Evaluation of shear connectors in composite bridges

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An important note for the reader

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Abbreviations and acronyms

50MAX	heavy vehicle with one more axle than conventional 44-tonne vehicle combinations
AISC	American Institute of Steel Construction
AS	Australian standard
BS	British standard
f’c	concrete compressive strength (MPa)
fy	tensile strength
fs	allowable stress in tension
fv	basic allowable mean shear stress on structural steel webs
Fv	allowable stress in tension in vertical stirrups
HERA	NZ Heavy Engineering Research Association
HPMV	high-productivity motor vehicle
MoW	Ministry of Works
NZS	New Zealand standard
RFP	request for proposal
SH	state highway
ULS	ultimate limit state
WP2	Work Package 2

Contents

Executive summary

There are approximately 270 bridges on New Zealand's state highway network and many more bridges on local roads with steel concrete composite superstructures. Most of these consist of reinforced concrete decks connected to braced steel I-beams, with welded channels or studs used to provide longitudinal shear connection. Over 70% of these bridges were constructed between 1950 and 1970, of which approximately three quarters were designed by the Ministry of Works. Based on their design live loading (typically H20-S16 or H20-S16-T16), the majority of these bridges are expected to be capable of supporting full high-productivity motor vehicle loading. However, significant variability currently exists in the assessed live load capacity of composite bridges, even when they are designed to identical design loadings.

This report first reviews international experiments for shear connectors and the development of design equations in different national standards. From an investigation of as-built records, the report also outlines the types, geometry and material properties of historic shear connectors in New Zealand and identifies the design standards for the historic bridges. The most common type of shear connector is the welded channel which was used in more than 67% of the historic composite bridges. A further 24% of bridges utilised the welded V-angles connector, while the remaining consist of shear studs, bent plates and riveted angles.

The performance of different existing design models for channel shear connectors is evaluated through reliability analysis prior to the development of a new equation, which ensures that the target margin of safety is achieved.

The proposed evaluation procedure for composite bending capacity adopts the Eurocode-based solution which covers a wide range of application. Developed from modern composite beam theories, the solution provides more design options for engineers when dealing with full/partial shear connection, ductile/non-ductile shear connectors, compact/non-compact steel sections and the degree of shear connection. Further requirements are given for the propped and unpropped construction methods.

The evaluation procedure given is an extension of the existing evaluation steps in the NZ Transport Agency's *Bridge manual* (3rd edition). The procedure incorporates the newly developed design equation of the channel shear connectors. A self-contained evaluation procedure is prepared in appendix H which can be used as an independent guide.

Fully worked examples are also given to support the proposed evaluation procedure and enable comparisons to be made with the existing methods in NZS3404/AS5100.6, where only the rigid plastic theory-based method is allowed. While similar results are achieved for cases where full shear connection is provided, for beams with non-ductile shear connectors or a lower degree of shear connection η, the proposed evaluation procedure provides bending moment capacities between 33% and 100% of that advised using the current NZS 3404 and AS5100.6 for the range of η between 0.7 and 0.85.

Abstract

There are approximately 270 bridges on New Zealand's state highway network and many more bridges on local roads with steel concrete composite superstructures. From an investigation of as-built records, most of these consist of reinforced concrete decks connected to braced steel I-beams, with welded channels or studs used to provide longitudinal shear connection. Over 70% of these bridges were constructed between 1950 and 1970, of which approximately three quarters were designed by the Ministry of Works. Significant variability currently exists in the assessed live load capacity of composite bridges, even when they are designed to identical design loadings.

This report reviews international experiments for shear connectors and the development of design equations in different national standards. A new equation for channel shear connectors was developed and evaluated through reliability analysis to ensure the target margin of safety was achieved.

An evaluation procedure for composite bending capacity is proposed in this report, incorporating the newly developed design equation of the channel shear connectors and adopting the Eurocode-based solution, which covers a wide range of application. The evaluation procedure is an extension of the existing evaluation steps in the NZ Transport Agency's *Bridge manual* (3rd edition).

1 Introduction

Steel-concrete composite construction comprises a reinforced concrete deck slab on top of steel girders. It is widely adopted in highway bridges since the construction considerably increases the beam load capacity and stiffness. The shear connectors play an important role in transferring longitudinal shear force between the deck slab and the steel girders, ensuring the composite behaviour.

Historically the existing bridges in New Zealand carry many different types of shear connectors. Unfortunately, the majority of these shear connectors are not supported by the standards referenced in the NZ Transport Agency's (2014) *Bridge manual*. Although internationally there is some guidance for the evaluation of existing shear connectors, eg BD 61/10, this is better suited to a particular country's practices. The aim of this report was therefore to develop design guidance for evaluating the capacity of existing bridges by considering the performance of different historical shear connector types. As a result of this study, high-productivity motor vehicles (HPMVs) and 50MAX vehicles are expected to be given wider access to the existing highway network.

The research first studied international design equations and test methods for the resistance of various types of shear connectors. A review of the shear connector types in existing New Zealand highway bridges was then carried out by studying databases of international consultants and the NZ Transport Agency's (2009) *Bridge data system structural guide*. The channel shear connector was identified to be the major type of shear connector. Utilising structural reliability analysis together with the international test data obtained in the study, a design equation was developed for the resistance of the local channel shear connectors.

A new design method for composite beam bending resistance, incorporating the proposed new design equation for channel shear connectors, is introduced in this report. It is to be accepted in the forthcoming AS/NZS 2327 as a general solution for composite structures in New Zealand.

A revised evaluation procedure, which integrates the new design method, was also developed based on the existing *Bridge manual* procedure.

A worked example using the newly proposed evaluation procedure is provided in appendix F of this report. The worked example is based on an exemplar project in New Zealand. It also compares the results with the existing methods such as AS 5100.6 and NZS 3404 and demonstrates that the new method gives a more accurate prediction.

2 Review of current design and evaluation practices in New Zealand and overseas for shear connectors

This chapter reviews current practice for the design of new and evaluation of existing shear connectors in composite construction. To ensure the review is well focused, particular attention is paid to the shear connection types used in New Zealand. More information is given about these in chapter 3.

2.1 Historical review of the development of shear connectors for steel-concrete composite beams

In 1922, the Dominion Bridge Company in Canada conducted tests on two floor panels consisting of a concrete slab and two steel I-beams encased in concrete. In reporting the results of these tests, Mackay et al (1923) wrote: 'While such beams have hitherto been designed on the assumption that the entire load … is carried by the steel, it was thought that the steel and concrete might really act together so as to form a composite beam …'. During this period, tests on encased composite beams were also being carried out in the UK and USA. In each investigation the experimental results indicated, so long as the bond between the steel and the concrete was not lost, the encased beam behaved as though there was complete interaction. However, it was appreciated by the investigators that the strength of such beams would be seriously impaired by the loss of bond, and once this natural bond had been broken, further composite action was not possible. Consequently, in order to provide security against premature failure, mechanical connection devices were introduced to augment natural bond. As bridge construction practice gradually moved away from full encasement towards a concrete slab supported on top of steel beams, investigations began to place more emphasis on the mechanical connection between the concrete and the steel and rely less on natural bond.

The first systematic studies with mechanical shear connectors were made in Switzerland with the development of the Alpha System. In this form of construction, the transfer of longitudinal shear, from the steel beam to the concrete slab, was satisfied by the provision of a round bar formed into a helix. The helix, otherwise known as a spiral connector, was welded to the top flange of the steel section at the points of contact along the length of the beam (see figure 2.1(e)). Static tests of eight specimens with helical connectors were reported by Roš (1934). The tests involved two double T-beams tested with two concentrated loads, four T-beams subjected to axial loads applied at different eccentricities and two special specimens devised for determining the shear transfer capacity of the spirals. These last two specimens consisted of a short section from an I-beam and two concrete slabs which were connected to each flange of the steel beam with a helical connector. The specimens were supported on the ends of the slabs and the load was applied axially to the steel beam. Slip at the interface between the steel member and the two slabs was recorded to determine the load-slip characteristics of the mechanical shear connection. This type of test specimen is now almost universally used for tests on mechanical connectors and is referred to as the push out or push test, see (figure 2.2).

Figure 2.1 Typical shear connectors according to CP 117-1: 1965

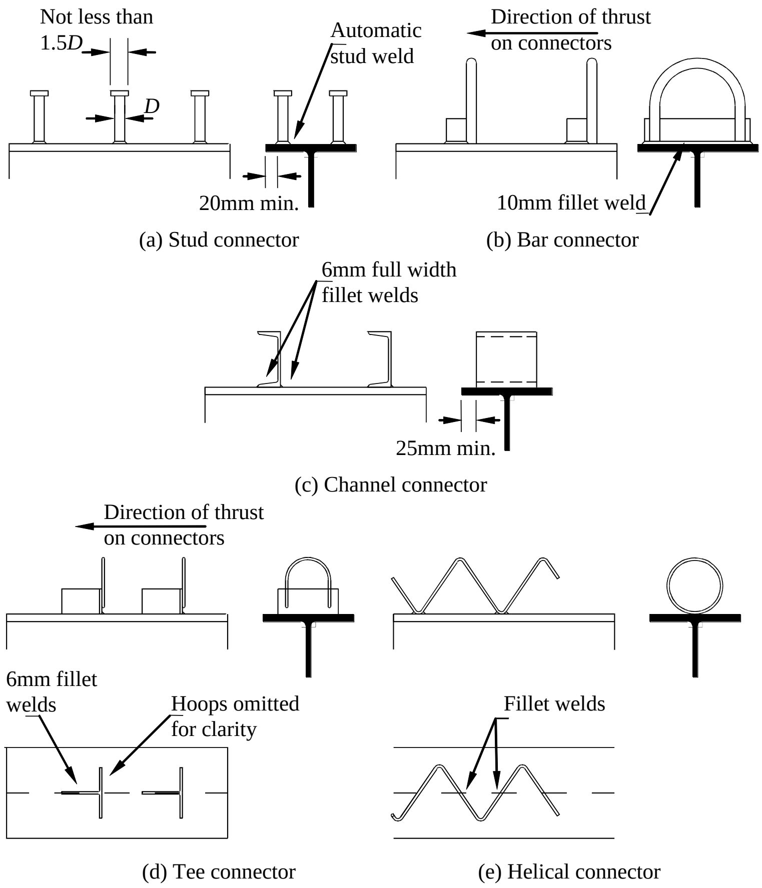
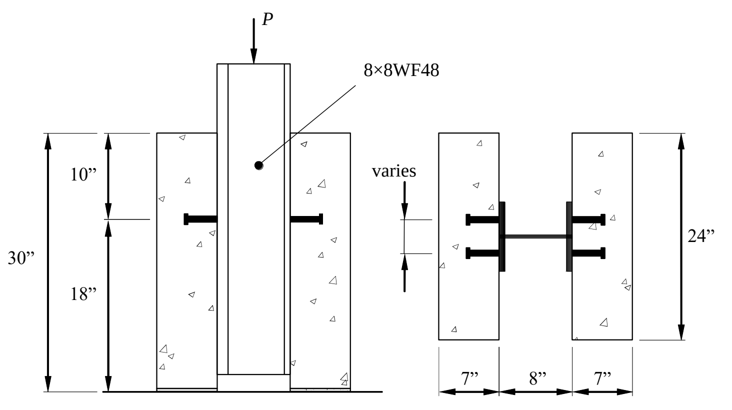


Figure 2.2 Push specimen (Viest 1956b)



In the period between 1935 and 1951, after the early studies on helical connectors, European investigators turned their attention to other forms of mechanical connection devices, eg Bar and Tee connectors (see figure 2.1(b) and (d)). These types of mechanical devices, primarily used in British bridge construction, were denoted as stiff connectors since they provided almost complete interaction by preventing slip at the steel flange/concrete slab interface by transferring the shear forces primarily by bearing on the concrete due to their relative stiffness.

In North America between 1939 and 1958, engineers turned towards shear connectors which required less fabrication, for example, the channel connector shown in figure 2.1(c). These types of mechanical connecting devices were termed flexible connectors due to the fact they allowed a certain amount of slip at the steel flange/concrete slab interface and, for design purposes were idealised as a flexible dowel in an elastic medium. This assumption led to semi-empirical formulae relating the maximum stress to the concrete strength and the connector width, in addition to the flange and web thickness. In 1952 two studies were carried out by Siess et al (1952) and Viest et al (1952) to compare the performance of so-called stiff connectors with that of flexible connectors. From these investigations it was found that, when considering the load-slip performance obtained from push-out tests, the stiff connectors were superior to the flexible types. However, the differences were much smaller than had been expected by the investigators and from beam tests it was found flexible connectors could, in fact, provide adequate shear connection to develop full composite action.

The development of the electric drawn arc stud welding apparatus in 1954 allowed another type of flexible connector known as the headed stud connector (see figure 2.1(a)), to be rapidly fastened to the top flange of steel beams. This development was accompanied by extensive investigations in the USA between 1956 and 1959 at the University of Illinois (Viest 1956a; 1956b) and Lehigh University (Thürlimann 1959) using push-out tests, of the type shown in figure 2.2. From these research programmes it was found that the behaviour of the headed stud connector was virtually identical to that of the channel connector. Furthermore, due to its advantages over the channel connector (eg the rapid installation technique, and the fact that they were equally strong and stiff in shear in all directions normal to the axis of the stud), the stud shear connector became one of the most popular types of connecting device to be used in composite construction.

During this period, the strength of headed shear connectors was almost entirely found from push-out tests of the general type discussed above. Although investigators like Viest and Siess (1954) acknowledged the necessity for an ultimate load design method for connectors, they appreciated that the contemporary method of designing composite bridge beams was based on working stresses. Therefore, the design strength of shear connectors was based on a 'critical load' or 'useful capacity', which was determined by subjecting a push test specimen to cycles in load until the residual slip reached a specified value. According to the American Code of Practice at the time (AASHO 1957) this was 0.003in (0.076mm), which generally corresponded to half of the ultimate capacity.

An important observation is that, up to 1959, no generally agreed guidance was in place with regard to the suitable proportioning of the push-out test specimens, either on a national or international level. This fact can at least be partly attributed to the need for research to keep up with the rapid developments occurring in industry. Consequently, at this point various sizes, configurations and fabrication techniques for the test specimens had been employed in the examination of the load-slip performance of mechanical shear connectors. Also, an obvious question regarding the behaviour of shear connectors is whether their behaviour in push-out tests was comparable to that which would occur in a full-scale beam. In an attempt to address this question, Thürlimann (1959) studied the behaviour of headed studs in beams and companion push-out tests. From the results of these tests Thürlimann concluded the push-out test produced essentially the same conditions as normally existed in actual composite beams.

In 1959, investigations in the USA by Culver and Coston (1959), Culver et al (1960; 1961) and UK by Adekola (1959); Balakrishnan (1963) and Chapman and Balakrishnan (1964) were directed towards the behaviour of composite beams in buildings. As mentioned above, most previous research had been concerned with the elastic approach based on specified working stresses in the steel and concrete; these tests investigated the feasibility of the concept of ultimate strength applied to both the bending strength of composite beams together with their shear connection. From the studies in the USA at Lehigh University

(Culver and Coston 1959; Culver et al 1960; 1961), it was found that, provided the total strength of the shear connection was adequate to resist the ultimate compressive force in the concrete, the magnitude of the slip at the interface did not significantly affect the development of the ultimate moment of resistance. It appeared that the only limitation to the amount of slip allowed at ultimate load was the amount the connectors could deform without fracturing. It was shown that since flexible connectors possess sufficient ductility to redistribute the horizontal shear force, then all the connectors were equally loaded prior to failure. In addition, the investigation in the USA concluded that using this limit state approach, rather than the previous elastic permissible stress design, allowed the shear connectors to be spaced uniformly along the length of the beam, irrespective of whether the applied loading was concentrated or uniformly distributed (ie elastic perfectly plastic behaviour).

The research programmes conducted in the UK at Imperial College between 1959 and 1964 (Adekola 1959; Balakrishnan 1963; Chapman and Balakrishnan 1964) further confirmed the feasibility and advantages of the limit state concept, by stating that the ultimate capacity of a shear connector formed a more satisfactory basis for designing such connectors than the 'useful capacity' concept, based on a limiting residual slip. Moreover, emphasis was placed on ensuring that in practice the shear connection should not precipitate flexural failure of the beam. It was therefore proposed in 1964 by Chapman and Balakrishnan (1964) together with Yam and Chapman (1968) that the shear connectors should be loaded to only 80% of their ultimate capacity at ultimate moment (this recommendation was included in CP 117-1: 1965 and its successor BS 5950-3.1: 1990. Furthermore, the results obtained at Imperial College, like those at Lehigh University, confirmed that when the shear connectors are designed on an ultimate load basis, they can be uniformly spaced along the length of the beam, even when the shear force diagram is triangular, ie when the beam is subjected to a uniformly distributed load. Also, an important phenomenon was reported by Adekola (1959) in his tests on one-eighth and one-quarter scale T-beams; when low levels of transverse reinforcement were provided, a splitting related failure of the concrete slab along the plan centreline of the beams was observed.

In 1960, some questions arose over Thürlimann's (1959) assumption that the push-out test produced the same conditions as in full-scale beams. Culver et al (1960), in comparing the results of four beam tests with their companion push-out specimens, observed that, although the results were too limited to establish the correct correlation between beam and push-out tests, the connection force in all the beams with an adequate number of shear connectors exceeded the connector force in the comparable push-out specimen by 39% or more. Further tests by Culver et al (1961) substantiated the conclusion that the behaviour of a shear connector in a push-out test was different from that in a beam specimen.

However, an investigation by Slutter and Driscoll (1965) examined in greater detail the behaviour of stud connectors in beams and push-out tests. The authors accepted that the performance of shear connectors in a beam and a push-out test were somewhat similar, but suggested there were some basic differences between the two types of specimens, which affected the test results and the reliability of the test data, namely:

1. The direct stresses present in the concrete due to bending in the slab of the beam were absent in a push-out test specimen.
2. The eccentricity of loading in the push-out test might result in low average ultimate connector strengths.
3. From experimental observations, the amount of slab reinforcement must be greater in a push-out specimen than in a beam to attain comparable ultimate connector strengths.

As a result of the above, Slutter and Driscoll undertook a careful study of testing techniques, test specimens and corresponding test results. In addition, to compare the resistance of the headed stud connectors properly, partial shear connection was employed in some of the beam tests (ie at the limit state the total compressive force developed from the shear connectors in the shear span would be less than that which would develop in the concrete flange if full connection were provided to mobilise the full moment capacity of the composite section). From the general behaviour of the specimens, the authors concluded that the highest results attained in push-out tests were a conservative approximation of the ultimate strengths of shear connectors in beams.

From the above summary of previous research work on composite beams with solid concrete slabs, it can be seen that most of the test data on the load-slip characteristics of shear connectors was as a result of push-out tests of the general type shown in figure 2.2. Ideally this load-slip curve should be found from full-scale beam specimens since it seems that in some cases the results from push tests did not fully represent the behaviour of a stud connector. However, although helpful in evaluating the actual load-slip performance of shear connectors, it would be difficult to conduct beam tests in sufficient numbers to investigate the sensitivity of different parameters and evaluate the performance of a design model using structural reliability analysis. As a consequence of this, it would appear that the push-out test is one of the only reliable methods for determining the load-slip characteristics of shear connectors and, if suitably proportioned, should provide a satisfactory means for determining the ultimate strength of headed stud connectors.

If the limit state concept is adopted, the shear studs may be uniformly positioned along the length of the beam because at ultimate load it has been shown from the tests discussed above (Culver and Coston 1959; Culver et al 1960; 1961), providing the studs possess sufficient ductility, they will redistribute the applied shear forces from near the supports to the span (until all the connectors are equally loaded prior to failure). Consequently, it would appear that a theoretical model based on the failure of a single connector is justifiable with regards to the limit state concept.

2.2 Historical development of the code defined push-out test

In 1960, the Drafting Committee for the British Code of Practice for Composite Construction was formed and began its work by preparing a summary of existing knowledge, specifications and construction practice (Institution of Structural Engineers 1964). In the last part of this publication the committee listed various areas of research which urgently needed to be investigated for implementation in the Code of Practice. The first of these areas was the standardisation of a specimen for determining the load-slip characteristics of shear connectors. From the review of previous work, the Drafting Committee (Institution of Structural Engineers 1964) stated that the behaviour of a push-out test could vary considerably depending on the form of specimen and the method of loading, as well as on the dimensions, reinforcement and concrete strength of the slabs. For these reasons it was difficult to correlate the results from the considerable number of push-out tests on various types of shear connector that had been performed in different laboratories. Consequently, the committee proposed that the following list of variables should be considered in the definition of a standard specimen:

1 Form of specimen: For this point, the committee acknowledged that by far the most commonly adopted form was the push-out test shown in figure 2.2, where the central joist is pushed between two slabs which are firmly bedded at the base. However, concern was expressed that the slabs were not free to separate, although the committee conceded that separation was not entirely prevented, since rotation of the slabs typically occurred prior to failure.

2 The number of shear connectors to be employed in the push-out test: In this case the committee stated that for a shear connector such as a channel or bar, a single shear connector would probably be satisfactory, but in the case of a headed stud a single connector would not be sufficient. This point was probably due to the limit state concept discussed above, in that it would be desirable to have a specimen configuration which enabled redistribution of load to occur between the connectors, as would occur in the shear span of a composite beam where there is a monotonic distribution of force.

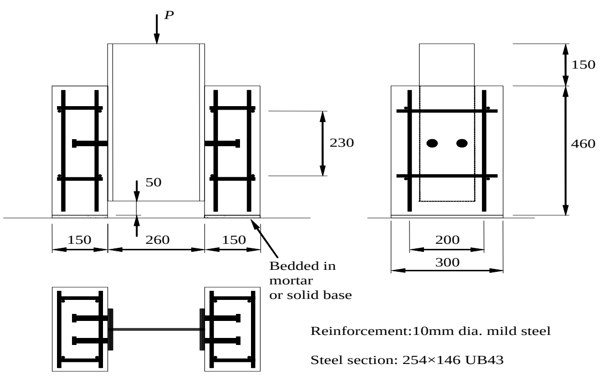
3 The size and quality of the concrete slab: On this point the committee was concerned that an ex cathedra pronouncement defining the form of the slab would cause unsatisfactory results for prototype testing of other forms of beam cross-section, eg deck-slab, haunched and pre-cast slab elements.

4 Reinforcement: In this case the committee was primarily concerned that enough reinforcement should be specified in the new code so as to prevent secondary failures, ie longitudinal splitting (Adekola 1959; Robinson 1967) from affecting the resistance of the shear connection.

5 The dimensions of the steel member: This last point was concerned with the possibility of the shear connector being influenced by the thickness of the beam flange. Later, a study made by Goble (1968) demonstrated that the full static strength of a headed stud connector can be developed if the ratio of the stud diameter to that of the steel flange thickness is less than approximately 2.7.

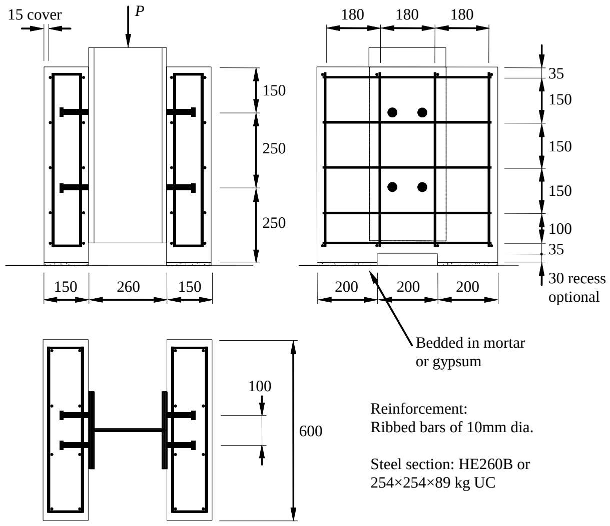
The standardised specimen configuration chosen to fulfil the above variables was introduced in the UK in 1965 with the publication of CP 117-1: 1965. A metricated version of this test arrangement was later introduced in 1979 with the publication of BS 5400-5: 2005 (see figure 2.3) and referred to in 1990 by BS 5950-3.1 without comment on the need to modify it when profiled steel sheeting is employed. This test has two variables with respect to the amount of reinforcement and the size of slab: it can either conform to the arrangement shown in figure 2.3 or, should conform to the details which will be present in the beams to which the test is related (ie point (3) above).

Figure 2.3 Standard push test according to CP 117-1: 1965 and BS 5400-5: 2005



During the drafting of Eurocode 4 (ENV 1994-1-1) a wide scatter was again found in the results of tests from previous investigations reported in the UK and overseas on the shear strength of headed stud connectors. This scatter was attributed by Johnson and Anderson (1993) together with Roik et al (1989) to the differences in sizes of the test specimens, the methods in casting, the methods of testing and the lack of reported test data on the ultimate tensile strength of the stud material. The British Standard push-out specimen in figure 2.3 has been recently criticised by Johnson and Anderson (1993) since, due to the very small slabs, it has a tendency to split longitudinally as the light, mild steel reinforcement bars are poorly anchored. In addition, the connectors are only located at one level, which reduces the effects of redistribution of load from one slab to the other, and so gives the resistance of the weaker of the two connectors (Oehlers and Johnson 1987). In an attempt to overcome these shortcomings, Eurocode 4 presents the standard test as shown in figure 2.4.

Figure 2.4 Standard push test according to Eurocode 4 (ENV 1994-1-1)



The significant changes in this specimen compared with the British standard push-out test are as follows:

1 The slabs have the same thickness of 150mm but have been increased in size from 460 × 300mm to 650 × 600mm. This increased width allows the transverse reinforcement to be better anchored and avoids the potential of low results due to longitudinal splitting.

2 The transverse reinforcement is increased from four 10mm diameter mild steel bars to 10 high-yield 10mm diameter bars. In addition to the prevention of longitudinal splitting, the increased stiffness offered by the high-yield bars is intended to simulate the transverse restraint by the in-plane stiffness of the slab that exists in T-beams (Slutter and Driscoll 1965). Thus, the extra reinforcement is not intended to reproduce the reinforcement provided in a beam.

3 The flange width of the steel beam is increased from 146mm to 260mm, so that wider channel and block connectors may be tested, and the lateral spacing of shear studs is standardised.

4 The connectors are provided in two levels in each slab which enables greater load redistribution. The test therefore gives the mean resistance of at least four connectors and attempts to better simulate the redistribution occurring in the shear span of a beam.

5 Each concrete slab must be cast in the horizontal position, as would occur in practice. Many specimens in the past were cast with the slabs vertical, which caused a risk of the concrete below the connectors to be poorly compacted. This was thought to be one of the reasons for the high scatter of previously reported results, because in a push test most of the load on a connector is transferred at the base of the studs, in a region which is most likely to have an air pocket or weak concrete when vertical casting is employed. This variable was examined in greater detail in Japan by Maeda et al (1983; Maeda 1996), who studied the effect of concrete placing techniques on headed stud connectors subjected to fatigue and static push-out tests. The results of these tests, which in turn influenced the drafting of the Japanese Industrial Standards (JIS B1198-1982) in 1982, confirmed the Eurocode 4 requirements for horizontal casting.

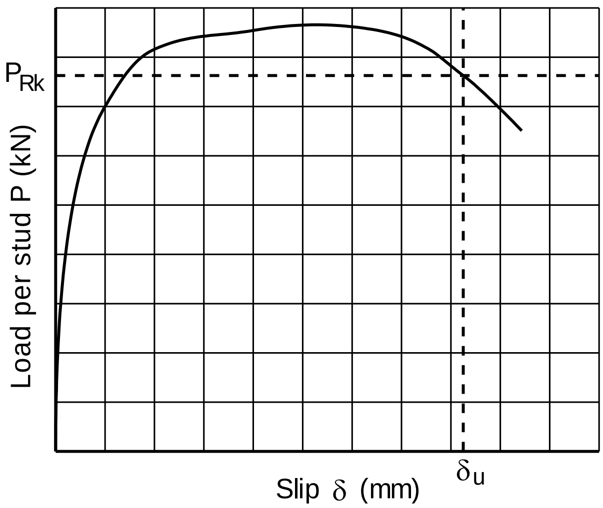
6 The optional recess at the base of the push specimen slabs. During the drafting of Eurocode 4 (ENV 1994-1-1), members of the committee were unable to agree on a standard base condition for the push

test, since the adoption of a single country's test configuration would cause the values of a considerable number of results from another country to be invalidated. The British standard test specimen in figure 2.3 typically has a continuous base which causes the shear forces from the connectors to be more concentrated in the centre of the specimen. However, the German standard specimen includes a recess at the base of the slab in an attempt to cause a dispersion of shear forces comparable to that which would occur in a full-scale beam (Roik and Hanswille 1983; 1987) by distributing the forces outwards from the centre-line of the specimen into the concrete flange.

Like the British Codes of Practice (BS 5400-5: 2005; BS 5950: 1990; CP 117-1: 1965) the Eurocode 4 (ENV 1994-1-1) specimen has two variables with respect to the amount of reinforcement and the size of slab; in that it can either conform to the arrangement shown in figure 2.4, or it should conform to the details of the beams to which it is related. In addition, there is again no comment on the need to modify the test when profiled steel sheeting is employed.

In addition to defining the standard push test, Eurocode 4 also provides rules for evaluating the characteristic values for shear connectors (based on the lower 5% fractile). According to Eurocode 4, if three nominally identical tests are carried out, and the deviation of any individual result from the mean value does not exceed 10%, the characteristic resistance of a shear connector P_{Rk} is defined as 0.9 times the minimum failure load per stud (see figure 2.5). The ductility of a shear connector is measured by the slip capacity δ_u, which is defined in Eurocode 4 as the slip value at the point where the characteristic resistance of the connector intersects the falling branch of the load-slip curve (see figure 2.5). The characteristic slip capacity δ_{uk} is taken as 0.9 times the minimum test value of δ_u. Alternatively, the characteristic properties of a shear connector can be determined by a statistical evaluation of all of the results according to BS EN 1990: 2002.

Figure 2.5 Determination of characteristic resistance and slip capacity from load-slip curve measured from a push test

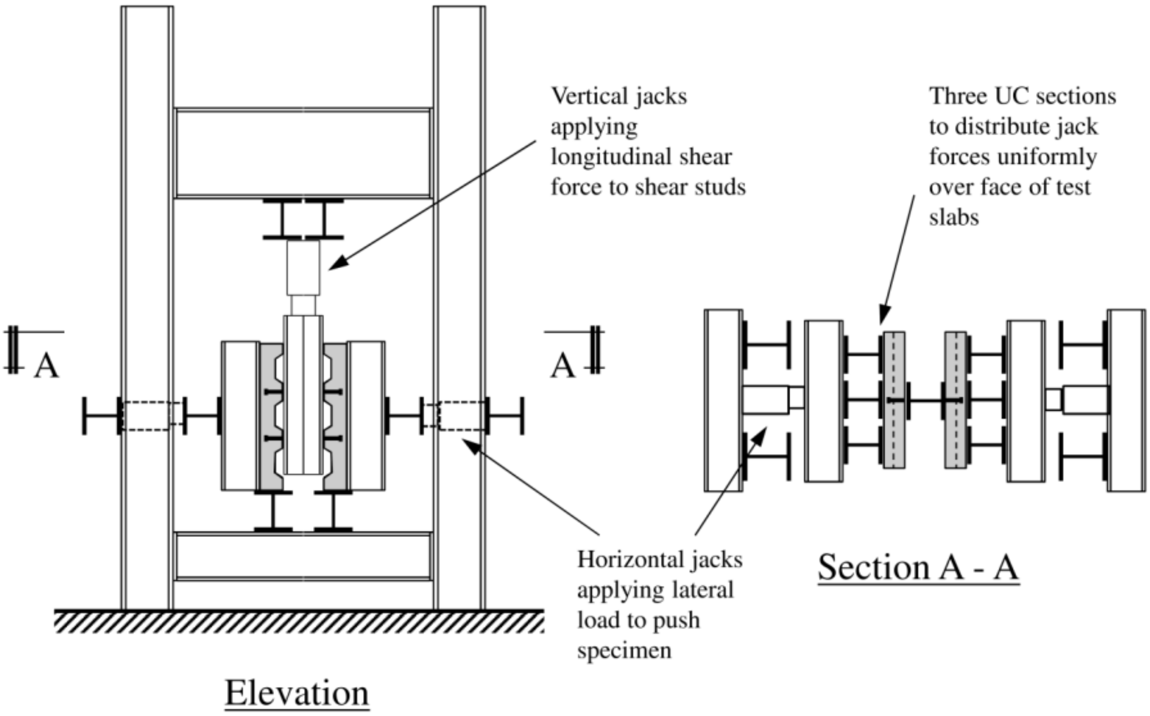


The rules for partial shear connection given in BS 5950-3.1 and Eurocode 4 are based on extensive numerical analyses, which considered composite beams over a wide range of spans and section types (Aribert 1997; Johnson and Molenstra 1991). To enable the designer to assume all of the studs are equally loaded at the ultimate limit state (ie the shear connection is fully plastic), the minimum degree of shear connection in these standards is based on an assumed ductility limit. In Eurocode 4, a connector may be taken as ductile if the characteristic slip capacity δ_{uk} is at least 6mm.

From the above, it would appear that previous investigations on stud connector strengths which employed the standard British test (see figure 2.3), have caused a considerable scatter in the results, which has in turn lead to lower design values. Therefore the characteristic resistances of stud connectors in solid slabs in both BS 5400-5 (2005) and BS 5950-3.1 (1990), which were established on results from push tests of this type (Menzies 1971) may be slightly conservative. Moreover, it would appear that even when the Eurocode 4 test (see figure 2.4) is employed, a clear case exists for the need to standardise specimens with profiled deck-slabs. With respect to comments in (i) regarding the form of specimen in figure 2.3, due to the fact that in both types of code-defined tests, the slab specimens are directly bedded down onto the floor, variations in the frictional forces which must develop at the interface between the base of the specimen and the floor, might be a further reason for the high scatter in results (Hicks and McConnel 1997).

More recently, a number of researchers have proposed an improved push test specimen for composite slabs using profiled steel sheeting (Easterling et al 1993; Bradford et al 2006; Hicks and McConnel 1997), which delivers more representative load-slip performance of headed studs within a composite beam. Figure 2.6 shows one example of the improved push test specimen, where a normal force is applied to the face of the test slabs.

Figure 2.6 Improved push specimen, according to Hicks and Smith (2014)



2.3 Past research on the theoretical strength of shear connectors

2.3.1 Shear stud connectors

As discussed in the previous sections, the early design philosophy for composite beams was based on a working stress approach where the design strength of the shear connectors was described by a 'critical load' or 'useful capacity' which was determined by measuring the residual slip from push tests. More efficient designs were later achieved by the introduction of the limit state concept where it was assumed that at ultimate load the shear connectors were all equally loaded in the shear span prior to flexural failure (ie plasticity theory). To achieve a consistent means of theoretically determining the ultimate strength of headed stud connectors various research programmes were undertaken in the 1970s in the UK and USA.

One of the first comprehensive attempts in the UK to establish the ultimate strength of shear connectors in solid concrete slabs was undertaken by Menzies (1971). Menzies carried out push tests of the type shown in figure 2.3 with specimens employing 229mm deep normal weight and lightweight concrete slabs. These tests investigated the fatigue and static strengths of three types of shear connector: Stud, Channel and Bar connectors (see figure 2.1(a) to (c)). The main objective of this experimental programme was to explain the reason for some discrepancies in shear connector resistance obtained from earlier reported tests when compared with the linear relationship between the static connector strength and the concrete strength which at this time was assumed by CP 117-1: 1965 and CP 117-2: 1967. In addition, this code only presented the linear relationship for cases of composite beams where normal weight concrete was employed which effectively negated the use of lightweight concrete in building construction. Moreover, this relationship was established from only a small number of tests where, in some cases the concrete strength was not a main variable. Thus, in this new series of tests, the strength was varied for both types of concrete to between 20 and 40N/mm^2.

From the experimental results of this test programme the maximum load per stud was plotted against the water- and air-cured cube strengths for the normal weight concrete specimens. Whereas, for the lightweight concrete specimens, the maximum load per stud was only plotted against the water stored cube strengths. In both cases regression lines with 90% confidence limits were superimposed over these experimental results to provide a linear relationship between the stud and cube strengths.

With regard to the effect on the stud resistance of varying the density of the concrete, the lightweight specimens resulted in 10% less strength than studs embedded in normal weight concrete of the same compressive strength. In comparing the new linear relationship for the normal weight concrete specimens with the values presented in CP 117, the code strengths were found to be above the 90% confidence limits. Menzies also corroborated this new linear relationship for both lightweight and normal weight concrete specimens by comparing the results of other push tests conducted in the UK and USA. Conversely, when comparing the normal density specimens with the code predictions, Menzies found in approximately 50% of the cases the connector strength was less than the appropriate values specified in the code. This led the author to recommend a revision of the values in CP 117. The linear relationship presented by Menzies between stud and cube strength was later incorporated into BS 5400-5: 2005 and together with a further 10% reduction for studs in solid lightweight concrete slabs, by BS 5950-3.1: 1990.

Also in 1971, another thorough investigation of the ultimate shear strength of headed stud connectors embedded in solid concrete slabs was carried out at Lehigh University in the USA by Ollgaard et al (1971). They undertook 48 push tests of the type shown in figure 2.2 with specimens employing normal weight or lightweight concrete. The main objective of this experimental programme was to obtain a mathematical relationship between the ultimate shear strength of the stud connector and the material properties of the concrete. This mathematical relationship was empirical in nature in that multiple regression analyses were undertaken using a least squares fit for the results. Since in all 48 tests the stud material was taken from the same batch, the tensile strength of the shear connectors was not considered to be a factor in the regression analysis. Therefore, the ratio of the shear strength to the cross-sectional area of the headed stud connector, Q_u/A_s was used as the dependent variable while the concrete properties were considered as independent variables. The initial general exponential model which considered all the concrete properties was presented by the authors as:

$$Q_u / A_s = e f_c^{\,a} f_{ct}^{\,b} E_c^{\,c} w^d \qquad \text{(Equation 2.1)}$$

Where:

a, b, c and d are the general exponents for the concrete variables

e is a constant

f_c is the cylinder strength of the concrete

f_{ct} is the tensile strength of the concrete

E_c is the elastic modulus of the concrete and w is the concrete density.

In order to obtain linear equations for the regression analyses the authors linearised equation 2.1 by making a logarithmic transformation and presented 15 models which represented all possible combinations of the four independent concrete variables. In comparing the different models the authors conceded that although the best correlation was found from the model which considered all of the four independent variables, there was little difference in correlation between two models which considered only two concrete variables. From this comparison, it was decided by the authors that the combination of compressive strength and the modulus of elasticity of the concrete provided a reasonable estimate of the ultimate resistance of headed stud connectors embedded in both normal weight and lightweight concrete, which in turn led them to present the following equation:

$$Q_u / A_s = 1.106 f_c^{\,0.3} E_c^{\,0.44} \qquad \text{(Equation 2.2)}$$

Since, for design purposes, it is more desirable to use equations with more convenient exponents, the authors conducted a further series of analyses to determine the effect of rounding off the exponents of the two models which considered only two independent variables. From this analysis the following equation was recommended for design purposes:

$$Q_u = 0.5 A_s \sqrt{f_c E_c} \qquad \text{(Equation 2.3)}$$

Finally, using the results from a previous research programme (Driscoll and Slutter 1961), equation 2.3 was assumed to be valid provided the following condition was met:

$$h/d \geq 4 \qquad \text{(Equation 2.4)}$$

Where:

h and d is the height and diameter of the stud respectively.

