girder. The cantilever is supported by a steel truss system integral with the precast slab. This means the precast deck requires no permanent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

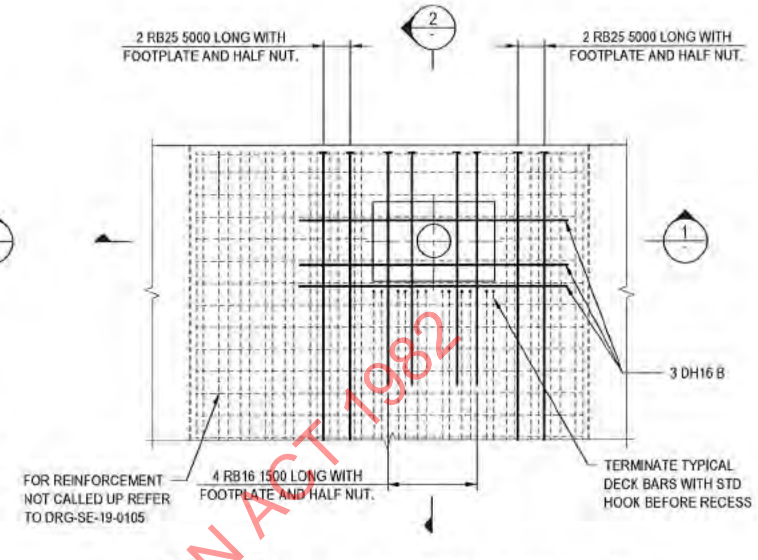


C PANEL TYPE S
REINFORCEMENT DETAILS
1:25(A1), 1:50(A3)

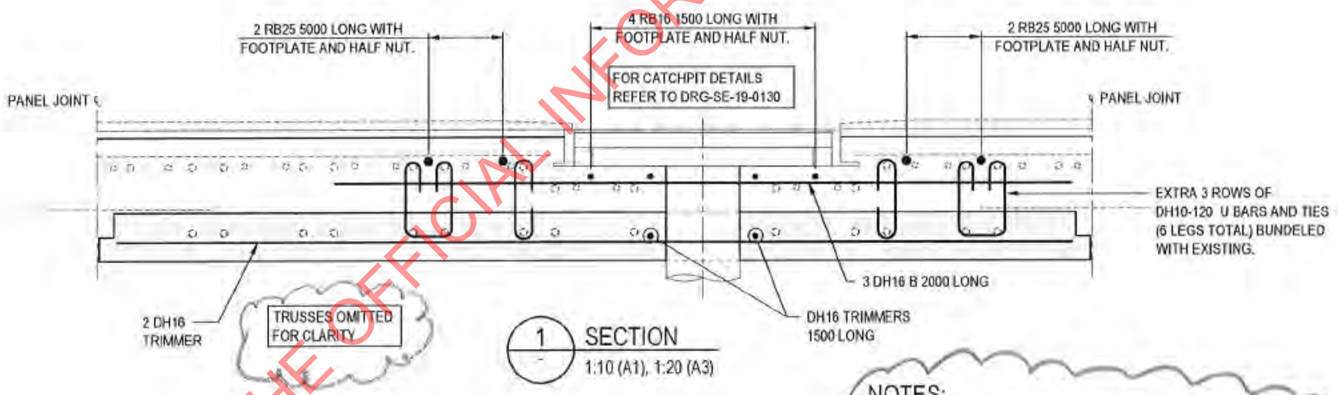
NOTE:
REFER TO SECTIONS FOR U BAR AND TIE LAYOUT



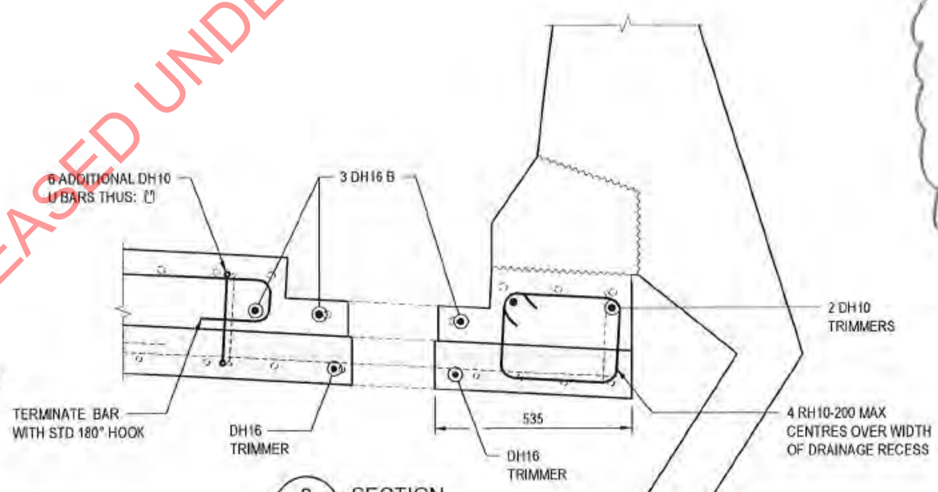
C PLAN- PRECAST PANEL AT CATCHPIT
ADDITIONAL REINFORCEMENT
1:25 (A1), 1:50 (A3)



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:

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 - 7DAYS 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 SLAB 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.

NOTES:

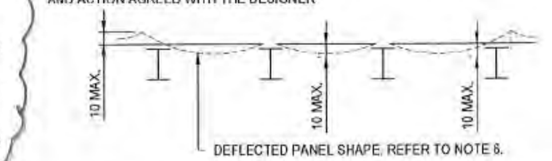
TEMPORARY TRUSS DESIGN.

