STRENGTH TESTING OF BRIDGE BEAMS AFFECTED BY ALKALI AGGREGATE REACTION

R.B. STEVENSON, G.J. BEATTIE
Works Central Laboratories,
Lower Hutt, New Zealand

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AN IMPORTANT NOTE FOR THE READER

CONTENTS

EXECUTIVE SUMMARY

Eight hollow-core precast-prestressed concrete beams were removed and recovered in good condition from the Pahurehure Inlet Bridge No. 2 on the Auckland Southern Motorway, New Zealand, in 1988. The beams had been affected by alkali aggregate reaction (AAR), a reaction between alkalis in the cement and the aggregates in the concrete.

The opportunity arose in 1992 to compare the theoretical flexural strength of the beams with their measured strengths to quantify the effect of AAR. One of the beams was test loaded in flexure until failure, using a second beam as a reaction frame for the applied load.

The flexural strength at failure was between 4% and 9% greater than the theoretical ultimate flexural strength of the beam and approximately 30% greater than that which would be required by the then current (1991) bridge design standards.

The results show that the effect of AAR on the flexural strength of the beam was not significant because the strength at failure was greater than the theoretical ultimate flexural strength.

The prestressing steel quantity present meant that a ductile failure should have occurred, but the increase in steel strength as the strain increased beyond yield strain caused the beam to fail ultimately by crushing of the concrete.

The effect of AAR on the shear strength of the prestressed beam was not investigated in this project but the recommendation is that shear effect is investigated.

ABSTRACT

Eight hollow-core precast-prestressed concrete beams were removed and recovered in good condition from the Pahurehure Inlet Bridge No. 2 on the Auckland Southern Motorway, New Zealand, in 1988. The beams had been affected by alkali aggregate reaction (AAR), a reaction between alkalis in the cement and the aggregates in the concrete. One of the beams was test loaded in flexure until failure, using a second beam as a reaction frame for the applied load.

The aim of the test was to determine whether the flexural strength had been affected by AAR. The flexural strength at failure was between 4% and 9% greater than the theoretical ultimate flexural strength and approximately 30% greater than that which would be required by the then current (1991) bridge design standards.

The effect of AAR on the flexural strength of the beam was not significant because the strength at failure was greater than the theoretical ultimate flexural strength.

1. INTRODUCTION

The Pahurehure Inlet Bridge No. 2 on State Highway 1 at ERP 355/8.4 on the Auckland Southern Motorway, New Zealand, was dismantled in 1988 after extensive cracking was identified to have been caused by reaction of alkalis in the cement with the aggregates used in the concrete. This phenomenon is known as alkali aggregate reaction (AAR). When an alkali aggregate reaction occurs in concrete the alkali silica gel so formed expands, causing cracking in the concrete. In the Pahurehure Inlet Bridge No. 2 deck beams, St John (1988) identified lithoidal rhyolite as the principal reactive component.

The single span bridge was built about 1963 as part of the Auckland Southern Motorway. It crossed a channel joining two estuaries and had been exposed to salty air all its life.

Eight precast-prestressed beams were removed and recovered in good condition in 1988 for the then National Roads Board (now Transit New Zealand) to examine the effectiveness of two surface applied coatings. These coatings had been designed to prevent water from entering the concrete through the many fine cracks produced by AAR.

In 1992 this earlier project was coming to its end and an opportunity arose for comparing the theoretical flexural strength of the beams with their present measured strength to quantify the effect of AAR.

The objective of the study was to load test one beam, and use a second beam as a reaction frame for the loads applied to the beam under test. The beams were jacked apart until failure of the test beam occurred so that the failure load could be compared with the theoretical ultimate flexural strength, and with the beam failure moment which would be required by the then current design standards (*Bridge Manual: design and evaluation*, Transit New Zealand 1991). The test setup is shown in Figure 1.

At the time of dismantling (1988), longitudinal cracking was evident along the outside edge of the outside beam and on the undersides of the deck beams. These cracks were clearly visible on the two beams used for the test. There was no evidence of transverse cracking on the undersides of the beams (soffits), probably because the expansive effect of the AAR in the longitudinal direction was being resisted by the prestressing forces in the beams. There was no evidence of rust staining on the beam soffit caused by penetration of water to the prestressing strands. On the sides there was some evidence of corrosion of vertical shear reinforcement because of thin concrete cover (10 mm or less). More detailed description of the cracking is provided by St John (1988).

Figure 1. Reaction beam resting on timber dunnage on test beam before commencement of the test and before rams and loadcells are placed.

2. THEORETICAL ANALYSES

Before undertaking the tests, the exact details of the beams were not known because all the design calculations and drawings of the Pahurehure Inlet Bridge No. 2 beams had been destroyed in a fire some years ago. Photographs of the units indicated that they were similar to beams on another bridge, at Slippery Creek located further south along the Auckland Southern Motorway. Cross-sectional drawings and beam spans for the Slippery Creek bridge were available, and these were used in the preliminary calculations when considering the feasibility of inverting one beam so that it could be used as a reaction beam.

At the time of testing, accurate cross-sectional dimensions were made of the beams, and the locations and sizes of the prestressing strands were recorded and used in other calculations.

Cross-sectional details are presented in Figure 2. The prestressing steel consisted of three layers of ⅜" (9.52 mm) diameter Bridon Seven-Wire prestressing strand. The top layer contained four strands, the middle layer 14 strands, and the lower layer 18 strands. Details of the shear reinforcement were not established.

Cores of concrete were taken from the beams stored adjacent to the test beam, in order to obtain a representative compressive strength for the concrete. Four 100 mm diameter cores were tested for compressive strength information. However, two core results were disregarded as their length to diameter ratios were less than 1. The remaining two cores gave compressive strengths of 52.3 MPa and 49.8 MPa respectively, yielding a mean compressive strength, f'_c, of 51 MPa.