Equation 2.3 was adopted in the AISC Code of Practice (AISC 1986) and later in many other international codes for composite construction. In some cases an upper limit was added, related to the ultimate tensile strength of the stud material itself, as:

$$Q_u = 0.7\left(\pi d^2/4\right) f_u \qquad \text{(Equation 2.5)}$$

Where:

f_u is the ultimate tensile strength of the stud material.

Oehlers and Johnson (1987) also carried out a similar linear regression analysis and presented the following functional relationship for predicting the shear strength of a stud connector P_p as:

$$P_p = KA\left(E_c / E_s\right)^{\alpha} f_{cu}{}^{\beta} f_u{}^{\gamma}$$ (Equation 2.6)

Where:

α, β and γ are the material exponents

K is a constant

A is the area of the shank of the stud

E_s is the elastic modulus of the stud material

f_{cu} is the cube strength of the concrete.

After calculation of the exponents from the regression analysis the authors suggested the following formula should be used for determining the mean shear strength of a headed stud embedded in a solid concrete slab:

$$P_p = 4.9 A f_u \left[\frac{E_c}{E_s}\right]^{0.40} \left[\frac{f_{cu}}{f_u}\right]^{0.35}$$ (Equation 2.7)

Roik et al (1989) conducted a statistical calibration study of push test results to establish satisfactory safety factors for the draft version of Eurocode 4. In this version of Eurocode 4 the equations used to determine the predicted strengths of headed studs were essentially based on the work of Ollgaard et al (1971). For the statistical analysis, Roik et al (1989) required that the predicted values must describe the shear resistance correctly at the mean. On this basis, equations 2.6 and 2.7 were re-written in the following form to establish mean shear resistance values:

$$P_p = 0.36 d^2 \sqrt{f_c E_{cm}}$$ (Equation 2.8)

and

$$P_p = 0.85\left(\pi d^2 / 4\right) f_u$$ (Equation 2.9)

Where:

E_{cm} is the mean elastic modulus for concrete.

In the study, 76 test results from other research programmes, which also included the original Menzies (1971) experimental data, were compared with equations 2.8 and 2.9 in addition to the Oehlers and Johnson equation 2.7. In most cases Roik et al (1989) found the previously reported experimental data often neglected to state the mean elastic modulus of the concrete and the ultimate strength of the stud material itself. Thus, in order to overcome the problem when the strength of the stud material was not reported, the average value stated by the stud manufacturers was employed. While, for the cases when the mean elastic modulus of the concrete was not reported, the following Eurocode 2 (DD ENV 1992-1-1) equation was used:

$$E_{cm} = 9.5\left(f_c + 8\right)^{1/3}$$ (Equation 2.10)

On carrying out the statistical evaluation of equations 2.7, 2.8 and 2.9, it was found that the mean model factors (experimental failure load/predicted failure load) for the Ollgaard et al (1971) and Oehlers and Johnson (1987) equations were 1.1 and 1.01 with a variance of 2.2% and 1.6% respectively. Roik et al

(1989) also found in both approaches there was no dependence on the stud diameter and so in later tests they made no distinction between experimental values for different stud diameters.

On completing this statistical exercise, the authors concluded there was little difference between the calculation model for the draft of Eurocode 4 (equations 2.8 and 2.9) and the model according to Oehlers and Johnson (equation 2.5). However, concern was expressed that both models had the disadvantage of not being based on a mechanical model and only being derived empirically. Nevertheless, since there was no alternative model available Roik et al (1989) recommended the draft Eurocode 4 formulae should be adopted, due to the fact they were more easily applicable and more widely known in current practice. From the analyses conducted they recommended that for design, equations 2.8 and 2.9 should be used and divided by a uniform partial safety factor of γ_M =1.2. However, prior to the publication of the prENV and ENV versions of Eurocode 4 this material safety factor together with equations 2.8 and 2.9 were further modified to enable a single value for γ_M, subsequently denoted γ_V (V for shear), of 1.25 to be recommended for all types of shear connection. Following this modification the current Eurocode 4 design equations for establishing the shear strength P_{Rd} of headed stud connectors embedded in solid concrete slabs took the following form:

$$P_{Rd} = \frac{0.29d^2\sqrt{f_c E_{cm}}}{\gamma_V} \qquad \text{(Equation 2.11)}$$

or

$$P_{Rd} = \frac{0.8 f_u \left(\pi d^2/4\right)}{\gamma_V} \qquad \text{(Equation 2.12)}$$

whichever is smaller.

More recently, Hicks and Jones (2013) conducted a structural reliability study for the draft AS/NZS 5100.6 that extends these earlier studies to include 113 push tests with concrete strengths up to 91MPa. From considering the performance of the existing design equations for stud failure contained within Eurocode 4 (equation 2.10), AS 2327.1, AS 5100.6, NZS 3404.1 and ANSI/AISC 2010 it was found they may, in general, be safely extended to include higher strength concretes. However, the existing equations for concrete failure (equation 2.11) produced predictions that were over-optimistic. From this study, and considering the variability of material strengths given in both the current Australian and New Zealand product standards, the following design equations have been incorporated within the current draft of AS/NZS 5100.6.

$$P_{Rd} = \phi_V\, 0.29d^2\sqrt{f_c E_{cm}} \qquad \text{(Equation 2.13)}$$

or

$$P_{Rd} = \phi_V\, 0.89 f_u \left(\pi d^2/4\right) \qquad \text{(Equation 2.14)}$$

whichever is smaller.

Where:

ϕ_V is the reduction factor = 0.8.

2.3.2 Channel shear connectors

The channel shear connector has a number of advantages over other types of connectors as the use of an expensive machine is not required for its installation. Also, because of their higher shear resistance, very few channel connectors are required, which avoids the cluster usually caused by headed studs (Pashan 2006).

Slutter and Driscoll Jr (1962) carried out 17 push-out tests on the fatigue strength of channel shear connectors in composite beams. The concrete strength used varied from 24.9 to 43.9MPa, and the specimens were loaded until the shear connectors could no longer support the maximum load. It was observed that the majority of the failures occurred due to channel fracture. The compressive strength of the concrete did not have an adverse effect on its ultimate strength; rather, the stress range was the most important factor. Slutter and Driscoll Jr (1962) proposed using the following equation in calculating the ultimate strength of a channel connector (in pounds).

$$Q_u = 550(h + 0.5t)w\sqrt{f_c^{'}} \qquad \text{(Equation 2.15)}$$

Where:

h is the average flange thickness (in inches)

t is the thickness of the web (in inches) and *w* represents the length of the channel (in inches)

$f_c^{'}$ is the compressive strength of the concrete (in psi).

Equation 2.15 was the basis for the following expression given in the current Canadian standard CSA S16-01 for evaluating the resistance of a channel shear connector in SI units:

$$q_{rs} = 45(t + 0.5w)L_c\sqrt{f_c^{'}} \qquad \text{(Equation 2.16)}$$

Where:

t is the flange thickness of the channel (in mm)

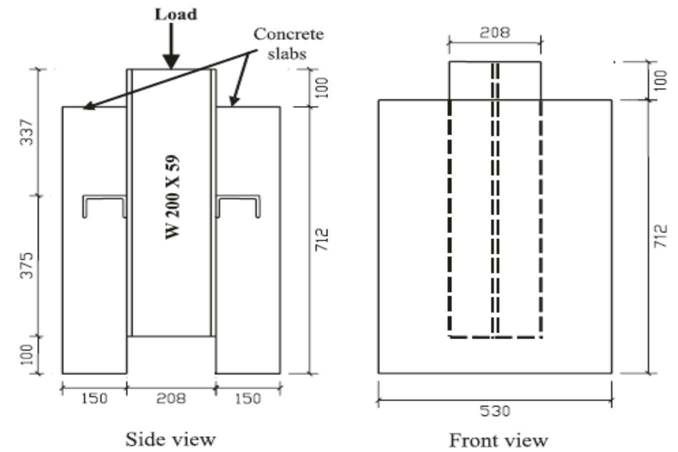
w is the web thickness of the channel (in mm)

L_c is the length of the channel (in mm)

$f_c^{'}$ is the compressive cylinder strength of concrete (in MPa).

Menzies (1971) undertook 15 push-out tests on channel connectors embedded in solid concrete having varying compressive strengths. He also reversed the direction of the channel connectors to extend the results reported by previous researchers. The results led Menzies to propose a change to CP 117-1: 1965 and CP 117-2: 1967 for the ultimate strength of channel shear connectors. The existing design recommendation for channel connector specified the connector should be placed in the same direction as the applied load as shown in figure 2.7; however, the removal of this restriction would be advantageous to simplifying the design and eliminating the possibility of connectors being welded to beams with the wrong orientation. The slip measurement indicated there was little difference in the ductility of the channel connector when placed in a reversed direction.

Figure 2.7 Geometrical parameters of push-out specimen using channel shear connectors (Pashan and Hosain 2009)



The ENV version of Eurocode 4 (ENV 1994-1-1 (1994)) did not provide an equation for channel shear connectors; however, provisions were made for an angle shear connector by equation 2.17.

$$P_{rd} = \frac{10 \times b \times h^{3/4} \times f_{ck}{}^{2/3}}{\gamma_v}$$ (Equation 2.17)

Where:

b is the length of the angle (mm)

h is the height of the upstanding leg of the angle (mm)

f_{ck} is the characteristic concrete compressive strength (MPa).

The recommended value for the partial safety factor γ_v is taken as 1.25 for ultimate limit state (ULS) conditions.

The New Zealand standard (NZS 3404.1: 1997) equation is similar to the equation 2.16 for the ultimate resistance of channel shear connectors, except that the multiplier is reduced from 45 to 36.5. However, the use of equation 2.18 was limited to concrete strengths greater or equal to 20MPa and a concrete density of 2,300kg/m^3.

$$q_r = 36.5(t_{fsc} + 0.5t_{wsc})L_{sc}\sqrt{f_c'}$$ (Equation 2.18)

Where:

the average flange thickness of the channel is t_{fsc} (mm)

the web thickness of the channel is t_{wsc} (mm)

the length of the channel connector is L_{sc} (mm)

f_c' is the concrete cylinder strength (MPa).

The British Standard (BS 5400-5: 2005) and Australian Standard (AS 5100.6: 2004) provide values for the static shear strength of channel shear connectors having different concrete compressive strengths in tabular form as shown in tables 2.1 and 2.2 respectively.

Table 2.1 Nominal static strength of channel connectors (BS 5400-5: 2005)

Connector size	Connector material properties	Nominal static strength per connector (kN) and concrete cube strength			
		20MPa	30MPa	40MPa	50MPa
127mm x 64mm x 14.9kg x 150mm	Grade S275 of BS EN 10025-3 (2004)	351	397	419	442
102mm x 51mm x 10.4kg x 150mm	Grade S275 of BS EN 10025-3 (2004)	293	337	364	390
76mm x 38mm x 6.7kg x 150mm	Grade S275 of BS EN 10025-3 (2004)	239	283	305	326

Table 2.2 Nominal static strength of channel connectors (AS 5100.6: 2004)

Channel size (mm)	f'_{cy} = 20MPa	f'_{cy} = 25MPa	f'_{cy} = 32MPa	f'_{cy} = 40MPa
125 TFC	113	118	128	143
100 TFC	95	100	110	125
75 TFC	79	84	93	106

Note: f'_{cy} is the characteristic cylinder strength of concrete at the age being considered. The values in the table above are appropriate for channel shear connectors having a yield stress greater than or equal to 260 MPa and a length in excess of 50mm.

More recently, a comprehensive research programme on channel shear connectors was carried out by Pashan (2006) and Pashan and Hosain (2009) where 78 push-out tests were undertaken to evaluate the behaviour of channel shear connectors embedded in both solid and composite slabs. The testing was done in two different phases. Phase 1 consisted of 36 push-out tests in three batches of 12 specimens having equal channel height of 127mm; six specimens in each batch had solid concrete slabs, while the other six had composite slabs with wide ribbed metal. Phase was 2 similar to phase 1 with the only difference being the channel height, which was 102mm. The push-out specimen consisted of two identical reinforced concrete slabs having an average compressive strength of between 21.45 and 40.95MPa attached to the flanges of a steel section (W200x59) by means of channel shear connectors. Figure 2.7 shows the push-out test setup.

Two types of failure modes were observed: channel fracture and failure by crushing-splitting of the concrete. The specimens with a metal deck had a common type of failure caused by a concrete shear plane from the high concentration stresses within a smaller area. The failure mode was mostly governed by the compressive strength of the concrete for channels having equal lengths. Channel web fracture was observed for concrete with a higher strength, while those with lower strengths had failure due to concrete crushing-splitting. A new design equation was proposed after comparing the experimental values with those predicted using equation 2.16, as this was found to produce very conservative predictions for channels with smaller lengths. The new proposed design equation 2.19 produced an average of 10% difference in the predicted values when compared with the experimental data.

$$q_u = (336w^2 + 5.24LH)\sqrt{f_c^{'}} \qquad \text{(Equation 2.19)}$$

Where:

L is the length of the channel (mm)

w is the web thickness (mm)

$f_c^{'}$ is the compressive strength of the concrete

H is the height of the channel.

A regression analysis was carried out to develop a design equation for specimens with wide-ribbed metal deck slabs. The parameters used were the same as the solid concrete slab with the introduction of width to depth ratio of the rib metal deck. The results obtained from the statistical analysis of the values predicted by equation 2.20 showed 11.8% absolute difference from the experimental data with channels having heights of 102mm only.

$$q_u = (1.7LH \tfrac{wd}{hd} + 275.4w^2)\sqrt{f_c^{'}}$$ (Equation 2.20)

Where:

$\frac{wd}{hd}$ is the width to depth ratio of the rib metal deck.

The American code AISC (2010) provides the following equation for calculating the shear resistance of a hot-rolled channel connector embedded in solid concrete slab, which is claimed to have been adapted from equation 2.15.

$$Q_n = 0.3(t_f + 0.5t_w)l_a\sqrt{f_c^{'}E_c}$$ (Equation 2.21)

Where:

l_a is the length of the channel connector (mm)

t_f is the flange thickness (mm)

t_w is the web thickness of the channel (mm)

$f_c^{'}$ is the compressive strength of concrete (MPa)

E_c is the modulus of elasticity of concrete, taken as $E_c = 0.043 w_c^{1.5}\sqrt{f_c'}$

w_c is the density of concrete (1,500kg/m³ < w_c ≤ 2,500kg/m³).

Maleki and Bagheri (2008) carried out a series of experimental push-out tests on channel shear connections under monotonic and cyclic loading. The tests were carried out with four different types of concrete specimen, namely: plain concrete (C); reinforced concrete (RC); fibre reinforced concrete (FRC); and engineered cementitious composite (ECC). From the test results it was observed that the monotonic shear strength of most specimens were higher than their cyclic strength by about 10% to 23%, which was in agreement with research carried out by Shariati et al (2013). All specimens subjected to cyclic loading could not withstand up to 90% of their corresponding monotonic loading. The experimental results for the ultimate shear strength were compared with the Canadian CSA S16-01 and American AISC (2010) provisions given by equations 2.16 and 2.21, respectively. It was concluded the Canadian code was in better agreement, and more conservative than the American equation 2.21, for all cases. A parametric study carried out numerically by Maleki and Bagheri (2008) using ANSYS to investigate the variations in concrete strength, channel dimensions and the orientation of the channel indicated that changing the orientation of the channel affected the ultimate strength by an average of 16%, which undermined the previous research done by Menzies (1971). The results also showed that the flange and web thickness of the channel, length of the channel and concrete strength were very important parameters in determining the ultimate strength of the channel shear connector.

An equation for the ultimate strength of channel shear connectors was derived after experimental investigations by Maleki and Mahoutian (2009) on the behaviour of channel shear connectors embedded in fibre-reinforced concrete. It was predicted that the fibre reinforcement had no adverse effect on the compressive strength of the concrete, therefore, a comparison was made between results obtained and the Canadian equation 2.16. The shear capacity was on average found to be 26% lower than channel connectors embedded in normal concrete as predicted by the Canadian equation 2.16. Therefore a new equation 2.22 was proposed for channels embedded in concrete reinforced with polypropylene fibres.

$$q_n = 27.2(t_f + 0.5t_w)L_c\sqrt{f_c^{'}} \qquad \text{(Equation 2.22)}$$

Shariati et al (2011) carried out tests on the behaviour of channel connectors embedded in normal concrete and lightweight concrete to determine its shear resistance. It was concluded that lightweight concrete could be used for composite construction, but the resistance tended to reduce when plain concrete was replaced with lightweight concrete. Also, lightweight concrete has a significant effect on the ductility and load-displacement performance of the composite beam. Further research carried out by Shariati et al (2012) on the monotonic and cyclic behaviour of channel connectors embedded in high strength concrete indicated that in all specimens, failure was due to channel fracture. Specimens with higher channel connectors had a higher ultimate load value and were more flexible than channel connections with lower height. Baran and Topkaya (2012) undertook further investigations on the behaviour of channel shear connectors with different heights and lengths. The result was compared with the design equation given in the American (AISC 2010) and Canadian CSA S16-01 standards. It was observed that both equations 2.16 and 2.21 were too conservative in determining the ultimate shear capacity of the channel connector, so equation 2.23 was proposed which is based on the observed deformation pattern utilising plastic analysis.

$$q_n = 0.25 \times F_1 \times F_2 \times f_c^{'} \times L_c \times H + \frac{2 \times t_w^2 \times L_c}{H} f_u \qquad \text{(Equation 2.23)}$$

$$F_1 = 7.2 - 0.023 \times L_c \qquad \text{(Equation 2.24)}$$

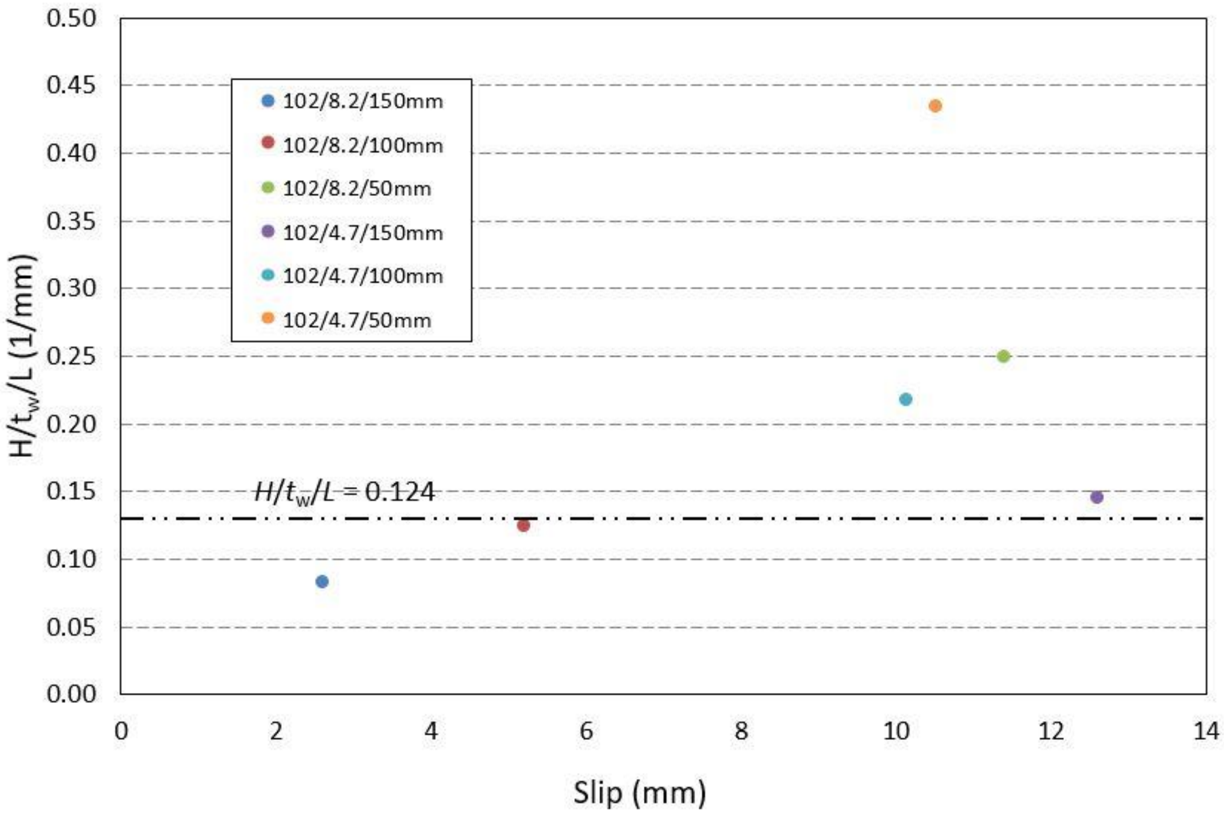
$$F_2 = 1.5 - 0.005 \times H \qquad \text{(Equation 2.25)}$$

Where:

f_u is the tensile strength of steel (MPa).

A study was undetaken in order to investigate the variables that could influence the ductility (slip) of channel shear connectors. The experiments by Pashan and Hosain (2009) were chosen for the study. Figure 2.8 shows the effect of the unit web slenderness of channel shear connectors on the ductility of the connectors. The unit web slenderness is defined as the ratio of the height of the channel shear connectors H over the web thickness t_w per unit length of the channel shear connectors L, ie $H/t_w/L$. Figure 2.8 suggests a rough criterion may be set up by which the channel shear connectors are deemed to be ‘ductile’ (ie slip at failure is larger than 6mm) as long as the unit web slenderness $H/t_w/L$ is larger than 0.124.

Figure 2.8 Effect of unit web slenderness of channel shear connectors on connector ductility (Pashan and Hosain 2009)



2.4 Conclusion

In this chapter, a review of existing tests and design expressions for the main connector types (identified in chapter 3) has been undertaken. In summary, the following observations can be made:

- Although the push test has been widely used since 1934, the form of the test specimen has varied considerably until the standard test given in ENV 1994-1-1 was published in 1994. As a consequence of this, some of the historical experimental data must be treated with caution in case the form of the test may have affected the results; this is particularly true of the test specimen given in BS 5400-5, where the poorly anchored transverse reinforcement is known to have resulted in low results from longitudinal splitting.
- To enable plastic design principles to be used at the ultimate limit state, the shear connectors should have sufficient slip capacity, which enables a redistribution of connector forces, and permits the designer to assume that each connector is loaded equally at failure (thereby allowing him to space the connectors equally). From the comprehensive numerical analyses that formed the basis of BS 5950-3.1 and Eurocode 4, a shear connector may be taken to be ductile if the characteristic slip capacity δ_{uk} is greater than or equal to 6mm.
- A great deal of testing has been undertaken on headed stud connectors and, because their performance is well understood, this is one of the reasons why many modern standards only consider this connector type. Experimental data from 113 push tests were used to calibrate the headed stud connector equations in the draft AS/NZS 5100.6. By utilising the historical material strengths given in

chapter 3, it should be possible to evaluate appropriate strength reduction factors for different periods of construction, so that the equations may be used for the assessment of existing bridges.

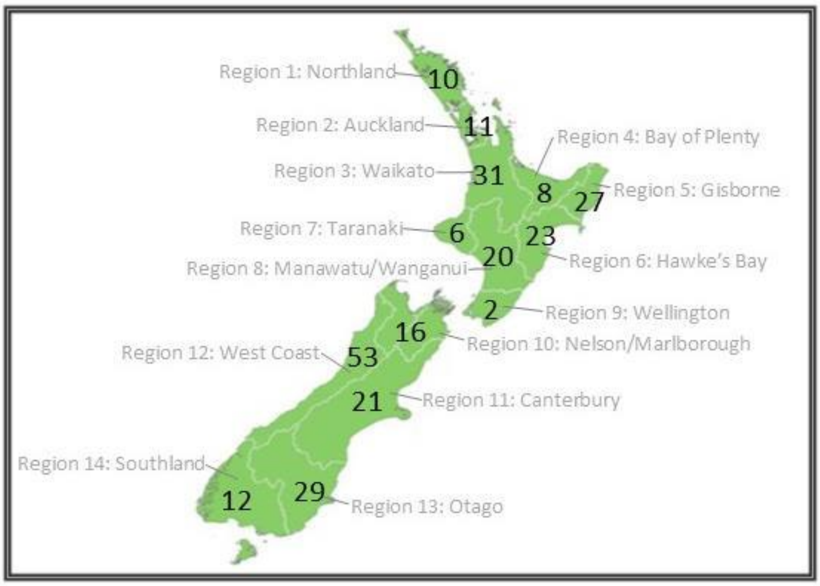
In chapter 3, the variety of proposed design equations suggests a unified design model has yet to be developed. During this review, experimental data from 150 push tests was collected, which will be used to identify a suitable candidate for the design model, before evaluating the appropriate strength reduction factor(s) using New Zealand concrete types.

3 Review of historical forms of shear connectors used in bridges in New Zealand

3.1 Introduction

There are approximately 270 steel-concrete composite bridges on New Zealand's state highway network, mostly consisting of reinforced concrete decks connected to braced steel I-beams, with welded channels or studs used to provide longitudinal shear connection. Figure 3.1 indicates the distribution of these bridges throughout the country, showing particularly high numbers in the central North Island and on the West Coast of the South Island.

Figure 3.1 Map showing number of composite bridges with each NZ Transport Agency territorial region, according to NZ Transport Agency (2009) *Bridge data system structural guide*



Over 70% of New Zealand's composite state highway bridges were constructed between 1950 and 1970, of which approximately three quarters were designed by the Ministry of Works. Based on their design live loading (typically H20-S16 or H20-S16-T16), the majority of these bridges are expected to be able to safely support full high-productivity motor vehicle (HPMV) loading. However, significant variability currently exists in the assessed live load capacity of composite bridges, even when they have been designed to identical design loadings.

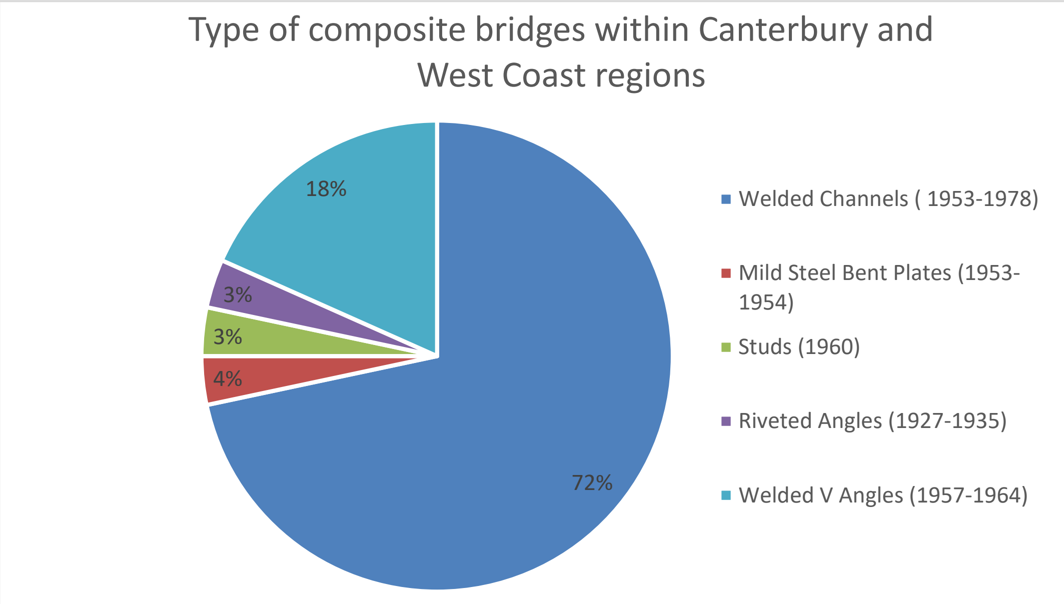
Even though design rules for shear connectors are well developed, many existing bridge shear connectors do not meet current design standards. For example, some shear cleats exceed the maximum spacing requirements, and more unusual cleat arrangements do not have a method for assessment within current Australian or New Zealand design standards. There is little guidance on how to assess composite bridges that do not meet current design standards.

3.2 Canterbury and West Coast case study

Over a quarter of New Zealand's composite bridges are located within the Canterbury and West Coast regions. These regions also have extensive as-built records that are readily available. For these reasons a case study was undertaken of composite bridges within NZ Transport Agency regions 11 and 12 (Canterbury and West Coast). This case study included an investigation into the type of shear connectors and the reinforcement layout in concrete decks for composite bridges.

From the current data in NZ Transport Agency (2009), there are 74 bridges with composite superstructures within regions 11 and 12. However, based on further detailed review, only 60 of these bridges contain details of composite connections on their available as-built drawings. From these 60 bridges, details of the type of shear connectors and deck reinforcement were recorded. The majority of the bridges have a steel I beam superstructure. However, a select few have a truss superstructure. Figure 3.2 indicates the percentages of the different types of composite bridges within the Canterbury and West Coast regions. From the graph, 72% of the bridges contain welded channels while welded V-angles occur in 18% of the bridges. Shear stud connectors, mild steel bent plates and riveted angles occur in the remaining 10% of the bridges.

Figure 3.2 Proportion of different types of shear connectors in composite bridges located in the Canterbury and West Coast regions (out of the 60 bridges sampled)



To further understand the structural details of the composite bridges, drawings of each type of composite connection were investigated. This included the type of connector (eg channel), cross-sectional dimensions and spacing along the beams. A summary of the results is provided in the sections below, with a more comprehensive list contained in appendix A (published separately at www.nzta.govt.nz/resources/research/reports/602).

3.2.1 Welded channels

Figure 3.3 indicates the proportion of different cross-section dimensions for the 43 welded channel composite bridges in the Canterbury and West Coast regions. A large majority of these bridges contain 4" x 2" (102mm x 51mm) x 7.09 lb (10.5kg/m) channels. The spacing of channels along the beams ranges from 1'–2' (305mm–610mm), with 2' being the most common spacing. The length of the channels ranges from 5" (127mm) to 9.5" (241mm), with 8" (203mm) being the most common length (23 out of 43). The channels are welded on to the beams with 3/16" (4.8mm) continuous fillet welds; from 1978 these became 5mm continuous fillet welds. Figure 3.4 shows a typical detail for the 4" x 2" x 7.09lb channels. Only one bridge had two different size channels on one continuous beam.

Figure 3.3 Proportion of sizes of welded channel shear connectors in composite bridges in the Canterbury and West Coast regions (out of 44 bridges)

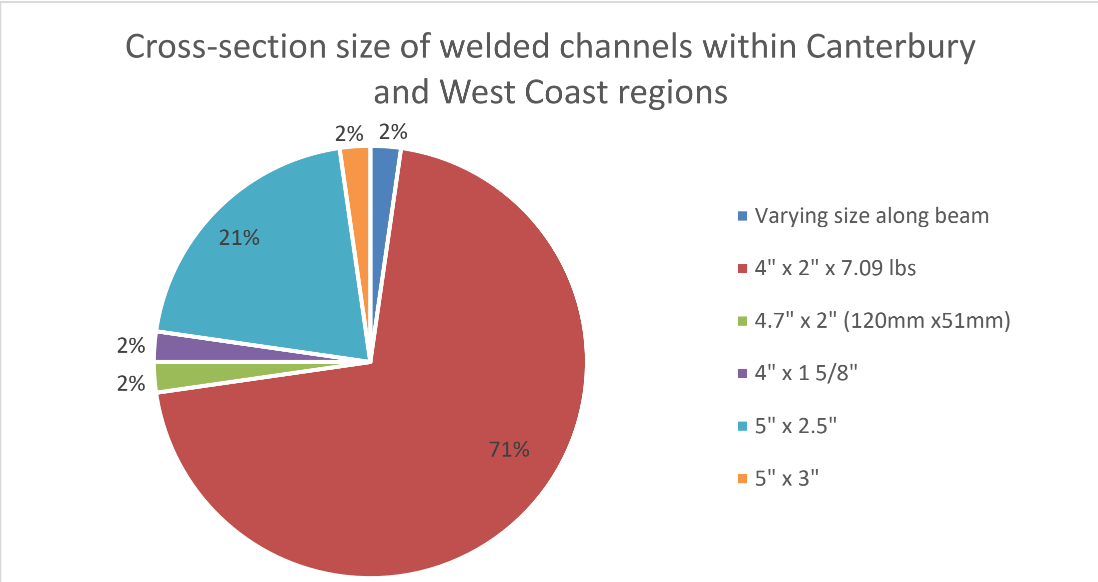
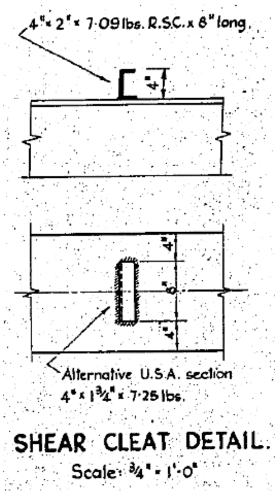


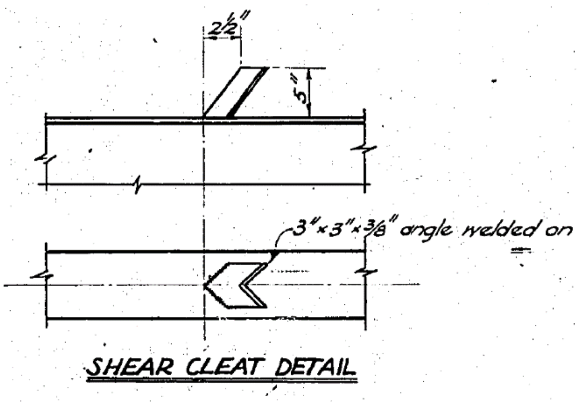
Figure 3.4 Most common welded channel shear cleat detail (refer appendix D for full drawings) (SH7 Arnold River Bridge, 1961, HCH 1538/9)



3.2.2 Welded V-angles

Welded V-angles are the second most common shear connector. They comprise 3” x 3” x 3/8” (76mm x 76mm x 9.5mm) angles, 5” (127mm) high on an angle as seen in figure 3.5 and are typically at varying spacing along the span of the beam, from 1’ to 2’. The angles are welded on to the beams with 3/16” continuous fillet welds.

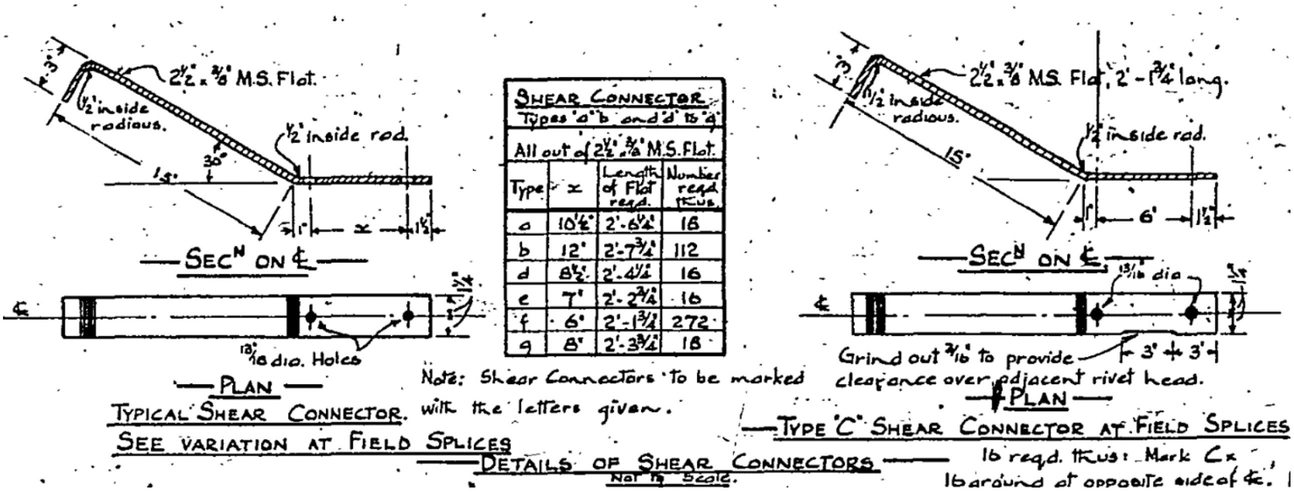
Figure 3.5 Common mild steel equal angle shear connectors (Refer appendix D for full drawings) (SH7 Black Water Creek Bridge, 1960, HCH 1776/2)



3.2.3 Mild steel bent plates

Two bridges have mild steel bent plates as shear connectors. The shape of these plates varies over the length of the beam; however, all shear connectors are constructed out of 2.5" x 3/8" (63mm x 9.5mm) plates (see figure 3.6 for details). The spacing between the shear connectors also varies along the length of the beams. The bent plates are connected to the steel beams with a ¾" diameter site rivets.

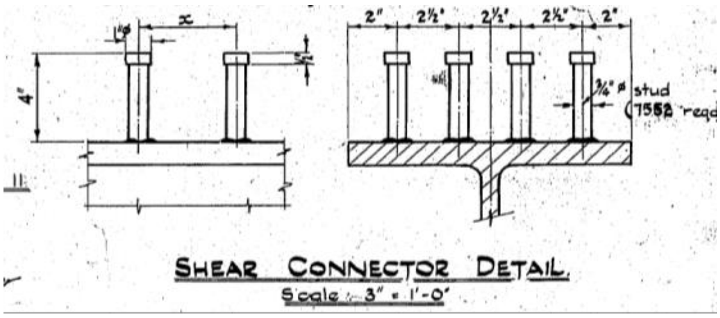
Figure 3.6 Mild steel plate shear connector detail (refer appendix D for full drawings) (SH8 Opuha River (Skiptons) Bridge, 1953, HCH710/IR)



3.2.4 Studs

Only two of the bridges have stud shear connectors. Connectors are ¾" (19mm) diameter with a 1" (25mm) diameter heads. One bridge has four studs at 2.5" (64mm) transverse spacing while the other has only two studs at 3.5" (89mm) transverse spacing. All welds are 3/16" (4.8mm) continuous fillet welds. Figure 3.7 shows the detail of a shear stud connector.

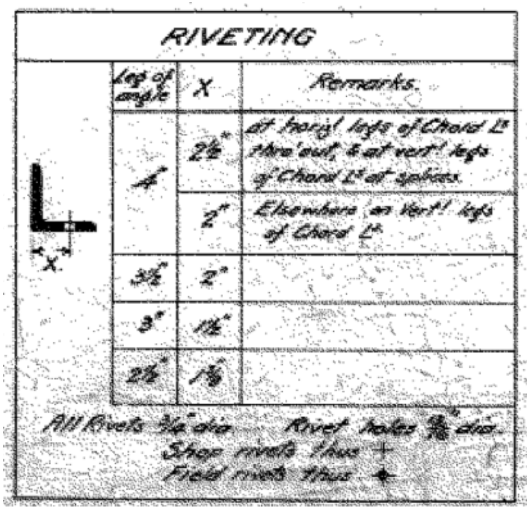
Figure 3.7 Stud shear connector detail (refer appendix D for full drawings) (SH7 Little Grey River (Mawheraiti), 1960, HCH 1436/14)



3.2.5 Riveted angles

Only two of the bridges sampled have riveted angle shear connectors. These are both 3" x 2" x ½" (76 x 51 x 12.7mm) angles at various spacings. The length of the rivets is either 7.5" or 8" (190 or 203mm). Figure 3.8 shows one of the details for the angle shear connectors.