1. DESIGN OF THE PRECAST PLANKS AND THE TEMPORARY REINFORCING TRUSS (INCLUDING TRUSS LOCATION) IS THE CONSTRUCTORS RESPONSIBILITY. AS DEFINED BY BA 36/90 AND AS 3610. 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.

   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.

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.
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



DEFLECTED PANEL SHAPE. REFER TO NOTE 6.

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 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.
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	s 9(2)(a)			14.05.18

Original Scale (A1) AS SHOWN	Design	s 9(2)(a)	16.11.17
Reduced Scale (A3) HALF SIZE	Drawn		05.11.17
	Dsg Verifier		27.09.18
	Drg Check		27.09.18
Approved For Construction	s 9(2)(a)	Date	01.10.18

* Refer to Original Hardcopy for Signature

NZ TRANSPORT AGENCY WAKA KOTAHI

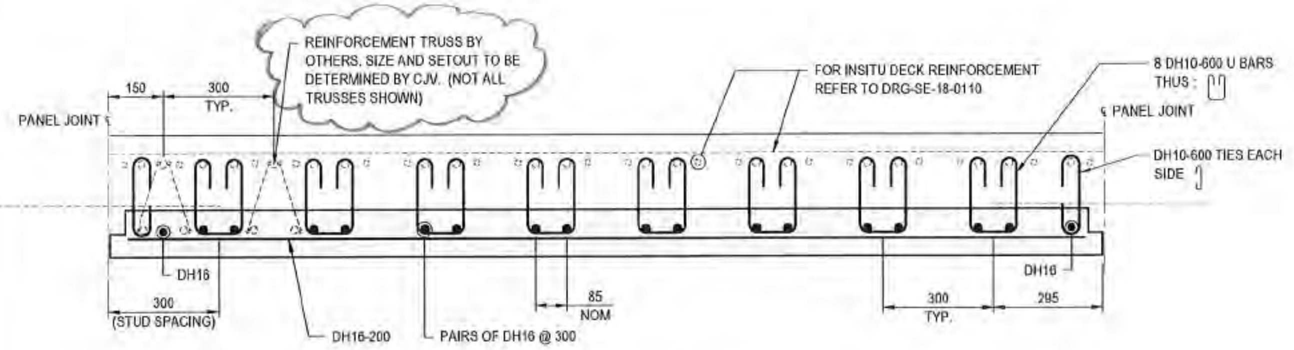
PŪHOI TO WARKWORTH MOTORWAY

NX2 NORTHERN EXPRESS GROUP

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

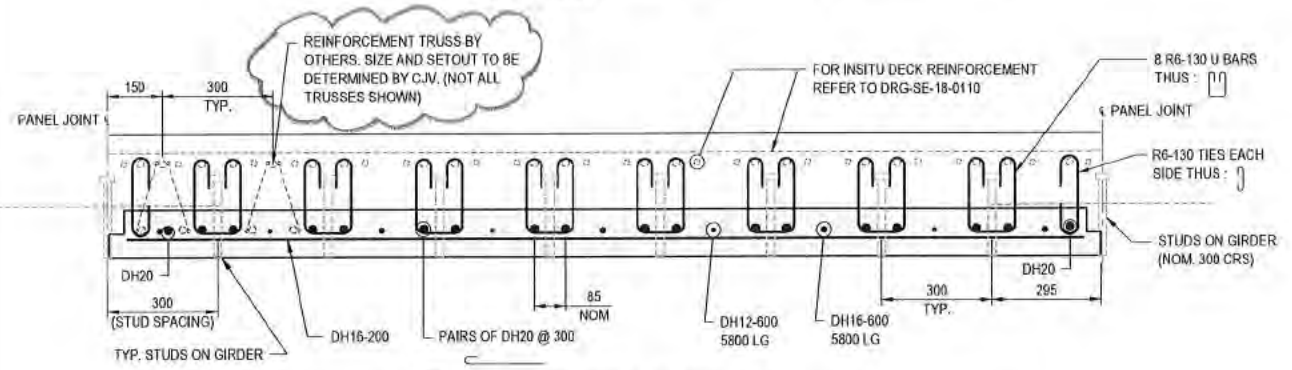
ORIGINAL SIZE A1 DO NOT SCALE

Document No. C:\USERS\PUBLIC\DOCUMENTS\P2WK\SE-A\J-PW-AGARY\OSANO\MS0052\P2WK-DRG-SE-19-0107.DWG



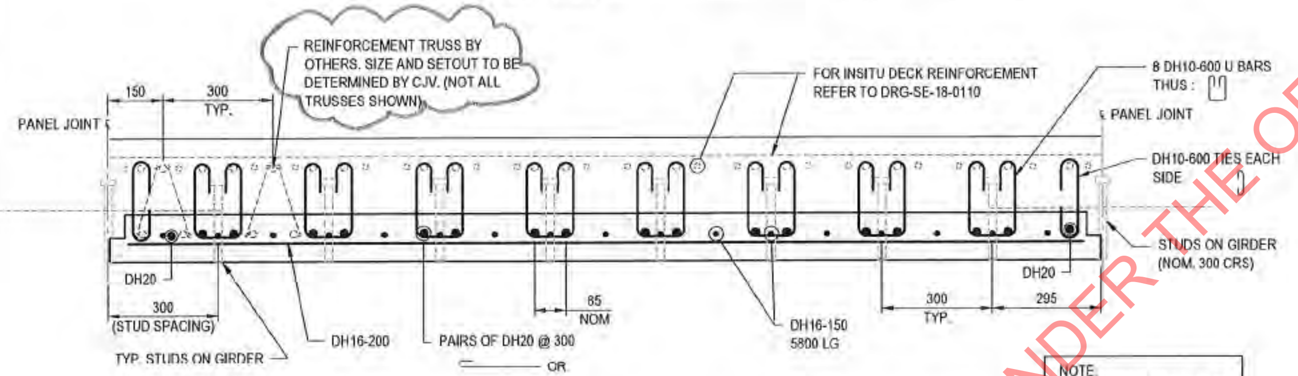
1 / 0105 PANEL TYPES A, A1, F AND H REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