The bending moment at which first flexural cracking of the beam soffit would occur was calculated for the beam, assuming that prestress losses at the time of testing would be of the order of 20%. The second moment of area of the uncracked beam section, I, was calculated after transforming the prestressing steel area into an equivalent concrete area. Using an assumed maximum concrete tensile stress of 3.57 MPa ($0.5\sqrt{f'_c}$), the cracking bending moment was calculated to be 352 kNm.

To calculate the bending moment at the onset of yield of the prestressing strands, the initial yield strain in the bottom layer of prestressing steel needed to be established. A strain (ε) of 0.007 was obtained from British Ropes Ltd (1962). Assuming that the beam was cracked at the onset of yield and that the strain distribution was linear through the section, the bending moment at the onset of yield of the prestressing strands was calculated to be 753 kNm.

Assuming that the prestress losses would be of the order of 20% and the concrete compressive strength would be 51 MPa, the theoretical ultimate flexural strength of the section was calculated by iteration of the depth to the neutral axis, c, using the following procedure.

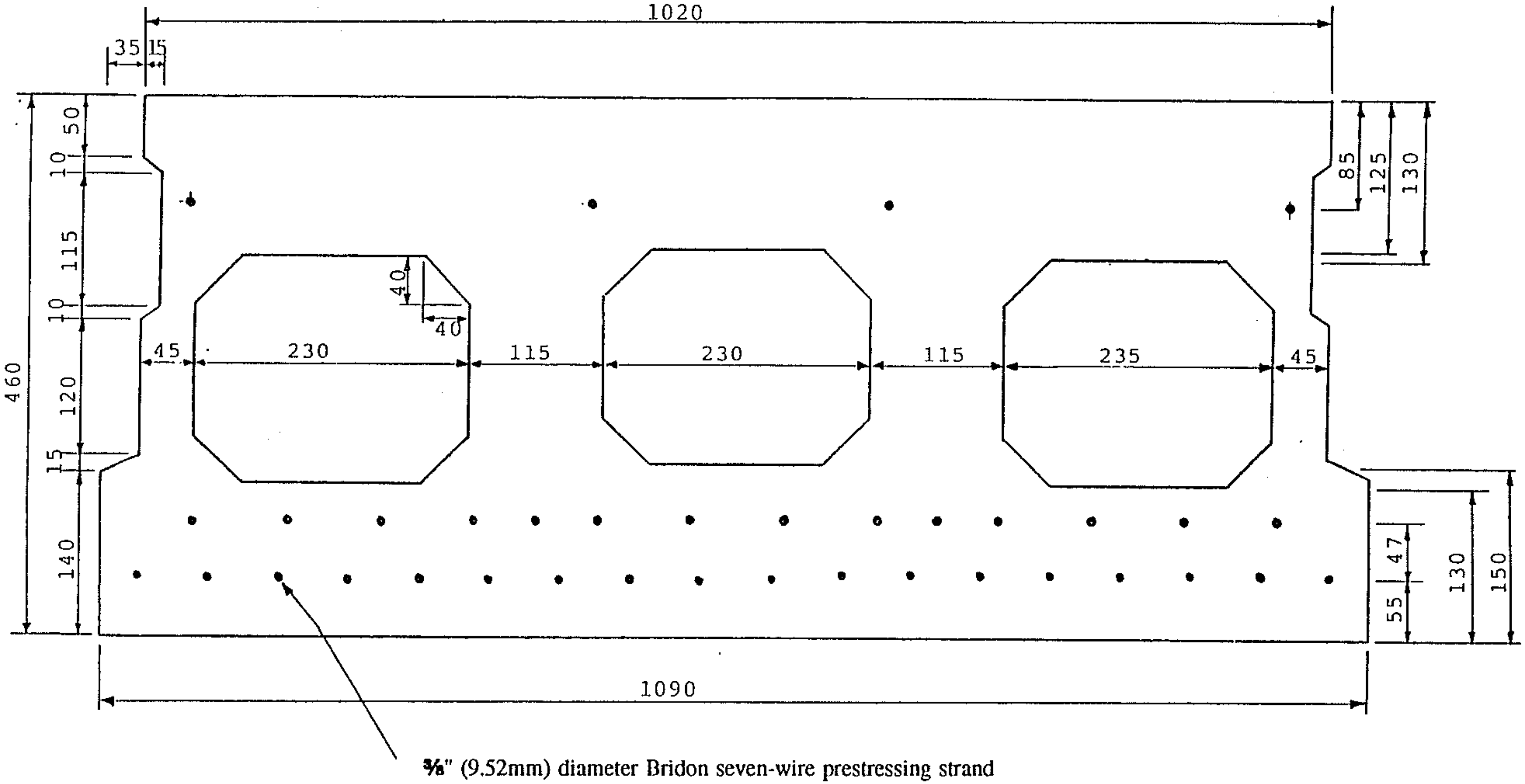
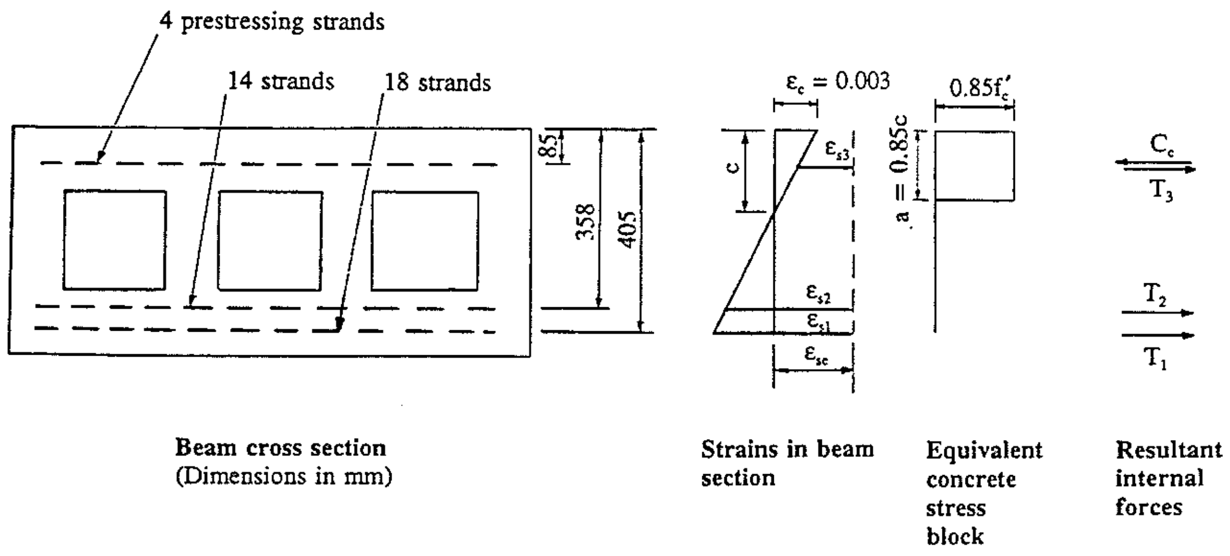