Figure 3.8 Riveted angle shear connector detail (refer appendix D for full drawings) (SH1 Hurunui River Bridge, 1927, PWD G3833)



RIVETING

Leg of angle	X	Remarks.
4"	2½"	at horiz'l legs of Chord L^s thro'out, & at vert'l legs of Chord L^s at splices.
	2"	Elsewhere on Vert'l legs of Chord L^s
3½"	2"	
3"	1¾"	
2½"	1⅜"	

All Rivets ¾" dia. Rivet holes 13/16" dia.
Shop rivets thus +
Field rivets thus ✦

3.2.6 General spacing

Approximately half the bridges have the shear connectors spaced evenly along the span of the bridge while the other half have varying shear connector spacings. The most common shear connector spacing is 2' (610mm). The spacing of channels is generally smaller towards the ends of the span and larger at mid-span, which corresponds to the elastic longitudinal shear flow distribution at the interface between the beams and concrete deck.

3.2.7 Deck reinforcement

The majority of all the composite bridges have the following deck reinforcement:

- Transversely: typically 5/8" (16mm) diameter rebar with spacing ranging from 6"–9" (152–229mm), with 8" (203mm) the most common spacing.

- Longitudinally: typically ½" (12.7mm) diameter with spacing ranging from 9"–14" (229–356mm), with the most common spacing 12" (305mm).

There are a few decks with ½" (12.7mm) diameter rebar both longitudinally and transversely. In these cases the transverse spacing is approximately 5" (127mm).

3.3 North Island case studies

In addition to the Canterbury and West Coast case study, two North Island case studies were completed: the first involved a small collection of composite bridges that contained information in the Opus on-line database; and the second was a case study of the composite bridges in the Gisborne and Hawke's Bay regions.

3.3.1 Opus database case study

This case study utilised information within the Opus on-line database, which indicated that the majority of the composite bridges had channel or stud connectors, with a few angle connectors. The shear stud connectors were mainly on bridges constructed post-1990 with a typical stud diameter of 17.8 or 22mm. Almost all the examples of the channel connectors were 6" (152mm) long, 4" x 2" x 7 lb (102 x 51mm x 10.5kg/m) channels. This is a very similar result to the Canterbury and West Coast case study. See appendix B for more details on this study.

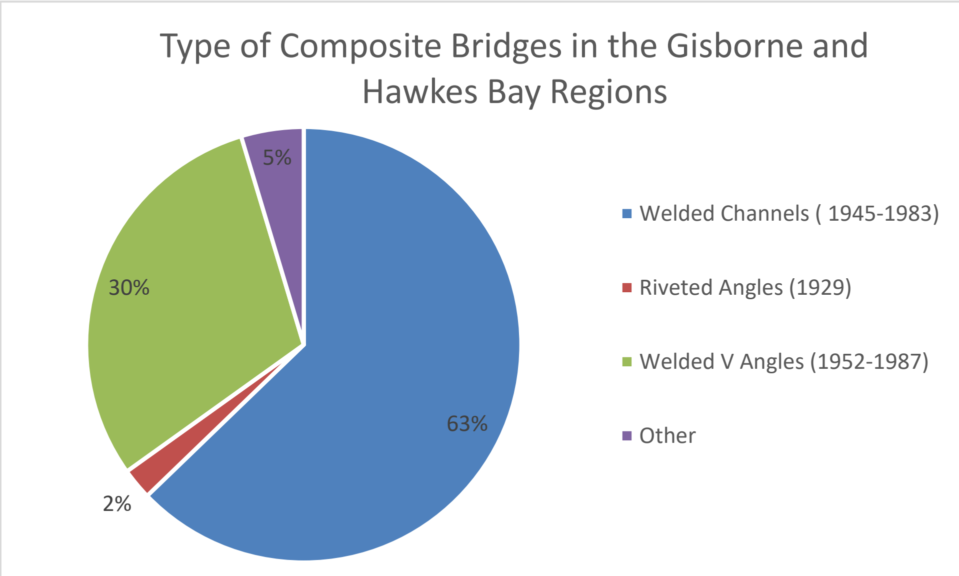
3.3.2 Gisborne and Hawke's Bay case study

A large number of composite bridges are located within regions 5 and 6 (Gisborne and Hawke's Bay respectively), with readily available as-built records, from which the type of shear connectors used for composite bridges was investigated.

From the available data in NZ Transport Agency (2009), there are 50 bridges with composite superstructures within regions 5 and 6. However, based on further detailed reviews, only 43 of these bridges contain details of composite connections on their available as-built drawings. From these 43 bridges, details of the type of shear connectors are given in appendix C. The bridge superstructures all comprise steel girders.

Figure 3.9 indicates the percentages of the different types of composite bridges within the Gisborne and Hawke's Bay regions. From the graph 63% of the bridges contain welded channels while welded V-angles cover 30% of the bridges. Riveted angles and other types of connectors make up the remaining 7% of the bridges. Other bridge shear connectors include one bridge with a combination of welded channels and welded vs angles, with another bridge having welded UB halves. The overall proportion of connector types is very similar to the results from the Canterbury and West Coast case study. See appendix C for more details of this study.

Figure 3.9 Proportion of different types of shear connectors in composite bridges located in the Gisborne and Hawke's Bay regions (out of the 43 bridges sampled)



3.4 Standard drawing

Three sets of standard drawings were investigated:

1 Ministry of Works (MoW) (1957 and 1959) Standard plans for highway bridges, folders nos 1 & 2 1957–1959 (H20-S16-44 design loading)

2 Ministry of Works (MoW) (1978) Standard plans for highway bridge components (HN-HO-72 design)

3 Works Consultancy Services (1990) Highway bridge standard plans (HN-HO-72 design).

3.4.1 Standard MoW H20-S16-44 composite designs

For H20-S16-44 design loading there are three different types of standard shear connectors: studs, welded V-angles and welded channels. In these drawings there is no mention of specific design standards used; however, they state: 'This design is based on materials and workmanship being in accordance with current specifications of the Ministry of Works'.

The normal design stresses (working stress design) for composite bridges are specified as follows:

- concrete: 1,000Psi (6.9MPa)
- steel beams: 20,000Psi (137.9MPa)
- structural steel: 20,000Psi (137.9MPa)
- reinforcing steel: 22,000Psi (151.7MPa)

Refer to tables 3.1, 3.2 and 3.3 for the corresponding characteristic concrete compressive strength and steel yield stresses. The above structural steel design stresses include the shear connectors design stresses.

The studs are 2 x ¾" (19mm) diameter with 1" (25mm) diameter heads, 5" (127mm) high at 3½" (89mm) spacing transversely and 1'3" (381mm) and 2' (610mm) spacing longitudinally for a 60' (18m) span. Figure 3.7 is an example of this type of standard MoW detail.

All the welded V-angles are 3" x 3" x ⅜" (76 x 76 x 9.5mm) angles, 5" (127mm) high with varying spacing. This is the same as all the Canterbury and West Coast bridges found in the case study as illustrated in figure 3.5. The spans range from 20' to 40' (6–12m).

Welded channel connectors are all 4" x 2" x 7lb (102 x 51mm x 10.5kg/m) RSCs, 8" (203mm) long (or an alternative American size of 4" x 1 ¾" x 7.25lb (102 x 45mm x 10.8kg/m)). The spacing of the channels depends on the span (which ranges from 35'–100' (10–30m), with the channel spacing increasing with the increasing span length and beam depth. It was noted that once the span reaches 50' (15m) the spacing of the channels is a constant 2' (610mm).

All deck reinforcements for these bridges appear to be 5/8" (16mm) diameter rebar at 8" to 9" (203-229mm) spacing transversely and 1/2" (12.7mm) diameter rebar longitudinally.

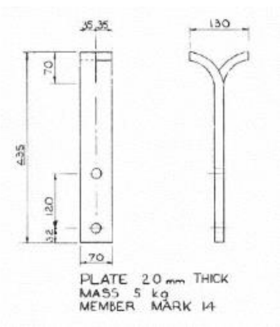
3.4.2 Standard MoW HN-HO-72 composite designs post-1978

After 1978, a new type of standard shear connector for the top chord of trusses and back-to-back PFCs was implemented. Figure 3.10 shows the mild steel plate truss shear connector.

The material strengths for truss bridges with mild steel plate truss shear connectors are specified as follows:

- concrete compressive strength (f'_c): 25MPa
- structural steel (f_y): grade 43A or grade 50B as specified
- steel plate shear connector (f_y): grade 50B
- reinforcing steel (f_y): deformed steel grade 275MPa.

Figure 3.10 MoW standard mild steel plate truss shear connector (refer appendix D for full drawings)



The only other shear connector used is a channel cleat. This is a 152 x 76 x 18kg/m channel, 150mm long (125mm long for 8m span). The spacing of the channels ranges from 225mm–500mm with spans ranging from 8m–22m. There is no obvious trend between the channel spacing and beam length as the spacing is dependent on a range of variables including the beam steel grade and dimensions. It was also noted the spacing decreases when the grade of the steel in the beam decreases.

The steel sections are designed to conform to BS 4 or BS 4848. The material strengths for bridges with channel connectors are specified as follows:

- concrete compressive strength (f'_c): 25MPa

- steel beams (f_y): grade 250LO (or 43C) or grade 350LO (or 50C)
- structural steel (f_y): 250MPa (AS 1204) or grade 43 (BS 4360)
- reinforcing steel (f_y): deformed steel grade 275MPa.

The above structural steel design material strength includes the shear connector strengths.

3.4.3 Standard Works Consultancy Services HN-HO-72 composite designs post-1990

Since 1990, the only standard shear connectors are welded channels, which are the same as previously (152 x 76 x 18kg/m channel, 150mm long). The spacing varies along the length of each beam ranging from 150mm to 450mm for 8–18m spans. There is no obvious trend between the channel spacing and beam length as the spacing is dependent on a range of variables including the beam steel grade and dimensions.

The drawings state 'this design is based on materials and workmanship being in accordance with specification CD405'. Steel beams are to conform to the specification given below, while all other steel sections are to conform to BS 4 or BS 4848. The material strengths for bridges with channel connectors are specified as follows:

- concrete compressive strength (f'_c): 25MPa
- steel beams (f_y): grade 250LO (AS 1204)/43C (BS 4360), or grade 350LO (AS 1204)/50C (BS 4360)
- structural steel (f_y): 250MPa (AS 1204) or grade 43 (BS 4360)
- reinforcing steel (f_y): deformed steel grade 300 MPa.

The above structural steel design material strength includes the shear connector strengths.

3.4.4 Non-standard designs

Following the privatisation of the Ministry of Works in 1988, a number of composite bridges have been designed that do not match any of the historic standard drawings listed above. These typically comprise welded stud connectors, with steel and concrete specifications varying depending on the specific design requirements.

3.5 Material strengths

As part of this work package, a brief review of historic material strengths was undertaken, as summarised below.

The NZ Transport Agency (2014) *Bridge manual* SP/M/022 (3rd edition) (referred to henceforth as the *Bridge manual*) and Transit NZ (2002) *Bridge overweight rating and posting weight limits assessment* were used to evaluate historic design strengths for concrete and structural steel.

3.5.1 Concrete strengths

A summary of the historic design standards for concrete production are:

- NZS 2086: 1967 Ready mixed concrete productions
- NZS 3104: 1983 Specification for concrete production – high grade and special grade

- NZS 3104: 1991 Specification for concrete production – high grade and special grade
- NZS 3104: 2003 Specification for concrete.

The coefficient of variation and minimum target mean strength of various concretes strengths are stated within the above codes. For certain codes this is based on the frequency of testing used by the concrete production plant.

A summary of the historic design standards for concrete are:

- NZS 95, Part V: 1939
- BS CP114: 1948 The structural use of normal reinforced concrete in buildings
- BS CP114: 1957 The structural use of normal reinforced concrete in buildings
- NZS 1900: 1964 Chapter 9.3: Design and construction, concrete
- NZS 3010P: 1970 Code of practice for reinforced concrete design
- NZS 3101.1 and 2: 1982 Code of practice for the design of concrete structures
- NZS 3101 Parts 1 and 2: 1995 The design of concrete structures
- NZS 3101.1 and 2: 2006 Concrete structures standard.

Prior to the 1960s New Zealand did not have its own concrete design codes and British standards were used. Following the 1960s New Zealand standards have been maintained and are still updated and amended today.

The historic strengths for concrete are summarised in table 3.1. These are based on the standard bridge design requirements at the time.

Table 3.1 Estimated strengths of concrete based on specification at the time

Construction date	Specified strength (MPa)	Normal design stress[(a)]	Allowable overstress[(a)]
Prior to 1932	14	4.1	6.2
1933 to 1940	17	5.5	7.6
1941 to 1970	21	6.9	9.0
1971 and later	25	–	–
1990 and later	Varies	–	–

Note: [(a)] Working stress design method

Target concrete strengths are typically 8MPa higher than the specified strengths shown in table 3.1 above. Results from concrete cores taken from existing structures have also demonstrated that the actual strength of concrete is typically much higher than specified in the original design (drawings) and as suggested in table 3.1, but is also highly variable even throughout the same bridge. Variations of up to 30MPa within a single bridge are not uncommon. However, over time as the methods for concrete mixing and quality control have improved the variability has reduced. For this reason it is very difficult to accurately estimate the actual historical concrete strengths based only on the age of the structure. Mean concrete strengths from testing of actual bridge concrete cores are typically up to 50% higher than the lower bound strengths specified for design.

3.5.2 Structural steel strengths

The historic structural steel strength as stated in Transit NZ (2002) are summarised in table 3.2. The yield stress values approximately align with the values stated in the *Bridge manual*.

Table 3.2 Historic structural steel strengths

Date built	Yield stress	Normal design stress[a]	Allowable overstress[a]
Prior to 1935	30,000 psi (206MPa)	f_s=16,000 psi (110MPa) f_v= 11,000 psi (75MPa)	21,000 psi (144MPa) 14,500 psi (99MPa)
1936 to 1940	30,000 psi (206MPa)	f_s= 18,000 psi (124MPa) f_v=12,500 psi (86MPa)	24,000 psi (165MPa) 16,500 psi (113MPa)
1941 to 1970	33,000 psi (227MPa)	f_s= 20,000 psi (137MPa) f_v=13,500 psi (93MPa)	27,000 psi (186MPa) 18,000 psi (124MPa)
1970 to 1980s	250, 275, 345, 350MPa	–	-
1990s onward	Varies	-	-

Note: [a] Working stress design method

f_s= allowable stress in tension, extreme fibre of structural steel

f_v= basic allowable mean shear stress on structural steel webs

A summary of the historic design standards for steel structures over the last 100 years are listed below.

- BS 4: Specification for structural steel sections, various editions from 1903–2005
- BS 15: 1906 Standard specification for structural steel for bridge and general building construction
- BS 15: 1912 Standard specification for structural steel for bridge and general building construction
- BS 15: 1930 Standard specification for structural steel for bridge and general building construction
- BS 548: 1934 High tensile structural steel for bridges and general building construction
- War emergency amendment to BS 548 1942 (withdrawn in 1965)
- BS 968:1941 (war emergency standard) High tensile (fusion welding quality) structural steel for bridges and general building purposes.
- War time amendment no.1 to BS 968 1943
- BS 15: 1936 Standard specification for structural steel for bridge and general building construction
- CF (15) 7376: 1941 War emergency revision to BS 15
- BS 15: 1948 Structural steel
- BS 15: 1961 Mild steel for general structural purposes
- BS 968: 1962 High tensile (fusion welding quality) structural steel for bridges or general building purposes
- BS 4360:1968 Weldable structural steels
- Amendment slip no.1 to BS4360 1968
- BS 4360 Part 2 1969 (metric units issued)
- BS 4848: 1972 Hot-rolled structural steel sections

- BS 4360: 1972 Weldable structural steels
- BS 4360: 1979 Weldable structural steels
- NZS 3404: 1977 (publisher SANZ Wellington) Code of design for steel structures (with commentary)
- AS 1204: 1980 Structural steels – ordinary weldable grades
- NZS 3404 Parts 1 and 2: 1989 Steel structures standard
- BS 4848: 1991 Hot-rolled structural steel sections
- NZS 3404 Parts 1 and 2: 1992 Steel structures standard
- NSZ 3404 Parts 1 and 2: 1997 Steel structures standard
- NSZ3404.1:2009 Steel structures standard – materials, fabrication and construction.

From the early 1900s until the 1970s these standards were based on the British standards. After 1970 New Zealand standards (NZS) were implemented, which are routinely updated and amended.

From the early 1970s, structural steel strengths used for standard Ministry of Works bridges were typically specified as either grade 43 (fy = 275MPa), grade 50 (fy = 345MPa), grade 250, or grade 350 (or similar equivalents). Shear channels and studs were typically specified as grade 250.

Mean steel yield strengths from testing of actual samples of historic steel are typically around 10% higher than the lower bound strength specified in design.

3.5.3 Reinforcing steel strengths

The historic reinforcing steel strength as stated in Transit NZ (2002) are summarised in table 3.3. The yield stress values roughly align with the values stated in the *Bridge manual*.

Table 3.3 Historic reinforcing steel strengths

Date built	*Bridge manual*	Yield stress	Normal design stress *	Allowable overstress*
Prior to 1932	210	30,000 psi (206 MPa)	f_s=16,000 psi (110 MPa) f_v=16,000 psi (110 MPa)	24,000 psi (165 MPa) 21,500 psi (148 MPa)
1933 to 1940	250	30,000 psi (206 MPa)	f_s= 18,000 psi (124 MPa) f_v=18,000 psi (124 MPa)	25,000 psi (172 MPa) 24,000 psi (165 MPa)
1941 to 1966	250/275	33,000 psi (227 MPa)	f_s= 20,000 psi (137 MPa) f_v=20,000 psi (137 MPa)	27,000 psi (186MPa) 27,000 psi (186 MPa)
1967 and later	275	40,000 psi (275 MPa)	f_s= 20,000 psi (137 MPa) f_v=20,000 psi (137 MPa)	27,000 psi (186 MPa) 27,000 psi (186 MPa)
1990 and later		Varies		

Note: *Working stress design method

f_s= allowable stress in tension, extreme fibre of main bars in reinforced concrete

F_v= allowable stress in tension in vertical stirrups in reinforced concrete

3.6 Capacity of composite bridges

Out of the 60 bridges sampled in Canterbury and West Coast, four were rated as restrictive to HPMVs during the previous 2012 national screening. One of these bridges has subsequently been strengthened and one has been re-analysed, resulting in a significant increase in the assessed capacity.

Current ULS design methods, also used for assessment require shear cleats to be designed assuming the full section of the main steel beams reaching yield. Many design codes, including the New Zealand steel code (NZS 3404: 1997) allow shear cleats to be spaced evenly provided they can resist the ultimate design force through their connection interface. Partial shear connection is also allowed for within the New Zealand steel code. Longitudinal shear forces from the serviceability limit state are required to be checked to ensure fatigue design requirements are met. However, for many low-volume state highways, fatigue of shear connectors may not be an issue.

Many historic composite bridges designed using working stress methods end up with having significantly higher live load capacity when assessed using an ULS method. Capacity increases are particularly large for structures with large dead loads that were not propped during construction. Shorter span structures with smaller beams designed using working stress methods also often have far more shear connectors than required using an ULS method of assessment. Conversely, bridges with deep sections that were designed to be propped during construction may result in lower ULS capacities when compared with their working stress design.

3.7 Failure investigation

An investigation was undertaken to determine whether any bridges on the New Zealand highway system exhibited signs of shear connector failure or distress. A request was sent out to the different regions managed by Opus on behalf of the NZ Transport Agency for feedback on the above matter. The findings of this request are given below.

3.7.1 Bridges in the regions

The feedback from the responding regional managers was as follows:

3.7.1.1 Regions 5 and 6

Region 5 and 6 covers the Gisborne and Hawke’s Bay area, where the type of shear connector used in composite bridges was found to be:

- 63% channel connectors
- 30% welded V-angles
- 5% riveted angles
- 2% other forms, such as a combination of welded channels and V-angles, as well as welded UB halves.

There are no signs of distress or failure identified to date on the 45 bridges submitted by the regional manager.

3.7.1.2 Region 10

Region 10 covers the Nelson, Tasman and Marlborough area, where the type of shear connector used in composite bridges was found to be:

- 55% welded V-angles

- 33% channel connectors
- 12% horizontal angles.

There were no signs of distress or failure identified on the nine bridges submitted by the regional manager.

It was noted that in addition to the above bridges, there are 10 steel non-composite bridges over 80 years old with no shear connectors. It appears they simply rely on the adhesion bond between the steel and cast-in-situ concrete slab. This indicates the current loading on these bridges is within the original designed loading, thus there have been no signs of distress or even the breaking of that bond.

3.7.1.3 Region 11 and 12

Region 11 and 12 covers the Canterbury and the West Coast areas, where the type of shear connector used in composite bridges is given in chapter 2.

There were no signs of distress or failure identified on the 60 bridges submitted by the regional manager. Two other bridges with signs of distress were identified, but both were originally designed as non-composite, hence they were not included in this review.

3.7.1.4 Region 13

Region 13 covers the Otago area, where the type of shear connector used in composite bridges was found to be:

- 32% welded V-angles
- 32% channel connectors
- 16% shear studs
- 20% other forms, such as bolted concrete panels, fishtail plates and welded spiral rebar.

Out of the 50 bridges submitted, six are exhibiting signs of distress and failure of the shear connection. These are:

- Welded V-angle: cracks in the concrete slab were noted on three bridges. These were erected in 1956, 1958 and 1960. The 1958 bridge has signs of rust staining, while both the 1956 and 1958 bridges, are exhibiting voids between the steel and concrete slab. See figure 3.11 for details.

Figure 3.11 Examples of distress signs noted

- While there is no evidence of the failure of the welded angle shear connector, there was breakage of the grout around the welded angle on the 1960 bridge; which may have been caused by excessive longitudinal movement between the concrete deck and steel girder. This could be attributed to the lack of reinforcement around the shear connector, as shown in figure 3.12. Alternatively, it could have been caused by serviceability related issues, such as poor specification and/or construction.

Figure 3.12 Breakage of the grout around the welded angle

- Bolted concrete panels: Three bolted concrete panels all exhibited signs of the breakage of the bolts connecting the concrete panels to the steel girder. These bridges were all erected in 1983, and it is believed they failed due to lack of an adequate number of bolts to provide a required shear connection. See figure 3.13.

Figure 3.13 Example of failed bolts providing the shear connection between the concrete slab and steel girder

3.7.2 Discussion

Out of 164 bridges that were part of this review, only six were identified with signs of failure in the vicinity of the shear connector, which corresponds to less than 4% of the bridges reviewed (all of which are located in region 13 (Otago)). Three of the affected bridges utilise welded angles, while the other three had failed bolted shear connectors. These signs may be attributed to poor detailing, specification and/or construction, in addition to overloading of the bridge at some point in the past.

Based on the above, the majority of composite bridges in New Zealand are performing satisfactorily, with the three welded angles showing signs of distress only, while the connector is still intact. It should be noted that none of the bridges utilising channel connectors were identified as having any failure signs or other related issues.

3.8 International reports on the performance of shear connectors in composite bridges

Through membership of the International Association for Bridge and Structural Engineering (IABSE) Working Commission 2 (IABSE WC2) 'Steel, Timber and Composite Structures' and the European Convention for Constructional Steelwork Technical Committee 11 (ECCS TC11) 'Composite Structures', one of the authors of this report has used the following international network to obtain shear connector performance data in composite bridges:

Name	Company	Country
Roman Geier	Schimetta	AT
Alessio Pipinato	University Padova	IT
Javier Jordán	Pedelta	ES
Andrea Frangi	ETHZ	CH
Tobias Lehnert	Dillinger	DE
Ulrike Kuhlmann	University of Stuttgart	DE
Calvin Schrage	NSBA	US
Wilen Heikki	Ruukki	FI
Oliver Hechler	ArcelorMittal	LU
Gerhard Hanswille	University of Wuppertal	DE
Thomas Petraschek	OEBB	AT
Jon Solemsli	Norconsult	SE
Jon Halden	Ramboll	NO
Laurence Davaine	Ingérop	FR
Ilkka Vilonen	Ramboll	FI
Daniel Lõhmus	Ramboll	EE
Rasmus Walter	Ramboll	DK
Morten de la Motte	Ramboll	DK
Mike Needham	Ramboll	UK
Nirmalya Bandyopadhyay	STUP	IN
Luigino Dezi	Univeristy PM	IT
Manuel Escamilla	ACL	ES
Bert Hesselink	Movares	NL
Vesa Jarvinen	Ruukki	FI
Markus Knobloch	ETHZ	CH
Roberto Leon	Virginia Tech	US
Hans Petursson	Trafikverket	SE
Heinz Pircher	TDV	AT
Gianluca Ranzi	University Sydney	AU
Roman Safar	CTU	CZ
Paul Skelton	Hardesty & Hanover	US
Richard Stroetmann	TU Dresden	DE
Jörgen Thor;	Brandskyddslaget	SE
Tina Vejrum	COWI	DK
Xin Zhao	Tongji University	CN
Bert Snijder	University Eindhoven	BE
Eduardo Batista	UFRJ	BR

Name	Company	Country
Anne Blom	Movares	NL
Graziano Leoni	University of Camerino	IT
Shunichi Nakamura	Tokai University	JP
Marion Rauch	Germanischer Lloyd	DE
Hans de Backer	University of Ghent	BE
Chris Hendy	Atkins	UK
Miguel Ortega	IDEAM	ES
Peter Collin	LTU	SE

From the replies received, it appeared there had been no reported failures of channel shear connectors in composite bridges. In Germany only a very small number of bridges have been constructed using this type of shear connection. Most of these bridges were erected between 1950 and 1960 and are still under traffic. Moreover, in the USA, channel shear connectors have been used so rarely, removal of the design provisions in the AISC specification has been considered.

3.9 Conclusions

A review of historic shear connectors has indicated the following:

- The vast majority (~67%) of the investigated historic shear connectors comprise welded channels (typically 4” x 2” (102mm x 51mm)).
- Design and evaluation criteria for shear channels and headed studs are well developed; however, additional research should be considered. This should include determining the extent of partial composite action that can be achieved.
- Most of the remaining bridges (~24%) have shear connectors comprising welded V-angles. Analysis methods for this type of shear connector are not well documented, and additional research on its performance and behaviour is recommended.
- Standard Ministry of Works bridge designs make up approximately three quarters of existing composite bridges constructed between 1950 and 1970 on New Zealand state highways.
- Shear studs have gained more popularity since the late 1990s and have been seen as a more economical solution than the previously popular welded channels.
- The spacing of shear cleats in many historic composite bridges has been determined to match the elastic longitudinal shear flow distribution at the interface between the beams and concrete deck (current New Zealand design practice assumes elastic perfectly plastic behaviour of the shear connection to enable the shear connectors to be spaced uniformly, which is unique when compared with other international design standards such as BS 5400-5 and BS EN 1994-2).

The maximum spacing of historic shear cleats is typically less than the maximum spacing limits given for new design within NZS3404: 1997 (800mm) and AS5100.6:2004 (600mm, or three times the thickness of the slab, or four times the height of the shear connector, whichever is the least). However, it should be noted the results of full-scale tests on composite beams recommend, when using plastic design principles (Hicks 2007; Couchman 2015), that the shear connector spacing is consistent with AS5100.6: 2004 to prevent uplift effects.

- Actual tested material strengths are highly variable. In particular, the range of concrete strengths can vary by more than 30MPa even within a single bridge that has been designed assuming one type/strength of concrete. However, the mean strength is typically about 50% higher than the specified concrete strength.

- Expected mean steel strengths are typically 10% higher than the lower bound figures given in the *Bridge manual*. These design values are outlined in chapter 6 of this report.
- ULS analysis of composite bridges often provides significant gains in capacity above historic working stress design capacities.
- More testing is required to determine post-elastic performance at the historic shear connectors and the overall capacity of composite sections they are part of.
- A review of possible failures of the different shear connectors identified six bridges with signs of failure in the vicinity of the shear connector distressed or failed shear connector. Three bridges utilising welded angles exhibited such signs via cracks in the concrete, while another three bridges utilising bolted concrete panels, exhibited broken bolts. These signs may be attributed to poor detailing, specification and/or construction, in addition to overloading of the bridge at some point in the past. None of the bridges utilising channel connectors were identified with any issues.

From an international consultation on the performance of channel shear connectors, it appears there have been no reported failures of this type of connector.

4 Review of recent research outputs from New Zealand and overseas for shear connectors

The push test data from the research presented in chapter 2 is considered in this chapter and compared with the predictions given by the different international design equations. The performance of the different international design equations is assessed by calculating the values of the required capacity factor using current material standards and comparing this with the target capacity factor. The intention of this section is to identify a suitable candidate for the design model that can be used with the historical material strengths presented in chapter 3, so that assessments of existing composite bridges may be undertaken.

4.1 Overview of capacity factor design and reliability analysis

The only rational basis for deciding safety margins for the ultimate resistance of structural components or members is data from failures. If the design method is probabilistic, the data should be interpreted statistically; it is only from laboratory testing that failures occur in sufficient numbers for this to be done. In probability-based design, the probability of failure P_f is the basic reliability measure that is used in international standards such as ISO 2394: 2015. An alternative measure is the reliability index, which is used in the head code to the Eurocode suite EN 1990: 2002 and is related to the probability of failure P_f by:

$$P_f = \Phi(-\beta) \qquad \text{(Equation 4.1)}$$

Where:

Φ is the cumulative distribution function of the standardised normal distribution

β is the reliability index.

For ULS considerations, the target reliability index given in ISO 2394 and EN 1990 for a 50-year reference period is β = 3.8. Design values of resistances are defined so that the probability of having a more unfavourable value is as follows:

$$P(R \leq R_d) = \Phi(-\alpha_R \beta) \qquad \text{(Equation 4.2)}$$

Where:

α_R is the first order reliability method sensitivity factor for resistance given in ISO 2394.

Values of β for a different reference period can be calculated using the following expression:

$$\Phi(\beta_n) = [\Phi(\beta_1)]^n \qquad \text{(Equation 4.3)}$$

Where:

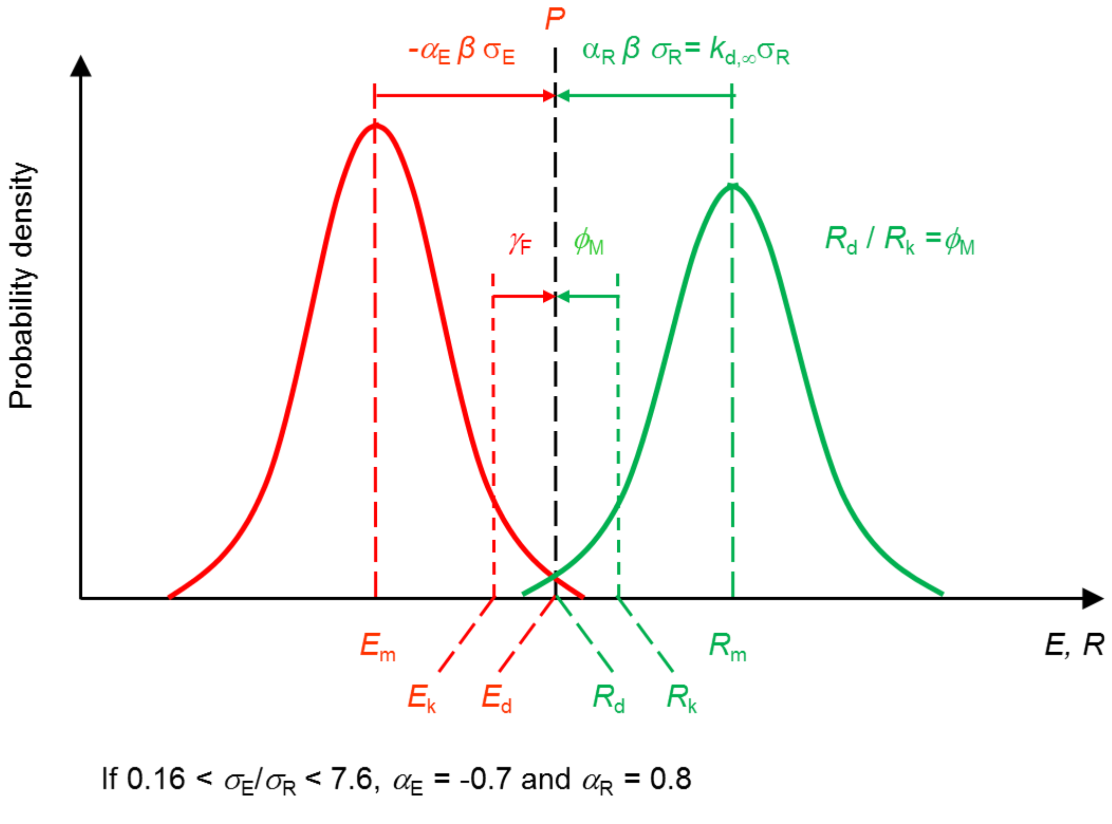
β_n is the reliability index for a reference period of n years

β_1 is the reliability index for one year.

While it is possible to calculate capacity factors for different reference periods using equation 4.3 (or when a structure has a higher consequence of failure), this is normally considered impractical as it would be tedious for designers to implement this in practice. As a result of this, internationally, it is normal to provide a higher reliability index through using tighter quality control and management measures through 'execution standards'. The international execution standard for reinforced concrete structures is ISO 22966: 2009. For steel structures, ISO/CD 17607: 2015 is still under development, but is based on the widely used European execution standard EN 1090-2: 2008.

The application of the first order reliability method is presented graphically in figure 4.1. The design point *P* is defined by the design value of the effects from actions E_d coinciding with the design value for resistance R_d. According to ISO 2394, if the ratio of the standard deviation of the actions σ_E and resistance σ_R is within certain limits (see figure 4.1), the design value is given by the reliability index β multiplied by the first order reliability method sensitivity factor α_E and α_R for actions and resistance, respectively. The advantage of this approach is that, the load factor γ_F and capacity factor ϕ_M can be evaluated separately for actions and resistance, respectively.

Figure 4.1 Design point P and reliability index β according to the first order reliability method for action effect E and resistance R as random variables having a normal distribution



Both ISO 2394 and EN 1990 give α_R = 0.8 for a dominating resistance parameter. Therefore, the design value for resistance corresponds to the product $\alpha_R\beta$ = 0.8 × 3.8 = 3.04 (equivalent to a probability of the actual resistance falling below the design resistance of 1 in 845 = 0.0012). The remaining safety is achieved in the specification of actions.

The design resistance R_d can be derived in two ways, either by direct determination from:

$$R_d = (R_m - k_{dn}\sigma_r) = R_m(1 - k_{dn}V_r) \quad \text{(Equation 4.4)}$$

Where:

R_m is the sample mean value

k_{dn} is the design fractile factor from equation 4.5 or 4.6 (for a probability of 0.0012, k_{dn} = 3.04 when $n = \infty$.),
σ_r is the sample standard deviation

V_r is the sample coefficient of variation [NB coefficient of variation = (standard deviation)/(mean value)].

Or by assessing a characteristic value, which is then divided by a partial safety factor as follows:

$$R_k == \phi R_k = \phi(R_m - k_n\sigma_r) = \phi R_m(1 - k_n V_r) \quad \text{(Equation 4.5)}$$

Where:

R_k is the lower characteristic resistance (also known as the fifth percentile resistance in AS/NZS 1170: 2002

k_n is the characteristic fractile factor from equation 4.6 or 4.7 (for a probability of 0.05, k_n = 1.64 when $n = \infty$.)

ϕ is the capacity factor which accounts for uncertainties of the basic variables contained within the equation for the design model, ie material and geometrical uncertainties, as well as uncertainties in the theoretical resistance function when compared with experimental values from tests ($\phi = R_d / R_k$).

For design assisted by testing, the design or characteristic value is based on the prediction method, which is a procedure for estimating a population's fractile from an available sample of limited size n. If the coefficient of variation of the population is known, the fractile factor is calculated from:

$$k_n = -u_p(1/n + 1)^{1/2} \quad \text{(Equation 4.6)}$$

Where:

u_p is the p fractile of the standardised normal distribution

n is the size of the population.

Alternatively, if the coefficient of variation of the population is unknown, the fractile factor is calculated from:

$$k_n = -t_p(1/n + 1)^{1/2} \quad \text{(Equation 4.7)}$$

Where:

t_p is the p fractile of the known student t-distribution (with $v = n - 1$ degrees of freedom)

n is the size of the population.

In most international standards such as EN 1990, it is preferable to assume that the coefficient of variation is known (equation 4.6). Therefore, the coefficient of variation of the population is assumed to be known in the analysis presented in this section.

4.1.1 Design assisted by testing and evaluation of capacity factor ϕ

A method for evaluating the design resistance of steel structures from tests was developed by Bijlaard et al (1988), which is based on the principles of ISO 2394 and has subsequently been implemented within EN 1990, annex D as the standard evaluation procedure for all materials. This methodology was used as the basis for the design equations and associated capacity factors within the draft AS/NZS 5100.6 and AS/NZS 2327 (Hicks and Jones 2013). For brevity, the methodology is not repeated here, but an overview is given by Hicks and Jones; it is also proposed that the methodology will be implemented within a normative appendix to AS/NZS 2327.

Based on observation of actual behaviour in tests and on theoretical considerations, a 'design model' is selected, leading to a resistance function. The efficiency of this model is then checked by means of a statistical interpretation of all available test data. If necessary, the design model is adjusted until sufficient correlation is achieved between the theoretical values and the test data.

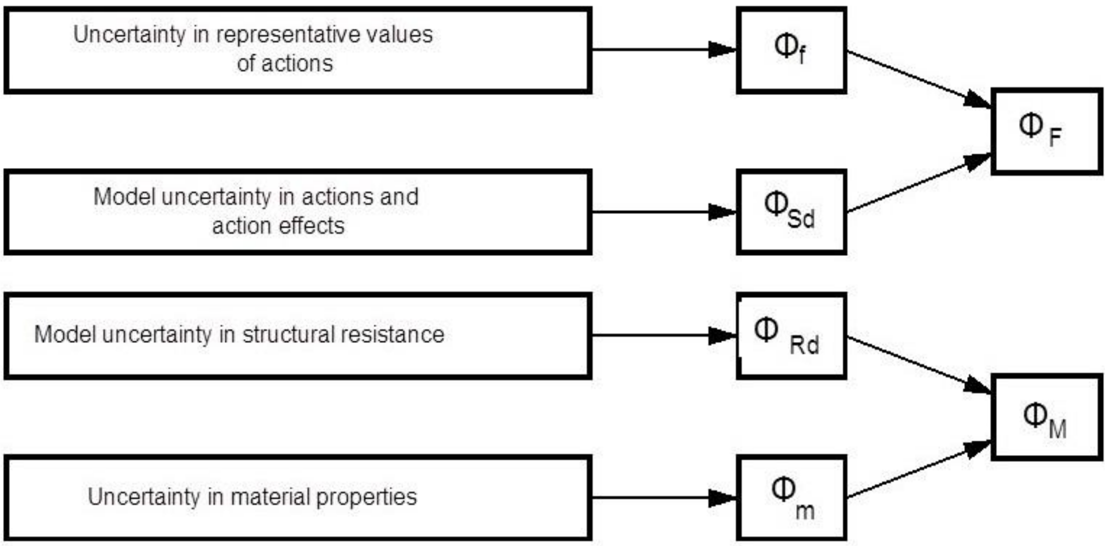
The variation in the prediction of the design model is also determined from the tests (ie the variation of the 'error' term δ). This variation is combined with the variations of the other variables in the resistance function. These include:

- variation in material strength and stiffness
- variation in geometrical properties.

The characteristic resistance is determined taking account of the variations of all the variables.

The design value is also determined from the test data and, hence, the ϕ-factor to be applied to the characteristic resistance function is obtained. The capacity factors and load factors are indicated in figure 4.2.