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



2 / 0105 PANEL TYPES A, A1, F AND H REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

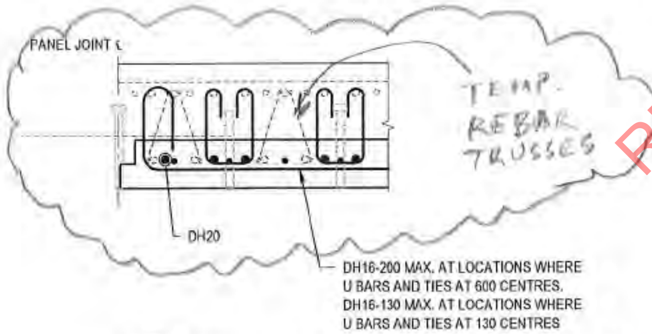
NOTE:
PANEL TYPE A DRAWN, PANELS A1, F AND H SIMILAR



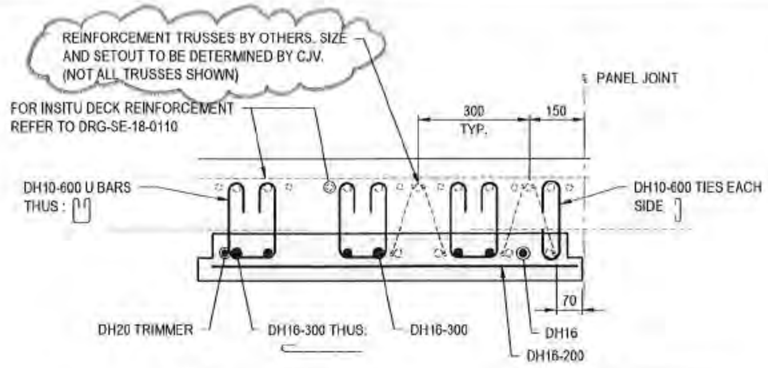
3 / 0105 PANEL TYPES B, B1, G AND J REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

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.

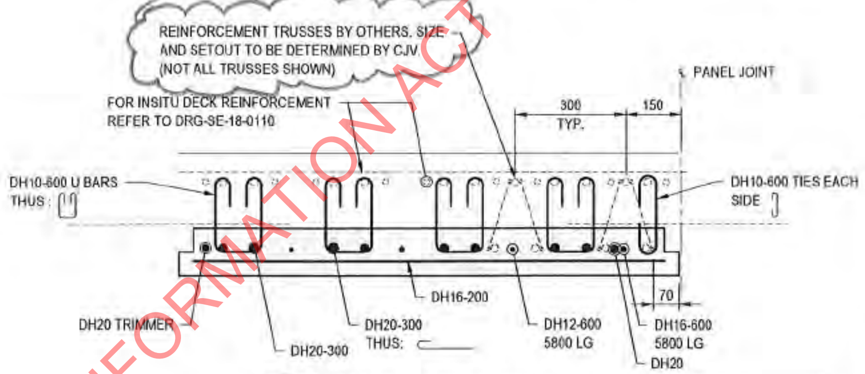


A PROPOSED CJV ALTERNATIVE OPTION TO REPLACE END TIE WITH DH16 HOOK BAR
1:10(A1), 1:20(A3)



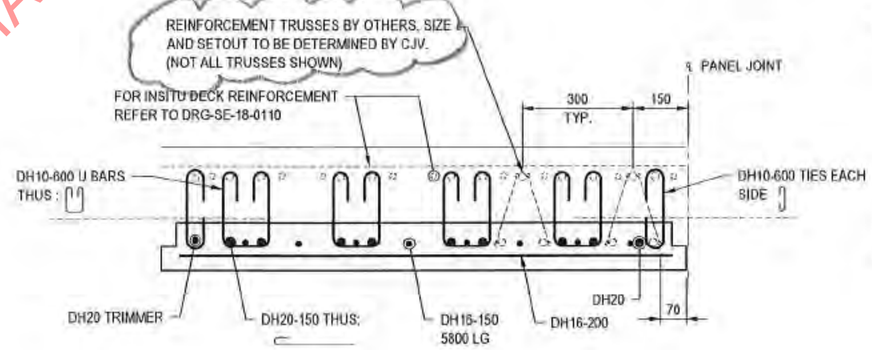
4 / 0106 PANEL TYPE C AND E REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

NOTE:
PANEL TYPE C DRAWN, PANEL E SIMILAR



5 / 0106 PANEL TYPE C AND E REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

NOTE:
PANEL TYPE C DRAWN, PANEL E SIMILAR



6 / 0106 PANEL TYPE D REINFORCEMENT DETAILS
1:10(A1), 1:20(A3)

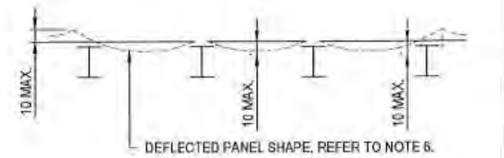
NOTES:

1. GENERAL
   - 1.1. REFER DRG-SE-18-0002 FOR GENERAL NOTES.
   - 1.2. FOR INFORMATION RELATING TO SERVICE PENETRATIONS AND SPACING, REFER TO DRG-SE-18-0135.
   - 1.3. INSITU SLAB SHALL BE POURED TO ACHIEVE A TOTAL DECK SLAB THICKNESS OF 335mm RATHER THAN TO A GEOMETRIC LEVEL.
2. 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 - 7DAYS 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 335mm THICK DECK SHALL BE U5 FINISH APART FROM BARRIER AREA WHERE A TYPE B CONSTRUCTION JOINT TO NZS 3109 IS REQUIRE.
   - 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.