Figure 2a. Cross sectional details of the tested beam (dimensions in mm).



ε_c	=	maximum compressive strain in the concrete (assumed to be 0.003)
ε_{se}	=	strain in the prestressing strands after losses
$\varepsilon_{s1}, \varepsilon_{s2}, \varepsilon_{s3}$	=	strains in the prestressing strands at the theoretical ultimate flexural strength
c	=	depth to the neutral axis of the section (mm)
a	=	depth of equivalent concrete stress block (mm)
f'_c	=	compressive strength of the concrete (N/mm^2)
C_c	=	compression force in the equivalent concrete stress block (kN)
T_1, T_2, T_3	=	tension forces in the layers of prestressing strands (kN)

Figure 2b. Calculation of theoretical ultimate flexural strength of the beam section.

From British Ropes Ltd (1962) the typical modulus of elasticity, E, for Bridon 7 wire prestressing strand is 20,040 kg/mm^2 (= 1.96×10^5 MPa). Also from this reference, the stressing load, F_s, is given as 6,668 kg, and the strand cross-sectional area, A_s, as 51.6 mm^2.

Therefore after 20% losses, $\varepsilon_{se} = \dfrac{6{,}668 \times 0.8}{20{,}040 \times 51.6} = 0.00516$

A value of c was assumed. By trigonometry, the strains in the prestressing strands were determined. The values of T_1, T_2 and T_3 were then determined using:

$$T = \varepsilon.E.A_s.n$$

where n = number of strands in the layer.

If ε was found to be greater than 0.08, the graph of load versus extension on page 31 of British Ropes Ltd (1962) was used to determine the corresponding load, T_{py}, in the strand and then the values of T_1 T_2 and T_3 were determined using:

$$T = T_{py}.n$$

A number of values of c were trialled to find iteratively a solution to the internal force equilibrium equation, $C_c = T_3 + T_2 + T_1$. With a value of 85 mm for c, a close match between the left and right sides of the equation was obtained.

The theoretical ultimate flexural strength was then calculated to be 1067 kNm. At this strength the forces in the bottom layer of steel were nearing the breaking load given in British Ropes Ltd (1962). A sensitivity analysis was conducted to ascertain the variation in strength caused by variations in the concrete compressive strength, the prestress losses and the peak compression strain in the concrete. Little difference in the flexural strength occurred with an assumed loss figure of 15%. A similar observation was made when the peak concrete strain was increased to 0.004. With an assumed concrete compressive strength of 60 MPa, the flexural strength only increased to 1096 kNm.

Thus the ultimate flexural strength of the beam was expected to be in the order of 1050 to 1100 kNm.

At the limiting concrete strain of 0.003, the strains in the two bottom layers of prestressing strands were well beyond the yield strain, indicating that a ductile failure (typical of an under-reinforced beam) ought to occur.

As stated in the Introduction, cracking was parallel to the prestressing strains. The flexural strength of the section is provided by a force couple between the concrete in compression at the top of the beam and tension in the prestressing strands at the bottom of the beam. The preload from the prestressing force on the beam had prevented any transverse cracks from developing. Such cracks would have had little effect on the flexural strength of the beam, but would have caused an increase in the flexibility of the beam because they would have to close before the concrete could properly resist the flexural compressive forces. The shear strength of the beam would not be reduced by the presence of the longitudinal cracks because they were not along the transverse potential shear failure planes.

3. METHOD OF TEST

At the site where the beams had been stored following dismantling of the bridge, five beams had been supported at their ends on two others, and an eighth beam had rested on the ground adjacent to the others.

Two beams were selected from the five supported beams for the test. Because of the eccentricity of the prestressing force required to resist normal gravity loads in the beams, it was necessary to turn one beam upside down before positioning it above the other. Preliminary calculations indicated that this beam could be temporarily supported on its side at its ends and at mid-span without becoming unstable. Similarly, this beam was calculated to be stable when inverted provided that the two supporting points were within the middle third of its length, and that the crane slings were also positioned within the middle third.

Cribs of timber dunnage were used to support the upper beam above the lower beam (Figures 1 and 3) before the commencement of jacking.