Figure 4.2 Capacity factors and load factors



From figure 4.2, the capacity factor is given by the following definition, which enables a calibration to be undertaken for any structural element composed of more than one material:

$$\phi_{M,i} = \phi_{Rd}\phi_{m,i} \quad i \geq 1 \tag{Equation 4.8}$$

Where:

ϕ_{Rd} is the capacity factor associated with the uncertainty of the resistance model (according to ISO 2394 ϕ_{Rd} should, in general, be $\phi_{Rd} \geq 1.0$)

$\phi_{m,i}$ is the partial factor for the material property.

For example, for structural concrete, the basic variable f'_c is considered as a characteristic value based on the lower 5% fractile, so that:

$$\frac{f'_{cd}}{f'_c} = \phi_{m,c} = \frac{f_{cm}\exp\left(-k_{d,\infty}V_{fc} - 0.5V_{fc}^2\right)}{f_{cm}\exp\left(-k_{\infty}V_{fc} - 0.5V_{fc}^2\right)} = \exp\left[\left(k_{\infty} - k_{d,\infty}\right)V_{fc}\right] \tag{Equation 4.9}$$

Where:

$k_{d,\infty}$ is the design fractile factor from equation 4.5 when $n = \infty$ ($k_{d,\infty} = 3.04$)

k_{∞} is the characteristic fractile factor from equation 4.6 when $n = \infty$ ($k_{\infty} = 1.64$)

V_{fc} is the coefficient of variation of the compressive strength of the concrete given by the appropriate territory's material standard (in New Zealand, NZS 3104: 2003).

4.1.2 Target capacity factor for ULS design

Although the value of the capacity factor is dependent on the permitted variability of a material strength within a particular territory, the different capacity factors for shear connectors used in international design standards are presented below. This comparison is useful as it gives an appreciation for the expected capacity factors that should be delivered by the respective design equations presented in chapter 2.

4.1.2.1 NZS 3404: 1997

According to NZS 3404, the capacity factor (strength reduction factor) for shear connectors is $\phi_{sc} = 1.0$ and $\phi_{sc} = 0.75$ for positive moment and negative moment regions, respectively. It will be shown below that the value of $\phi_{sc} = 1.0$ is extremely optimistic compared with other international standards as, in general, $\phi_{sc} \leq 1.0$. This value implies there is a significant amount of conservatism in the design equations compared with tests.

4.1.2.2 AS 5100.6: 2004

According to AS 5100.6, the capacity factor (strength reduction factor) for shear connectors is $\phi_{sc} = 0.85$, irrespective of whether they are contained within positive moment or negative moment regions. However, for other connection components subjected to shear, the capacity factor is $\phi_{sc} = 0.80$.

4.1.2.3 CSA-S16-09

According to CSA-S16-09, the capacity factor (resistance factor) for shear connectors is $\phi_{sc} = 0.80$, irrespective of whether they are contained within positive moment or negative moment regions.

4.1.2.4 AISC 2010

According to the AISC specification AISC 2010, the capacity factor (resistance factor) for shear connectors is $\phi_{sc} = 0.75$, irrespective of whether they are contained within positive moment or negative moment regions.

4.1.2.5 BS 5400-5 and BS 5950-3.1

The nominal static strengths of shear connectors presented in tabular for within BS 5400-5 and BS 5950-3.1 share the same basis, as they were evaluated from push tests undertaken by Menzies (1971) (see chapter 2). Given that plastic design of the shear connectors in BS 5400-5 is not permitted, BS 5950-3.1 is the appropriate standard for assessing the appropriate capacity factor. According to BS 5950-3.1, for shear connectors in positive moment regions $\phi_{sc} = 0.8$. While, for shear connectors in negative moment regions $\phi_{sc} = 0.6$.

4.1.2.6 EN 1990 and EN 1994-1-1 (Eurocode 4)

In the structural Eurocodes the values of the partial factors γ_M have been harmonised depending on the resistance criterion. Therefore, the design equations sometimes have a conversion factor η included within them to ensure that, when used with the harmonised partial factor, the equations deliver the correct design values. For steel structures, for yielding of the cross-section or stability failure, the target value $\gamma_M = 1.0$. For fracture or shear failure, the target value $\gamma_V = 1.25$ (V for shear) and is used in Eurocode 4 for all types of shear connectors irrespective of whether they are contained within positive moment or negative moment regions. This partial factor is equivalent to a capacity factor of ϕ_{sc} = 0.80 ($\phi = 1 / \gamma_M$), which is identical to that used in CSA-S16-09.

4.1.2.7 AS/NZS 5100.6

In the development of the design rules for headed studs (Hicks and Jones 2013), in accordance with international design standards, it was decided to harmonise the capacity factors for connectors subjected to shear. Therefore, a capacity factor of ϕ_{sc} = 0.80 has been selected.

4.1.3 General comments

From the above review of international standards, it appears the capacity factor for shear connectors is normally taken to be ϕ_{sc} = 0.80. Given the fact that this value has also been selected in the forthcoming AS/NZS 5100.6 for headed stud connectors, the target value for channel shear connectors should be ϕ_{sc} = 0.80.

The above ϕ_{sc} values in different international standards are concluded in table 4.1.

Table 4.1 Summary of ϕ_{sc} values in different international standards

Standard	ϕ_{sc}
NZS 3404: 1997	Positive moment region: 1.0 Negative moment region: 0.75
AS 5100.6: 2004	In both positive and negative moment regions Shear connectors: 0.85 Other types of connection in shear: 0.80
CSA S16-09	In both positive and negative moment regions 0.80
AISC 360-10	In both positive and negative moment regions 0.75
BS 5400-5 BS 5950-3.1	Positive moment region: 0.8 Negative moment region: 0.6
EN 1990 EN 1994-1-1	In both positive and negative moment regions 0.80
AS/NZS 5100.6	In both positive and negative moment regions 0.80

4.2 Evaluation of capacity factor for channel shear connectors embedded in solid concrete slabs

This section considers the 150 push tests collected in the literature review (chapter 2). In some cases, the concrete compressive strengths were lower than those established by NZS 3101 for structural concrete, which resulted in these experimental results being removed from the database. In addition, tests were removed when not all of the variables had been reported so the equations could be used. As a consequence of this, the database was reduced to 84 push tests (see table 4.2).

Table 4.2 Pushout tests parameters from the literature

S/N	Specimen	Reference	P_e(KN)	$f$cm (MPa)	fu(MPa)	Ecm(Mpa)	tf(mm)	tw(mm)	Lc(mm)	H(mm)
1	A1a	Pashan 2006	602.6	32.2	501.15	33436	8.1	8.3	152.4	127
2	A1b	Pashan 2006	603.6	32.2	501.15	33436	8.1	8.3	152.4	127
3	A2a	Pashan 2006	472.1	32.2	501.15	33436	8.1	8.3	101.6	127
4	A2b	Pashan 2006	474.1	32.2	501.15	33436	8.1	8.3	101.6	127
5	A3a	Pashan 2006	288.85	32.2	501.15	33436	8.1	8.3	50.8	127
6	A3b	Pashan 2006	295.8	32.2	501.15	33436	8.1	8.3	50.8	127
7	A4a	Pashan 2006	563.75	32.2	501.15	33436	8.1	4.8	152.4	127
8	A4b	Pashan 2006	576.7	32.2	501.15	33436	8.1	4.8	152.4	127
9	A5a	Pashan 2006	436.25	32.2	501.15	33436	8.1	4.8	101.6	127
10	A5b	Pashan 2006	464.15	32.2	501.15	33436	8.1	4.8	101.6	127
11	A6a	Pashan 2006	250.5	32.2	501.15	33436	8.1	4.8	50.8	127
12	A6b	Pashan 2006	256.95	32.2	501.15	33436	8.1	4.8	50.8	127
13	B1S	Pashan 2006	368.5	21.5	501.15	30290	8.1	8.3	150	127
14	B2S	Pashan 2006	330.15	21.5	501.15	30290	8.1	8.3	100	127
15	B3S	Pashan 2006	236.55	21.5	501.15	30290	8.1	8.3	50	127
16	B4S	Pashan 2006	408.85	21.5	501.15	30290	8.1	4.8	150	127
17	B5S	Pashan 2006	336.65	21.5	501.15	30290	8.1	4.8	100	127
18	B6S	Pashan 2006	224.85	21.5	501.15	30290	8.1	4.8	50	127
19	C1S	Pashan 2006	694.7	41.0	468.9	35190	8.1	8.3	150	127
20	C2S	Pashan 2006	516.45	41.0	468.9	35190	8.1	8.3	100	127
21	C3S	Pashan 2006	313.75	41.0	468.9	35190	8.1	8.3	50	127
22	C4S	Pashan 2006	677.3	41.0	468.9	35190	8.1	4.8	150	127
23	C5S	Pashan 2006	486.05	41.0	468.9	35190	8.1	4.8	100	127
24	C6S	Pashan 2006	262.95	41.0	468.9	35190	8.1	4.8	50	127
25	D1S	Pashan 2006	403.4	21.2	468.9	30236	7.5	8.2	150	102
26	D2S	Pashan 2006	326.7	21.2	468.9	30236	7.5	8.2	100	102
27	D3S	Pashan 2006	239.05	21.2	468.9	30236	7.5	8.2	50	102
28	D4S	Pashan 2006	396.4	21.2	468.9	30236	7.5	4.7	150	102
29	D5S	Pashan 2006	301.8	21.2	468.9	30236	7.5	4.7	100	102
30	D6S	Pashan 2006	201.2	21.2	468.9	30236	7.5	4.7	50	102
31	E1S	Pashan 2006	583.65	34.8	500.265	33960	7.5	8.2	150	102
32	E2S	Pashan 2006	488.05	34.8	500.265	33960	7.5	8.2	100	102
33	E3S	Pashan 2006	345.6	34.8	500.265	33960	7.5	8.2	50	102
34	E4S	Pashan 2006	542.8	34.8	500.265	33960	7.5	4.7	150	102
35	E5S	Pashan 2006	433.25	34.8	500.265	33960	7.5	4.7	100	102
36	E6S	Pashan 2006	244	34.8	500.265	33960	7.5	4.7	50	102
37	F1S	Pashan 2006	485.05	28.6	500.265	32428	7.5	8.2	150	102
38	F2S	Pashan 2006	375.5	28.6	500.265	32428	7.5	8.2	100	102

S/N	Specimen	Reference	P_e(KN)	$f$cm (MPa)	fu(MPa)	Ecm(Mpa)	tf(mm)	tw(mm)	Lc(mm)	H(mm)
39	F3S	Pashan 2006	268.9	28.6	500.265	32428	7.5	8.2	50	102
40	F4S	Pashan 2006	450.2	28.6	500.265	32428	7.5	4.7	150	102
41	F5S	Pashan 2006	358.55	28.6	500.265	32428	7.5	4.7	100	102
42	F6S	Pashan 2006	222.1	28.6	500.265	32428	7.5	4.7	50	102
43	P10(1)	Menzies 1971	264	28.5	500	32428	6.9	6.55	102	76
44	P11(1)	Menzies 1971	263.5	28.5	500	32428	6.9	6.55	102	76
45	P12(1)	Menzies 1971	267.5	28.5	500	32428	6.9	6.55	102	76
46	P13	Menzies 1971	178.4	20.2	500	30040	6.9	6.55	102	76
47	P14	Menzies 1971	176.4	20.2	500	30040	6.9	6.55	102	76
48	P15	Menzies 1971	197.3	20.2	500	30040	6.9	6.55	102	76
49	P16	Menzies 1971	198.8	31.3	500	33253	6.9	6.55	102	76
50	P17	Menzies 1971	201.3	31.3	500	33253	6.9	6.55	102	76
51	P18	Menzies 1971	194.8	31.3	500	33253	6.9	6.55	102	76
52	P19	Menzies 1971	219.2	40.4	500	35072	6.9	6.55	102	76
53	P20	Menzies 1971	222.2	40.4	500	35072	6.9	6.55	102	76
54	P21	Menzies 1971	232.2	40.4	500	35072	6.9	6.55	102	76
55	C1-M	Maleki and Bagheri 2008a; 2008b	83.9	27.5	360	24804	8.5	6	50	100
56	RC1-M	Maleki and Bagheri 2008a; 2008b	129.3	36.8	360	28693	8.5	6	50	100
57	RC2-M	Maleki and Bagheri 2008a; 2008b	98.8	34.2	360	27661	8.5	6	30	100
58	C1	Maleki and Mahoutian 2009	85	27.5	360	32000	8.5	6	50	100
59	RC1	Maleki and Mahoutian 2009	129.5	38.6	360	34720	8.5	6	50	100
60	RC2	Maleki and Mahoutian 2009	99	28.8	360	32520	8.5	6	30	100
61	N	Shariati et al 2011	128.3	47.1	360	35530	7	5	50	100
62	RC1	Shariati et al 2011	131.5	47.1	360	35530	7	5	50	100
63	RC2	Shariati et al 2011	92.7	47.1	360	35530	7	5	30	100
64	S65-50	Baran and Topkaya 2012	218.4	31.8	501	33360	7.5	5.5	50	65
65	S80-50	Baran and Topkaya 2012	252.6	33.3	467	33660	6	6	50	80
66	S100-50	Baran and Topkaya 2012	291.6	32.2	470	33440	8.5	6	50	100
67	S120-50	Baran and Topkaya 2012	293.1	39.9	465	34980	9	7	50	120
68	S140-50	Baran and Topkaya 2012	302.6	36.7	451	34340	10	7	50	140

S/N	Specimen	Reference	P_e(KN)	$f$cm (MPa)	fu(MPa)	Ecm(Mpa)	tf(mm)	tw(mm)	Lc(mm)	H(mm)
69	S65-75	Baran and Topkaya 2012	292.6	34.7	501	33940	7.5	5.5	75	65
70	S80-75	Baran and Topkaya 2012	310.4	33.8	467	33760	6	6	75	80
71	S100-75	Baran and Topkaya 2012	345.1	36.7	470	34340	8.5	6	75	100
72	S120-75	Baran and Topkaya 2012	385.2	32.7	465	33540	9	7	75	120
73	S140-75	Baran and Topkaya 2012	401.3	32.9	451	33580	10	7	75	140
74	S65-100	Baran and Topkaya 2012	320.6	34.0	501	33800	7.5	5.5	100	65
75	S80-100	Baran and Topkaya 2012	328	34.5	467	33900	6	6	100	80
76	S100-100	Baran and Topkaya 2012	378.3	33.4	470	33680	8.5	6	100	100
77	H10050–M	Shariati et al 2011	197.7	82.0	410	34677	8.5	6	50	100
78	H7550–M	Shariati et al 201	196.1	82.0	410	34677	7.5	5	50	75
79	N10030–M	Shariati et al 2011	115.5	63.0	410	30376	8.5	6	30	100
80	N7530–M	Shariati et al 2011	111.1	63.0	410	30376	7.5	5	30	75
81	C10050–M	Shariati et al 2013	152.5	35.7	360	29234	8.5	6	50	100
82	C7550–M	Shariati et al 2013	139.7	35.7	360	29234	7.5	5	50	75
83	C10030–M	Shariati et al 2013	112.3	35.7	360	29234	8.5	6	30	100
84	C7530–M	Shariati et al 2013	109.5	35.7	360	29234	7.5	5	30	75

As the purpose of this research was to present design equations for use in evaluating the resistance of bridges in New Zealand, the material variability and geometric tolerances were confined to this territory. As a starting point, the compressive concrete strength variability was based on the requirements given in NZS 3104 to identify the most suitable equation for use in current design before adjustments were made to account for the uncertainty in historical material strengths. Due to their being the most widely used the performance of the following design equations were considered:

- NZS 3404 (equation 2.18)
- CSA-S16-09 (equation 2.16)
- Pashan and Hosain (equation 2.19)
- AISC 2010 (equation 2.21)
- Baran and Topkaya (equation 2.23 to 2.25).

The results from the reliability analyses are presented in table 4.3, as well as the value of the required capacity factor ϕ_M. The capacity factor associated with the uncertainty of the resistance model ϕ_{Rd} is also presented to provide an indication of whether the design model is entirely appropriate (according to ISO 2394 ϕ_{Rd} it should, in general, be $\phi_{Rd} \leq 1.0$).

Table 4.3 Test results

S/N	NZS 3404		CSA S16-09		Pashan and Hosain 2009		AISC 2010		Baran and Topkaya 2012	
	ϕ_{RD}	ϕ_{M^*}	ϕ_{RD}	ϕ_{M^*}	ϕ_{RD}	ϕ_{M^*}	ϕ_{RD}	ϕ_{M^*}	ϕ_{RD}	ϕ_{M^*}
1	0.852	0.739	0.689	0.598	0.498	0.464	0.599	0.558	0.555	0.379
2	0.849	0.736	0.690	0.598	0.497	0.463	0.601	0.559	0.566	0.379
3	0.851	0.738	0.691	0.599	0.497	0.463	0.602	0.561	0.444	0.378
4	0.847	0.735	0.690	0.599	0.498	0.464	0.599	0.558	0.440	0.381
5	0.847	0.735	0.689	0.598	0.495	0.461	0.599	0.558	0.362	0.379
6	0.847	0.735	0.689	0.598	0.494	0.460	0.598	0.557	0.363	0.378
7	0.847	0.735	0.686	0.595	0.498	0.464	0.599	0.558	0.480	0.379
8	0.849	0.737	0.687	0.596	0.499	0.464	0.599	0.558	0.482	0.381
9	0.846	0.734	0.686	0.595	0.498	0.464	0.599	0.558	0.379	0.380
10	0.845	0.733	0.686	0.595	0.498	0.464	0.599	0.558	0.377	0.380
11	0.845	0.733	0.686	0.595	0.495	0.461	0.598	0.557	0.313	0.379
12	0.844	0.732	0.686	0.595	0.495	0.461	0.598	0.557	0.312	0.376
13	0.860	0.734	0.699	0.597	0.500	0.462	0.602	0.556	0.636	0.375
14	0.860	0.734	0.698	0.596	0.499	0.461	0.601	0.556	0.509	0.374
15	0.856	0.731	0.696	0.594	0.494	0.456	0.600	0.554	0.424	0.373
16	0.858	0.732	0.696	0.594	0.500	0.462	0.601	0.555	0.547	0.376
17	0.857	0.731	0.695	0.593	0.500	0.461	0.599	0.553	0.438	0.376
18	0.855	0.730	0.695	0.593	0.496	0.458	0.596	0.551	0.366	0.374
19	0.821	0.722	0.665	0.585	0.485	0.455	0.585	0.549	0.485	0.368
20	0.821	0.722	0.667	0.586	0.484	0.453	0.586	0.549	0.380	0.367
21	0.822	0.723	0.665	0.585	0.480	0.450	0.583	0.547	0.312	0.366
22	0.819	0.720	0.665	0.585	0.486	0.456	0.582	0.546	0.415	0.367
23	0.819	0.720	0.664	0.584	0.485	0.455	0.583	0.547	0.326	0.366
24	0.817	0.719	0.662	0.582	0.482	0.452	0.584	0.547	0.269	0.365
25	0.856	0.730	0.694	0.591	0.497	0.459	0.598	0.552	0.633	0.370
26	0.856	0.730	0.692	0.590	0.494	0.456	0.596	0.550	0.514	0.371
27	0.851	0.725	0.690	0.588	0.490	0.452	0.596	0.550	0.428	0.367
28	0.849	0.724	0.692	0.590	0.496	0.458	0.594	0.548	0.536	0.371
29	0.851	0.725	0.691	0.589	0.496	0.458	0.596	0.551	0.435	0.371
30	0.848	0.722	0.687	0.586	0.491	0.453	0.592	0.547	0.365	0.371
31	0.810	0.711	0.659	0.578	0.477	0.447	0.576	0.539	0.511	0.357
32	0.810	0.711	0.657	0.576	0.475	0.445	0.577	0.540	0.405	0.356
33	0.808	0.709	0.656	0.575	0.470	0.441	0.576	0.539	0.335	0.354
34	0.806	0.707	0.655	0.574	0.479	0.448	0.574	0.537	0.434	0.357
35	0.809	0.710	0.655	0.574	0.477	0.447	0.576	0.539	0.344	0.356
36	0.804	0.706	0.654	0.573	0.473	0.443	0.574	0.538	0.284	0.357

S/N	NZS 3404		CSA S16-09		Pashan and Hosain 2009		AISC 2010		Baran and Topkaya 2012	
	ϕ_{RD}	ϕ_{M^*}	ϕ_{RD}	ϕ_{M^*}	ϕ_{RD}	ϕ_{M^*}	ϕ_{RD}	ϕ_{M^*}	ϕ_{RD}	ϕ_{M^*}
37	0.878	0.758	0.713	0.615	0.514	0.478	0.618	0.574	0.611	0.398
38	0.880	0.759	0.713	0.616	0.513	0.477	0.620	0.576	0.494	0.400
39	0.876	0.756	0.711	0.614	0.509	0.473	0.617	0.573	0.410	0.397
40	0.876	0.756	0.710	0.613	0.515	0.478	0.619	0.575	0.520	0.399
41	0.876	0.756	0.711	0.614	0.514	0.477	0.617	0.573	0.417	0.398
42	0.875	0.755	0.709	0.612	0.509	0.473	0.617	0.573	0.347	0.398
43	0.854	0.735	0.691	0.595	0.499	0.463	0.599	0.556	0.493	0.381
44	0.852	0.734	0.691	0.595	0.498	0.462	0.600	0.557	0.495	0.378
45	0.852	0.734	0.690	0.594	0.498	0.462	0.599	0.556	0.491	0.379
46	0.830	0.687	0.678	0.561	0.478	0.435	0.568	0.517	0.563	0.337
47	0.833	0.690	0.676	0.560	0.478	0.435	0.573	0.521	0.563	0.338
48	0.833	0.690	0.677	0.561	0.477	0.434	0.573	0.522	0.566	0.338
49	0.901	0.771	0.730	0.625	0.526	0.487	0.632	0.585	0.567	0.413
50	0.902	0.772	0.728	0.623	0.525	0.486	0.631	0.584	0.566	0.415
51	0.900	0.770	0.727	0.623	0.524	0.485	0.631	0.584	0.567	0.414
52	0.839	0.728	0.681	0.590	0.492	0.458	0.592	0.551	0.450	0.374
53	0.837	0.726	0.680	0.589	0.492	0.459	0.593	0.552	0.453	0.372
54	0.838	0.727	0.679	0.589	0.493	0.459	0.593	0.553	0.452	0.374
55	0.868	0.741	0.705	0.601	0.500	0.462	0.606	0.560	0.431	0.383
56	0.811	0.701	0.659	0.569	0.472	0.438	0.573	0.533	0.349	0.349
57	0.853	0.735	0.692	0.596	0.490	0.455	0.602	0.558	0.368	0.380
58	0.868	0.741	0.703	0.600	0.501	0.463	0.606	0.560	0.430	0.383
59	0.831	0.719	0.673	0.583	0.482	0.449	0.587	0.546	0.358	0.364
60	0.879	0.751	0.714	0.610	0.505	0.467	0.616	0.570	0.409	0.396
61	0.838	0.734	0.678	0.593	0.493	0.461	0.597	0.558	0.290	0.383
62	0.826	0.715	0.670	0.580	0.483	0.450	0.584	0.543	0.304	0.365
63	0.878	0.751	0.713	0.609	0.506	0.468	0.612	0.566	0.351	0.397
64	0.839	0.728	0.681	0.591	0.485	0.452	0.592	0.551	0.370	0.375
65	0.853	0.741	0.693	0.602	0.498	0.464	0.606	0.565	0.334	0.390
66	0.845	0.733	0.687	0.596	0.492	0.458	0.599	0.558	0.351	0.378
67	0.809	0.710	0.657	0.577	0.475	0.445	0.576	0.540	0.311	0.357
68	0.836	0.730	0.677	0.592	0.488	0.456	0.594	0.556	0.352	0.372
69	0.810	0.705	0.657	0.572	0.472	0.441	0.575	0.537	0.371	0.357
70	0.798	0.694	0.648	0.563	0.468	0.437	0.566	0.528	0.320	0.345
71	0.832	0.727	0.677	0.591	0.490	0.458	0.592	0.554	0.359	0.372
72	0.855	0.742	0.694	0.602	0.499	0.465	0.606	0.565	0.398	0.384
73	0.861	0.747	0.697	0.606	0.502	0.468	0.608	0.566	0.417	0.387

S/N	NZS 3404		CSA S16-09		Pashan and Hosain 2009		AISC 2010		Baran and Topkaya 2012	
	ϕ_{RD}	ϕ_{M^*}	ϕ_{RD}	ϕ_{M^*}	ϕ_{RD}	ϕ_{M^*}	ϕ_{RD}	ϕ_{M^*}	ϕ_{RD}	ϕ_{M^*}
74	0.803	0.698	0.650	0.566	0.471	0.439	0.571	0.532	0.401	0.351
75	0.806	0.702	0.653	0.569	0.474	0.442	0.572	0.533	0.358	0.351
76	0.864	0.751	0.701	0.609	0.506	0.471	0.611	0.570	0.428	0.393
77	0.756	0.694	0.613	0.563	0.453	0.434	0.555	0.532	0.233	0.344
78	0.753	0.691	0.611	0.561	0.450	0.431	0.553	0.530	0.237	0.345
79	0.797	0.713	0.646	0.578	0.466	0.441	0.575	0.543	0.263	0.362
80	0.794	0.710	0.644	0.576	0.464	0.439	0.574	0.543	0.265	0.364
81	0.835	0.718	0.678	0.582	0.484	0.448	0.587	0.544	0.377	0.363
82	0.835	0.717	0.675	0.580	0.483	0.448	0.587	0.544	0.375	0.363
83	0.830	0.714	0.673	0.579	0.479	0.444	0.583	0.540	0.353	0.361
84	0.827	0.711	0.672	0.577	0.474	0.439	0.582	0.539	0.351	0.364
ρ	0.914		0.914		0.941		0.921		0.901	
b	1.482		1.202		0.83		1.141		0.711	
Vrt	7.03%		7.00%		6.51%		8.52%		9.47%	
Vδ*	26.15%		26.15%		22.52%		26.11%		27.54%	
Vr	27.07%		27.07%		23.44%		27.47%		29.12%	
φM*	0.728		0.590		0.456		0.552		0.373	

As can be seen from table 4.2, none of the design models performed very well and this is reflected in the low calculated capacity factors of 0.73, 0.59, 0.46, 0.55 and 0.37 for NZS 3404, CSA S16, Pashan and Hosain (2009), AISC (2010) and Baran and Topkaya (2012), respectively. The capacity factor of 0.73 for the design equation given in NZS 3404 is a particular concern as the recommended value is $\phi M^*=1.0$ (see section 3.1.2), which suggests that the current steel structures standard is on the unconservative side by almost 30%. From the magnitude of the correction factor *b*, the predictions given by both NZS 3404 and CSA S16 (AISC (2010)) are on the conservative side with values greater than unity. However, the reason for the punishing capacity factors that have been calculated is the very large scatter in the tests compared with the prediction with a coefficient of variation of the error terms (experimental tests), eg for NZS 3404, V_δ =26.2%. This is unusual as, typically, V_δ is relatively low and the main contributing term to the total coefficient of variation for resistance V_r is the theoretical coefficient of variation for resistance V_{rt} (determined from Monte Carlo simulations by varying the basis variables within the design equation according to tolerances and statistical properties of material strengths). The proposals by Pashan and Hosain (2009) and Baran and Topkaya (2012) provide the most unconservative predictions, which is reflected in the value of the correction factor b < 1.0 and a very punishing capacity factor. Owing to this poor performance, this design equation is discounted in future work.

From the above, two options exist for selecting a suitable design model for estimating the resistance of channel shear connectors in composite bridges:

1 On considering push test data, develop a completely new design model for channel shear connectors that more accurately reflects its behaviour (which will permit higher design capacities to be calculated).

2 Retain the existing design model given internationally by NZS 3404, CSA S16-09 and AISC 2010, but reduce the value of the multiplier so the target capacity factor of ϕ_{sc} can be used.

Due to budgetary constraints, we pursued the latter option (2) in the next phase of the project.

4.3 Conclusions

- From the database of push test results using channel shear connectors developed in chapter 2 (identified in chapter 3 as the most widely used form of shear connector in existing composite bridges), 84 tests were selected for analysis.
- The performance of the design equations given in NZS 3404, AISC 2010 and CSA S16-09, having all been developed from the same basis, have been evaluated from structural reliability analyses. Unfortunately, all design models perform badly with calculated capacity factors of ϕ_{sc}=0.73, 0.59 and 0.55 (which is substantially lower than the recommended value of ϕ_{sc}=1.0 given in NZS 3404), suggesting that current design values are overoptimistic (ie on the unsafe side). The design model recently proposed by Baran and Topkaya (2012) performed even worse and this model has been discounted for future work.
- While two options are open for selecting a more appropriate design model for estimating the shear resistance of channel shear connectors in composite bridges, due to budgetary constraints, it is proposed that a correction factor is applied to the NZS 3404 equation to ensure the target capacity factor is delivered. It should be noted, however, that the proposed correction factor will reduce the existing design capacity of channel shear connectors, but this is expected due to the poor predictions given by the design model when compared with test data.

5 Basis of proposed design methodology

5.1 Introduction

With the proposal for HPMV and 50MAX vehicles to have a wider access to the existing highway network, the capacity of the historic composite bridges based on the allowable stress design philosophy is in question. However, it can be envisaged that these composite structures will still exhibit higher section bending capacity than used to be expected. Benefitting from modern limit state design methods for composite structures, it has been acknowledged that the material plastic strength can be mobilised to create post-elastic bending capacity to an extent which relies on a collective behaviour of the ductility of the shear connectors, the degree of the shear connection and the steel section classifications. From chapters 2 and 3, it is known that in the majority of situations, historic shear connectors in New Zealand composite bridges may be considered ductile, so that partial shear connection design can be applied. The bending capacity of the bridges will thereby be governed by the strength of the shear connectors.

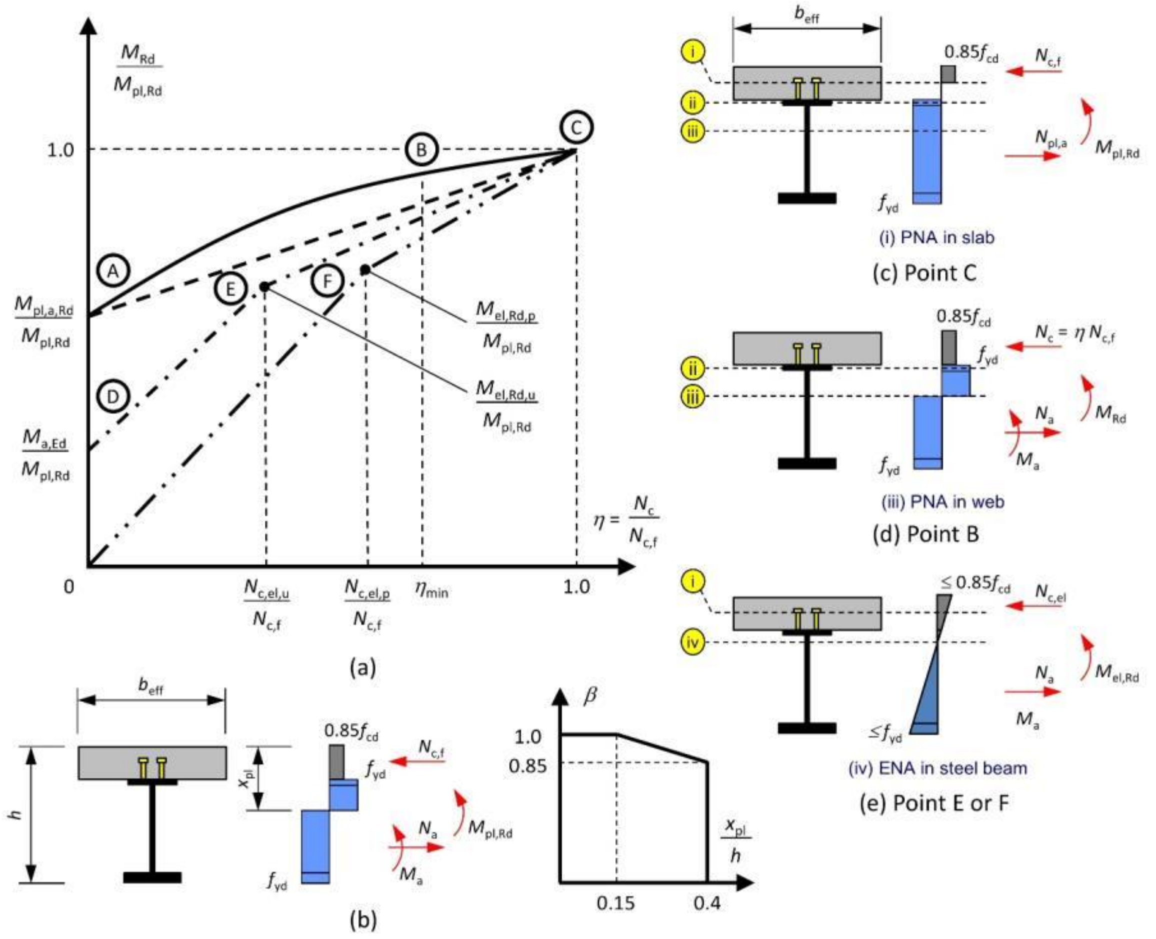
In helping engineers assess bridges with appropriate solutions, a brief introduction of the philosophy of the bending design for the composite beams is presented in section 5.2, followed by the presentation of design formulae in section 5.3. Remarks are made in section 5.4 as to the application of the design method to the assessment of existing composite bridges in New Zealand. This leads to an assessment procedure which is detailed in chapter 6, combined with the outcome of chapter 4 on the design capacity of channel shear connectors.

5.2 Codified design methods for the bending capacity of composite bridges

The relevant national standards in Australasia are AS 2327.1-2003 and NZS 3404:1997, which cope with bending capacity of composite structures assuming ductile shear connectors only with partial/full shear connection under limitation of the minimum degree of shear connection. On the contrary, Eurocode 4 provides a one-stop design solution covering a full spectrum of different shear connector behaviour and section classifications. It can be foreseen that this solution will be adopted in the forthcoming AS/NZS 2327: 2015 and be accepted as a general solution for composite design in New Zealand. The design philosophy from draft AS/NZS 2327 is extracted below.

The bending resistance of composite beams may be evaluated using rigid plastic theory, non-linear theory and elastic analysis. When the effective composite cross-section is compact, rigid plastic theory may be used for beams with full shear connection or partial shear connection that have connectors with sufficient deformation capacity to assume ideal plastic behaviour of the shear connection. The different design methods permitted are shown graphically in figure 5.1, together with the corresponding stress distributions for a composite beam with a solid slab.

Figure 5.1 Design method given in draft AS/NZS 2327



Notes to the figure:

Curve ABC – rigid plastic theory (equilibrium method)

Line AC – simple interpolation method

Line DEC – non-ductile and un-propped construction method

Line 0FC – non-ductile and propped construction method

The degree of shear connection is defined by:

$$\eta = N_c / N_{c,f} \qquad \text{(Equation 5.1)}$$

Where:

N_c is the design value of the compressive force in the concrete given as nP_{Rd}

$N_{c,f}$ is the design value of the compressive force in the concrete with full shear connection (which is the lesser of $A_a f_{yd}$ and $0.85 f_{cd} b_{eff} h_t$)

n is the number of shear connectors from the point of zero moment to the point of maximum moment

A_a is the cross-sectional area of the steel beam

f_{yd} is the design yield strength of the steel ($f_{yd} = \phi f_y$)

f_{cd} is the design compressive strength of the concrete ($f_{cd} = \phi_c f_c'$)

P_{Rd} is the design resistance of a shear connector ($P_{Rd} = \phi_V P_{Rk}$).

Full shear connection occurs at point C in figure 5.1, which corresponds to $\eta = 1.0$. From equilibrium of the stress blocks, the three possible positions for the plastic neutral axis are shown by (i), (ii) and (iii) in figure 5.1c. In this case, the plastic moment resistance $M_{pl,Rd}$ is evaluated using the strength reduction factors ϕ and ϕ_c, respectively.

The Eurocode 4 rules to evaluate the minimum degree of shear connection η_{min} are based on two independent studies (Aribert 1997; Johnson and Molenstra 1991) (point B in figure 5.1a), where the required slip was determined from numerical analyses of composite beams using various spans, cross-sections and degrees of shear connection. The rules are limited to situations where the required slip did not exceed the characteristic slip capacity of a shear connector, which was taken to be 6mm. Shear connectors were deemed to be 'ductile' in those situations.

The minimum degree of shear connection η_{min} can be obtained from the following expression:

a) For steel sections with equal flanges:

$$L_{ef} \le 25m : \eta_{min} = \max\left\{1 - \left(\frac{350}{f_y}\right)(0.75 - 0.03L_{ef}); 0.4\right\} \quad \text{(Equation 5.2)}$$

$$L_{ef} > 25m : \eta_{min} = 1.0$$

b) For steel sections having a bottom flange with an area equal to three times the area of the top flange:

$$L_{ef} \le 20m : \eta_{min} = \max\left\{1 - \left(\frac{350}{f_y}\right)(0.30 - 0.015L_{ef}); 0.4\right\} \quad \text{(Equation 5.3)}$$

$$L_{ef} > 20m : \eta_{min} = 1.0$$

Where:

L_{ef} is the distance in sagging bending between points of zero bending moment in metres.

For steel sections having a bottom flange with an area exceeding the area of the top flange but less than three times that area, η_{min} may be determined from equations 5.2 and 5.3 by linear interpolation.

For $\eta \geqslant \eta_{min}$ with a compact steel section and ductile shear connectors, the simple *interpolation method* may be used, where the design moment resistance M_{Rd} is evaluated by finding η and linearly interpolating between points A and C in figure 5.1a (point A is given by the design plastic moment resistance of the structural steel section $M_{pl,a,Rd}$ alone). By comparison, the *equilibrium method* based on the rigid-plastic theory can be used with the given convex curve ABC which is a less conservative alternative. Between A and C, the plastic neutral axis has two possible positions within the steel section given by (ii) and (iii) in figure 5.1d. For beams with partial shear connection and ductile shear connectors, M_{Rd} is evaluated using strength reduction factors ϕ, ϕ_c *and* ϕ_V for the structural steel, the concrete and the shear connection, respectively.

The design lines AC and ABC are based on the assumption the effective areas of the steel and concrete can reach their design strengths before the concrete begins to crush. AS/NZS 2327 and the forthcoming AS/NZS 5100.6 assumes there may be a possibility for premature crushing of the concrete if the nominal steel strength is f_y=420MPa or f_y=460MPa and the ratio x_{pl}/h is greater than 0.15. In these circumstances, the design resistance moment should be multiplied by the reduction factor β given in figure 5.1b. For x_{pl}/h beyond 0.4, the composite beam shall be treated as having a non-ductile shear connection using the methods below.

When $\eta < \eta_{min}$, or for cases when the characteristic slip capacity of an individual shear connector is less than 6mm, the shear connection is deemed to be 'non-ductile'. The design line for unpropped and propped construction is given in figure 5.1a by lines DEC and 0FC, respectively based on the *non-linear method*. Point F is defined by the design elastic moment resistance for propped construction $M_{el,p,Rd}$ and $\eta_{el,p}$, which corresponds to the point where the stresses in the outermost fibre of the section reach f_{cd} or f_{yd}, as shown in figure 5.1e. For point E, initial stresses from the bending moment applied to the structural steel section $M_{a,Ed}$ during the construction stage at point D reduce the design elastic moment resistance $M_{el,u,Rd}$ and the corresponding value of $\eta_{el,u}$.

Design lines DE and 0F are also referred to as the *elastic method* which was widely used under allowable stress design philosophy prior to the genesis of the USL design method. As one of the solutions allowable in AS/NZS 2327, it applies when the shear connection is designed to resist the longitudinal shear flow using elastic principles. The non-linear and elastic methods apply for all steel section classifications.