NOTES:

TEMPORARY TRUSS DESIGN.

1. DESIGN OF THE PRECAST PANELS AND THE TEMPORARY REINFORCING TRUSS (INCLUDING TRUSS LOCATION) IS THE CONSTRUCTORS RESPONSIBILITY. AS DEFINED BY BA 36/90 AND AS3610, 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.
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 ADDEQUATE 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

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.

HOLD 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.

FOR CONSTRUCTION

RELEASED UNDER THE OFFICIAL INFORMATION ACT 1982

No	Revision	By	Chk	Appd	Date
1	FOR CONSTRUCTION	s 9(2)(a) [redacted]			24.09.18

Original Scale (A1) AS SHOWN
Reduced Scale (A3) HALF SIZE
Design / Drawn / Dsg Verifier / Drg Check: s 9(2)(a) [redacted] 16.10.17 / 05.11.17 / 27.09.18 / 27.09.18
Approved For Construction s 9(2)(a) [redacted] Date 01.10.18
* Refer to Original

NZ TRANSPORT AGENCY WAKA KOTAHI

PŪHOI TO WARKWORTH MOTORWAY

NX2 NORTHERN EXPRESS GROUP

Subject: 12-12 PŪHOI VIADUCT
Title: PRECAST DECK PLANKS REINFORCEMENT DETAILS - SHEET 3
Discipline: STRUCTURAL
Drawing No: P2Wk-DRG-SE-18-0107
Rev: 1

ORIGINAL SIZE A1 DO NOT SCALE
IF IN DOUBT ASK

From: Barry Wright <Barry.Wright@nzta.govt.nz>
Sent: Thursday, 9 April 2020 5:27 PM
To: s 9(2)(a) [redacted]
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Thanks s 9(2)(a) [redacted]

I have raised with the key person who is looking into it so will see what happens

Happy Easter
Barry

From: s 9(2)(a)
Sent: Thursday, 9 April 2020 2:33 PM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi Barry

The NZTA investigation should discover the following:

Problem	Cause
1. Temporary Rebar Truss top chord buckling during topping pour preparations or during the pour. There is a considerable gap between the top chord axial capacity and the demands placed up them.	Acciona Ingeneria undertook a SAP 2000 FE analysis and accounted for the rebar truss and the precast concrete as both having linear elastic properties. That is the full 130mm thick precast concrete panel was modelled to carry maximum tension at the panel soffit and compression at the precast top surface. The error being the concrete cannot reliably carry any tension and only about 40mm depth of the concrete precast is immediately above the neutral axis is available to carry some small compression. Given the truss top chord is about 140mm above the neutral axis and at 'extreme compression fibre' then the vast majority of the compression loads are carried by the top chord. This prevents the high compressive strains in the concrete precast 100mm below the top chord. That is the concrete compression contributes a negligible of flexural capacity. Acciona results indicate that the precast concrete panel carry about half the required flexural capacity hence the compression demand in the top chord is about half what it should be. Acciona have taken no reasonable account of damaged to the top chord (e.g. say from a bundle of rebar being lowered onto the precast during supply for fixing the permanent deck reinforcement). Damage further reduces the top chord capacity. The erroneous Acciona analysis also indicates the precast panel deflections are much less than by elastic theory. The poorly performed load testing where the kentledge acted as a rudimentary but effective 'top flange' meant precast panels didn't deflect as much as elastic analysis indicates resulting in the precast panels not being cambered and added to the confidence that the precast panel will be suitable and disguised the top chord buckling failure mode.
2. High deck curvatures at locations of fatigue fractured Temporary Rebar Truss. There seems to be a gap between the allowable fatigue stress range and the actual stress range based on reasonable assumptions of detail category and number of load cycles. The longer	The designers clearly understood that the Temporary Rebar Truss would suffer from fatigue effects as it is mentioned in the Construction drawings. It is therefore concluded that the designers assumed that fatigue cracking will occur through the welds only; not the truss chords. (Please note that 'Temporary' is regarded as a misnomer because the trusses are permanently and completely embedded within the permanent deck slab.) Assuming a Detail Category E (from AASHTO and AREMA bridge codes) and an estimated 1 million load cycles of 6tonne axles per year it is was calculated the Temporary Reinforcement Truss will fracture

term effects on deck stiffness of a heavily reinforced slab have not been fully considered.	within the first year of service. The annual number of load cycles was based on a number of 9 axle A and B trains and 4 axle trucks using two lanes causing maximum cyclic stresses of the deck transverse reinforcement. Research into fatigue design shows the fractures start as micro cracking at locations of steel embrittlement (from welding causing a heat affected zone) and concentration of stresses (stress risers). The invisible micro cracks propagate to form threshold cracks and these can further propagate suddenly to compromise the entire section or top and bottom truss chords in this instance. Supposedly the truss are not contributing to the bending strength of the deck slab, but they do contribute to the deck stiffness. A sudden fracture of the truss chords results in an abrupt change in deck stiffness. Simple elastic beam theory shows the inverse of radius of curvature is proportional to the inverse of beam stiffness. In the viaduct deck slabs this is likely to result in sharper curvatures and high strains in the concrete at points of abrupt change in deck stiffness. This is likely to result in the localised deterioration of cover concrete and so on.
3. Slender girder webs in sagging regions. Various bridge codes allow web depth to thickness ratios of between 133 to 152 to prevent web bend buckling. These ratios are exceeded with ratios of 165 to 167 being provided in the viaducts.	Simply not accounted for by the designer or checked by the reviewer.