At the ends of the beams, double channels and Macalloy tension rods were used to transfer the reaction forces from the top beam to the ends of the bottom beam. The test load was applied at the centre span point by two 30 tonne rams resting on spreader beams (Figures 3 and 4). These rams were controlled independently but their loads were maintained at the same level throughout the test.

Between each ram and the upper beam a strain-gauged loadcell was coupled to a strain indicator. The product of the indicated strain and the calibration constant for the loadcell gave the applied load.

To measure the displacements of the two beams at the centre and the ends, a total of 12 wooden rulers were fixed to the side faces of the beams. Two automatic dumpy levels were used to record the change in displacement at each measuring point. The first level was used to record the changes at one end of the beam and at its centre. The second recorded the changes at the other end and also at the centre, thus providing a double check. Each level was periodically checked against datum points remote from the test area to ensure that no movement of the level itself had occurred.

As the applied load was increased, gradually the dead weight of the upper beam transferred from the timber dunnage supports onto the rams. As the load increased beyond the dead weight of the upper beam, the reaction was shared by the Macalloy tension rods at the beam ends (Figures 3 and 4).

The applied load was increased in the first cycle until it reached 290 kN, and then allowed to reduce to 32 kN. In the second cycle the load was increased until failure of the bottom beam occurred, which was at a peak load of 365.4 kN.

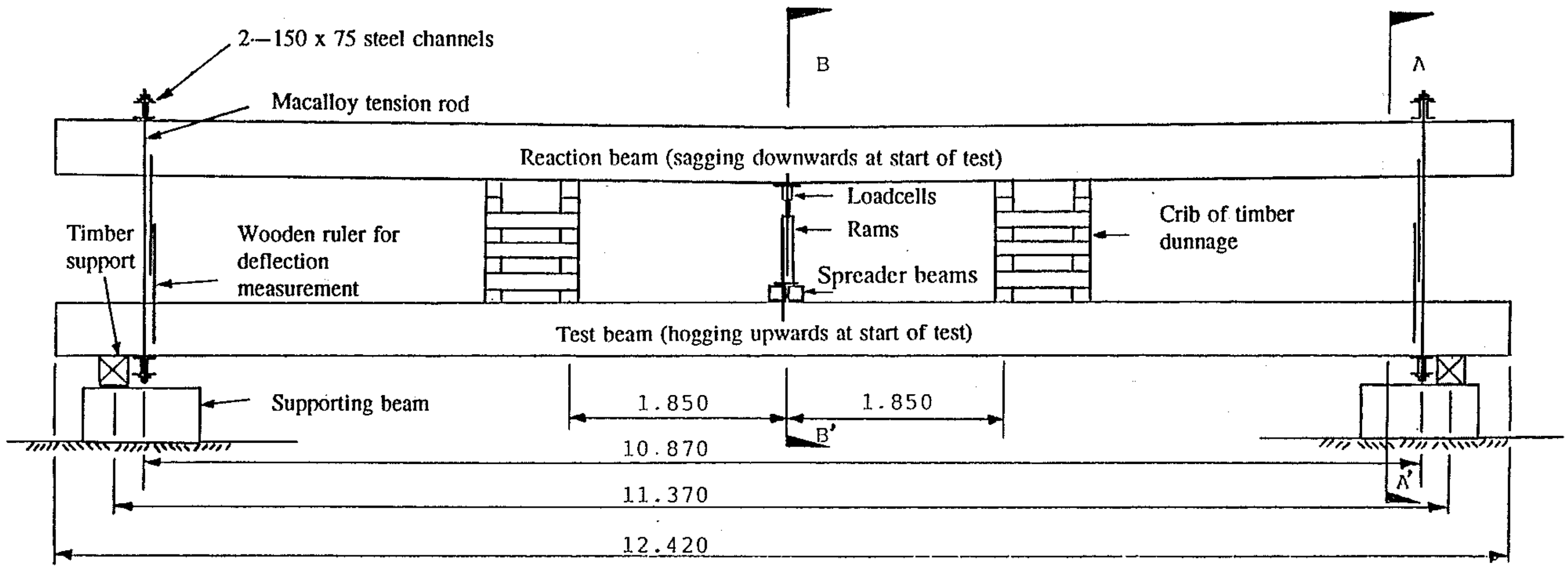


Figure 3. Elevation of test setup (dimensions in metres). A – A', B – B' positions of cross sections in Figure 4.