It is recommended table 5.1 of AS 5100.6:2004 be used when deciding the classifications of the composite bridge sections. It is anticipated that the table will be adopted in the draft AS/NZS 2327.

5.3 Design formulae for bending capacity

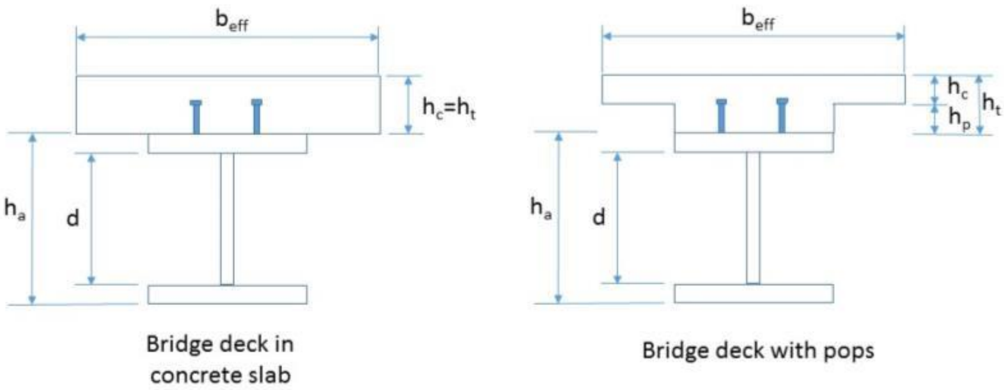
Having introduced the design philosophy of AS/NZS 2327 in section 5.2, the design formulae of the section bending capacity is presented below. Formulae are given for the steel beams with symmetric I- or H-sections with equal flanges. For other cross sections, the bending capacity may be evaluated using rectangular stress blocks and bending moments about the plastic neutral axis. This sub-section should be read in conjunction with the terms given in AS/NZS 2327 and section 5.2 together with the following supplementary definitions.

$N_{pl,a} = A_a f_{yd}$	Design value of the plastic resistance of the structural steel section to normal force.
$N_{pl,c} = 0.85 f_{cd} b_{eff} h_c$	Design compression resistance of the concrete flange.
$N_{pl,d} = t_w d f_{yd}$	Design value of the plastic resistance of the clear depth of the steel web to normal force.
$N_{pl,f} = b t_f f_{yd}$	Design value of the plastic resistance of the steel flange to normal force.
$N_{pl,n} = N_{pl,a} - N_{pl,d} + N_{pl,o}$	Design value of the plastic resistance of structural steel section with a class 3 web to normal force.
$N_{pl,o} = 60 \varepsilon t_w^{\ 2} f_{yd}$	Design value of the plastic resistance of the remaining portion of the compression part of a not-compact steel web to normal force. $\epsilon = \sqrt{\frac{250}{f_y}}$
$N_{pl,w} = N_{pl,a} - 2N_{pl,f}$	Design value of the plastic resistance of the steel web to normal force.
$N_s = A_s f_{sd}$	Design value of the plastic resistance of the longitudinal steel reinforcement to normal force.
N_c	Compression force in the concrete determined by the capacity of the shear connectors nP_{Rd}.

N_{cf}	Design value of the compressive normal force in the concrete flange with full shear connection.
$N_{c,el}$	Compressive force in the concrete flange corresponding to moment $M_{el,Rd}$.
b_{eff}	Effective breadth of the concrete flange.
d	Clear depth of the steel web.
f_{yd}	Design yield strength of steel ($f_{yd}=\phi f_y$).
f_{cd}	Design compressive strength of concrete ($f_{cd}=\phi_c f_c'$).
f_{sd}	Design yield strength of steel reinforcement ($f_{sd}=\phi f_{sy}$).
h_a	Depth of structural steel section.
h_c	For profiled sheetings, the height of concrete slab above crests, i.e. $h_c=h_t-h_p$. When solid slab is used, $h_c=h_t$.
h_p	The overall depth of the sheet excluding embossments; $h_p=0$ when solid slab is used.
h_s	Distance from the top flange of the steel beam to the centroid of the longitudinal reinforcement in tension.
h_t	Overall depth of the concrete slab.

A sketch showing section depths and widths using the notations is shown as below.

Figure 5.2 Notations for section dimensions adopted in section 5.3



5.3.1 Non-composite beam design

5.3.1.1 $M_{pl,a,Rd}$, point A in figure 5.1

$$M_{pl,a,Rd} = Z_{ep} f_{yd} \quad \text{(Equation 5.4)}$$

Where:

Z_{ep} is the plastic modulus of the steel section alone or determined as per Cl.5.1.3 of AS 5100.6: 2004.

5.3.2 Composite beams design with full shear connection

5.3.2.1 $M_{pl,Rd}$, point C in figure 5.1

Sagging bending

Case 1: $N_{pl,c} < N_{pl,w}$ (plastic neutral axis in web – iii in figure 5.1c)

a) Cross-section is compact:

The depth of the plastic neutral axis, measured down from the extreme fibre of the concrete flange in compression, is given as (which may be used to classify the web according to table 4.1 of AS 5100.6:2004):

$$x_{pl} = h_t + \frac{h_a}{2} - \frac{N_{pl,c}}{N_{pl,d}}\frac{d}{2} \qquad \text{(Equation 5.5)}$$

$$M_{pl,Rd} = M_{pl,a,Rd} + N_{pl,c}\frac{(h_a + h_t + h_p)}{2} - \frac{N_{pl,c}{}^2}{N_{pl,d}}\frac{d}{4} \qquad \text{(Equation 5.6)}$$

b) Cross-section with compact flanges, but with not-compact web that is further reduced to an effective cross-section into compact section with an effective web according to Cl.5.1.4 of AS 5100.6:2004:

$$M_{pl,Rd} = M_{pl,a,Rd} + N_{pl,c}\frac{(h_a + h_t + h_p)}{2} - \frac{\left(N_{pl,c}{}^2 + (N_{pl,d} - N_{pl,c})(N_{pl,d} - N_{pl,c} - 2N_{pl,o})\right)}{N_{pl,d}}\frac{d}{4} \qquad \text{(Equation 5.7)}$$

Case 2: $N_{pl,c} \geq N_{pl,w}$ (plastic neutral axis in flange

a) $N_{pl,a} > N_{pl,c}$ (plastic neutral axis in steel flange – ii in figure 5.1c).

$$M_{pl,Rd} = N_{pl,a}\frac{h_a}{2} + N_{pl,c}\left(h_t - \frac{h_c}{2}\right) - \frac{(N_{pl,a} - N_{pl,c})^2}{N_{pl,f}}\frac{t_f}{4} \qquad \text{(Equation 5.8)}$$

b) $N_{pl,a} \leq N_{pl,c}$ (plastic neutral axis in concrete flange – i in figure 5.1c).

$$M_{pl,Rd} = N_{pl,a}\left(\frac{h_a}{2} + h_t - \frac{N_{pl,a}}{N_{pl,c}}\frac{h_c}{2}\right) \qquad \text{(Equation 5.9)}$$

Hogging bending

Case 1: Plastic neutral axis in web

a) Cross-section is compact, $N_s < N_{pl,w}$:

The depth of the plastic neutral axis, measured down from the extreme fibre of the concrete flange in tension, is given as (which may be used to classify the web according to table 5.1 of AS 5100.6:2004):

$$x_{pl} = h_t + \frac{h_a}{2} - \frac{N_s}{N_{pl,d}}\frac{d}{2} \qquad \text{(Equation 5.10)}$$

$$M_{pl,Rd} = M_{pl,a,Rd} + N_s\left(\frac{h_a}{2} + h_s\right) - \frac{N_s{}^2}{N_{pl,d}}\frac{d}{4} \qquad \text{(Equation 5.11)}$$

b) Cross-section with compact flanges, but with not-compact web, which is further reduced to a compact cross-section with an effective web according to Cl.5.1.4 of AS 5100.6:2004.

$N_s < N_{pl,o}$: (Equation 5.12)

$$M_{pl,Rd} = M_{pl,a,Rd} + N_s\left(\frac{h_a}{2} + h_s\right) - \frac{\left(N_s^{\ 2} + \left(N_{pl,d} + N_s\right)\left(N_{pl,d} + N_s - 2N_{pl,o}\right)\right)}{N_{pl,d}}\frac{d}{4}$$

Where:

$M_{pl,a,Rd}$ is obtained from equation 5.4.

Case 2: Plastic neutral axis in flange

a) Cross-section is compact $N_s \geq N_{pl,w}$

i) Plastic neutral axis in steel flange $N_{pl,a} > N_s$:

$$M_{pl,Rd} = N_{pl,a}\frac{h_a}{2} + N_s h_s - \frac{\left(N_{pl,a} - N_s\right)^2}{N_{pl,f}}\frac{t_f}{4}$$ (Equation 5.13)

ii) $N_{pl,a} \leq N_s$ (plastic neutral axis outside steel beam).

$$M_{pl,Rd} = N_{pl,a}\left(\frac{h_a}{2} + h_s\right)$$ (Equation 5.14)

b) Cross-section with compact flanges, but with non-compact web which is further reduced to a compact cross-section with an effective web according to Cl.5.1.4 of AS 5100.6:2004 $N_s \geq N_{pl,o}$:

i) Plastic neutral axis in steel flange $N_{pl,n} > N_s$:

$$M_{pl,Rd} = N_{pl,n}\frac{h_a}{2} + N_s h_s - \frac{\left(N_{pl,n} - N_s\right)^2}{N_{pl,f}}\frac{t_f}{4}$$ (Equation 5.15)

ii) Plastic neutral axis outside steel beam $N_{pl,n} \leq N_s$

$$M_{pl,Rd} = N_{pl,n}\left(\frac{h_a}{2} + h_s\right)$$ (Equation 5.16)

5.3.3 Composite beam with partial shear connection

5.3.3.1 M_{Rd} of any point along curve ABC in figure 5.1 – equilibrium method (rigid-plastic theory)

For sagging bending

Case 1: $\eta N_{c,f} < N_{pl,w}$ (plastic neutral axis in web)

a) Cross-section is compact:

The depth of the plastic neutral axis, measured down from the extreme fibre of the concrete flange in compression, is given as (which may be used to classify the web according to table 5.1 of AS 5100.6:2004):

$$x_{pl} = h_t + \frac{h_a}{2} - \frac{\eta N_{c,f}}{N_{pl,d}} \frac{d}{2}$$ (Equation 5.17)

$$M_{Rd} = M_{pl,a,Rd} + \eta N_{c,f} \left(\frac{h_a}{2} + h_t - \frac{\eta N_{c,f}}{N_{pl,c}} \frac{h_c}{2} \right) - \frac{(\eta N_{c,f})^2}{N_{pl,d}} \frac{d}{4}$$ (Equation 5.18)

b) Cross-section with compact flanges, but with web being not-compact, which is further reduced to a compact cross-section with an effective web according to Cl.5.1.4 of AS 5100.6:2004:

$$M_{Rd} = M_{pl,a,Rd} + \eta N_{c,f} \left(\frac{h_a}{2} + h_t - \frac{\eta N_{c,f}}{N_{pl,c}} \frac{h_c}{2} \right) - \frac{\left((\eta N_{c,f})^2 + (N_{pl,d} - \eta N_{c,f})(N_{pl,d} - \eta N_{c,f} - 2N_{pl,o}) \right)}{N_{pl,d}} \frac{d}{4}$$ (Equation 4.19)

Where:

$M_{pl,a,Rd}$ is obtained in equation 5.4.

Case 2: $\eta N_{c,f} \geq N_{pl,w}$ (plastic neutral axis in steel flange)

$$M_{Rd} = N_{pl,a} \frac{h_a}{2} + \eta N_{c,f} \left(h_t - \frac{\eta N_{c,f}}{N_{pl,c}} \frac{h_c}{2} \right) - \frac{(N_{pl,a} - \eta N_{c,f})^2}{N_{pl,f}} \frac{t_f}{4}$$ (Equation 5.20)

5.3.3.2 M_{Rd} of any point along line AC in figure 5.1 – simple interpolation method

$$M_{Rd} = M_{pl,a,Rd} + \eta (M_{pl,Rd} - M_{pl,a,Rd})$$ (Equation 5.21)

Where:

$M_{pl,a,Rd}$ is obtained from equation 5.4 and $M_{pl,Rd}$ is obtained from equations 5.5 to 5.16.

5.3.4 Composite beam with non-linear design

5.3.4.1 $M_{a,Ed}$, point D in figure 5.1 for unpropped construction method

$$M_{a,Ed} = Z_e f_s$$ (Equation 5.22)

Where:

Z_e is the steel effective elastic section modulus

f_s is the stress at extreme fibre of the steel section under construction load.

5.3.4.2 $M_{el,Rd}$, point E in figure 5.1 for unpropped construction method

$$M_{el,Rd} = M_{a,Ed} + k M_{c,Ed}$$ (Equation 5.23)

5.3.4.3 $M_{el,Rd}$, point F in figure 5.1 for propped construction method

$$M_{el,Rd} = kM_{c,Ed} \quad \text{(Equation 5.24)}$$

Where:

$M_{a,Ed}$ is obtained from equation 5.22

$M_{c,Ed}$ is the part of the design bending moment applied to the composite section

k is the lowest factor such that any of the design stress limit

f_{cd}, f_{yd}, f_{sd} can be reached for concrete in compression, structural steel in tension or compression and in reinforcement in tension or compression respectively.

5.3.4.4 M_{Rd} of any point along line DEC in figure 5.1 for unpropped construction method – non-linear method

$$M_{Rd} = M_{a,Ed} + \left(M_{el,Rd} - M_{a,Ed}\right)\frac{N_c}{N_{c,el}} \text{ for } N_c \leq N_{c,el} \quad \text{(Equation 5.25)}$$

$$M_{Rd} = M_{el,Rd} + \left(M_{pl,Rd} - M_{el,Rd}\right)\frac{N_c - N_{c,el}}{N_{c,f} - N_{c,el}} \text{ for } N_{c,el} \leq N_c \leq N_{c,f} \quad \text{(Equation 5.26)}$$

Where:

$M_{a,Ed}$ is obtained from equation 5.22

$M_{el,Rd}$ is obtained from equation 5.23

$M_{pl,Rd}$ is derived from equation 5.5 to 5.15.

5.3.4.5 M_{Rd} of any point along line 0FC in figure 5.1 for propped construction method – non-linear method

$$M_{Rd} = M_{el,Rd}\frac{N_c}{N_{c,el}} \text{ for } N_c \leq N_{c,el} \quad \text{(Equation 5.27)}$$

$$M_{Rd} = M_{el,Rd} + \left(M_{pl,Rd} - M_{el,Ed}\right)\frac{N_c - N_{c,el}}{N_{c,f} - N_{c,el}} \text{ for } N_{c,el} \leq N_c \leq N_{c,f} \quad \text{(Equation 5.28)}$$

Where:

$M_{el,Rd}$ is obtained from equation 5.24

$M_{pl,Rd}$ is derived from equations 5.5 to 5.15.

5.3.4.6 M_{Rd} of any point along line DE in figure 5.1 for unpropped construction method – elastic method

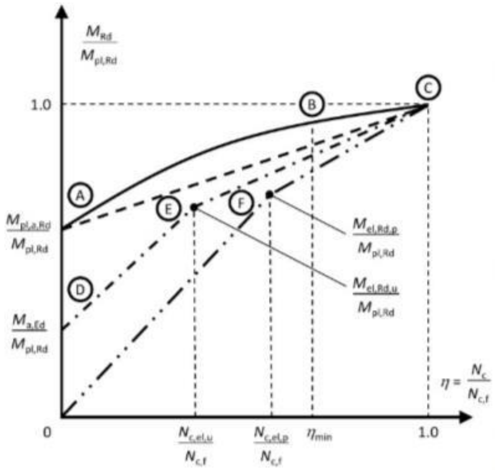
$$M_{Rd} = M_{a,Ed} + \left(M_{el,Rd} - M_{a,Ed}\right)\frac{N_c}{N_{c,el}} \text{ for } N_c \leq N_{c,el} \quad \text{(Equation 5.29)}$$

5.3.4.7 M_{Rd} of any point along line 0F in figure 5.1 for propped construction method – elastic method

$$M_{Rd} = M_{el,Rd} \frac{N_c}{N_{c,el}} \text{ for } N_c \leq N_{c,el} \qquad \text{(Equation 5.30)}$$

Table 5.1 summarises the application of design equations to different combinations according to the ductility of the shear connectors, degree of shear connections and steel section classifications. It also provides a quick reference for bridge engineers to select assessment options.

Table 5.1 Assessment options for bending capacity of composite beams according to draft AS/NZS 2327

<table>
<tr><td colspan="2" rowspan="3">
</td><td colspan="4">Ductility of shear connectors</td></tr>
<tr><td colspan="2">Ductile</td><td colspan="2">Non-ductile</td></tr>
<tr><td>Options[b]</td><td>Equations no.</td><td>Options</td><td>Equations no.</td></tr>
<tr><td rowspan="13">Steel section classification</td><td rowspan="8">Compact[a]</td><td>A</td><td>5.4</td><td>N/A</td><td>N/A</td></tr>
<tr><td>C</td><td>5.5 to 5.16</td><td>C</td><td>5.5 to 5.16</td></tr>
<tr><td>ABC[c]</td><td>5.18 to 5.20</td><td rowspan="3">DEC</td><td rowspan="3">5.25 to 5.26</td></tr>
<tr><td>AC[c]</td><td>5.21</td></tr>
<tr><td>DEC</td><td>5.25 to 5.26</td></tr>
<tr><td>0FC</td><td>5.27 to 5.28</td><td>0FC</td><td>5.27 to 5.28</td></tr>
<tr><td>DE</td><td>5.29</td><td>DE</td><td>5.29</td></tr>
<tr><td>0F</td><td>5.30</td><td>0F</td><td>5.30</td></tr>
<tr><td rowspan="5">Not-compact</td><td>C</td><td>5.5 to 5.16</td><td>C</td><td>5.5 to 5.16</td></tr>
<tr><td>DEC</td><td>5.25 to 5.26</td><td>DEC</td><td>5.25 to 5.26</td></tr>
<tr><td>0FC</td><td>5.27 to 5.28</td><td>0FC</td><td>5.27 to 5.28</td></tr>
<tr><td>DE</td><td>5.29</td><td>DE</td><td>5.29</td></tr>
<tr><td>0F</td><td>5.30</td><td>0F</td><td>5.30</td></tr>
</table>

Notes:

[a] The not-compact web may be treated as compact if the web section is reduced according to Cl.5.1.4 of AS 5100.6:2004.

[b] Assessment options are referred to as curves in figure 5.1.

Curve ABC – equilibrium method (rigid-plastic theory)

Line AC – simple interpolation method

Lines DEC/0FC – non-linear method (unpropped/propped construction)

Lines DE/0F – elastic method (unpropped/propped construction)

[c] These two options (excluding point C) only apply when degree of shear connection is larger than the minimum value, ie $\eta \geq \eta_{min}$ and when the steel section is compact. The rest of the options in the table apply regardless of this limitation. The shear connection is deemed to be non-ductile if $\eta < \eta_{min}$. even if the individual connectors meet certain criteria and are considered ductile. When steel beam is non-compact, similar rules to those for non-ductile shear connection apply.

When points A and C are calculated, alternative equations in sections 5.3.1 and 5.3.2 can be used respectively.

5.4 Conclusions

The review of the historical forms of shear connectors in chapter 3 revealed that the majority of shear connectors in New Zealand bridges are welded channel shear connectors. They comprise 71% and 63% of the total bridges studied in the South Island and the North Island respectively. Among the rest of the shear connectors, welded V-angles account for 18% and 30% of the bridges for the two regions, respectively. It has already been concluded in chapter 2 that the channel shear connectors are deemed to be ductile if they have a relatively short length (less than 150mm) with thin webs (less than 8mm). The push tests shown in chapter 2 have shown the shear studs also exhibit sufficient ductility. It can also be expected that under the modern traffic loadings, the load resistance of existing bridges may be increased by utilising partial shear connection theory where the composite bending resistance is limited by the resistance of the shear connectors.

With the section classifications, degree of the shear connection and the minimum degree of the shear connection being determined, assessment options can be chosen from section 5.3 and table 5.1. An assessment procedure in line with the current *Bridge manual* is presented in chapter 6 incorporating the assessment options from this chapter.

Consistent with the composite column provisions in AS 5100.6, instead of assigning a single global strength reduction factor, the design equations have been allocated with individual strength reduction factors φ for the individual material components. As the channel shear connector is the major type for the historic composite bridges in New Zealand, a reliability analysis has been carried out in this chapter to identify a formula that delivers the required margin of safety demanded by ISO2327 and AS 5104. The output of the analysis will also be applied in the assessment procedure given in chapter 6.

6 Assessment procedure for existing New Zealand composite bridges

6.1 Introduction

The *Bridge manual* is currently being used for the evaluation of local bridges and culverts. The manual refers to NZS 3404.1:1997 when the bending capacity of composite steel bridges is assessed. As discussed in section 5.2, NZS 3404 only addresses the capacity of composite structures made up of compact steel sections with shear connectors providing partial/full shear connection. These limitations set up a barrier in choosing appropriate options when engineers encounter various conditions of shear connectors and steel sections.

Previous sections of this report have identified the majority of composite bridges would have channel shear connectors providing partial shear connection under modern traffic evaluation loads. Pending AS/NZS 2327 and AS/NZS 5100.6, section 5.3 and table 5.1 of the report provide engineers with multiple assessment options dealing with variety of steel section classifications and the degree of the shear connection. An assessment procedure, adapted from table 7.1 of the *Bridge manual*, is presented in section 6.2.

When deciding the capacity of the channel shear connectors, equation 2.18 is recommended. Table 4.2 shows the strength reduction factor φ_{M^*}=0.728 has to be applied to the equation to deliver the required target reliability index. Compared with the reduction factor φ=0.85 as suggested in table 3.2 of AS 5100.6:2004 (note φ_{sc}=1.0 in table 13.1.2 of NZS 3404.1:1997 appears to be very unconservative), this finding indicates equation 2.18 is slightly unconservative. For consistency with the rules for headed stud connectors, by using the results presented in chapter 4, while still retaining the form of the existing design equation, the design value of channel shear connectors should be calculated from the following equation:

$$P_{Rd} = \phi_V \cdot 31.2(t_{fsc} + 0.5t_{wsc})L_{sc}\sqrt{f_c^{'}} \qquad \text{(Equation 6.1)}$$

Where φ_V=0.85:

t_{fsc} is the average flange thickness of the channel

t_{wsc} is the web thickness of the channel

L_{sc} is the length of the channel connector

f_c' is the concrete cylinder strength.

For an elastic calculation of the composite cross-section resistance (eg for fatigue design), it is necessary to ensure elastic behaviour of the beam, and the relative slip between the slab and the steel beam must be limited. This limitation is achieved by reducing the design value given by equation 6.1 by the factor k_s (EN 1994-2), whence:

$$k_s P_{Rd} = k_s \cdot \phi_V \cdot 31.2(t_{fsc} + 0.5t_{wsc})L_{sc}\sqrt{f_c^{'}} \qquad \text{(Equation 6.2)}$$

Where:

k_s is reduction factor for the shear resistance of a stud connector, taken as k_s = 0.75.

Owing to the fact that fatigue design is outside the scope of this research report, reference should be made to AS 5100.6 for further information.

A worked example prepared by OPUS according to the assessment procedures outlined in section 6.2 is provided in appendix F.

6.2 Assessment procedures

To represent the majority cases in New Zealand, this assessment procedure is for the bending capacity of the composite bridges and assumes an equivalent span in sagging bending moment as defined in the forthcoming AS/NZS 2327 and AS/NZS 5100.6. Both headed studs and channel shear may be considered. For simply supported bridges, the equivalent span is equal to the effective bridge span. Symmetric steel section is assumed in the equations presented in section 6.3. The evaluation procedure given in table 7.1 of the *Bridge manual* is replicated here with adaptation made where necessary to suit the purpose of this section.

Step 1: Carry out site inspection

Identify structural deterioration of the bridge structure, including, among others, the conditions of the shear connectors. As a reference, the review in chapter 4 concludes that the shear connectors in a majority of composite bridges in New Zealand perform robustly, with only a very small number of bridges exhibiting signs of distress at the shear connection.

Step 2: Determine appropriate material strengths

When deciding the material strength, information from sections 3.4 and 3.5 and appendix G of this report may be consulted alongside section 7.3 of the *Bridge manual*.

Step 2a: Identify types of the shear connectors and determine the ductility of the shear connectors

Check whether the shear connectors may be considered ductile. Headed shear studs may be considered ductile when the overall height after welding is not less than four times the shank diameter d_{bs} and 16mm$<d_{bs}\leq$25mm. Channel shear connectors may be considered ductile if the channel unit web slenderness.

$$H/t_w/L>0.124 \qquad \text{(Equation 6.2)}$$

Where:

H, t_w and L are height, web thickness and length of the channel shear connectors, respectively (see figure 2.8).

Step 2b: Identify dimensions of the concrete slabs and steel sections and the bridge effective span for the calculation of the effective width of concrete flange

Use figure 6.1 to calculate the effective bridge span L_{ef}. For a simply supported bridge, the effective span equals to the effective bridge span. Figure 6.1 will be introduced within the forthcoming AS/NZS 2327 and AS/NZS 5100.6.

Step 2c: Calculate effective breadth of the concrete flange

At mid-span or an internal support the total effective width b_{eff} (see figure 6.1) may be determined as:

$$b_{eff} = b_0 + \Sigma b_{ei} \qquad \text{(Equation 6.3)}$$

Where:

b_0 = distance between the centres of the outstand headed stud connectors, or the width of the shear connector (when channel shear connectors are used).

b_{ei} = value of the effective width of the concrete flange on each side of the web and taken as $L_{ef}/8$ (but not greater than the geometric width b_i). The value b_i defines the distance from the outstand shear connector to a point mid-way between adjacent webs, measured at mid-depth of the concrete flange, except that at a free edge b_i is the distance to the free edge. The length L_{ef} is taken as the approximate distance between points of zero bending moment. For typical continuous composite beams, where a moment envelope from various load arrangements governs the design, and for cantilevers, L_{ef} may be taken from figure 6.1.

The effective width b_{eff} at an end support is determined as:

$$b_{eff} = b_0 + \Sigma\beta_i b_{ei} \qquad \text{(Equation 6.4)}$$

with

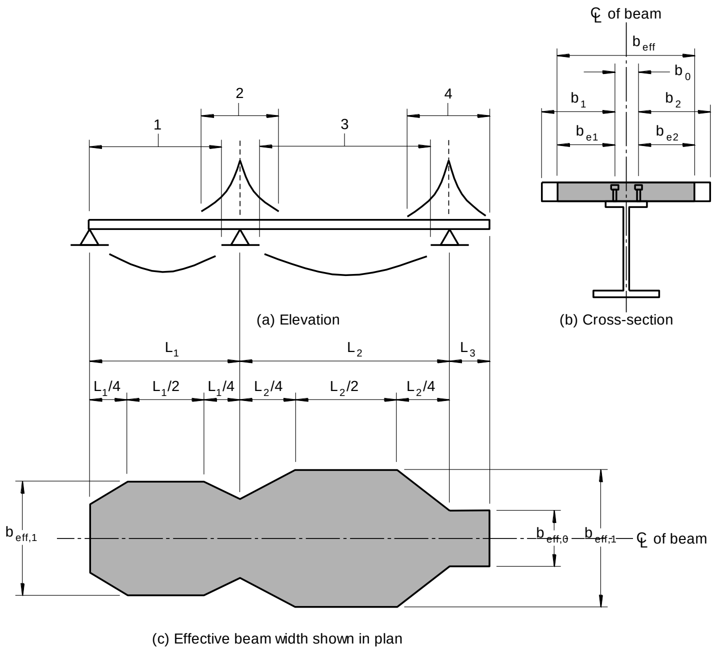
$$\beta_i = (0.55 + 0.025\ L_{ef} / b_{ei}) \qquad \text{(Equation 6.5)}$$

Where:

b_{ei} = effective width, see equation 6.3 of the end span at mid-span and L_{ef} is the equivalent span of the end span according to figure 6.1.

The distribution of the effective width between supports and mid span regions shall be assumed to be as shown in figure 6.1 or calculated by more refined analysis. Note that this rule will be adopted in the forthcoming AS/NZS 2327 and AS/NZS 5100.6.

Figure 6.1 Equivalent spans for calculation of the effective width of concrete flange



LEGEND:
L_{ef} = 0.85 L_1 for b_{eff} in segment 1
L_{ef} = 0.25 (L_1 + L_2) for b_{eff} in segment 2
L_{ef} = 0.70 L_2 for b_{eff} in segment 3
L_{ef} = 2 L_3 for b_{eff} in segment 4

Step 2d: Determine steel section classifications

Any section damage observed from step 1 shall be considered. Use table 5.1 of AS 5100.6:2004 to determine the section classifications.

Step 2e: Identify construction methods – propped or unpropped

Construction method may be identified in the as-built drawings. If the information is not available, it is reasonable to conservatively assume unpropped construction was used.

Step 3: Identify critical section(s) of the main supporting members and the critical effect(s) on them

Incorporating information from steps 2b and 2c, establish and run global grillage analysis to obtain the location of the maximum sagging bending moment in the composite beams under evaluation load, (eg 50MAX and HPMV) and bending moment at critical sections under critical vehicle loading closer to the support.

Step 3a: Determine the total number of shear connectors *n* between the locations of zero and maximum sagging moment in the bending moment envelope

For simply supported bridges, locations of zero moment are at supports. Note that under moving traffic loading, the location of the maximum bending moment of simply supported bridges may shift slightly from the mid-span. Use the lesser number of shear connectors between two segments either side of the location of the maximum bending moment.

For continuous beams, the lesser number of the shear connectors from both the location of zero moment (point of contraflexure) to the location of the maximum sagging moment should be used.

The location of the maximum sagging moment for both simply supported and continuous bridges may be determined from the envelope of bending moment. The corresponding contraflexture location in the continuous bridges should be determined from the bending moment distribution when the traffic load moves to where the maximum sagging moment in the envelope occurs.

Step 3b: Determine the degree of the shear connection η at maximum sagging moment location using equation 5.1 with the number of shear connectors obtained from step 3a if shear connectors are deemed to be ductile from step 2a.

Step 3c: Determine the minimum degree of shear connection $\eta_{\min}$ at maximum sagging moment location for shear connection ductility check if step 3b applies.

Use equations 5.2 and 5.3 to calculate the minimum degree of shear connection. The effective bridge span L_{ef} determined from step 2b may be used as the effective bridge span in the equations. Note that if η determined in step 3b is found to be lower than $\eta_{\min}$, ductile shear connection identified from step 2a is deemed non-ductile in choosing design equations in table 5.1.

Step 3d: Determine the total number of shear connectors *n* between the locations of zero and critical sections when the vehicle is closer to the supports

Repeat step 3a but calculate the total number of shear connectors between zero moment and critical sections.

Step 3e: Determine the degree of the shear connections η at critical sections using equation 5.1 with the number of shear connectors obtained from step 3d

Repeat steps 2b and 2c to derive the effective concrete area of the critical sections in order to use equation 5.1.

Step 4: Determine the overload capacity and/or the live load capacity at each critical main member section

Step 4a: Choose appropriate design curves

With the ductility of the shear connectors determined in step 2a, steel section classification determined in step 2d, degree of the shear connection obtained in step 3b, minimum degree of the shear connection determined $\eta_{\min}$ in step 3c and the construction methods identified in step 2e, the engineer can decide which assessment option in table 5.1 to pursue. If further assuming the steel section is compact (step 2d) and the construction method is unpropped (step 2e), the available options for assessing the composite bridge bending capacity are:

1 For ductile shear connectors with $\eta \geqslant \eta\text{min}$,

Curve ABC (equations 5.18 to 5.20)

Line AC (equation 5.21)

Line DEC (equations 5.25 to 5.26)

Line DE (equation 5.29)

Take equation 5.20 for example, it can be further elaborated as below:

$$\phi R_i = M_{Rd} = N_{pl,a}\frac{h_a}{2} + \eta N_{c,f}\left(h_t - \frac{\eta N_{c,f}}{N_{pl,c}}\frac{h_c}{2}\right) - \frac{\left(N_{pl,a} - \eta N_{c,f}\right)^2}{N_{pl,f}}\frac{t_f}{4} \qquad \text{(Equation 6.6)}$$

$$= \phi A_a f_y \frac{h_a}{2} + nP_{Rd}\left(h_t - \frac{nP_{Rd}}{\phi_c 0.85 f_c^{'} b_{eff} h_c}\frac{h_c}{2}\right) - \frac{\left(\phi A_a f_y h_a - nP_{Rd}\right)^2}{\phi b t_f f_y}\frac{t_f}{4}$$

2 For non-ductile shear connectors or $\eta < \eta\text{min}$

Line DEC (equations 5.25 to 5.26)

Line DE (equation 5.29)

Take equation 5.26 for example with further assumptions made as shown in figure 6.2, ie:

a The neutral axis of the composite section is located at the interface between the concrete slab and the steel section under the load increment after the construction stage.

b The design strength of concrete $f_{cd} = \phi_c 0.85 f_c'$ at the outermost compressive fibre is reached before the design strength of steel section $f_{yd} = \phi f_y$ at the outermost tensile fibre of the composite section is obtained.

Equation 5.26 can now be further elaborated as below:

$$\phi R_i = M_{el,Rd} + \left(M_{pl,Rd} - M_{el,Rd}\right)\frac{N_c - N_{c,el}}{N_{c,f} - N_{c,el}} \qquad \text{(Equation 6.7)}$$

$$= (Z_e f_s + \phi_c Z_{e,co} 0.85 f_c^{'}) + \left(M_{pl,Rd} - (Z_e f_s + \phi_c Z_{e,co} 0.85 f_c^{'})\right)\frac{nP_{Rd} - \phi_c \dfrac{0.85 f_c^{'} h_t b_{eff}}{2}}{\min(\phi A_d f_y, \phi_c 0.85 f_c^{'} h_t b_{eff}) - \phi_c \dfrac{0.85 f_c^{'} h_t b_{eff}}{2}}$$

Where:

$M_{el,Rd} = M_{a,Ed} + kM_{c,Ed}$ is from figure 6.2

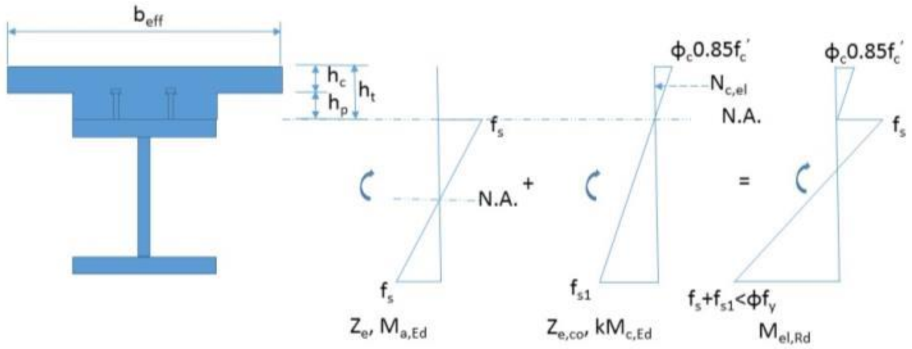
Z_e is the steel elastic section modulus

f_s is the stress at extreme fibre of the steel section under construction load

$Z_{e,co}$ is the elastic section modulus of the transformed composite section by taking into account the short term Young's modulus ratio between concrete and steel

$M_{pl,Rd}$ shall be obtained from equation 6.6 by substituting $\min(\phi A_d f_d, \phi_c 0.85 f_c^{'} h_t b_{eff})$ for nP_{Rd}, (ie bending capacity with full shear connection $\eta = 1.0$).

Figure 6.2 Non-linear method for non-ductile shear connector or $\eta < \eta_{min}$



To determine the design capacity, strength reduction factors have to be properly assigned to each material component in the design equations, where the strength reduction factors $\varphi=0.9$, $\varphi_C=0.6$, $\varphi_V=0.85$ for steel, concrete and shear connectors respectively as per table 4.2 of AS 5100.6: 2004. P_{Rd} is obtained from equation 6.1 for channel shear connectors.

Step 4b: Determine load capacity at critical sections

Use the section properties such as Z_e, $M_{pl,Rd}$, $N_{c,f}$ of each individual critical section in determining the section capacity when using the equations identified in step 4a. In particular, degree of shear connection η obtained from step 3b and step 3d should be used respectively for the sections at maximum bending moment and critical sections under vehicle loading closer to supports.

Steps 5 to 11

No adaptations are made to these steps in the *Bridge manual* and are therefore omitted here.

When assessing the composite bridges, the transverse reinforcement should also be assessed to prevent longitudinal shear failure of the concrete from the concentrated shear connector forces. In all cases, sufficient transverse reinforcement is required to ensure that the longitudinal shear resistance in the slab is greater than the force in the concrete compressive flange from the shear connectors. Cl.6.6.5 of AS 5110.6:2004 may be referred to as rules for the assessment.

6.3 Conclusions

The amended evaluation principles for the bending capacity of the composite bridges for the current *Bridge manual* provide bridge engineers with a wider range of assessment options, among which, as supplementary to NZS 3404, AS 2327.1 and AS 5100.6 in particular, are the non-linear and elastic methods applied to any degree of shear connection. These options are derived from the latest developments in the Australasian national standards including AS/NZS 2327 and AS/NZS 5100.6.

The economic significance of applying these principles is that the level of the repairing strategy relies entirely on the rating of the bridges, which is strongly influenced by the assessment options being chosen due to their inherent difference in conservativeness among these options. Both the transportation authority and consultants will therefore benefit from having more solutions to strike a balance with limitation of budgets.

The reliability analysis for the shear connectors in chapter 5 assumes material and geometric dimensions are time independent. Of the most concern in the bridge assessment is how certain safety margins can be ensured for the remaining service life of the structures under conditions of deteriorating material properties and damages observed in the assessed structures and whether the rating properly represents these conditions. The current *Bridge manual* has addressed this issue by allowing further reduction factors to the strength reduction factors. This is shown in table 7.5 of the manual. At this stage, it is still advised to use table 7.5 to adjust strength reduction factor according to the level of the damage to the channel shear connectors.

A sensitivity study for the comparison among the predictions as per the proposed assessment procedure, AS5100.6 and NZS3404 is provided in the worked example (see appendices E and F). It is clear that as the degree of shear connectivity is reduced, the difference of bending capacities between NZS3404 or AS5100.6 and the proposed assessment procedure increases. The primary reason for this is that, as the degree of shear connectivity reduces, the connection behaves in a more brittle manner. As a result, the distribution of stresses in the beam section (steel beam and concrete slab) will not be able to become plastic, but instead will be elastic. Therefore the proposed assessment procedure assumes an elastic distribution of stresses when determining the beam bending resistance. This which yields a lower value than NZS3404 and AS5100.6, which assume a plastic distribution regardless of degree of shear connectivity.

However, it should be noted that the proposed assessment procedure was developed to assist with determining the composite bending resistance only. It is therefore possible that, depending on the degree of shear connectivity, relative dimensions of the concrete slab and steel beam section, the bending resistance of the bare steel beam may be greater than the composite bending resistance. Therefore, in such scenarios, it is important for users of the proposed assessment procedure to determine the bare steel beam bending resistance, giving attention to the degree of lateral support provided by the concrete slab and/or intermediate bracing between adjacent bridge beams. Users should also consider the effectiveness of shear connectors which have yielded in providing lateral restraint to the top flange of the steel section, paying attention to the locations where they may have yielded (eg near support for simply supported beams).