Please keep me posted on the investigation. Before the lock down Acciona were reaching the point where the precast deck panels were to be transported to the Okahu viaduct from Wilsons Precast, Heritage Way, Otara, to lift onto the girders.

I have developed solutions for these issues should the NZTA investigation concur with our concerns.

On a personal note I believe these issues are too big to 'bury'. The issue becomes at what point do they get validated.

Kind regards

s 9(2)(a)

[redacted]

From: Barry Wright <Barry.Wright@nzta.govt.nz>
Sent: Thursday, 9 April 2020 11:55 AM
To: s 9(2)(a)
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi s 9(2)(a)

I think so

Will initiate and see what happens

Cheers
Barry

From: s 9(2)(a)
Sent: Thursday, 9 April 2020 10:52 AM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi Barry,

Is the process outlined below suitable?

That is find a way for the NZTA appointed investigator (from routine QA assessments) to approach s 9(2)(a) in the first instance based on what they discover from Acciona. If it is s 9(2)(a) then he should refer investigator queries to s 9(2)(a) Transport Lead, who will refer them to me if he is allowed to.

s 9(2)(a)

From: s 9(2)(a)
Sent: Tuesday, 7 April 2020 4:00 PM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi Barry,

Please see my responses in red below.

s 9(2)(a)

From: Barry Wright <Barry.Wright@nzta.govt.nz>
Sent: Tuesday, 7 April 2020 3:01 PM
To: s 9(2)(a) [redacted]
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi s 9(2)(a) [redacted]

Thanks for the update and the offer.

I think that I need to consult relevant senior project staff and engage a recognised independent person. Please do this. I would also suggest any site investigations only carried out at the end of the lock down when construction begins again but before any precast panels are lifted onto the girders, and; the NZTA should carry out its regulatory obligations in strict accordance with the appropriate QA procedures with this project but in a routine manner (Its $20M+ mistake with almost certain chance of being discovered – preferably before someone is injured or killed).

Would you be prepared to present your case to such a person. Initially this can be in confidence I think, but depending on subsequent events it may become obvious that you were involved. From the above Acciona s 9(2)(a) [redacted] will 'let slip' that s 9(2)(a) [redacted] had been involved which can then prompt your investigator to formally approach s 9(2)(a) [redacted] is our geotechnical engineer helping Acciona on geotech issues) for our assistance. In principle and ethically this should be forthcoming. This temporary rebar truss top chord buckling issue can then lead to the fatigue fracture/abrupt change in stiffness in deck, and the overly slender webs in the sagging regions of the main girders and anything else that may come up in the investigation.

Acciona is a platinum global client of s 9(2)(a) [redacted] Should s 9(2)(a) [redacted] choose not to participate then please let me know and NZTA and I can take it from there.

Is this OK with you?

Cheers
Barry

From: Barry Wright
Sent: Tuesday, 7 April 2020 3:01 PM
To: s 9(2)(a) [redacted]
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi s 9(2)(a) [redacted]

Thanks for the update and the offer.

I think that I need to consult relevant senior project staff and engage a recognised independent person.

Would you be prepared to present your case to such a person. Initially this can be in confidence I think, but depending on subsequent events it may become obvious that you were involved.

Is this OK with you?

Cheers
Barry

From: s 9(2)(a) [redacted]
Sent: Tuesday, 7 April 2020 2:41 PM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi Barry,

Confidential.

Acciona want to 'bury' these issues. They have confirmed the temporary rebar truss design with Acciona Ingeneria in Spain and received a letter of peer review from Case International (don't know them).

s 9(2)(a) [redacted] have performed a manual analysis (by myself) and had it confirmed by Strand 7 FE analysis independent of me in Auckland.

Given the discrepancy in our results we suggested that they raise the issue with the NZTA as Principal and Regulator. They maintain they are not obliged to 'close out' this temporary works design or notify the NZTA.

Please let me know if you want s 9(2)(a) [redacted] to assist with any NZTA investigation?

Kind regards

s 9(2)(a) [redacted]

From: s 9(2)(a) [redacted]
Sent: Monday, 23 March 2020 4:25 PM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi Barry,

We have finished the report into the Okahu and Puhoi Viaduct temporary reinforcement trusses.

Given various commercial imperatives I am not sure if or how it will be received by NX2.

Nevertheless the recommendations in summary are:

1. Act urgently and decisively
2. Revalidate, if able, the intended viaduct system by:-
   a. Getting the Spanish temporary works engineering to recalculate the safe capacity of the trusses
   b. Conduct further load testing this time to failure and without the kentledge bearing on the top chords
   c. Gather evidence of the longer term performance of welded trusses cast within the permanent deck
   d. Gather further evidence of the designers philosophy and peer review comments with regard to the temporary reinforcement trusses.
3. In parallel with 2 investigate an alternative deck system and recheck the super and sub structure design, cost and programme implications.

We have not recommended NX2 notify NZTA.

We have not offered any assistance with the above.

We have stated that we can not peer review the temporary reinforcement trusses as requested.