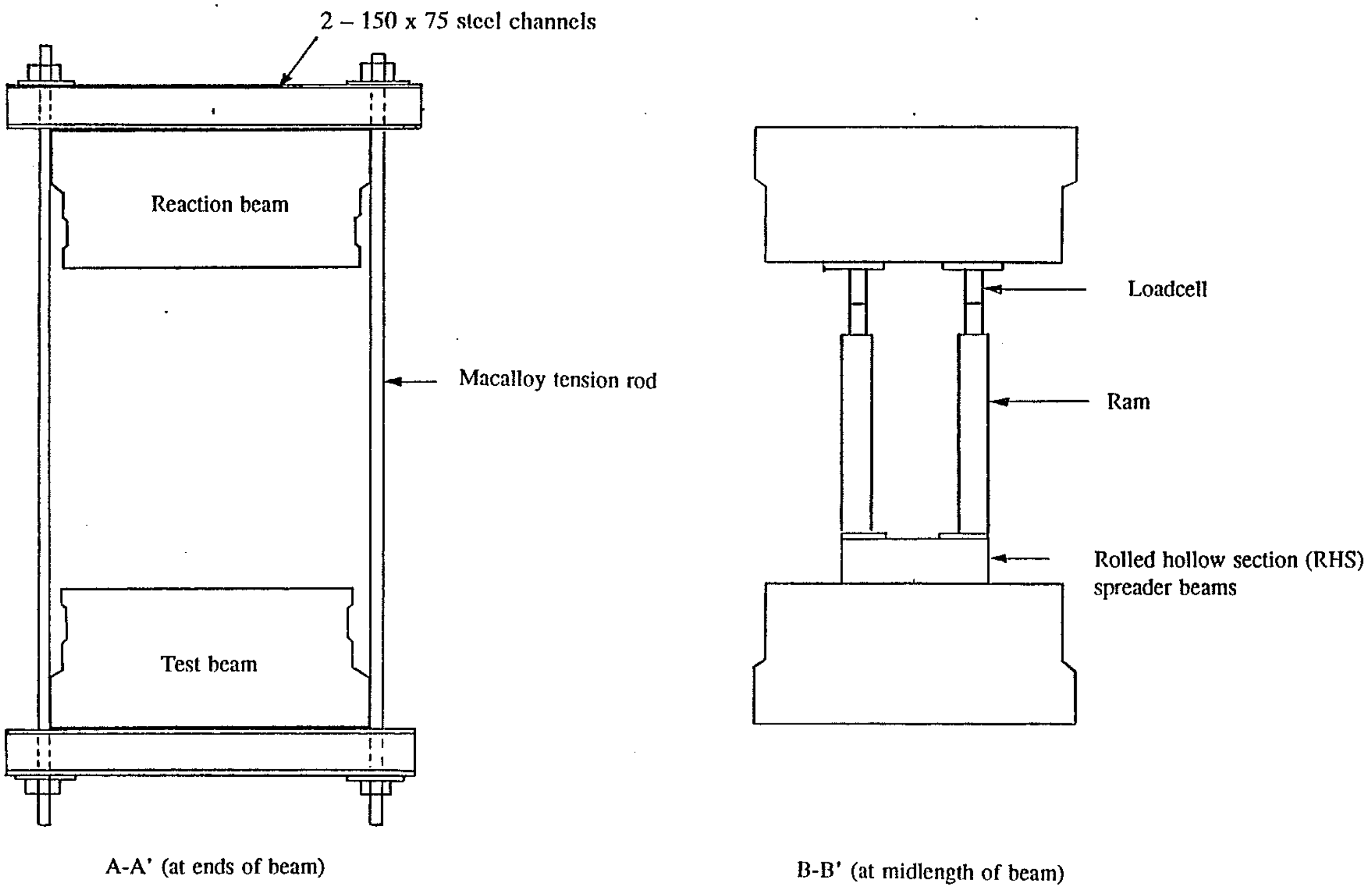


Figure 4. Cross sections A – A', B – B' of test setup (see Figure 3 for positions).

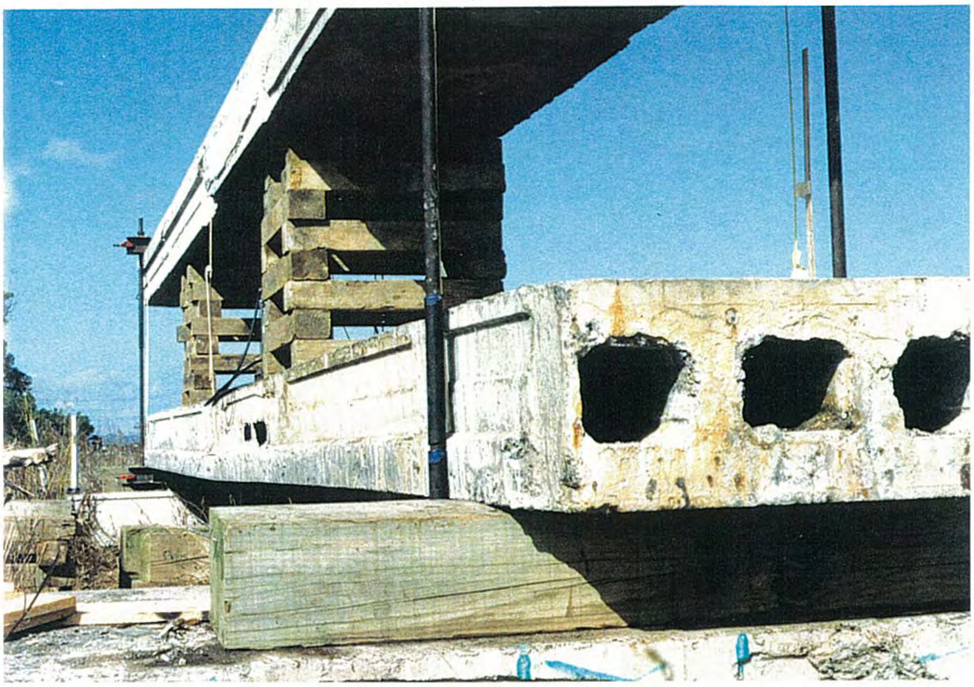
4. TEST RESULTS

4.1 Description of Behaviour

During the first cycle to 190 kN loading (bending moment of 941 kNm) the obvious upward curvature (hog), evident in both beams when placed the correct way up before the start of the test, had reduced to nothing in the reaction beam and had changed to a sag in the test beam, as can be seen in Figure 5. Flexural tensile cracking was visible on the soffit of the test beam over the centre 2 m of its span, with these cracks extending approximately 150 mm up the side faces of the beam.

Once the applied load had returned to 32 kN (bending moment of 372 kNm), these cracks had closed up but were still visible. Both beams had some residual deflection present when compared to this point in the first cycle.

Figure 5. Test at 290 kN in first cycle. Note sag in test beam.