7 Conclusions

The bridge assessment method for composite bridges in the *Bridge manual* referred initially to NZS 3404 and subsequently to AS 5100.6. The design equations in both NZS 3404 and AS 5100.6 are outdated with inadequate focus on the influence of the ductility and degree of shear connection on the beam resistance. The corresponding design equations for channel shear connectors, which this study has shown to be the major type of shear connectors in New Zealand's historic bridges, have not been calibrated to keep pace with recent international tests in terms of a safety margin.

This research project identified the types of shear connectors in New Zealand historic bridges from a substantial database study. The decision was then taken to focus on welded channel shear connectors, which are used in more than 67% of New Zealand bridges.

A new design equation for the resistance of welded channel shear connectors has been developed as part of this research. The equation retains the form of that in NZS 3404. The capacity reduction factor, however, has been re-evaluated through a comprehensive reliability analysis based on EN 1990 appendix D, against 84 international experiments explored by this research and implementing local material properties for the Australasian region. The analysis shows that the design resistance should be lower than the existing equations. A method to identify the ductility of the welded channel shear connectors has also been developed.

The report introduces a new design method to evaluate beam bending capacity which incorporates the newly proposed design equation for the channel shear connectors. The Eurocode-based design method is to be accepted in the forthcoming AS/NZS 2327 which will be a general solution in New Zealand for composite structures. While the existing NZS 3404 and AS 5100.6 solution assumes ductile shear connection and adopts rigid plastic theory only, the new method considers multiple design options depending on the degree of shear connection, ductility of the shear connection, steel section compactness, minimum degree of shear connection and construction method, ie propped and unpropped construction methods.

A revised evaluation procedure, adapted from the *Bridge manual*, has been developed incorporating the new design method. A worked example for an existing historic bridge located in New Zealand shows that NZS 3404 and AS 5100.6 overestimate the bridge capacity, the level of which increases with the reduced degree of shear connection.

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Appendix A: Canterbury and West Coast case study information

This is located at www.nzta.govt.nz/resources/research/reports/602

Appendix B: Opus database case study information

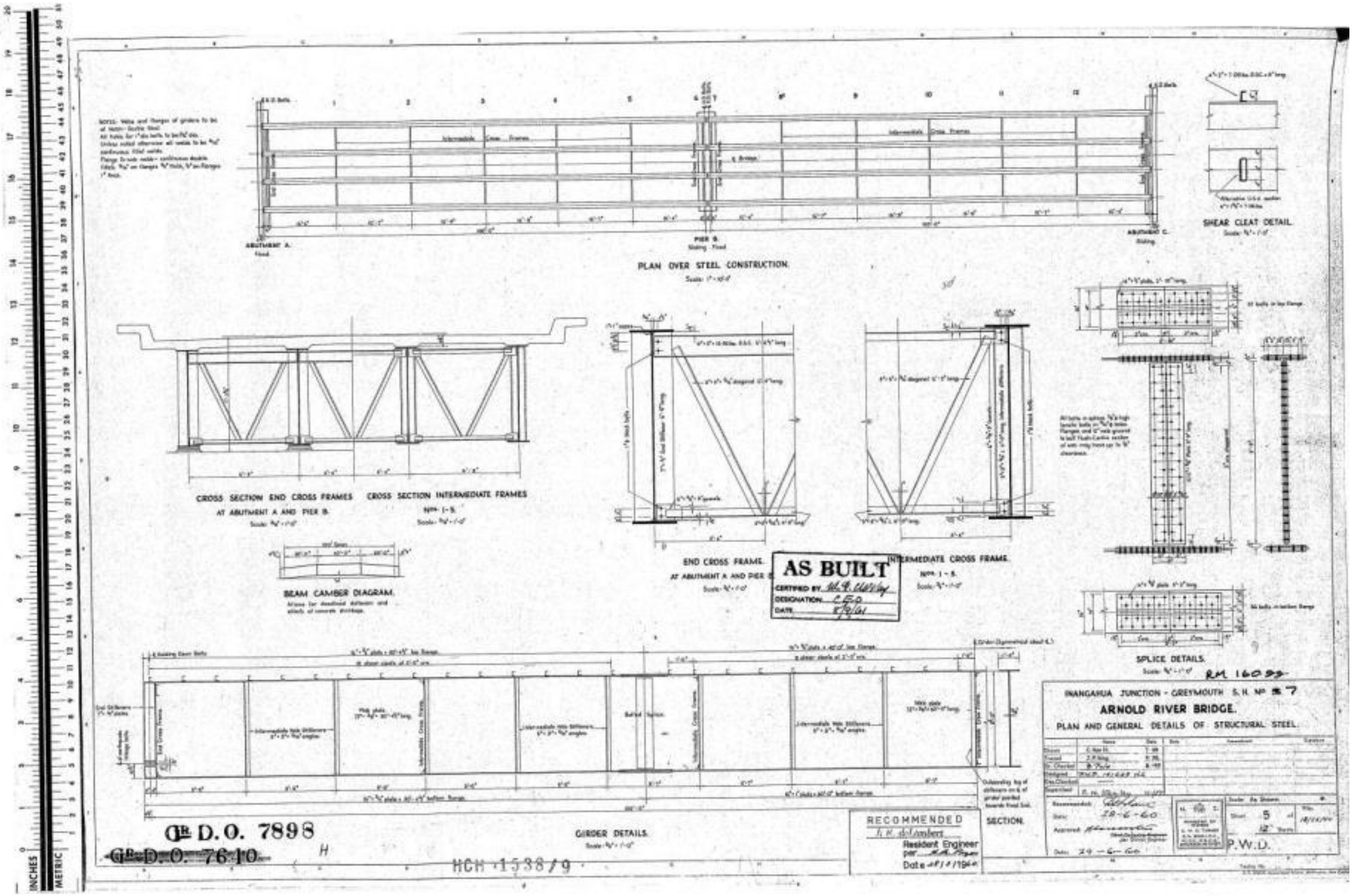
Bridge name	Location	Year of design	Beams/deck	Shear cleat/stud type	Designer	Ref
Bairds Road Overbridge	SH1 South Auckland	1953	27*10*102 RSJ with cast in situ deck	Shear cleats 4*2*7*6inches wide channel	Public Works Department	ADO26766
East Tamaki Overbridge	SH1 South Auckland	1953	27*10*102 RSJ with cast in situ deck	Shear cleats 2*4*7 channel, 6in wide	Public Works Department	ADO24608
Puhinui Road Overbridge	SH1 South Auckland	1954	27*10*102 RSJ with cast in situ deck	Shear cleats 2*4*7 channel, 6in wide	Public Works Department	ADO26768-5
Princes Street Overbridge	SH1 South Auckland	1954 & 1997 for repair	27*10*102 RSJ with cast in situ deck	Shear cleats 2*4*7 channel, 6in wide alt 3.5*3.5*3/8 angle & 102*51*10.4kg channel	Public Works Department	ADO24608 & 1/28/153/7104
Oparau Bridge Kawhia	Okupata-Omanawa Road Kawhia	1955	22*7*75 steel RSJ with cast in situ concrete deck	3.5in*3.5in*3/8 inch steel angle sections	Babbage and Shores	HDO 7135
Hill Road Underpass	SH1 South Auckland	1960	24*7.5*85 RSJ	3/4in Dia*4in shear studs	Ministry of Works	ADO29480
Te Reinga Bridge	Wairoa	1974	68in steel girder	6*3*12 channel*14in wide	Malcolm Sweet Parker and Holland	3/135/1/7924
Waipoua River Bridge	SH 12 RD 1 RS 89	1979	914*308*289UB Grade 50C	152*76*18 *150 long channels as shear cleats	Ministry of Works and Development	1/47/3/7104
Orewa River Bridge	SH 1 RS 288 Orewa to Silverdale	1998	Grade 250 steel Plate with precast concrete deck	19mm Dia studs, 200mm high	Beca Carter Hollings and Ferner	1/23/115/7914/211
Arapuni Headrace Bridge	Arapuni Road, South Waikato	2002	Grade 250 steel plate with precast deck units	22mm Dia studs, 150mm high	Opus	2/240/30/7204/1
Parker Lane bridge deck replacement	Buckland Road, Franklin	2009	530UB32	19mm Dia *100mm shear stud	Opus	1/1110/203/6104
Hamiltons Bridge widening	Bucklands Road, Franklin	2010	700WB30 steel section with cast in place deck	19mm Dia atuds 100mm high	Opus	1/1042/269/7104
Kaituna River Bridge	Tauranga Eastern Link Motorway	2011	Steel transom with precast concrete deck	22mm Dia studs 180mm high	URS/Opus	S-34-110

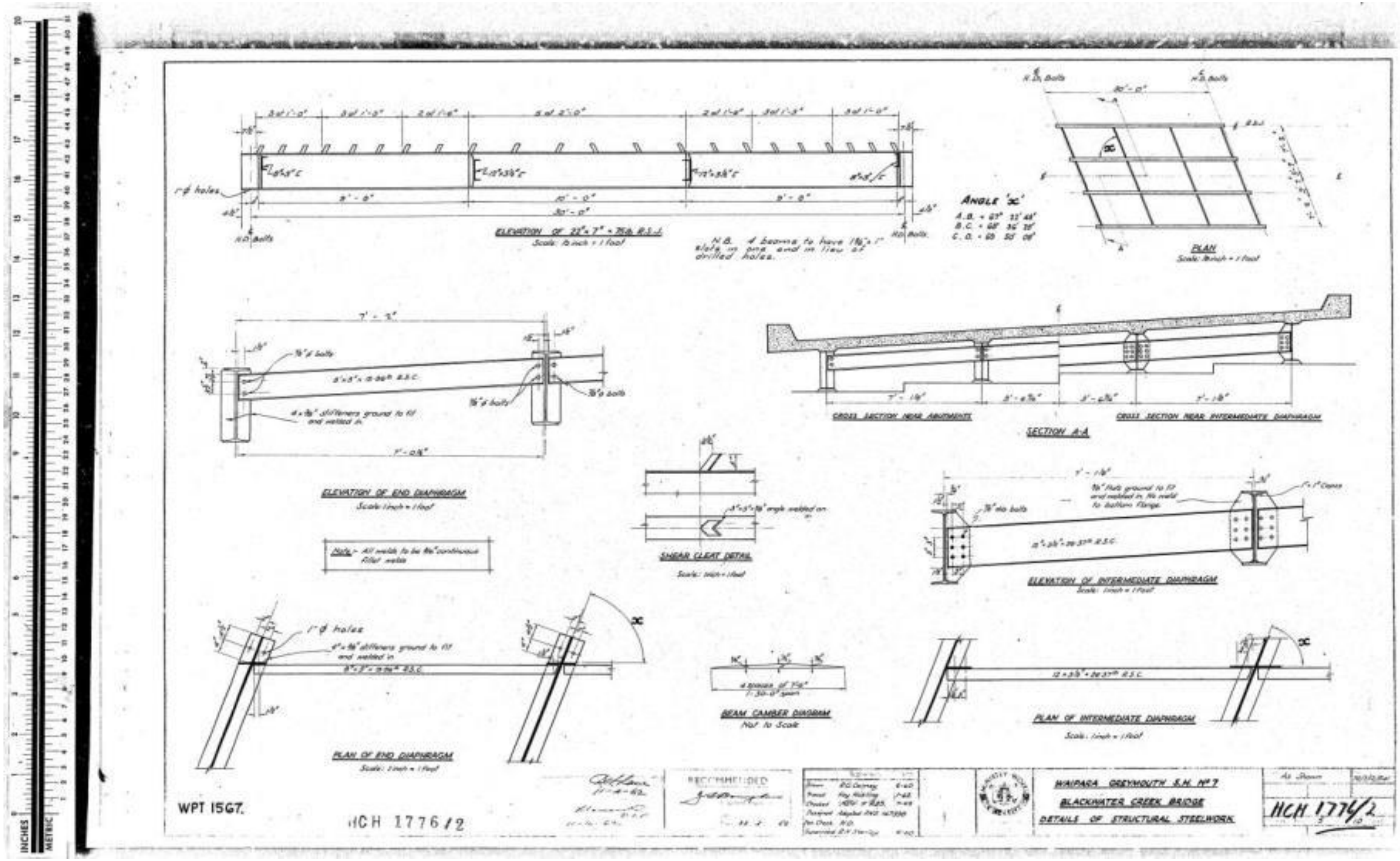
* Sourced from the Opus online database

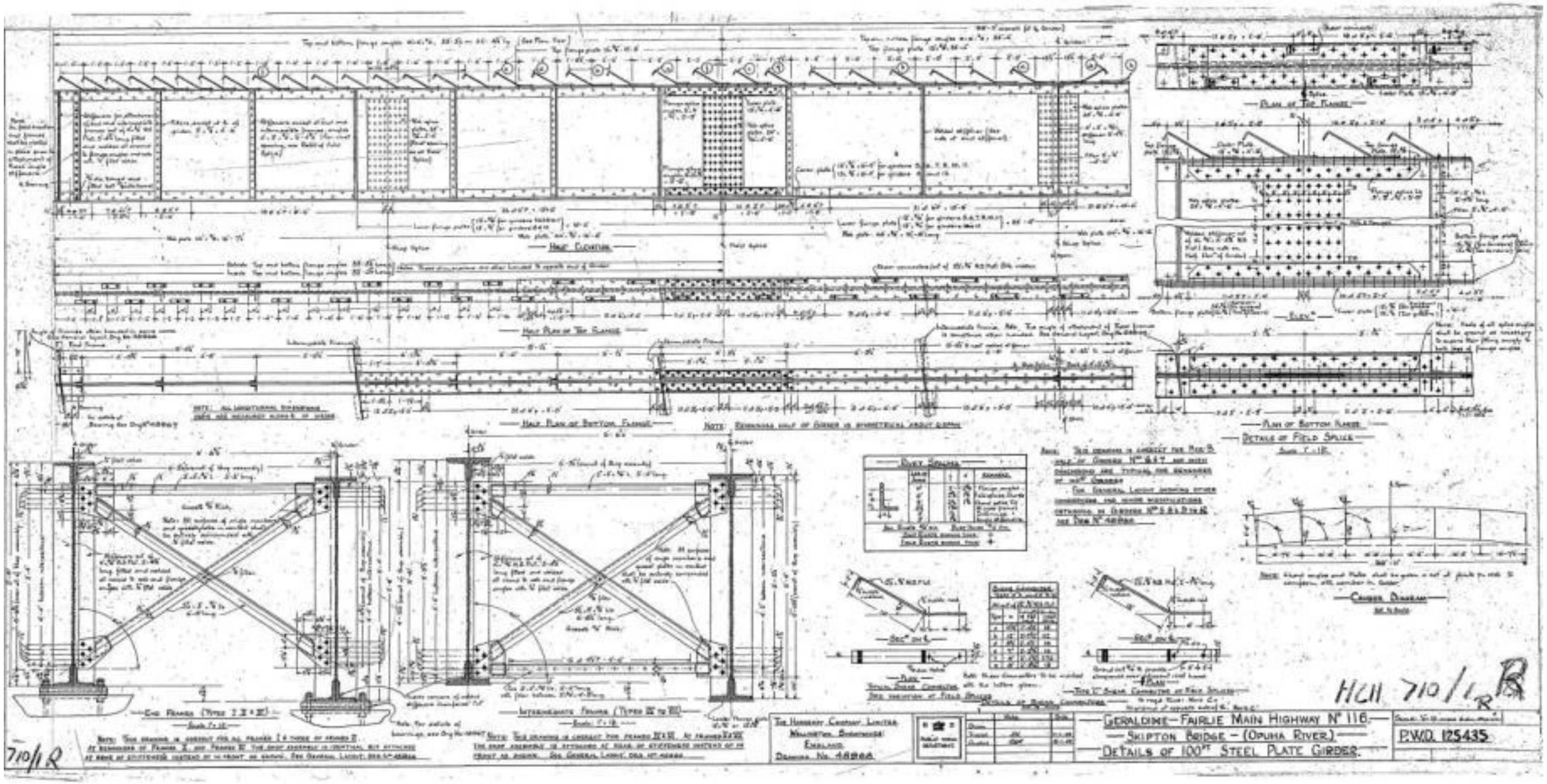
Appendix C: Gisborne and Hawke's Bay case study information

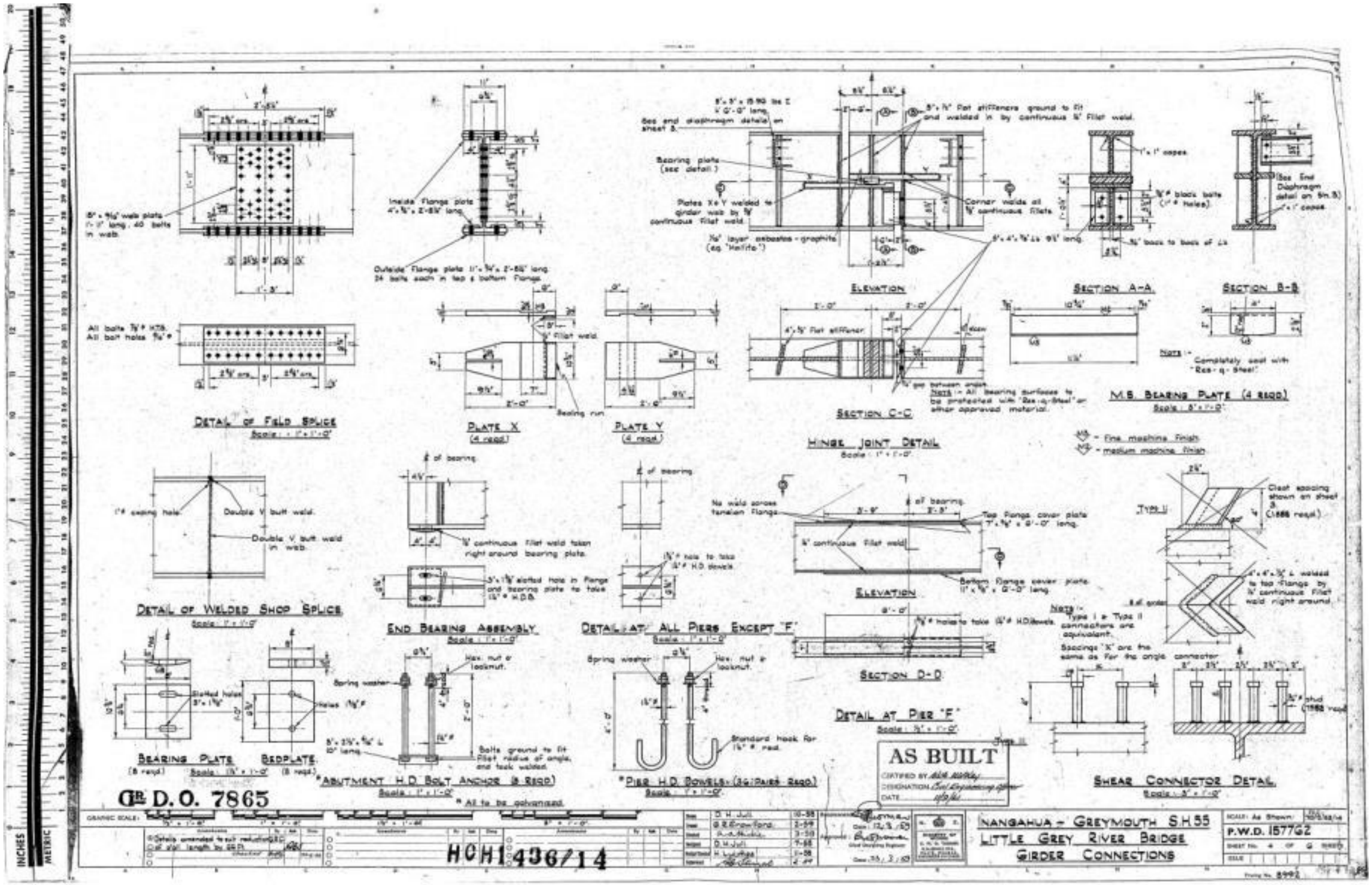
Structure ID	BSN	SH	RS	RP	Name	Year Constructed	Design Loading	Beam Type	Style	Comment
32582	6671	2	661	6.06	KARAMU CREEK BRIDGE	1929	traction engine	I beams	Rivetered Angles	
33593	2547	5	249	5.88	MUNNS BRIDGE	1945	H20_S16	I beams	Welded Channels	
33221	1320	35	132	0.02	OWEKA RIVER BRIDGE	1952	H20_S16	I beams	Welded V Angles	
33275	2630	35	263	0	MANGATUNA BRIDGE	1952	H20_S16	I beams	Welded V Angles	
32507	4087	2	406	2.69	HOLDSWORTH BRIDGE	1953	H20_S16	I beams	Welded V Angles	
33223	1383	35	132	6.25	MANGAOMEKA STREAM BRIDGE	1953	H20_S16_T16	I beams	Welded V Angles	
33266	2483	35	238	10.32	WAIPUTAPUTA STREAM BRIDGE	1953	H20_S16	I beams	Welded V Angles	
32581	6613	2	661	0.25	CLIVE RIVER BRIDGE	1954	H20_S16	I beams	Welded V Angles	
33224	1426	35	132	10.55	WAITAUKAKARI BRIDGE	1954	H20_S16	I beams	Welded V Angles	
32515	4475	2	443	4.5	WHATATUNA BRIDGE	1955	H20_S16	I beams	Welded Channels	
32493	3750	2	375	0	MORTLEMANS BRIDGE	1956	H20_S16	I beams	Welded Channels	
32545	5317	2	516	15.75	AWATERE NO 3 BRIDGE	1956	H20_S16	I beams	Welded Channels	
33331	1946	38	189	5.6	AWATERE STREAM BRIDGE No.2	1956	H20_S16	I beams	Welded Channels	
33332	1952	38	189	6.2	AWATERE BRIDGE NO. 1	1956	H20_S16	I beams	Welded Channels	
32514	4456	2	443	2.61	WAIPAOA RIVER BRIDGE	1957	H20_S16	Plate girders	Welded Channels	
32554	5676	2	562	5.62	MANGATURANGA BRIDGE	1957	H20_S16	I beams	Other	Welded Channels and Welded V Angles
33232	1622	35	159	3.17	TAURANGAKOAU BRIDGE	1957	H20_S16	I beams	Welded Channels	
33239	1910	35	190	0.98	PAOARUKU STREAM BRIDGE	1957	H20_S16	I beams	Welded Channels	
33580	1921	5	190	2.07	RUATITI BRIDGE (WAIPUNGA)	1957	H20_S16	Plate girders	Welded Channels	
32588	7138	2	707	6.82	WAIPAWA RIVER BRIDGE	1958	H20_S16	Plate girders	Welded Channels	
33246	2092	35	200	9.19	KOPUAROA NO 4 BRIDGE	1958	HN_HO_72	I beams	Welded V Angles	
33251	2205	35	213	7.47	TAKAPAU STREAM BRIDGE	1958	H20_S16	Plate girders	Welded Channels	
32501	3948	2	390	4.8	WAIHUKA No.2 BRIDGE	1959	H20_S16	Plate girders	Welded Channels	
32516	4488	2	443	5.75	TE ARAI RIVER BRIDGE	1959	H20_S16	Plate girders	Welded Channels	
32518	4560	2	443	13	CALCOTTS BRIDGE	1959	H20_S16	Plate girders	Welded Channels	
32539	5160	2	516	0	WAIKATUKU BRIDGE	1959	H20_S16	I beams	Welded V Angles	
33245	2052	35	200	5.23	MAKATOTE BRIDGE	1959	HN_HO_72	I beams	Welded Channels	
33267	2500	35	250	0	HIKUWAI BRIDGE NO 4	1959	HN_HO_72	Plate girders	Welded Channels	
32533	5008	2	497	3.76	OMANA STREAM BRIDGE	1961	H20_S16	I beams	Welded V Angles	
33272	2561	35	250	6.07	HIKUWAI BRIDGE NO 2	1961	HN_HO_72	Plate girders	Welded Channels	
33273	2563	35	250	6.25	HIKUWAI BRIDGE NO 1	1961	HN_HO_72	Plate girders	Welded Channels	
33584	2040	5	204	0	WAIONE BRIDGE	1961	H20_S16	Plate girders	Welded Channels	
33622	857	50	79	6.73	MANGATEWAI STREAM BRIDGE	1961	H20_S16	I beams	Welded V Angles	
32498	3900	2	390	0	PARIHOHONU BRIDGE	1962	H20_S16	Plate girders	Welded Channels	
33271	2556	35	250	5.57	HIKUWAI BRIDGE NO 3	1962	HN_HO_72	Plate girders	Welded Channels	
33609	577	50	49	8.68	MANGAMATE STREAM BRIDGE (GWAVAS)	1962	H20_S16	I beams	Welded V Angles	
32496	3858	2	375	10.75	ANZAC BRIDGE	1963	H20_S16	Plate girders	Welded Channels	
33621	831	50	79	4.1	TUKIPO STREAM BRIDGE	1964	H20_S16_T16	I beams	Welded Channels	
33290	3000	35	300	0	WAIOMOKO RIVER BRIDGE	1965	H20_S16_T16	Plate girders	Welded Channels	
33328	1879	38	179	8.91	FRASERTOWN BRIDGE	1970	H20_S16_T16	Plate girders	Other	Welded UB Half
32567	6054	2	592	13.36	SANDY CREEK BRIDGE	1980	HN_HO_72	I beams	Welded Channels	
33326	1802	38	179	1.15	DAVIES BRIDGE	1982	HN_HO_72	I beams	Unsure	
32562	5938	2	592	1.81	KINGS CREEK BRIDGE	1983	HN_HO_72	I beams	Welded Channels	
33250	2130	35	213	0	KOPUAROA No.1 STREAM BRIDGE	1987	HN_HO_72	I beams	Welded V Angles	

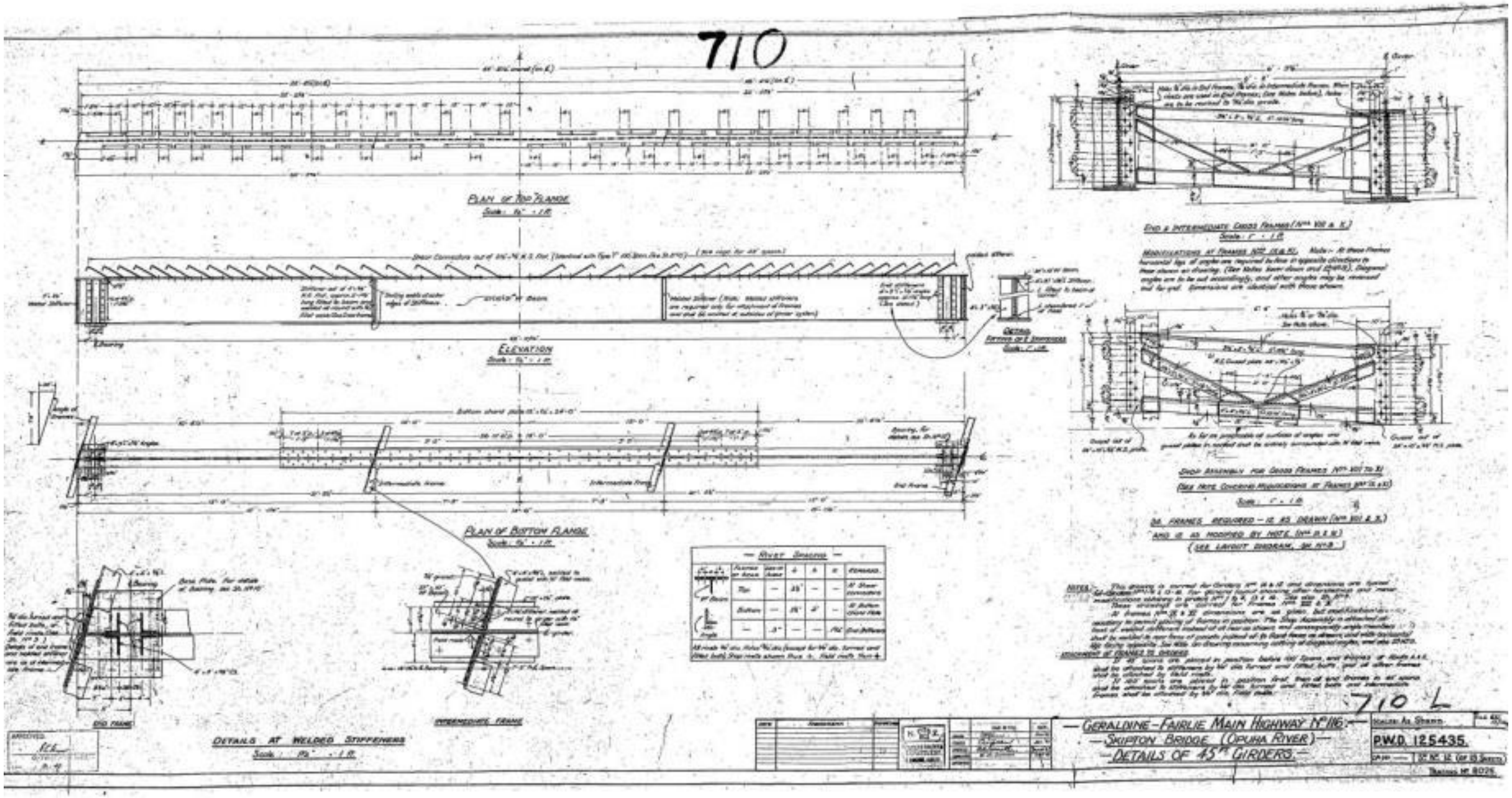
Appendix D: Drawings of composite bridge examples

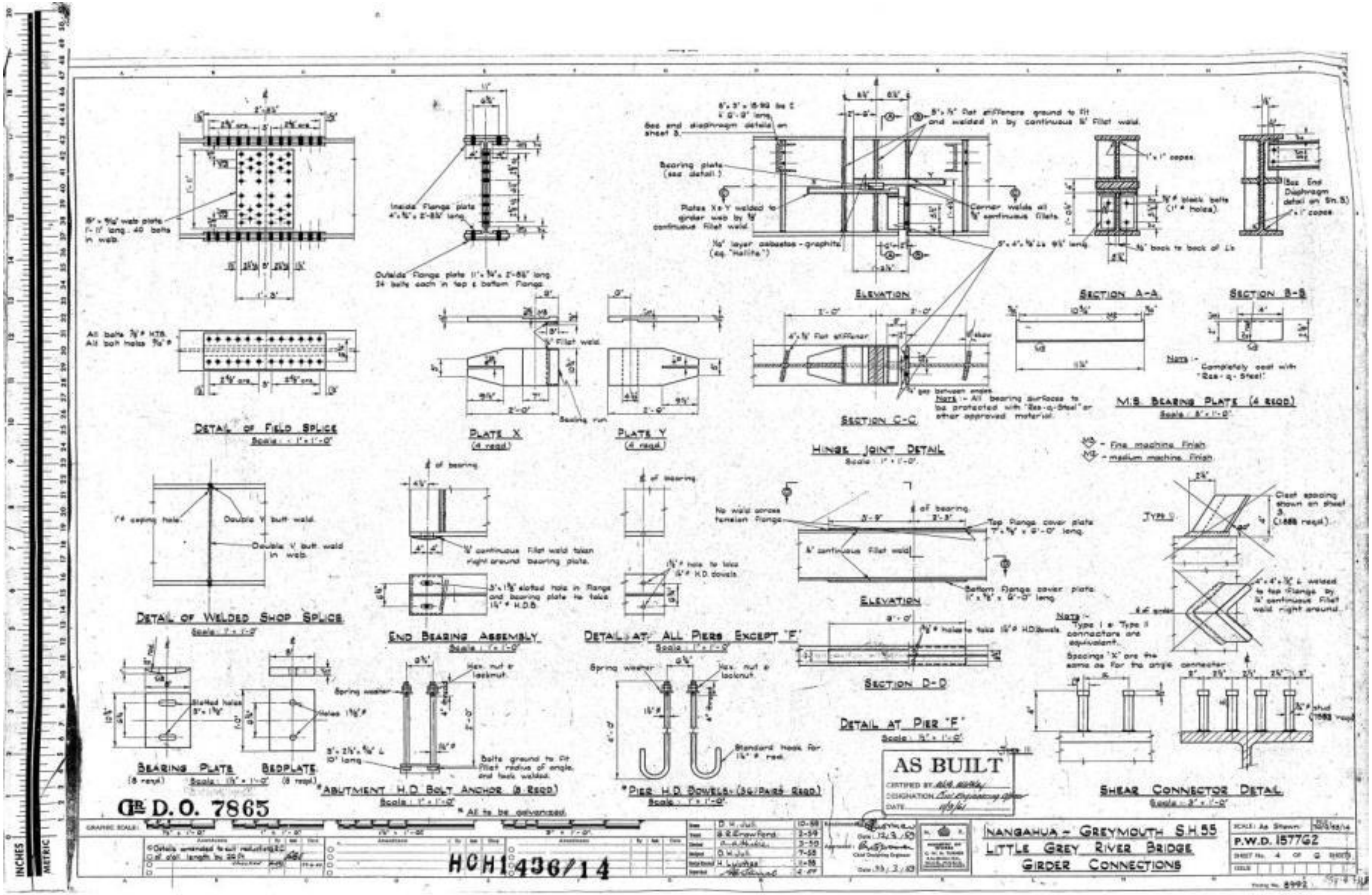


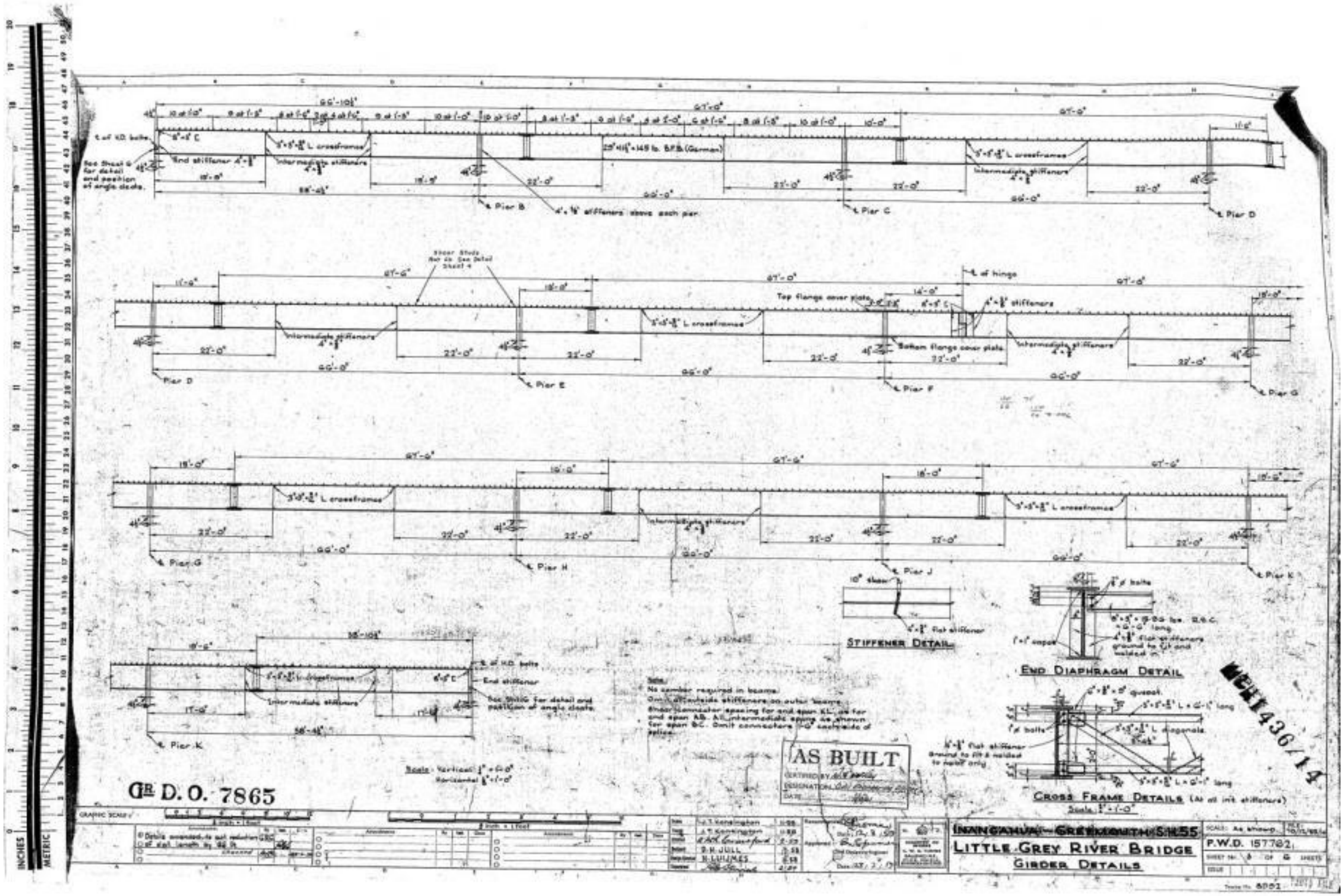

ELEVATION OF 22" x 7" x 75lb R.S.J.
PLAN
ANGLE 'x'
SECTION A-A
CROSS SECTION NEAR ABUTMENTS
CROSS SECTION NEAR INTERMEDIATE DIAPHRAGM
ELEVATION OF END DIAPHRAGM
SHEAR CLEAT DETAIL
ELEVATION OF INTERMEDIATE DIAPHRAGM
PLAN OF END DIAPHRAGM
BEAM CAMBER DIAGRAM
Not to Scale
PLAN OF INTERMEDIATE DIAPHRAGM
RECOMMENDED
WAIPARA GREYMOUTH S.H. No 7
BLACKWATER CREEK BRIDGE
DETAILS OF STRUCTURAL STEELWORK
WPT 15G7.
HCH 1776/2