If the NZTA wishes to investigate further then there should be official channels as either the Principal or regulator to do this.

s 9(2)(a) [redacted]

From: s 9(2)(a) [redacted]
Sent: Monday, 23 March 2020 11:59 AM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi Barry,

I have finished my report. For various commercial imperatives there is reluctance to advance it. Understandable.

However basically my recommendations to NX2 are:

1. Act urgently and decisively.
2. Defend the current design by:- getting the Spanish temporary works engineer to confirm the temporary reinforcement truss design; conduct further precast panel load testing to failure with the kentledge separated from the top chord; gather evidence of the long term performance of the decks with a welded trusses within them; gather complete evidence of the designers philosophy and peer review comments with respect to the temporary reinforcement trusses etc.
3. In parallel with 2 develop an alternative deck solution and recheck the super and sub structures, construction programme and cost estimate.

We have not suggested to NX2 to notify the NZTA.

We have not offered further engineering assistance to the NX2 on this serious matter.

The report is 'matter of fact' with no opinions.

There may be another way the NZTA could raise he temporary reinforcement truss design with NX2 through official channels.

Kind regards

s 9(2)(a) [redacted]

I'll still be working in Wellington this week except this afternoon Thursday which are 'work from home' days for if you want to meet me to discuss.

s 9(2)(a) [redacted]

From: s 9(2)(a)
Sent: Tuesday, 17 March 2020 4:31 PM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi Barry,

Here is the first draft ES for my report.

Executive summary

Upon a request by NX2 (NX2Group – PPP special purpose vehicle to deliver P2Wk project for the NZTA) for s 9(2)(a) 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 the 'lynch-pin' for both modes of failure.

By calculation and assessment s 9(2)(a) have confirmed these deck failure modes are serious. s 9(2)(a) first notified NX2 of these potentially dangerous failure modes on 6 March 2020 by email. Because of the deck failure modes discovered, s 9(2)(a) are unable to peer review the Temporary Reinforcement Truss (TRT).

This report formerly outlines to NX2, and if necessary, the NZTA, two potential failure modes associated with the viaduct decks. One failure mode is associated with the sudden failure of the precast deck panels during construction. The other failure mode is associated with a premature decline in deck durability during its service life.

The first and most serious failure mode eventuates from the TRT top chord buckling failure either in preparation or during the topping pour. Over the two viaducts there are over 17,400m^2 of precast deck panels that could collapse suddenly and fall down nearly 20m at the lowest point of ground level. The nature, chance and consequence of failure render the proposed deck system as an unacceptable risk. It is strongly recommended that NX2 prove the resilience and demonstrate sufficient margins of safety of the deck system to the NZTA 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 because should the first occur this will prevent the viaduct decks being constructed as currently intended.

It is extremely concerning that such deck failure modes could be overlooked in the viaduct design process beginning up to three years earlier, progressing through peer review and only 'picked-up' when construction of the viaduct decks is imminent.

Seven recommendations have been put forward. There is onus on NX2 to demonstrate to the NZTA 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.

I also note the heavy lift crane hire sub-contractor had to remove the crane keys from their machines at both viaduct sites at the beginning of last week to ensure outstanding invoices were paid. Worrying signs?

s 9(2)(a)

From: Barry Wright <Barry.Wright@nzta.govt.nz>
Sent: Tuesday, 17 March 2020 11:23 AM
To: s 9(2)(a) [redacted]
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi s 9(2)(a) [redacted]

Let's see what response you get to your email.

I guess they will either accept and address your points or maybe go elsewhere.

If they go elsewhere then we can "step in" somehow to ensure the points you raise are addressed satisfactorily

Cheers
Barry

From: s 9(2)(a) [redacted]
Sent: Tuesday, 17 March 2020 11:04 AM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Morning Barry,

If it transpires that NX2 don't want to engage s 9(2)(a) [redacted] to review the Temporary Reinforcement Truss system for the viaduct deck system, then at what point, if any, would NZTA step in?

The NX2 know from the attached email bad news is on the way.

s 9(2)(a) [redacted]

From: s 9(2)(a) [redacted]
To: [redacted]
Cc: [redacted]
Subject: P2Wr - Puhoi and Okahu Viaducts - Proposed deck system peer review
Attachments: image003.png

Dear s 9(2)(a) [redacted]

Thank you for 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) [redacted] 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.

Kind regards

s 9(2)(a)
[redacted]

s 9(2)(a)

[redacted]

From: s 9(2)(a) [redacted]
Sent: Monday, 16 March 2020 9:42 AM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Morning Barry,

Currently I am estimating the number of load cycles to fracture the temporary reinforcement truss and the resulting change in stiffness in the deck and therefore the change in curvature. This may be an academic exercise but will allow me to more clearly think through the situation. Then I will write a report. I have a colleague verifying my buckling calculations by a limit state method.

Based on my calculations it is conceivable that NX2 will assume the temporary reinforcement truss top chords act as ideal fully restrained segments and therefore have sufficient capacity to allow the deck pour; and further assume the likelihood of fatigue failure within the 25 year concession period is low enough to proceed with the design as it is. If they do so then it is likely to be contrary to our recommendations which will be based both on probability of failure and consequence of failure.

Cheers

s 9(2)(a) [redacted]

From: Barry Wright <Barry.Wright@nzta.govt.nz>
Sent: Monday, 16 March 2020 9:15 AM
To: s 9(2)(a) [redacted]
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi s 9(2)(a) [redacted]

Currently I am waiting to see if you can resolve differences with the designer.

If this cannot be achieved then I am not sure what will happen on site - some form of independent arbitration may be required? Or as you have intimated they might seek an alternative reviewer.