During the second cycle the beam deflections increased significantly as the load increased beyond the peak loading of the first cycle. As before, tension cracks opened on the lower beam soffit. Just before failure, diagonal cracking on the side face of the beam, indicative of an impending concrete crushing failure, was visible (Figure 6). The deflected shape is clearly visible in Figure 7.

Figure 6. View on side of bottom beam at loading point in the second cycle, just before failure. Note cracking pattern.

Figure 7. Test just before failure in the second cycle.
Note significant sag in test beam.

Collapse when it occurred was sudden and brittle. Adjacent to and under the ram spreader beams the concrete crushed, allowing the test beam to shorten as the strain in the lower prestressing strands was relieved. With no concrete compressive resistance provided for the flexural force couple in the test beam, the beam collapsed down onto a safety support. Even though the prestressing strands were yielding at collapse, as the strain increased so did the stress in the strands because there is no defined yield plateau in the stress-strain plot for Bridon strands (British Ropes Ltd 1962). At collapse, the maximum strain that the concrete could withstand was reached and crushing occurred. Soffit cover concrete fell off the beam, exposing the lower layers of prestressing steel (Figure 8). None of the prestressing strands had broken, and neither were they significantly affected by corrosion. At failure of the test beam, the mid-span moment was 1145 kNm.

Figure 8. View on underside of lower beam at mid-span. Note that cover concrete has been lost but all prestressing strands are intact.

When the beams were designed (approximately 1963) the governing basis of design for the prestressed beams was likely to have been the limiting flexural stresses on the section (e.g. zero tension at the bottom of the concrete section) rather than ultimate strength. The codes in use at the time specified a check of steel percentage to ensure that the beams were not over-reinforced. Over-reinforcing could result in a sudden concrete compression failure.

Since 1968 prestressed concrete beams have been required to be designed to fail in a ductile manner. A ductile failure in the test beam resulted because its prestressing steel percentage was found to lie between the percentage causing fracture of the prestressing steel and the percentage causing crushing of the concrete, as defined in the 1968 prestressed concrete standard recommendation (NZS 32:1968, SANZ 1968).

In this NZS 32:1968 standard, the process of a ductile failure is described as excessive elongation of the steel followed by crushing of the concrete. The very large cracks and the considerable deflection of the beam are considered by the standard to be ample warning of an impending failure.

A small amount of shear steel was present in the beams. Single R10 ties were evident at approximately 600 mm centres although the spacing of these was quite variable. Offcuts of prestressing strand were used as transverse shrinkage steel in both the top and bottom faces of the unit. In the bottom faces, these were straight lengths, but in the top faces they turned down into the side webs for approximately 75 mm.

4.2 Moment–Deflection Plots

As described in Methods of Test, loads and deflections were recorded for both reaction and test beams during for the test.

Using the procedure detailed in Appendix 1, at each load step the mid-span moment in each beam was calculated and plotted against the mid-span deflection. The graphs of moment versus deflection for the reaction and the test beams are presented in Figures 9 and 10 respectively. Neither graph exhibits any significant linear behaviour over the plotted range. However, the deflected shape of each beam before the application of the ram loads was taken as a zero point with all future deflections referenced to that state.

The extent of cracking related to AAR was mainly in the longitudinal direction, and therefore the non-linear behaviour of the plots is not expected to have been caused by the closing up of any transverse cracks.

The calculated cracking moment was 352 kNm. The dead weight of the reaction beam alone caused a bending moment in the test beam of 342 kNm. Because the dead weight was the lowest load that could be applied to the test beam, there was no opportunity to determine the relationship between moment and deflection below 342 kNm.

As the bending moment increased above the cracking moment, the curvature of the plot was indicative of an increasing flexibility of the beam caused by:

(a) crack opening, reducing the second moment of area of the beam; and

(b) above 753 kNm, yielding of the prestressing strands.

Figure 9. Reaction beam mid-span moment v deflection during testing.