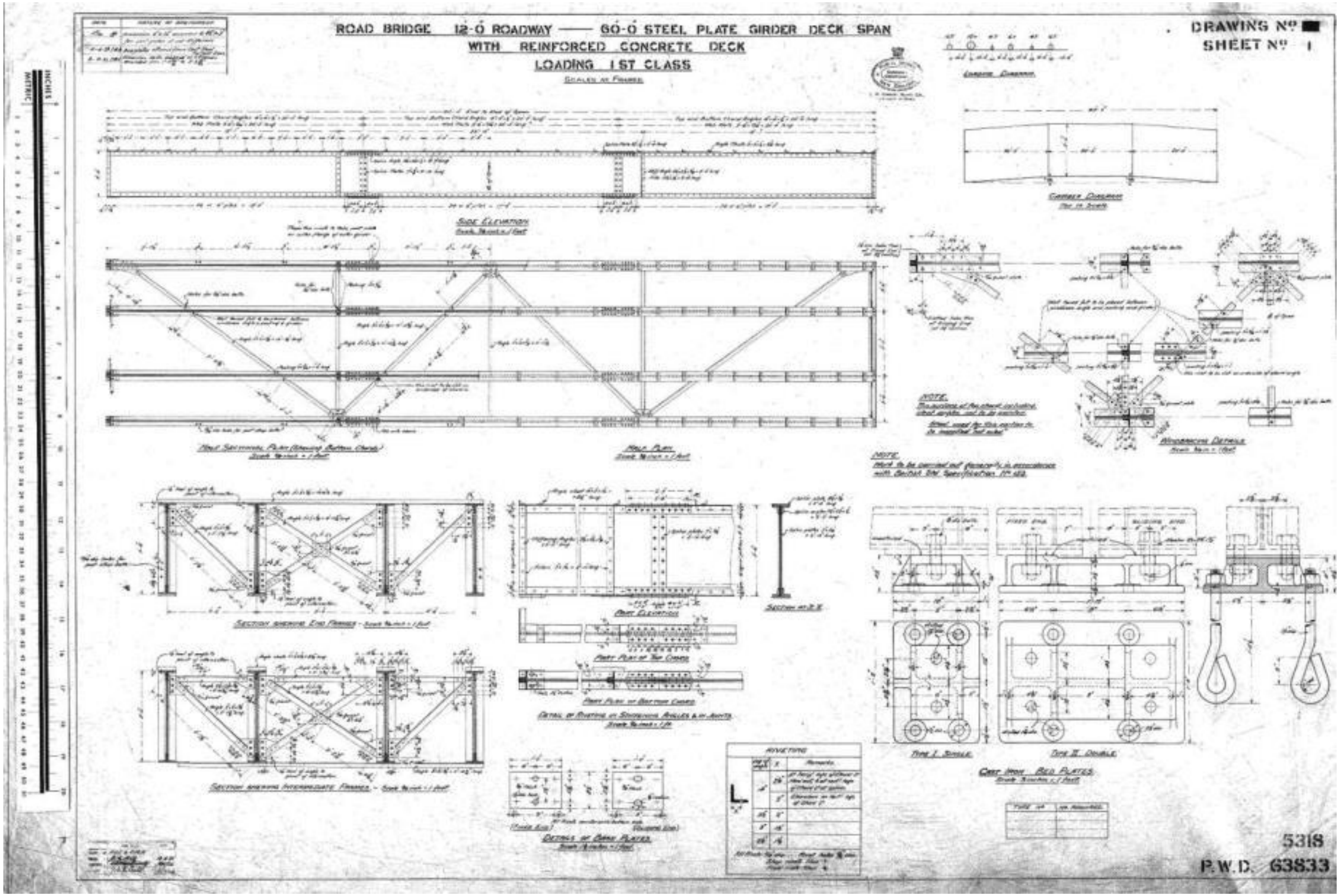

GERALDINE–FAIRLIE MAIN HIGHWAY Nº 116
SKIPTON BRIDGE – (OPUHA RIVER)
DETAILS OF 100FT STEEL PLATE GIRDER
P.W.D. 125435
HALF ELEVATION
HALF PLAN OF TOP FLANGE
HALF PLAN OF BOTTOM FLANGE
PLAN OF TOP FLANGE
PLAN OF BOTTOM FLANGE
DETAILS OF FIELD SPLICE
CAMBER DIAGRAM
END FRAMES
INTERMEDIATE FRAMES
RIVET SPACING
710/1R

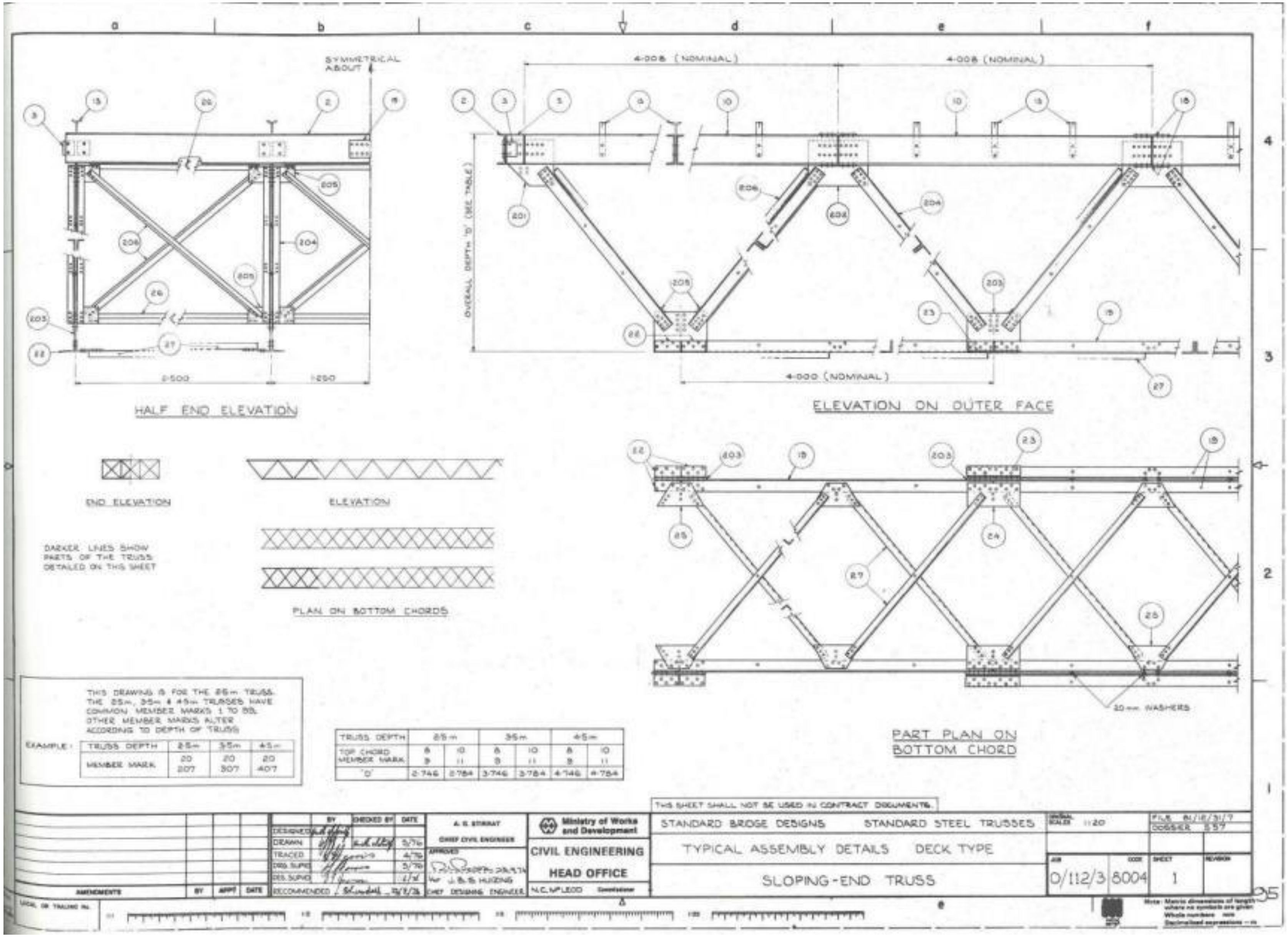

DETAIL OF FIELD SPLICE
PLATE X
PLATE Y
ELEVATION
SECTION A-A
SECTION B-B
SECTION C-C
HINGE JOINT DETAIL
M.S. BEARING PLATE (4 REQD)
DETAIL OF WELDED SHOP SPLICE
END BEARING ASSEMBLY
DETAIL AT ALL PIERS EXCEPT "F"
SECTION D-D
DETAIL AT PIER "F"
BEARING PLATE
BEDPLATE
ABUTMENT H.D. BOLT ANCHOR
PIER H.D. DOWELS
SHEAR CONNECTOR DETAIL
AS BUILT
GR D.O. 7865
HCH1436/14
INANGAHUA - GREYMOUTH S.H.55
LITTLE GREY RIVER BRIDGE
GIRDER CONNECTIONS
P.W.D. 157762

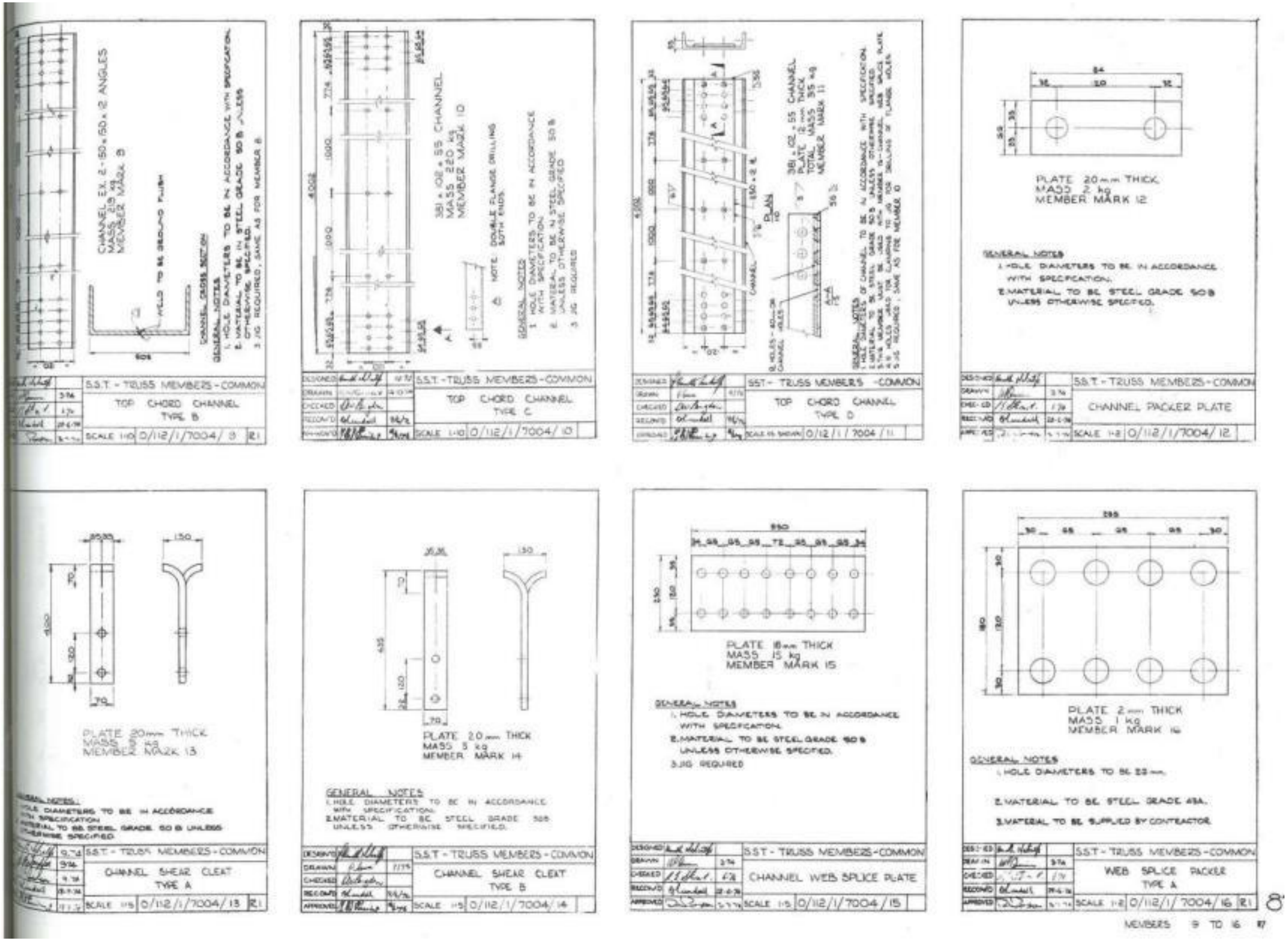

710
PLAN OF TOP FLANGE
ELEVATION
PLAN OF BOTTOM FLANGE
RIVET SPACING
DETAILS AT WELDED STIFFENERS
END FRAME
INTERMEDIATE FRAME
710 L
GERALDINE-FAIRLIE MAIN HIGHWAY Nº 116
SKIPTON BRIDGE (OPUHA RIVER)
DETAILS OF 45FT GIRDERS
P.W.D. 125435


DETAIL OF FIELD SPLICE
PLATE X
PLATE Y
ELEVATION
SECTION A-A
SECTION B-B
SECTION C-C
HINGE JOINT DETAIL
M.S. BEARING PLATE (4 REQD)
DETAIL OF WELDED SHOP SPLICE
END BEARING ASSEMBLY
DETAIL AT ALL PIERS EXCEPT 'F'
SECTION D-D
DETAIL AT PIER 'F'
BEARING PLATE
BEDPLATE
ABUTMENT H.D. BOLT ANCHOR
PIER H.D. DOWELS
SHEAR CONNECTOR DETAIL
AS BUILT
D.O. 7865
HCH1 436/14
NANGAHUA - GREYMOUTH S.H.55
LITTLE GREY RIVER BRIDGE
GIRDER CONNECTIONS
P.W.D. 157762


Pier B
Pier C
Pier D
Pier E
Pier F
Pier G
Pier H
Pier J
Pier K
STIFFENER DETAIL
END DIAPHRAGM DETAIL
CROSS FRAME DETAILS
AS BUILT
D.O. 7865
LITTLE GREY RIVER BRIDGE
GIRDER DETAILS
P.W.D. 157782


ROAD BRIDGE 12-0 ROADWAY —— 60-0 STEEL PLATE GIRDER DECK SPAN
WITH REINFORCED CONCRETE DECK
LOADING 1ST CLASS
DRAWINGS Nº [redacted]
SHEET Nº 1
METRIC
INCHES
5318
P.W.D. 63833


HALF END ELEVATION
ELEVATION ON OUTER FACE
END ELEVATION
ELEVATION
PLAN ON BOTTOM CHORDS
PART PLAN ON BOTTOM CHORD
DARKER LINES SHOW PARTS OF THE TRUSS DETAILED ON THIS SHEET
SYMMETRICAL ABOUT
4·008 (NOMINAL)
4·000 (NOMINAL)
2·500
1·250
20mm WASHERS
THIS SHEET SHALL NOT BE USED IN CONTRACT DOCUMENTS.
STANDARD BRIDGE DESIGNS
STANDARD STEEL TRUSSES
TYPICAL ASSEMBLY DETAILS DECK TYPE
SLOPING-END TRUSS
Ministry of Works and Development
CIVIL ENGINEERING
HEAD OFFICE
0/112/3 8004
1
1:20


S.S.T - TRUSS MEMBERS - COMMON
TOP CHORD CHANNEL TYPE B
0/112/1/7004/9
381 x 102 x 55 CHANNEL
MASS 220 kg
MEMBER MARK 10
TOP CHORD CHANNEL TYPE C
0/112/1/7004/10
TOP CHORD CHANNEL TYPE D
0/112/1/7004/11
PLATE 20mm THICK
MASS 2 kg
MEMBER MARK 12
CHANNEL PACKER PLATE
0/112/1/7004/12
PLATE 20mm THICK
MASS 5 kg
MEMBER MARK 13
CHANNEL SHEAR CLEAT TYPE A
0/112/1/7004/13
PLATE 20mm THICK
MASS 5 kg
MEMBER MARK 14
CHANNEL SHEAR CLEAT TYPE B
0/112/1/7004/14
PLATE 16mm THICK
MASS 15 kg
MEMBER MARK 15
CHANNEL WEB SPLICE PLATE
0/112/1/7004/15
PLATE 2mm THICK
MASS 1 kg
MEMBER MARK 16
WEB SPLICE PACKER TYPE A
0/112/1/7004/16
MEMBERS 9 TO 16

Appendix E: Worked example using the existing design provisions

STEEL BRIDGE 17-19 Cladding Place, Manukau City, 2104 Telephone: +64 (09) 262 2885 Fax: +64 (09) 262 2856 CALCULATION SHEET	Job No: 1-C1146.00	Sheet: 1 of 14	Rev: 04
	Job: TAR 14/07 Evaluation of shear connectors in composite bridges		
	Subject: Assessment example		
	Client: HERA	Made by: Mehedi Chowdhury	Date: 16/02/16
		Checked by: Dejan Novakov	Date: 17/02/16

This comparative assessment uses NZS 3404 to determine the sagging bending moment at the midspan of a 23m steel composite girder of Waipoua River Bridge.

STEEL BRIDGE
17-19 Cadding Place, Manukau City, 2104
Telephone +64 (09) 262 2885
Fax +64 (09) 262 2856
CALCULATION SHEET

Job No: 1-C1146.00	Sheet: 2 of 14	Rev: 04
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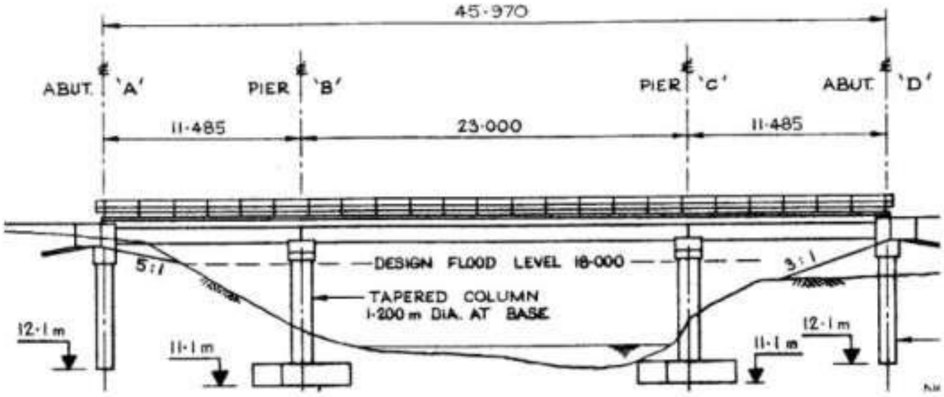
	Checked by: Dejan Novakov	Date: 17/02/16

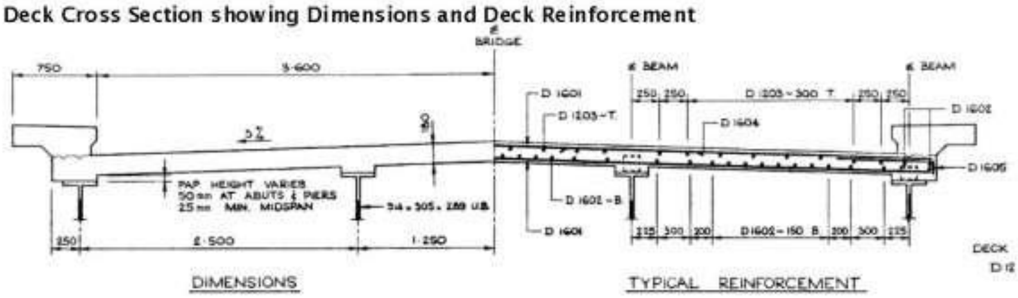
Waipoua River Bridge was designed in 1979 by the Ministry of Works and is located on State Highway 12 in Whangarei. It has three simply supported spans, with span arrangement of 11.485m + 23m + 11.485m. The bridge is on a 25 degree skew. The bridge superstructure consists of a 180mm thick insitu reinforced concrete slab supported by four structural steel girders at 2.5m c/c spacing. The main (middle) span girders are 914x305x289 UB with 152x76x18 - 150 long channel shear connectors with varying spacing (350mm over 6m from the supports and 500mm in the middle section of the beams).

The bridge elevation, cross section and main span girder elevation are shown below.

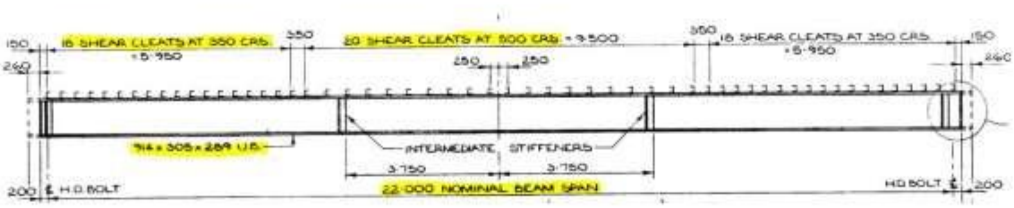
Bridge Elevation



Deck Cross Section showing Dimensions and Deck Reinforcement



Girder Elevation showing Shear Connectors



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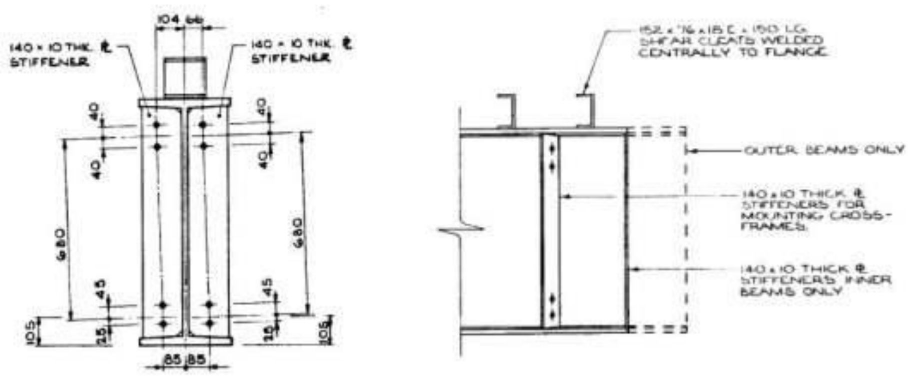
Girder Cross Section and Shear connector



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Example 1: BASED ON AS-BUILT NUMBER OF SHEAR CONNECTORS

As a comparison, the beam bending capacity will now be determined using NZS3404 Section 13...

First checking whether effective concrete slab width is same as before...

Effective slab width $b_{ec} := \min(0.25 \cdot L_{ef}, 2.5m) = 2500 \cdot mm$ Cl 13.4.2.1.1

So effective slab width is unchanged. Section properties therefore unchanged

Using Clause 13.4.5.2 the bending capacity of the composite section can be determined. However first need to determine whether shear connection provides full or partial composite action.

Area of steel section $A_s = 36514 \cdot mm^2$

Yield strength of steel section $f_y = 345 \cdot MPa$

Maximum tensile force in steel beam (when plastic) $F_{beam} = 12597 \cdot kN$

Compressive strength of concrete slab $f_c = 25 \cdot MPa$

Concrete slab thickness (from before) $t_{slab} = 180 \cdot mm$

Maximum compressive force in concrete slab $F_{slab} := 0.85 \cdot f_c \cdot b_{ec} \cdot t_{slab} = 9563 \cdot kN$

Strength reduction factor for shear connector $\phi_{sc} := 1.0$ Table 13.1.2(1)

Shear capacity of single shear connector (from before) $q_r := P_{Rk} = 288 \cdot kN$

Number of shear connectors between midspan and end support of beam $n = 28$

Nominal total capacity of shear connectors $R_{ss} := n \cdot q_r = 8069 \cdot kN$

As the shear connector capacity $\phi_{sc}R_{ss}$ is less than both F_{beam} and F_{slab}, the beam section may be considered to be **partially composite**. Therefore Case 3 and Eqn 13.4.5(6) is applicable. Cl 13.4.5.2(c)

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Internal compression force from area of concrete in compression $R_{cc} := \phi_{sc} \cdot R_{ss} = 8069 \cdot kN$ Eqn 13.4.5(7)

Internal compression force from area of steel section in compression $R_{sc} := \dfrac{A_s \cdot f_y - R_{cc}}{2} = 2264 \cdot kN$ Eqn 13.4.5(8)

Depth of equivalent rectangular stress block $a_c := \dfrac{R_{cc}}{0.85 \cdot f_c \cdot b_{ec}} = 151.88 \cdot mm$ Eqn 13.4.5(9)

Steel beam flange area $A_f := w \cdot tf = 9846 \cdot mm^2$

Thickness of flange in compression (assuming web is not in compression) $t_{fcomp} := \dfrac{R_{sc}}{f_y \cdot w} = 21.33 \cdot mm$

Less than flange thickness (32mm) so assumption valid

Thickness of top flange in tension $t_{ftension} := tf - t_{fcomp} = 10.67 \cdot mm$

Area of top flange in tension $A_{flangetension} := t_{ftension} \cdot w$

$A_{flangetension} = 3284 \cdot mm^2$

Depth of web $d_w := h - 2 \cdot tf = 862.6 \cdot mm$

Area of web of steel beam $A_w := d_w \cdot tw = 16821 \cdot mm^2$

Height of centroid of steel beam in tension

$$e1 := \frac{A_f \cdot \frac{tf}{2} + A_w \cdot \left(tf + \frac{d_{web}}{2}\right) + A_{flangetension} \cdot \left(tf + d_{web} + \frac{t_{ftension}}{2}\right)}{A_f + A_w + A_{flangetension}} = 364 \cdot mm$$

Height of centroid of steel beam in compression $e2 := h - t_{fcomp} \cdot 0.5 = 916 \cdot mm$

Height of centroid of compression block in slab $e3 := h + t_{slab} - 0.5 \cdot a_c = 1031 \cdot mm$

Lever arm between steel area in tension and steel area in compression $L1 := e2 - e1 = 552 \cdot mm$

Lever arm between steel area in tension and compression block in slab $L2 := e3 - e1 = 667 \cdot mm$

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		Checked by: Dejan Novakov	Date: 17/02/16

Nominal moment resistance $M_{Rc} := R_{sc} \cdot L1 + R_{cc} \cdot L2$

$M_{Rc} = 6628 \cdot kN \cdot m$ **Eqn 13.4.5(6)**

The strength reduction factor for composite slabs subject to flexure is (from Table 13.1.2)

$\phi_{flexure} := 0.85$

Design moment resistance $M_{Rd\_NZS} := \phi_{flexure} \cdot M_{Rc} = 5633 \cdot kN \cdot m$

COMPARISON OF FLEXURAL CAPACITIES COMPUTED ABOVE

Design flexural capacity using the NZTA research report $M_{Rd\_NZTA} = 5687 kN \cdot m$

Design flexural capacity using NZS 3404 $M_{Rd\_NZS} = 5633 \cdot kN \cdot m$

Ratio of capacities between AS5100.6 and NZTA research report $\dfrac{M_{Rd\_NZS}}{M_{Rd\_NZTA}} = 0.99$

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Investigating what the fully composite bending resistance is using NZS3404 and how many shear connectors would be required to achieve this

Using section 13.4.5.2 of NZS3404. As F_{slab} is less than F_{beam}, Case 2 is applicable.

Internal compression force from area of concrete in compression $R_{cc1} := F_{slab} = 9563 \cdot kN$ Eqn 13.4.5(7)

Internal compression force from area of steel section in compression $R_{sc1} := \dfrac{A_s \cdot f_y - R_{cc1}}{2} = 1517 \cdot kN$ Eqn 13.4.5(8)

Depth of equivalent rectangular stress block $a_{c1} := \dfrac{R_{cc1}}{0.85 \cdot f_c \cdot b_{ec}} = 180 \cdot mm$ Eqn 13.4.5(9)

Steel beam flange area $A_{flange1} := w \cdot tf = 9846 \cdot mm^2$

Thickness of flange in compression (assuming web is not in compression) $t_{fcompl} := \dfrac{R_{sc1}}{f_y \cdot w} = 14.29 \cdot mm$

Less than flange thickness (32mm) so assumption valid

Thickness of top flange in tension $t_{ftension1} := tf - t_{fcompl} = 17.71 \cdot mm$

Area of top flange in tension $A_{flangetension1} := t_{ftension1} \cdot w$

$A_{flangetension1} = 5448 \cdot mm^2$

Depth of web $d_{web1} := h - 2 \cdot tf = 862.6 \cdot mm$

Area of web of steel beam $A_{web1} := d_{web1} \cdot tw = 16821 \cdot mm^2$

Height of centroid of steel beam in tension

$$e11 := \frac{A_{flange1} \cdot \frac{tf}{2} + A_{web1} \cdot \left(tf + \frac{d_{web1}}{2}\right) + A_{flangetension1} \cdot \left(tf + d_{web1} + \frac{t_{ftension1}}{2}\right)}{A_{flange1} + A_{web1} + A_{flangetension1}} = 400.83 \cdot mm$$

Height of centroid of steel beam in compression $e21 := h - t_{fcompl} \cdot 0.5 = 919 \cdot mm$

Height of centroid of compression block in slab $e31 := h + t_{slab} - 0.5 \cdot a_{c1} = 1017 \cdot mm$

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	Subject: Assessment example		
	Client: HERA	Made by: Mehedi Chowdhury	Date: 16/02/16
		Checked by: Dejan Novakov	Date: 17/02/16

Lever arm between steel area in tension and steel area in compression $L11 := e21 - e11 = 518.62 \cdot mm$

Lever arm between steel area in tension and compression block in slab $L21 := e31 - e11 = 615.77 \cdot mm$

Nominal moment resistance $M_{Rc1} := R_{sc1} \cdot L11 + R_{cc1} \cdot L21$

$M_{Rc1} = 6675 \cdot kN \cdot m$ Eqn 13.4.5(6)

Design moment resistance $\Phi_{flexure} \cdot M_{Rc1} = 5674 \cdot kN \cdot m$

To achieve fully composite behaviour we need the shear connector capacity to equal the concrete slab compression force. Back calculating the number of shear connectors required to achieve this...

$n_{req} := \dfrac{F_{slab}}{q_r} = 33.18$ so say 34

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	Subject: Assessment example		
	Client: HERA	Made by: Mehedi Chowdhury	Date: 16/02/16
		Checked by: Dejan Novakov	Date: 17/02/16

Example 2: 70% SHEAR CONNECTION

As a comparison, the beam bending capacity will now be determined using NZS3404 Section 13...

First checking whether effective concrete slab width is same as before...

Effective slab width $b_{ec} := \min(0.25 \cdot L_{ef}, 2.5m) = 2500 \cdot mm$ Cl 13.4.2.1.1

So effective slab width is unchanged. Section properties therefore unchanged

Using Clause 13.4.5.2 the bending capacity of the composite section can be determined. However first need to determine whether shear connection provides full or partial composite action.

Area of steel section $A_s = 36514 \cdot mm^2$

Yield strength of steel section $f_y = 345 \cdot MPa$

Maximum tensile force in steel beam (when plastic) $F_{beam} = 12597 \cdot kN$

Compressive strength of concrete slab $f_c = 25 \cdot MPa$

Concrete slab thickness (from before) $t_{slab} = 180 \cdot mm$

Maximum compressive force in concrete slab $F_{slab} := 0.85 \cdot f_c \cdot b_{ec} \cdot t_{slab} = 9563 \cdot kN$

Strength reduction factor for shear connector $\phi_{sc} := 1.0$ Table 13.1.2(1)

Shear capacity of single shear connector (from before) $q_r := P_{Rk} = 288 \cdot kN$

Number of shear connectors between midspan and end support of beam $n = 20$

Nominal total capacity of shear connectors $R_{ss} := n \cdot q_r = 5763 \cdot kN$

As the shear connector capacity $\phi_{sc}R_{ss}$ is less than both F_{beam} and F_{slab}, the beam section may be considered to be partially composite. Therefore Case 3 and Eqn 13.4.5(6) is applicable. Cl 13.4.5.2(c)

Check for composite action

$$\text{composite\_test} := \begin{cases} \text{"Composite"} & \text{if } R_{ss} \ge 0.5 \cdot \min(F_{slab}, F_{beam}) \\ \text{"Non-composite"} & \text{otherwise} \end{cases}$$

composite_test = "Composite"

Therefore, compsite action is considered in determining the flexural capacity

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	Subject: Assessment example		
	Client: HERA	Made by: Mehedi Chowdhury	Date: 16/02/16
		Checked by: Dejan Novakov	Date: 17/02/16

Internal compression force from area of concrete in compression $R_{cc} := \phi_{sc} \cdot R_{ss} = 5763 \cdot kN$ Eqn 13.4.5(7)

Internal compression force from area of steel section in compression $R_{sc} := \dfrac{A_s \cdot f_y - R_{cc}}{2} = 3417 \cdot kN$ Eqn 13.4.5(8)

Depth of equivalent rectangular stress block $a_c := \dfrac{R_{cc}}{0.85 \cdot f_c \cdot b_{ec}} = 108.49 \cdot mm$ Eqn 13.4.5(9)

Steel beam flange area $A_{flange} = 9846 \cdot mm^2$

Thickness of flange in compression (assuming web is not in compression) $t_{fcomp} := \dfrac{R_{sc}}{f_y \cdot w} = 32.19 \cdot mm$

Less than flange thickness (32mm) so assumption valid

Thickness of top flange in tension $t_{ftension} := tf - t_{fcomp} = -0.19 \cdot mm$

Area of top flange in tension $A_{flangetension} := t_{ftension} \cdot w$

$A_{flangetension} = -58 \cdot mm^2$

Depth of web $d_{web} = 862.6 \cdot mm$

Area of web of steel beam $A_{web} = 16821 \cdot mm^2$

Height of centroid of steel beam in tension

$$e1 := \frac{A_{flange} \cdot \frac{tf}{2} + A_{web} \cdot \left(tf + \frac{d_{web}}{2}\right) + A_{flangetension} \cdot \left(tf + d_{web} + \frac{t_{ftension}}{2}\right)}{A_{flange} + A_{web} + A_{flangetension}} = 297 \cdot mm$$

Height of centroid of steel beam in compression $e2 := h - t_{fcomp} \cdot 0.5 = 911 \cdot mm$

Height of centroid of compression block in slab $e3 := h + t_{slab} - 0.5 \cdot a_c = 1052 \cdot mm$

Lever arm between steel area in tension and steel area in compression $L1 := e2 - e1 = 614 \cdot mm$

Lever arm between steel area in tension and compression block in slab $L2 := e3 - e1 = 756 \cdot mm$

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Nominal moment resistance $M_{Rc} := R_{sc} \cdot L1 + R_{cc} \cdot L2$

$M_{Rc} = 6451 \cdot kN \cdot m$ **Eqn 13.4.5(6)**

The strength reduction factor for composite slabs subject to flexure is (from Table 13.1.2)

$\phi_{flexure} := 0.85$

Design moment resistance $M_{Rd\_NZS} := \phi_{flexure} \cdot M_{Rc} = 5483 \cdot kN \cdot m$

COMPARISON OF FLEXURAL CAPACITIES COMPUTED ABOVE

Design flexural capacity using the NZTA research report $M_{Rd\_NZTA} = 1859 \cdot kN \cdot m$

Design flexural capacity using the NZS 3404 $M_{Rd\_NZS} = 5483 \cdot kN \cdot m$

Ratio of capacities between NZS3404 and NZTA research report $\dfrac{M_{Rd\_NZS}}{M_{Rd\_NZTA}} = 2.95$

---------- END OF EXAMPLE ----------

Example 3: 85% SHEAR CONNECTION

As a comparison, the beam bending capacity will now be determined using NZS3404 Section 13...

First checking whether effective concrete slab width is same as before...

Effective slab width $b_{ec} := \min(0.25 \cdot L_{ef}, 2.5m) = 2500 \cdot mm$ Cl 13.4.2.1.1

So effective slab width is unchanged. Section properties therefore unchanged

Using Clause 13.4.5.2 the bending capacity of the composite section can be determined. However first need to determine whether shear connection provides full or partial composite action.

Area of steel section $A_s = 36514 \cdot mm^2$

Yield strength of steel section $f_y = 345 \cdot MPa$

Maximum tensile force in steel beam (when plastic) $F_{beam} = 12597 \cdot kN$

Compressive strength of concrete slab $f_c = 25 \cdot MPa$

Concrete slab thickness (from before) $t_{slab} = 180 \cdot mm$

Maximum compressive force in concrete slab $F_{slab} := 0.85 \cdot f_c \cdot b_{ec} \cdot t_{slab} = 9563 \cdot kN$

Strength reduction factor for shear connector $\phi_{sc} := 1.0$ Table 13.1.2(1)

Shear capacity of single shear connector (from before) $q_r := P_{Rk} = 288 \cdot kN$

Number of shear connectors between midspan and end support of beam $n = 24$

Nominal total capacity of shear connectors $R_{ss} := n \cdot q_r = 6916 \cdot kN$

As the shear connector capacity $\phi_{sc} R_{ss}$ is less than both F_{beam} and F_{slab}, the beam section may be considered to be partially composite. Therefore Case 3 and Eqn 13.4.5(6) is applicable. Cl 13.4.5.2(c)

Check for composite action

$$\text{composite\_test} := \begin{cases} \text{"Composite"} & \text{if } R_{ss} \geq 0.5 \cdot \min(F_{slab}, F_{beam}) \\ \text{"Non-composite"} & \text{otherwise} \end{cases}$$

composite_test = "Composite"

Therefore, compsite action is considered in determining the flexural capacity

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Internal compression force from area of concrete in compression $R_{cc} := \phi_{sc} \cdot R_{ss} = 6916 \cdot kN$ Eqn 13.4.5(7)

Internal compression force from area of steel section in compression $R_{sc} := \dfrac{A_s \cdot f_y - R_{cc}}{2} = 2841 \cdot kN$ Eqn 13.4.5(8)

Depth of equivalent rectangular stress block $a_c := \dfrac{R_{cc}}{0.85 \cdot f_c \cdot b_{ec}} = 130.19 \cdot mm$ Eqn 13.4.5(9)

Steel beam flange area $A_{flange} = 9846 \cdot mm^2$

Thickness of flange in compression (assuming web is not in compression) $t_{fcomp} := \dfrac{R_{sc}}{f_y \cdot w} = 26.76 \cdot mm$

Less than flange thickness (32mm) so assumption valid

Thickness of top flange in tension $t_{ftension} := tf - t_{fcomp} = 5.24 \cdot mm$

Area of top flange in tension $A_{flangetension} := t_{ftension} \cdot w$

$A_{flangetension} = 1613 \cdot mm^2$

Depth of web $d_{web} = 862.6 \cdot mm$

Area of web of steel beam $A_{web} = 16821 \cdot mm^2$

Height of centroid of steel beam in tension

$$e1 := \frac{A_{flange} \cdot \frac{tf}{2} + A_{web} \cdot \left(tf + \frac{d_{web}}{2}\right) + A_{flangetension} \cdot \left(tf + d_{web} + \frac{t_{ftension}}{2}\right)}{A_{flange} + A_{web} + A_{flangetension}} = 332 \cdot mm$$

Height of centroid of steel beam in compression $e2 := h - t_{fcomp} \cdot 0.5 = 913 \cdot mm$

Height of centroid of compression block in slab $e3 := h + t_{slab} - 0.5 \cdot a_c = 1042 \cdot mm$

Lever arm between steel area in tension and steel area in compression $L1 := e2 - e1 = 581 \cdot mm$

Lever arm between steel area in tension and compression block in slab $L2 := e3 - e1 = 709 \cdot mm$

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Nominal moment resistance $M_{Rc} := R_{sc} \cdot L1 + R_{cc} \cdot L2$

$M_{Rc} = 6555 \cdot kN \cdot m$ **Eqn 13.4.5(6)**

The strength reduction factor for composite slabs subject to flexure is (from Table 13.1.2)

$\phi_{flexure} := 0.85$

Design moment resistance $M_{Rd\_NZS} := \phi_{flexure} \cdot M_{Rc} = 5572 \cdot kN \cdot m$

COMPARISON OF FLEXURAL CAPACITIES COMPUTED ABOVE

Design flexural capacity using the NZTA research report $M_{Rd\_NZTA} = 5680 kN \cdot m$

Ratio of capacities between AS5100.6 and NZTA research report $\dfrac{M_{Rd\_NZS}}{M_{Rd\_NZTA}} = 0.98$

Appendix F: Worked examples using new design guidance

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This appendix presents worked examples where sagging bending moment is calculated at the midspan of a 23m steel composite girder of Waipoua River Bridge, which is representative of the typical composite girder bridge stock in New Zealand. These examples also provide a comparative assessment of the bending resistance determined in accordance with the NZTA research report TAR 14/07 and AS 5100.6 (as referred to for bridges by the current NZ Bridge Manual) for different degrees of shear connection (v7). For comparative purposes only, the same examples is recalculated to NZS 3404.

The following shear connection criteria were considered:

- The first example considers the as-built number of shear connector which provides full composite action based on the degree of shear connection greater than or equal to one (v7 > 1.0), as per the NZTA research report.
- The second example considers 70% connectivity of the original as-built shear connection (v7 < v7min).
- The third example considers 85% connectivity of the original as-built shear connection (v7min < v7 <1.0).

