From: s 9(2)(a)
Sent: Thursday, 21 May 2020 12:01 PM
To: Barry Wright <Barry.Wright@nzta.govt.nz>
Subject: RE: Confidential Query

Hi Barry
I've been giving this issue some thought over the last few days, the following is my considered response to the two questions that you raise below.

1. The ability of the temp rebar truss to support the load during construction without buckling
   A properly designed "precast deck with welded rebar truss" should be fine for temporary loads. This system is used routinely in Australia and around the world and there are several proprietary brands such and SMorgan Transfloor and Humeslab. Which you can google if you want to find out more.
   The strength of the deck panels may be difficult to analytically prove because of the curves in the web members and interaction with the concrete, so unless the constructors are using a proven design (eg welded trusses from SMorgan or similar) I'd advocate constructing a trial panel(s) and proof load it to failure to prove, or otherwise, its strength.

2. Permanent effects as described below
   I think it is accepted industry practice that the welded trusses may fracture due to long - term fatigue stresses. Provided the trusses are not relied upon to resist design actions, that is they are sized only to resist construction loads and effectively ignored for the design of the bridge, then I believe this will not create a long-term issue for the bridge. I recall this topic being discussed at an AustRoads Conference many years ago – pretty sure the common view at the conference aligned with what I describe above.
   On the second point raised below about whether fracture of a weld or bar creates a stress concentration to the detriment of the permanent reinforcement, I agree only partly with this. My opinion is that fatigue fracture of a weld or bar in the truss will create a stress concentration in the permanent bars, but provided the permanent bars have been designed (including designed for fatigue) on the assumption that the truss bars are not present then that should not result in a premature fatigue failure of the permanent deck rebar.

Without understanding the design basis of the deck slabs and reviewing the design, I cannot comment definitively on whether they are safe in the temporary or permanent state.

If you wanted to reach out to your Aussie mates RTA, MainRoads, etc, they may be able to put your mind at ease.
I think it might have been s 9(2)(a) who talked about this at the Austroads Conference, he would be another person to contact.
If you do touch base with them I'd be keen to hear their feedback.

Kind regards
Peter

s 9(2)(a) Senior Bridge Engineer
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ROADLAB Enjoy the Journey

From: Barry Wright <Barry.Wright@nzta.govt.nz>
Sent: Tuesday, May 19, 2020 10:09 AM
To: s 9(2)(a) [redacted]
Subject: Confidential Query

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

I have a potential problem on the Puhoi to Warkworth Viaduct bridge decks that has been raised to me in confidence as described below.

I would appreciate your initial opinion on whether you see these as valid concerns if you can, based on the information here.

Please charge to WBS 60054489

There are 2 possible problems relating to the temporary rebar truss as shown attached, namely;

1. The ability of the temp rebar truss to support the load during construction without buckling
2. Permanent effects as described below

Cheers
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 Mott MacDonald 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.

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.