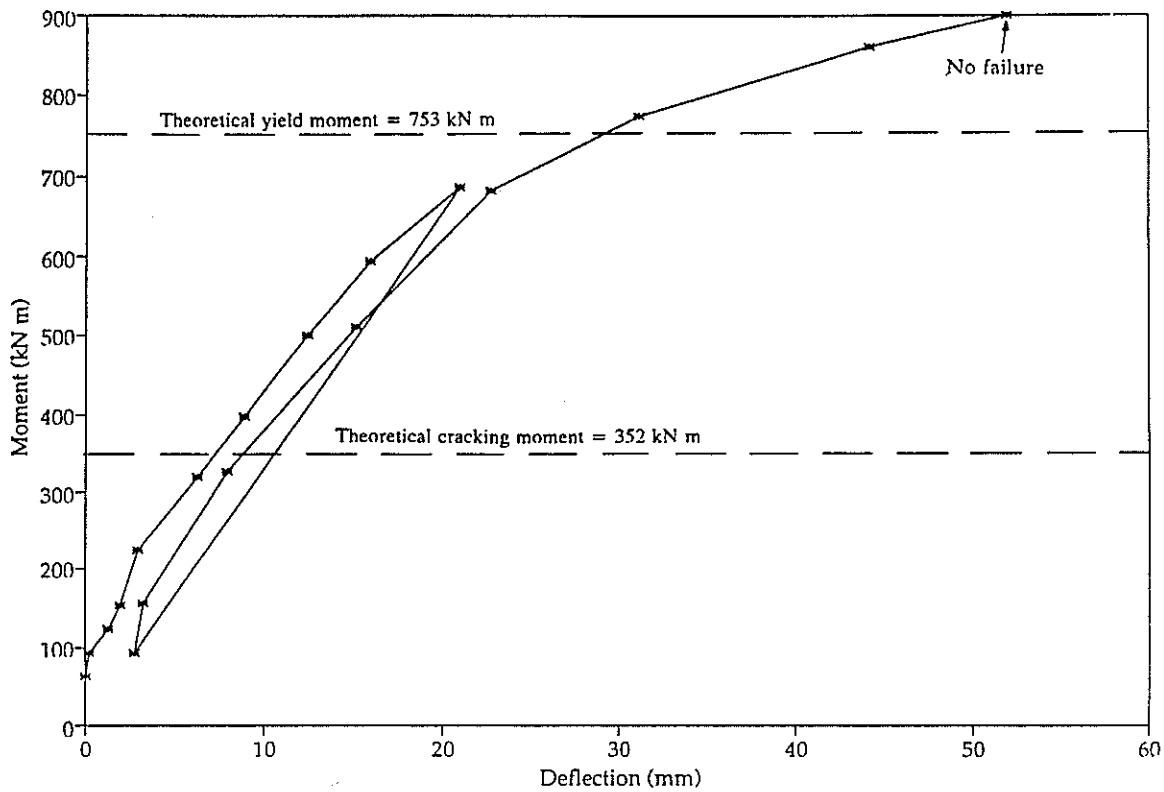
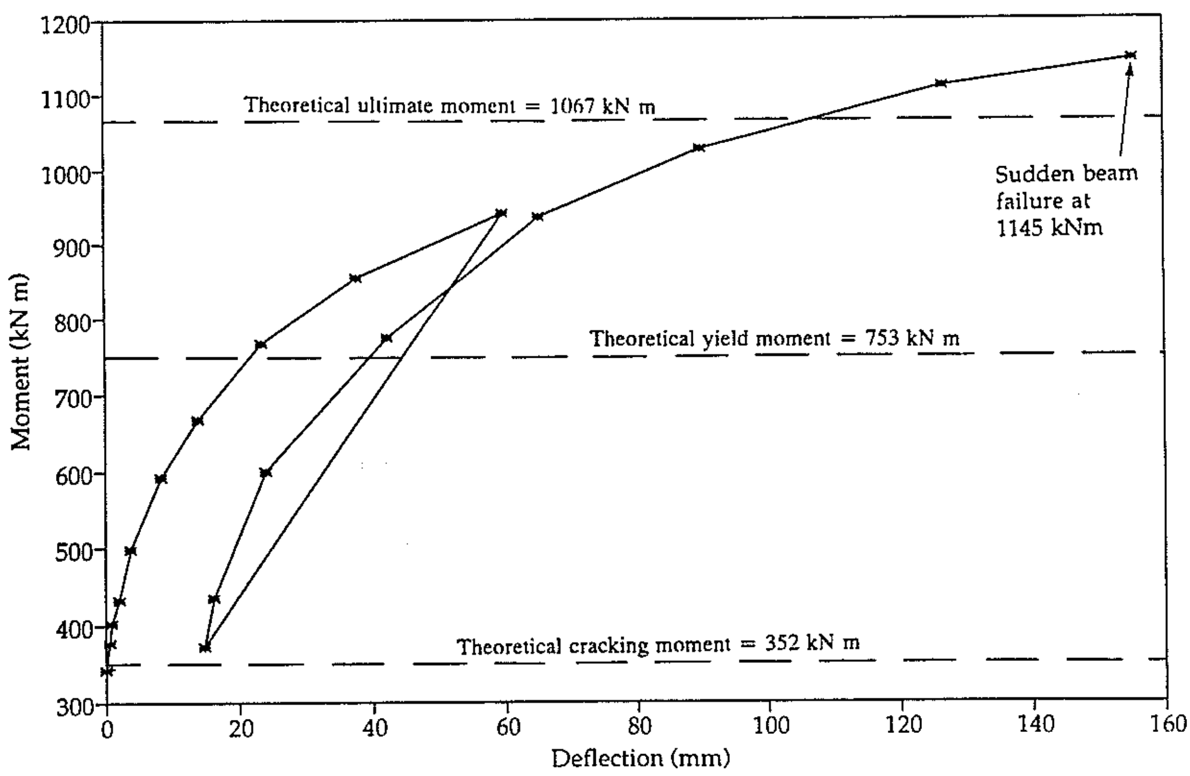


Figure 10. Test beam mid-span moment v deflection during testing.



5. COMPARISON OF TEST RESULTS WITH CALCULATED THEORETICAL ULTIMATE FLEXURAL STRENGTH

The maximum moment reached in the test beam was 1145 kNm, and the theoretical ultimate flexural strength was between 1050 and 1100 kNm. The correlation is considered to be good. The theoretical strength was calculated using the measured compressive strength of the concrete (51 MPa) and the strength properties of the steel which are given in British Ropes Ltd (1962).

6. COMPARISON OF TEST RESULTS WITH A BEAM DESIGNED TO 1991 STANDARDS

After discussions with staff of Works Consultancy Services' bridge design office (P. Stanford, pers. comm.), the completed deck of a bridge construction such as the Pahurehure Bridge was expected to behave as a single unit. Assuming this, the beam failure moment which would be required by the current design standards (*Bridge Manual: Design and Evaluation*, Transit New Zealand 1991), is calculated in Appendix 2, and it is 875 kNm. (It should be noted that for the calculation in Appendix 2 the author has had to make certain assumptions.)

This calculation indicates that the bending moment at which the test beam failed (1145 kNm) was approximately 30% greater than the beam failure moment which would be required by current design standards. (These 1991 standards use the bridge design loading HN-HO-72, originally introduced in 1972.)

As previously mentioned no information regarding the original design is available. However, it should be noted that:

- the original design criteria for the beam were probably based on working stress rather than ultimate strength, and
- the materials have probably tested stronger than the original design would have assumed.