I propose awaiting the final outcome of your interactions with the hope that agreement will be reached.

Cheers
Barry

From: s 9(2)(a) [redacted]
Sent: Friday, 13 March 2020 4:30 PM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi Barry,

Here is a table that outlines the permissible stress criteria of the temporary reinforcement truss top chords in the precast decks during pouring the topping. Any ratio above 1.0 or 1.05 is unacceptable. The outer precast panels have a cantilever which reduces the compression in the top chord at the critical section.

Situation		Okahu - Centre	Okahu - Outer	Puhoi - Centre	Puhoi – Outer
Fixed – Fixed & straight		0.87	0.75	1.06	0.92
Pinned – Pinned as a series of chords – a reasonable model		1.02	0.89	1.31	1.09
Missing restraint		1.60	1.39	1.96	1.71
Misalignment or crank (from damage to a 200mm long chord section between restraints and combining axial and bending)					
1/100	2mm	1.22	1.05	1.78	1.54
1/50	4mm	1.58	1.36	2.20	
1/33	6mm	1.93	1.67	2.66	
1/25	8mm	2.29	1.97	3.13	
1/20	10mm	2.64	2.27	3.60	
1/10	20mm	4.92	3.77	5.83	4.97

During topping pour they can expect span/deflection ratios of 160. Typically Live + Impact deflection ratios should be greater than 640 for comparison.

I offer no opinion as to the reasons behind designing such a slender precast panel or apparent reasons for oversights in not considering top chord buckling either in calculation or load testing (although I await test procedures and results).

I will have my calculations checked next week – perhaps using limit state (AS5100); I have used a working stress approach because of familiarity, speed, logic and transparency.

Even if the decks get poured safely and without major incident, that leaves the durability issues of the deck slab associated with change in stiffness (EI) at likely points of high moment (M) leading to sharp deck curvatures and high strains (1/R = M/EI). The change in stiffness is associated with fatigue fractured temporary reinforcement truss (at welds) embedded in the permanent deck slab and aligned transversely across the deck subjected to a high number of load cycles.

Cheers

s 9(2)(a) [redacted]

s 9(2)(a)

[redacted]

From: s 9(2)(a) [redacted]
Sent: Friday, 13 March 2020 11:07 AM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: Photos 12 March Okahu Viaduct and Okahu precast deck at Wilson Precasting, Otara - Confidential

Hi Barry,

Here are some photos from yesterday. We didn't get any closer to the girders than as shown.

I now have some Construction drawings (temporary reinforcement truss), verification measurements (by myself), and examples of use of this system elsewhere (provided by Acciona).

I am awaiting some information on the load testing procedures and results that where carried out on 3 of the Okahu deck panels.

Given all of the above I hold to my initial concerns.

I'm currently determining to degree of overstress in the top chords of the temporary reinforcement trusses and deflections during the topping pour. I can send you the numbers later today. I want to get my working checked next week.

The Acciona/NX2 Engineering Manager genuinely wants to know if they have a problem with the deck. s 9(2)(a) [redacted] will do a proposal to him along these lines. I think he has a lot of other issues to deal with.