7. CONCLUSIONS

Strength testing and calculations carried out on the AAR-affected prestressed concrete beam taken from the Pahurehure Inlet Bridge No. 2 show that the flexural strength at failure was:

- between 4% and 9% greater than the theoretical ultimate flexural strength, and
- approximately 30% greater than that which would be required by current design standards.

These results show that the effect of AAR on the flexural strength of the beam was minimal because the strength was greater than the theoretical ultimate flexural strength.

These results also suggest that the beams need not necessarily have been replaced. However, only one beam was tested and the effect of AAR on the shear strength of the beam was not investigated in this project.

While the prestressing steel quantity was determined to be sufficiently low to initiate a ductile yielding steel failure, the gradual increase in steel strength beyond its yield point with increasing strain ultimately caused the beam to fail by crushing of the concrete.

8. RECOMMENDATION

It is recommended that consideration is given to testing more such AAR-affected beams, and that this testing should include both the flexural and shear strength testing of several beams.

9. REFERENCES

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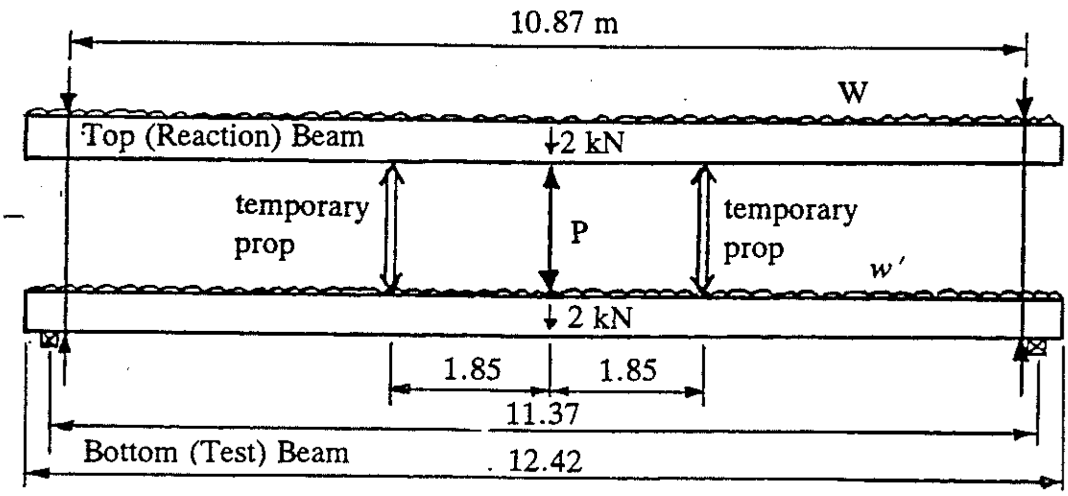
APPENDIX 1

CALCULATION OF
MID-SPAN BENDING MOMENTS IN THE BEAMS

CALCULATION OF MID-SPAN BENDING MOMENTS IN THE BEAMS

Diagram of Test Setup:



P = load applied by jacks
(dimensions in metres)

Calculation:

Beam x section = 0.34764 m^2

Assume concrete density is 24 kN/m^3

=> w = w′ = 0.34764 x 24 = **8.34 kN/m**

= distributed dead load along the beam (kN/m)

Filled voids over the centre of each beam = **2 kN**

=> total beam weight = 8.34 x 12.42 + 2 = **105.6 kN**

Bottom (Test) Beam Moments:

At any load below P = 105.6 kN

At any load above P = 105.6 kN

Top (Reaction) Beam Moments:

At any load below P = 105.6 kN

At any load above P = 105.6 kN

APPENDIX 2

CALCULATION OF BEAM FAILURE MOMENT REQUIRED BY 1991 DESIGN STANDARDS

CALCULATION OF BEAM FAILURE MOMENT REQUIRED BY 1991 DESIGN STANDARDS

Use the design loading requirements of the 1991 Transit New Zealand *Bridge Manual: Design and Evaluation.*

Assumptions regarding the load distribution to the beams are required because of the limited knowledge available about the original design. Assume that the beams act together as a unit with shear connection between beams. From an original drawing use a carriageway width of 10.4 m over 12.5 beams, which is required to be designed for three load lanes.

Weight of concrete including embedded steel = 25 kN/m^3
Cross sectional area of beam = 0.34764 m^2
Weight of beam = 0.34764 x 25 = 8.69 kN/m
Superimposed dead load (SDL) = 1.5 kN/m^2
SDL = 1.5 x 1.090 (beam width) kN/m beam = 1.64 kN/m
Dead load (DL) = 8.69 + 1.64 = 10.33 kN/m
Beam span = 12.2 m
DL moment = $wL^2/8$ = 10.33 x 12.2^2 / 8 = 192 kNm

HN (normal) moment with impact (I) = 847 kNm (per HN load element)
HO (overload) moment with impact (I) = 1446 kNm (per HO load element)

Consider normal live load (LL)
    HN in each of the three load lanes
    assume an eccentricity factor (EF) of 1.15 for the critical beams
    M = EF x load / no of beams = 1.15 x (3 x 847) / 12.5 = 234 kNm
Group 1A: M_u = 1.35(DL + 1.67 LL x I) = 1.35(192 + 1.67 x 234) = 787 kNm

Consider overload combination of traffic loads (OL)
    HO in one load lane and HN in each of the other two load lanes
Group 4: M_u = 1.35(DL + 1.1 OL x I)

Assume normal live load (LL) governs, hence M_u = 787 kNm

Design of concrete section includes a further factor of safety of 0.9, therefore
    787/0.9 = 875 kNm

Beam failure moment which would be required by current design standards = **875 kNm**