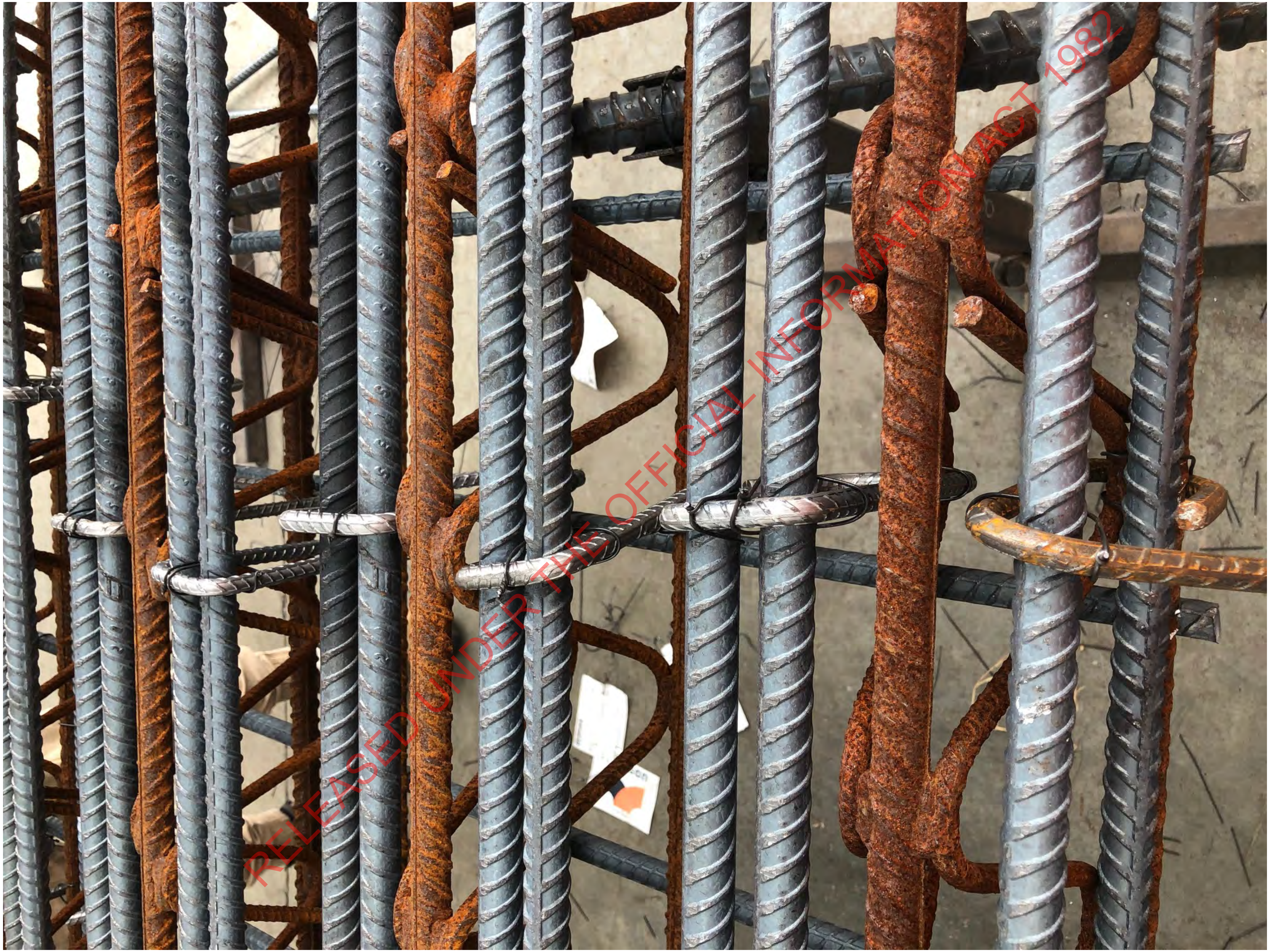
Cheers

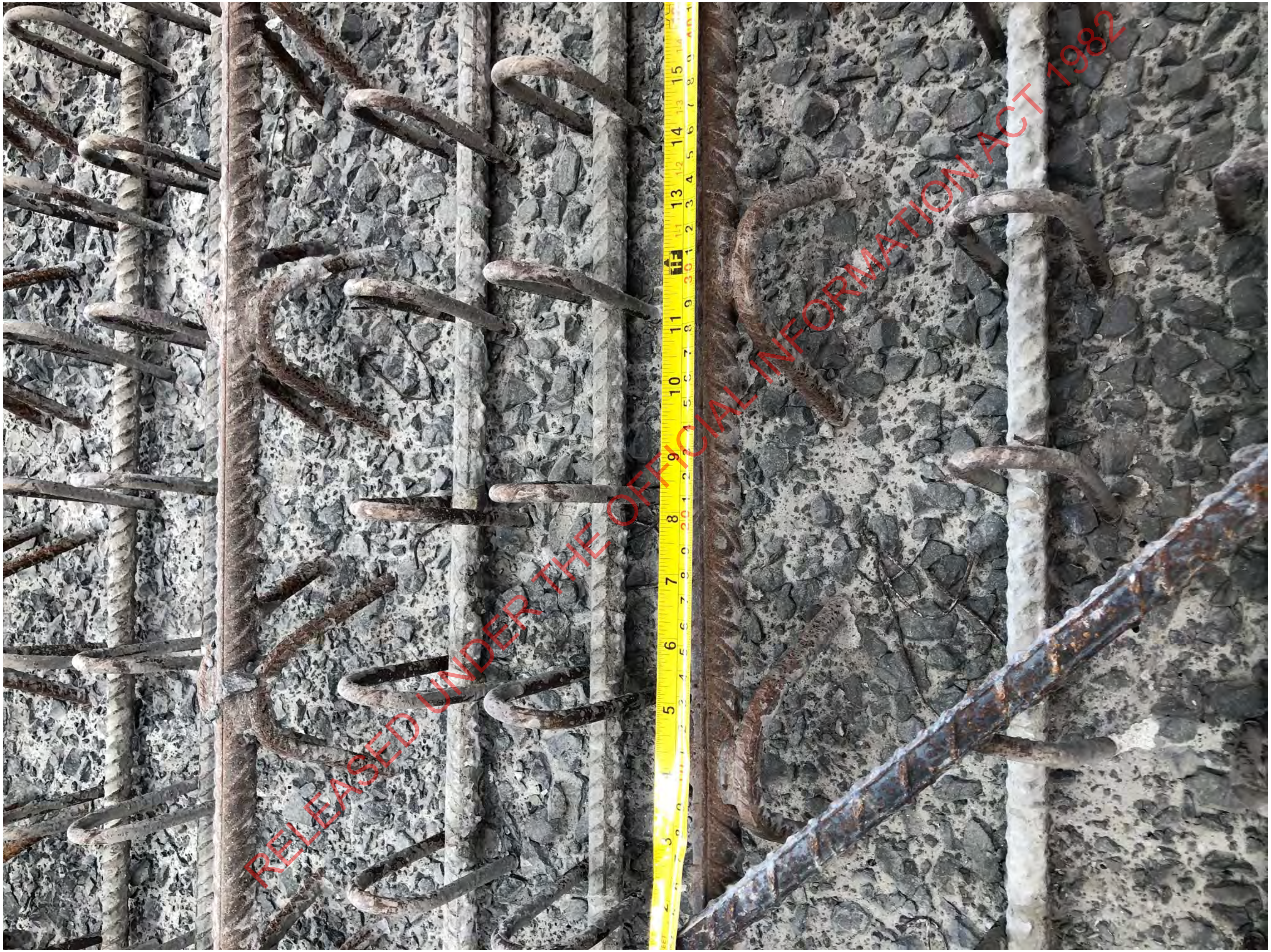
s 9(2)(a) [redacted]

s 9(2)(a) [redacted]


CAT
WHANGAREI MOXY HIRE
D8
B8
DZ054

s 9(2)(a)






s 9(2)(a)
s 9(2)(a)