For consistency in the bending moment calculation, capacity of the shear connectors was determined using Equation 5.1 of the NZTA research report for all the examples, irrespective of the design standards. Note that both AS 5100.6 and NZS 3404 recommend different equations for shear connector's capacity.

The summary of the assessment is presented below.

Summary of sensitivity study using non-linear method (Line DEC of report Figure 4.1)	No. of shear connectors between points of zero and maximum sagging moment	Degree of shear connectivity η	AS5100.6 bending capacity / Report bending capacity	NZS3404 bending capacity / Report bending capacity
No. in original example (based on as-built drawings)	28	$\eta > 1$	1.03	0.99
No. assumed in second example (~70% of as-built drawings)	0.7*28 = 20	$\eta < \eta_{min}$	3.16	2.95
No. assumed in third example (~85% of as-built drawings)	0.85*28 = 24	$\eta_{min} < \eta < 1$	1.04	0.98

Based on the above assessment, it is clear that as the degree of shear connectivity is reduced, the difference between the bending resistance obtained using NZS3404 or AS5100.6 and NZTA report guideline increases. The primary reason for this is that, as the degree of shear connectivity reduces, the connection behaves in a more brittle manner. As a result, the distribution of stresses in the overall beam section (steel beam and concrete slab) will not be able to become plastic, but instead, will be elastic. Therefore the NZTA report guidelines assume an elastic distribution of stresses when determining the beam bending resistance, which yields a lower value than NZS3404 and AS5100.6. The latter standards assume plastic distribution of stresses regardless of degree of shear connectivity.

However, it should be noted that the NZTA Report guidelines were developed to assist with determining the composite bending resistance only. It is therefore possible that, depending on the degree of shear connectivity and relative dimensions of the concrete slab and steel beam section, the bending resistance of the bare steel beam may be greater than that of the composite beam. In such scenarios, it is important for users of the NZTA report to determine the bare steel beam bending resistance, giving attention to the degree of lateral support provided by the concrete slab and/or intermediate bracing between adjacent bridge beams. Users should also consider the effectiveness of shear connectors which have yielded in providing lateral restraint to the top flange of the steel section, paying attention to the locations where they may have yielded (eg. near support for simply supported beams.

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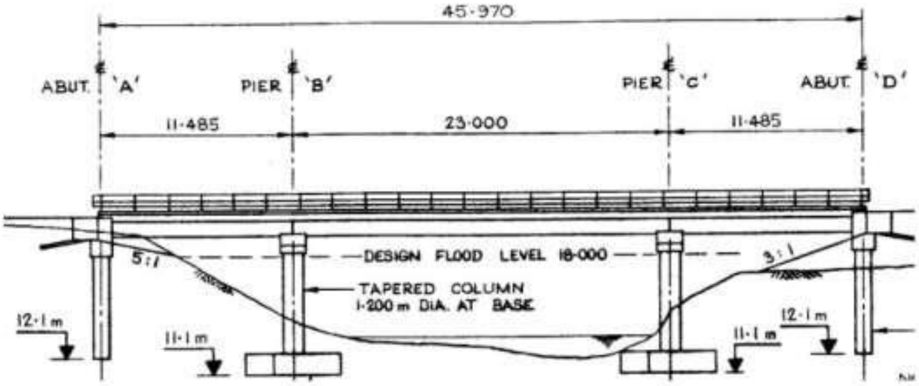
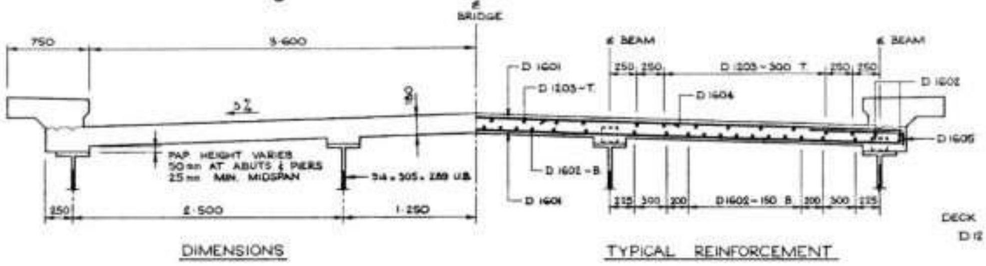
Waipoua River Bridge was designed in 1979 by the Ministry of Works and is located on State Highway 12 in Whangarei. It has three simply supported spans, with span arrangement of 11.485m + 23m + 11.485m. The bridge is on a 25 degree skew. The bridge superstructure consists of a 180mm thick insitu reinforced concrete slab supported by four structural steel girders at 2.5m c/c spacing. The main (middle) span girders are 914x305x289 UB with 152x76x18 - 150 long channel shear connectors with varying spacing (350mm over 6m from the supports and 500mm in the middle section of the beams).

The bridge elevation, cross section and main span girder elevation are shown below.

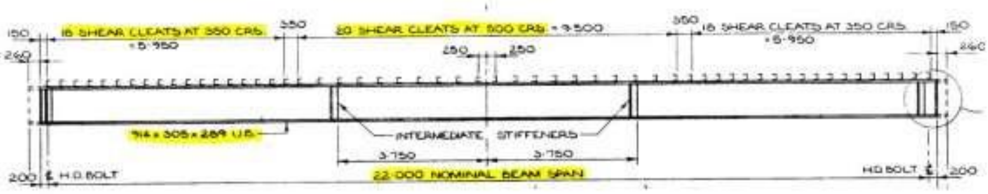
Bridge Elevation



Deck Cross Section showing Dimensions and Deck Reinforcement



Girder Elevation showing Shear Connectors



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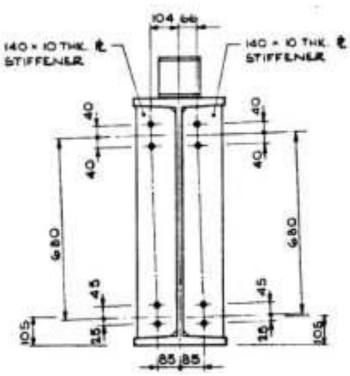
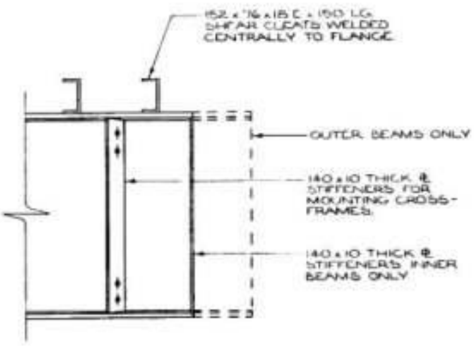
Girder Cross Section and Shear connector





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Example 1: BASED ON AS-BUILT NUMBER OF SHEAR CONNECTORS

Part 1 : COMPUTATION OF SAGGING MOMENT CAPACITY USING THE NZTA RESEARCH REPORT

The steps outlined below follow the procedures outlined in Section 5.2 of the report, which are largely based on Table 7.1 of the NZTA Bridge Manual.

Step 1 - Carry out site inspection

This would be undertaken as per NZTA Bridge Manual guidelines.

Step 2 - Determine appropriate material strengths

The design material strengths for the bridge components were obtained from the bridge drawings. No deterioration in the material strengths was assumed.

Structural steel yield strength (Grade 50C) $f_y := 345\text{MPa}$

Structural steel ultimate tensile strength (Grade 50C) $f_u := 450\text{MPa}$

Concrete compressive strength $f_c := 25\text{MPa}$

Yield strength of slab reinforcement $f_s := 275\text{MPa}$

Density of steel $\gamma_s := 7850\frac{\text{kg}}{\text{m}^3}$

Unit weight of concrete $\gamma_{conc} := 24\frac{\text{kN}}{\text{m}^3}$

Steel young's modulus $E_s := 205\text{GPa}$

Concrete young's modulus (based on NZS3101) $E_c := 3320\cdot\sqrt{\frac{f_c}{\text{MPa}}}\cdot 10^{-6}\,\text{GPa} + 6900\text{MPa}$

$E_c = 23.5\cdot\text{GPa}$

Step 2a - Identify types of the shear connectors and determine their ductility

For the bridge beams, 152x76x18 channel shear connectors are used with the following dimensions:

Height of channel $h_c := 152\text{mm}$

Width of channel $w_c := 76\text{mm}$

Flange thickness of channel $t_f := 9.14\text{mm}$

Web thickness of channel $t_w := 6.35\text{mm}$

Channel thickness considered $t_c := t_w$ $\quad t_c = 6.35\cdot\text{mm}$

Length of channel $l_c := 150\text{mm}$

Channel connectors are deemed to be ductile if $H/t_w/L > 0.124$ (Equation 5.2)

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$\text{DuctilityTest} := \left(\frac{h_c}{t_c}\right)\cdot\left(\frac{1}{l_c}\right)$ $\text{DuctilityTest} = 0.16\cdot\frac{1}{\text{mm}}$ which is greater than 0.124

Therefore, channel connectors are considered to be ductile.

Step 2b - Identify dimensions of concrete slab, steel sections and beam effective span for calculation of effective width of concrete flange

Effective bridge span (equal to simply supported span between bearings) $L_{ef} := 22\text{m}$

Concrete slab tributary width (consider inner beams) $b_{tributary} := 2.5\text{m}$

Concrete slab thickness $t_{slab} := 180\text{mm}$

Steel beam section height $h := 926.6\text{mm}$

Steel beam section width $w := 307.7\text{mm}$

Steel beam web thickness $tw := 19.5\text{mm}$

Steel beam flange thickness $tf := 32.0\text{mm}$

Step 2c - Calculate effective width of the concrete flange

Distance between outstand shear connectors (or length of channel connector) $b_0 := l_c = 150\cdot\text{mm}$

Geometric width of concrete flange on each side of the web $b_i := \frac{b_{tributary} - b_0}{2} = 1.18\times10^3\cdot\text{mm}$

Effective width of concrete flange on each side of the web (limited by geometric width) $\frac{L_{ef}}{8} = 2.75\,\text{m}$

$$b_{ei} := \min\left(\frac{L_{ef}}{8}, b_i\right)$$

$b_{ei} = 1175\cdot\text{mm}$ Note: in our example, this is the same for each side of the inner beam web

Total effective width $b_{eff} := b_0 + 2\cdot b_{ei}$ (Equation 5.3 which determines effective width at beam midspan)

$b_{eff} = 2.5\text{m}$

Note that equation 5.3 was used for two reasons (i) critical bending demand for a simply supported beam will be at midspan (ii) using equation 5.4 would yield a lower effective width value, a lower maximum potential compressive force in the concrete slab, and therefore potentially a higher degree of shear connectivity - therefore it is conservative to use equation 5.3

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Step 2d - Determine steel section classifications

In this step, using Section 5.1 of AS5100.6 (2004), the section is classified as either compact or non-compact. Note that universal beams are hot-rolled sections.

Top Flange

Flange outstand
$$b_{flange} := \frac{w - tw}{2}$$
$$b_{flange} = 144.1 \cdot mm$$

Flange slenderness
$$\lambda_{e\_flange} := \frac{b_{flange}}{tf} \cdot \sqrt{\frac{f_y}{250MPa}}$$
$$\lambda_{e\_flange} = 5.29$$

Flange yield slenderness limit (From Table 5.1 AS5100.6)
$$\lambda_{ey\_flange} := 16$$

Flange plasticity slenderness limit (From Table 5.1 AS5100.6)
$$\lambda_{ep\_flange} := 9$$

Flange slenderness ratio
$$\frac{\lambda_{e\_flange}}{\lambda_{ey\_flange}} = 0.33$$

Web

Web outstand
$$b_{web} := h - 2 \cdot tf$$
$$b_{web} = 862.6 \cdot mm$$

Web slenderness
$$\lambda_{e\_web} := \frac{b_{web}}{tw} \cdot \sqrt{\frac{f_y}{250MPa}}$$
$$\lambda_{e\_web} = 51.97$$

Web yield slenderness limit (From Table 5.1 AS5100.6)
$$\lambda_{ey\_web} := 115$$

Web plasticity slenderness limit (From Table 5.1 AS5100.6)
$$\lambda_{ep\_web} := 82$$

Web slenderness ratio
$$\frac{\lambda_{e\_web}}{\lambda_{ey\_web}} = 0.45$$

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Overall section

Web has higher slenderness ratio, therefore section slenderness properties are based on web

Section slenderness $\lambda_s := \lambda_{e\_web}$

$\lambda_s = 52$

Section plasticity limit $\lambda_{sp} := \lambda_{ep\_web}$

$\lambda_{sp} = 82$

Section yield limit $\lambda_{sy} := \lambda_{ey\_web}$

$\lambda_{sy} = 115$

As the section slenderness is less than the section plasticity limit, the section may be classed as being "compact"

Step 2e - Identify construction methods - propped or unpropped

The drawings do not provide any information on the construction methods used for the beams. Assume unpropped construction (conservative).

Step 3 - Identify critical section(s) of the main supporting members and the critical effect(s) on them

This step involves determining the maximum sagging moment in the composite beams under the evaluation load. This would involve undertaking a grillage analysis.

Typically for a simply supported bridge, maximum sagging moment would be at midspan. However as the bridge has a skew, the maximum moment will tend to be closer towards the obtuse corners. For the purposes of this example, will assume that maximum moment occurs at midspan of the beams.

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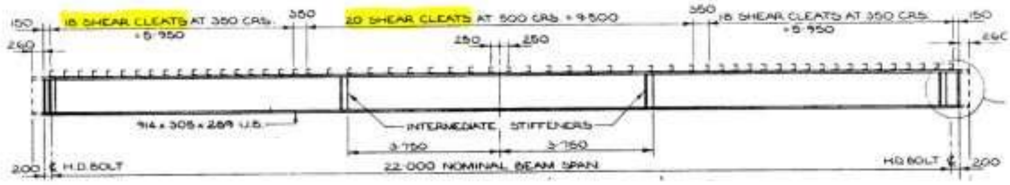
Step 3a - Determine the total number of shear connectors "n" between the locations of zero and maximum sagging moment



Number of shear connectors "n" $n := 18 + 0.5 \cdot 20$

$n = 28$

Step 3b - Determine the degree of the shear connection "η" if the shear connectors are deemed to be ductile

Determining design resistance of shear connector (Equation 5.1 of the NZTA research reprot)...

$$P_{Rk} := 31.2\left(t_f + 0.5 \cdot t_w\right) \cdot l_c \cdot \sqrt{\frac{f_c}{MPa}} \, MPa$$

$$P_{Rk} = 288 \cdot kN$$

$$\phi_v := 0.80$$

$$P_{Rd} := \phi_v \cdot P_{Rk}$$

$$P_{Rd} = 231 \cdot kN$$

Determining the design value of the compressive force in the concrete...

$$N_{c1} := n \cdot P_{Rd}$$

$$N_{c1} = 6455 \cdot kN$$

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Determining the design value of the compressive force in concrete slab when not limited by the shear connection...

Cross sectional area of steel beam $\quad A_s := (h - 2 \cdot tf) \cdot tw + 2 \cdot w \cdot tf$

$A_s = 36514 \cdot mm^2$

Strength reduction factor for steel $\quad \phi := 0.9 \quad$ (AS 5100.6 Table 3.2)

Design yield strength of steel $\quad f_{yd} := \phi \cdot f_y$

$f_{yd} = 310.5 \cdot MPa$

Strength reduction factor for concrete $\quad \phi_c := 0.6 \quad$ (AS 5100.6 Table 3.2)

Design compressive strength of concrete $\quad f_{cd} := \phi_c \cdot f_c$

$f_{cd} = 15 \cdot MPa$

Design compressive force in concrete with full shear connection $\quad N_{cf} := \min\left(A_s \cdot f_{yd}, 0.85 \cdot f_{cd} \cdot b_{eff} \cdot t_{slab}\right)$

$N_{cf} = 5738 \cdot kN$

Therefore the degree of shear connection is $\quad \eta := \dfrac{N_{c1}}{N_{cf}} \quad$ (Equation 4.1)

$\eta = 1.13 \quad$ as n is greater than or equal to 1, shear connection capacity does not govern (Full shear connection)

Therefore $\quad N_c := \min\left(N_{c1}, N_{cf}\right) = 5738 \cdot kN$

Step 3c - Determine the degree of the shear connection "η_{min}"

The minimum degree of shear connection η_{min} can be determined as shown below for steel sections with equal flanges.

$$\eta_{min} := \max\left[1 - \left(\frac{350MPa}{f_y}\right) \cdot \left(0.75 - 0.03\frac{L_{ef}}{m}\right), 0.4\right] \quad \text{(Equation 4.2 since } L_{ef} < 25m \text{)}$$

$\eta_{min} = 0.91 \quad$ therefore shear connection is ductile as $\eta > \eta_{min}$

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Step 4 - Determine the overload capacity and/or live load capacity at each critical main member section

The beams were classed to be "compact" with ductile shear connectors. As $\eta > \eta_{min}$, the shear connection is also deemed to be ductile as per note 3 of Table 4.1. Also, as $\eta > 1$, full shear connection can be achieved, so equations in section 4.3.2 of the report apply. Which equations to use depends on the location of the plastic neutral axis.

Depth of concrete slab $\quad h_{cs} := t_{slab} = 180 \cdot mm$

$h_t := t_{slab} = 180 \cdot mm$ ignoring "pops"

The height of "pops" varies along the girder length from 50mm at support to 25mm at mid span. Inclusion of "pops" will marginally increase the bending capacity.

Depth of structural steel section $\quad h_a := h = 926.6 \cdot mm$

Design value of plastic resistance of steel section to normal force $\quad N_{pla} := A_s \cdot f_{yd} = 11337 \cdot kN$

Design compression resistance of the concrete flange $\quad N_{plc} := 0.85 \cdot f_{cd} \cdot b_{eff} \cdot h_{cs} = 5738 \cdot kN$

Design plastic resistance of steel section flange $\quad N_{plf} := w \cdot tf \cdot f_{yd} = 3057 \cdot kN$

Design plastic resistance of steel section web $\quad N_{plw} := N_{pla} - 2 \cdot N_{plf} = 5223 \cdot kN$

As $N_{plc} > N_{plw}$ and $N_{pla} > N_{plc}$, equation 4.8 applies

Beam bending resistance

$$M_{plRd} := N_{pla} \cdot \frac{h_a}{2} + N_{plc} \cdot \left(h_t - \frac{h_{cs}}{2} \right) - \frac{\left(N_{pla} - N_{plc}\right)^2}{N_{plf}} \cdot \frac{tf}{4}$$

$$M_{plRd} = 5687 \cdot kN \cdot m$$

<u>For comparison, will assess the bending resistance using the "non-linear method" of the report</u>

Using the non-linear method should yield the same bending resistance as Equation 4.8 as full shear connection is achievable

Look at construction loading...

Construction pressure loads: Live load $\quad LLcon := 1kPa$

Wet concrete $\quad DLconc := \gamma_{conc} \cdot t_{slab} = 4.32 \cdot kPa$

UDL for bare steel beam $\quad DLsteel := 289 \frac{kg}{m} \cdot g = 2.83 \cdot \frac{kN}{m}$

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Load tributary width $\quad TribWidth := 2.5m$

Construction load UDL $\quad UDLcon := (1.49LLcon + 1.35DLconc)\cdot TribWidth + 1.35DLsteel$

(Load factors are taken from Table 3.2, Bridge Manual)

$$UDLcon = 22.13\cdot\frac{kN}{m}$$

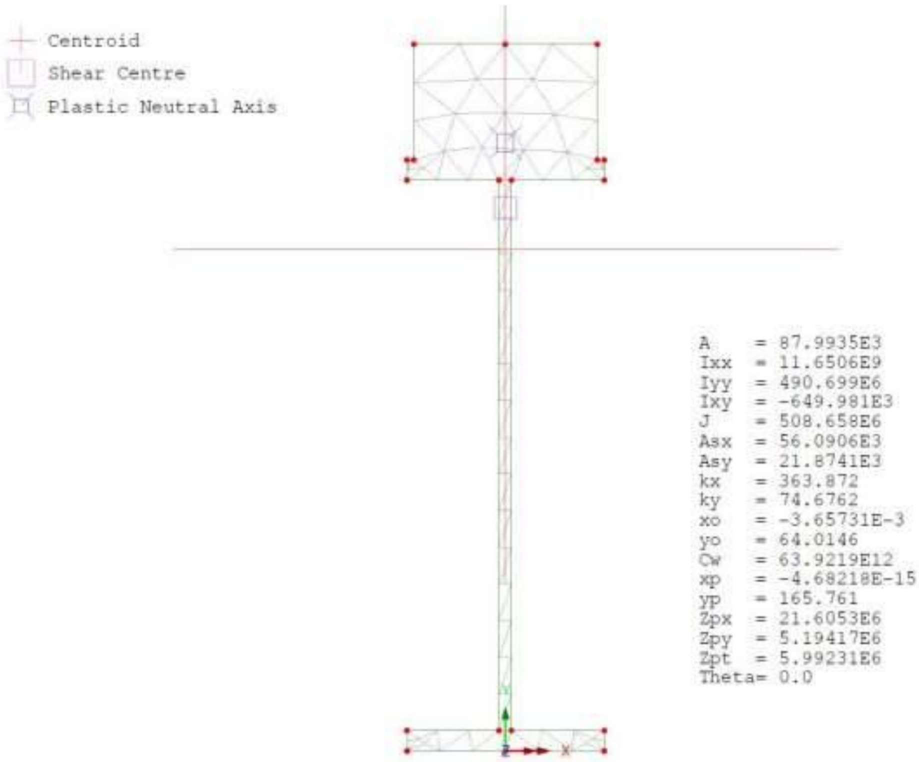
Construction load moment $\quad Mcon := \frac{UDLcon\cdot L_{ef}^{2}}{8}$

$$Mcon = 1339\cdot kN\cdot m$$

Point D in Figure 4.1 of report (Equation 4.22) $\quad M_{aEd} := Mcon = 1339\cdot kN\cdot m$

For equation 4.23, need to determine the design bending moment applied to the composite section. Normally structural analysis would be done to determine this demand, however for the purposes of this example, this demand will be assumed.

Design bending moment applied to composite section (assumed for example purposes) $\quad M_{cEd} := 1500kN\cdot m$



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For equation 4.23, "k" value also has to be determined, which is governed by whether concrete reaches design compressive limit (fcd), structural steel reaches yield stress (fyd), or slab reinforcement reaches yield stress (fsd). Need to use the composite section properties to determine "k".

Design conc. slab compressive strength (from before) $f_{cd} = 15 \cdot MPa$

Design yield strength of steel beam (tension and compression, from before) $f_{yd} = 310.5 \cdot MPa$

Strength reduction factor for concrete slab reinforcement $\phi_s := 0.85$ (NZS 3404 Table 13.1.2)

Design yield strength of slab reinforcement $f_{sd} := \phi_s \cdot f_s = 233.75 \cdot MPa$

Composite section properties (based on equivalent steel section)...

Modular ratio $modular\_ratio := \dfrac{E_s}{E_c} = 8.72$

Equivalent width of effective concrete slab $\dfrac{b_{eff}}{modular\_ratio} = 287 \cdot mm$

Composite section second moment of area $Ixx := 11.65 \times 10^9 mm^4$ refer transformed section properties picture on previous page

Steel beam section elastic modulus (non composite) $Zbeam := 10833000mm^3$

Composite section centroid height (within steel beam height) $yc := 787mm$

Height of steel beam extreme tension fibre $hst := 0mm$

Height of steel beam extreme comp. fibre $hsc := h = 927 \cdot mm$

Height of concrete extreme comp. fibre $hcc := t_{slab} + h = 1107 \cdot mm$

Height of concrete extreme comp. reinforcement bar (bar mark D1203) $hrc := hcc - 40mm - 16mm - \dfrac{12}{2}mm = 1045 \cdot mm$

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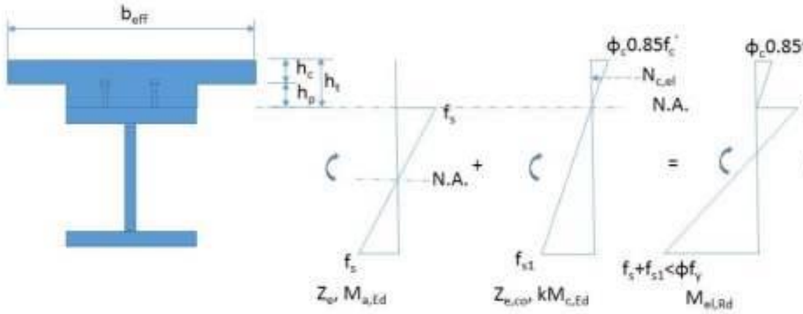

(Figure 5.2 of report)

Stress in top and bottom of beam due to construction loading

$$fs := \frac{M_{aEd}}{Zbeam} = 123.6 \cdot MPa$$

Additional moments acting on composite section required to reach design limit stresses:

Steel beam tension $$Mst := \frac{(f_{yd} - fs) \cdot Ixx}{(yc - hst)} = 2.77 \times 10^{3} \cdot kN \cdot m$$

Steel beam compression $$Msc := \frac{(f_{yd} - fs) \cdot Ixx}{(hsc - yc)} = 15598 \cdot kN \cdot m$$

Conc. slab compression $$Mcc := \frac{0.85 f_{cd} \cdot Ixx}{(hcc - yc)} = 464.76 \cdot kN \cdot m$$

Slab rebar compression. $$Mrc := \frac{f_{sd} \cdot Ixx}{(hrc - yc)} = 10571 \cdot kN \cdot m$$

From above, it is clear that under bending action, the concrete slab will reach its design compression limit first. Therefore "k" value will be based on this limiting stress/moment.

$$k := \frac{Mcc}{M_{cEd}} = 0.31$$

So $M_{elRd} := M_{aEd} + k \cdot M_{cEd} = 1.8 \times 10^{3} \cdot kN \cdot m$ (Equation 4.23)

For equation 4.25 or 4.26, the compressive force in the concrete slab when $M_{el,Rd}$ is applied needs to be determined.

Stress at bottom of concrete slab $$Stress\_btm := \frac{k \cdot M_{cEd} \cdot (h - yc)}{Ixx}$$

$Stress\_btm = 5.57 \cdot MPa$ (check: less than or equal to fcd -> OK)

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Stress at top of concrete slab

$$\text{Stress\_top} := \frac{k \cdot M_{cEd} \cdot (h + t_{slab} - yc)}{Ixx}$$

$$\text{Stress\_top} = 12.75 \cdot \text{MPa}$$

(check: less than or equal to $0.85f_{cd}$ -> OK)

Average stress in concrete slab

$$\text{Stress\_average} := \frac{\text{Stress\_btm} + \text{Stress\_top}}{2}$$

$$\text{Stress\_average} = 9.16 \cdot \text{MPa}$$

Effective concrete slab area

$$\text{ConcSlabArea} := b_{eff} \cdot t_{slab} = 0.45\,m^2$$

Slab concrete force, $N_{c,EL}$

$$N_{cEl} := \text{ConcSlabArea} \cdot \text{Stress\_average}$$

$$N_{cEl} = 4122 \cdot kN$$

From before $N_c = 5738 \cdot kN$

Therefore as Nc is greater than $N_{c,El}$ need to use equation 4.26 to determine beam moment resistance

$$M_{Rd} := M_{elRd} + (M_{plRd} - M_{elRd}) \cdot \frac{N_c - N_{cEl}}{N_{cf} - N_{cEl}} = 5687 \cdot kN \cdot m$$

(Check: same as plastic bending resistance previously determined using Equation 4.8 -> OK)

Step 5 onwards

No changes to Step 5 and onwards has been made by the report. Therefore following the end of Step 4 the engineer would follow the steps outlined in Table 7.1 of the NZTA Bridge Manual. i.e. undertake rating or posting analysis etc.

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Part 2 : COMPUTATION OF SAGGING MOMENT CAPACITY USING AS 5100.6

Steel section classification - compact or non-compact

As per AS5100.6 section 5.1

Top Flange

Flange outstand $b_{flange} = 144.1 \cdot mm$

Flange slenderness $\lambda_{e\_flange} = 5.29$

Flange yield slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ey\_flange} = 16$

Flange plasticity slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ep\_flange} = 9$

Flange slenderness ratio $\left(\frac{\lambda_{e\_flange}}{\lambda_{ey\_flange}}\right) = 0.33$

Web

Web outstand $b_{web} = 862.6 \cdot mm$

Web slenderness $\lambda_{e\_web} = 51.97$

Web yield slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ey\_web} = 115$

Web plasticity slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ep\_web} = 82$

Web slenderness ratio $\left(\frac{\lambda_{e\_web}}{\lambda_{ey\_web}}\right) = 0.45$

Overall section

Web has higher slenderness ratio, therefore section slenderness properties are based on web

Section slenderness $\lambda_s = 51.97$

Section plasticity limit $\lambda_{sp} = 82$

Section yield limit $\lambda_{sy} = 115$

As the section slenderness is less than the section plasticity limit, the section may be classed as being "compact"

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Effective width of concrete flange

As per AS5100.6 section 4.4.1

For girders which have composite flanges on both sides of the web, effective width shall be the least of:

one-fifth of the effective span length of the girder $b_1 := \frac{L_{ef}}{5} = 4.4m$

the distance centre-centre of girders $b_2 := 2.5m$

twelve times the least thickness of the slab $b_3 := 12 \cdot t_{slab} = 2160 \cdot mm$

Effective width of concrete flange $b_{eff\_AS} := \min(b_1, b_2, b_3) = 2160 \cdot mm$

Degree of shear connectivity

Based on Equation 5.1 of the NZTA research report, the characteristic resistance of shear connectors

$P_{Rk} = 288 \cdot kN$

Considering a capacity reduction factor (Table 3.2) $\phi_s = 0.85$

The design value of the shear connector $N_{cl\_AS} := n \cdot \phi_s \cdot P_{Rk} = 6858 \cdot kN$

Determining the design value of the compressive force in concrete deck slab with full shear connection

Cross sectional area of steel beam $A_s = 36514 \cdot mm^2$

Design strength of structural steel $f_{yd} = 310.5 \cdot MPa$

Design compressive strength of concrete $f_{cd} = 15 \cdot MPa$

Design compressive force in concrete with full shear connection

$$N_{cf\_AS} := \min(A_s \cdot f_{yd}, 0.85 \cdot f_{cd} \cdot b_{eff\_AS} \cdot t_{slab})$$

$$N_{cf\_AS} = 4957 \cdot kN$$

Degree of shear connector $\eta_{AS} := \frac{N_{cl\_AS}}{N_{cf\_AS}} = 1.38$ as n is greater than or equal to 1, shear connection capacity does not govern (Full shear connection)

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Bending capacity of composite beam

As per AS5100.6 section 6.3 ->
For compact beam, section capacity (Ms) = plastic moment capacity (Mp)

As per AS5100.6 section 6.4 ->
For beams continuously restrained by deck at compression flange level, beam moment capacity (Mb) = section capacity (Ms)

Determining the plastic moment capacity (Ms).
As per Clause 6.3.3, simple plastic theory shall be used to determine Ms.

Thickness of composite concrete slab $t_{slab} = 180 \cdot mm$

Effective width of concrete slab $b_{eff\_AS} = 2.16 \cdot m$

Maximum possible compressive force in concrete slab $F_{slab\_AS} := 0.85 \cdot f_c \cdot b_{eff\_AS} \cdot t_{slab} = 8262 \cdot kN$

Force transferred to concrete slab via shear connectors $R_{cc\_AS} := F_{slab\_AS} = 8262 \cdot kN$

Depth of equivalent rectangular stress block $a_{c\_AS} := \dfrac{R_{cc\_AS}}{0.85 \cdot f_c \cdot b_{eff\_AS}} = 180 \cdot mm$

Area of steel beam cross-section $A_s = 36514 \cdot mm^2$

Maximum possible tensile force in steel beam $F_{beam} := A_s \cdot f_y = 12597 \cdot kN$

Compressive force in steel beam
(As $F_{beam} > F_{slab}$) $R_{sc\_AS} := \dfrac{A_s \cdot f_y - R_{cc\_AS}}{2} = 2168 \cdot kN$

Area of top flange $A_{flange} := w \cdot tf = 9846 \cdot mm^2$

Thickness of flange in compression
(assuming web is not in compression) $t_{fcomp\_AS} := \dfrac{R_{sc\_AS}}{f_y \cdot w} = 20.42 \cdot mm$

Less than flange thickness (32mm)
so assumption valid

Thickness of top flange in tension $t_{ften\_AS} := tf - t_{fcomp\_AS} = 11.58 \cdot mm$

Area of top flange in tension $A_{flten\_AS} := t_{ften\_AS} \cdot w = 3564 \cdot mm^2$

$A_{flten\_AS} = 3564 \cdot mm^2$

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Depth of web $d_{web} := h - 2 \cdot tf = 862.6 \cdot mm$

Area of web of steel beam $A_{web} := d_{web} \cdot tw = 16821 \cdot mm^2$

Height of centroid of steel beam in tension

$$e_{1\_AS} := \frac{A_{flange} \cdot \frac{tf}{2} + A_{web} \cdot \left(tf + \frac{d_{web}}{2}\right) + A_{flten\_AS} \cdot \left(tf + d_{web} + \frac{t_{flten\_AS}}{2}\right)}{A_{flange} + A_{web} + A_{flten\_AS}} = 369 \cdot mm$$

Height of centroid of steel beam in compression $e_{2\_AS} := h - t_{fcomp\_AS} \cdot 0.5 = 916 \cdot mm$

Height of centroid of compression block in slab $e_{3\_AS} := h + t_{slab} - 0.5 \cdot a_{c\_AS} = 1017 \cdot mm$

Lever arm between steel area in tension and steel area in compression $L_{1\_AS} := e_{2\_AS} - e_{1\_AS} = 547 \cdot mm$

Lever arm between steel area in tension and compression block in slab $L_{2\_AS} := e_{3\_AS} - e_{1\_AS} = 647 \cdot mm$

Nominal moment resistance $M_{s\_AS} := R_{sc\_AS} \cdot L_{1\_AS} + R_{cc\_AS} \cdot L_{2\_AS}$

$M_{s\_AS} = 6536 \cdot kN \cdot m$

Strength reduction factor (Table 3.2) $\phi = 0.9$

Design moment resistance $M_{Rd\_AS} := \phi \cdot M_{s\_AS} = 5882 \cdot kN \cdot m$

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Part 3 : COMPARISON OF FLEXURAL CAPACITIES COMPUTED ABOVE

Design flexural capacity using the NZTA research report $M_{Rd\_NZTA} := M_{Rd} = 5687 \cdot kN \cdot m$

Design flexural capacity using AS 5100.6 $M_{Rd\_AS} = 5882 \cdot kN \cdot m$

Ratio of capacities between AS5100.6 and NZTA research report $\frac{M_{Rd\_AS}}{M_{Rd\_NZTA}} = 1.03$

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Example 2: 70% SHEAR CONNECTION ($n<n_{min}$)

Part 1 : COMPUTATION OF SAGGING MOMENT CAPACITY USING THE NZTA RESEARCH REPORT

The steps outlined below follow the procedures outlined in Section 5.2 of the report, which are largely based on Table 7.1 of the NZTA Bridge Manual.

Step 1 - Carry out site inspection

This would be undertaken as per NZTA Bridge Manual guidelines.

Step 2 - Determine appropriate material strengths

The design material strengths for the bridge components were obtained from the bridge drawings. No deterioration in the material strengths was assumed.

Structural steel yield strength (Grade 50C) $f_y := 345\text{MPa}$

Structural steel ultimate tensile strength (Grade 50C) $f_u := 450\text{MPa}$

Concrete compressive strength $f_c := 25\text{MPa}$

Yield strength of slab reinforcement $f_s := 275\text{MPa}$

Density of steel $\gamma_s := 7850\dfrac{\text{kg}}{\text{m}^3}$

Unit weight of concrete $\gamma_{conc} := 24\dfrac{\text{kN}}{\text{m}^3}$

Steel young's modulus $E_s := 205\text{GPa}$

Concrete young's modulus (based on NZS3101)

$$E_c := 3320\cdot\sqrt{\frac{f_c}{\text{MPa}}\cdot 10^{-6}}\text{GPa} + 6900\text{MPa}$$

$$E_c = 23.5\cdot\text{GPa}$$

Step 2a - Identify types of the shear connectors and determine their ductility

For the bridge beams, 152x76x18 channel shear connectors are used with the following dimensions:

Height of channel $h_c := 152\text{mm}$

Width of channel $w_c := 76\text{mm}$

Flange thickness of channel $t_f := 9.14\text{mm}$

Web thickness of channel $t_w := 6.35\text{mm}$

Channel thickness considered $t_c := t_w$ $t_c = 6.35\cdot\text{mm}$

Length of channel $l_c := 150\text{mm}$

Channel connectors are deemed to be ductile if $H/t_w/L > 0.124$ (Equation 5.2)

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$\mathrm{DuctilityTest} := \left(\frac{h_c}{t_c}\right)\cdot\left(\frac{1}{l_c}\right)$ $\mathrm{DuctilityTest} = 0.16\cdot\frac{1}{\mathrm{mm}}$ which is greater than 0.124

Therefore, channel connectors are considered to be ductile.

Step 2b - Identify dimensions of concrete slab, steel sections and beam effective span for calculation of effective width of concrete flange

Effective bridge span (equal to simply supported span between bearings) $L_{ef} := 22\mathrm{m}$

Concrete slab tributary width (consider inner beams) $b_{tributary} := 2.5\mathrm{m}$

Concrete slab thickness $t_{slab} := 180\mathrm{mm}$

Steel beam section height $h := 926.6\mathrm{mm}$

Steel beam section width $w := 307.7\mathrm{mm}$

Steel beam web thickness $tw := 19.5\mathrm{mm}$

Steel beam flange thickness $tf := 32.0\mathrm{mm}$

Step 2c - Calculate effective width of the concrete flange

Distance between outstand shear connectors (or length of channel connector) $b_0 := l_c = 150\cdot\mathrm{mm}$

Geometric width of concrete flange on each side of the web $b_i := \frac{b_{tributary} - b_0}{2} = 1.18\times 10^3\cdot\mathrm{mm}$

Effective width of concrete flange on each side of the web (limited by geometric width) $\frac{L_{ef}}{8} = 2.75\,\mathrm{m}$

$$b_{ei} := \min\left(\frac{L_{ef}}{8}, b_i\right)$$

$b_{ei} = 1175\cdot\mathrm{mm}$ Note: in our example, this is the same for each side of the inner beam web

Total effective width $b_{eff} := b_0 + 2\cdot b_{ei}$ (Equation 5.3 which determines effective width at beam midspan)

$b_{eff} = 2.5\,\mathrm{m}$

Note that equation 5.3 was used for two reasons (i) critical bending demand for a simply supported beam will be at midspan (ii) using equation 5.4 would yield a lower effective width value, a lower maximum potential compressive force in the concrete slab, and therefore potentially a higher degree of shear connectivity - therefore it is conservative to use equation 5.3

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Step 2d - Determine steel section classifications

In this step, using Section 5.1 of AS5100.6 (2004), the section is classified as either compact or non-compact. Note that universal beams are hot-rolled sections.

Top Flange

Flange outstand $b_{flange} := \frac{w - tw}{2}$

$b_{flange} = 144.1 \cdot mm$

Flange slenderness $\lambda_{e\_flange} := \frac{b_{flange}}{tf} \cdot \sqrt{\frac{f_y}{250MPa}}$

$\lambda_{e\_flange} = 5.29$

Flange yield slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ey\_flange} := 16$

Flange plasticity slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ep\_flange} := 9$

Flange slenderness ratio $\frac{\lambda_{e\_flange}}{\lambda_{ey\_flange}} = 0.33$

Web

Web outstand $b_{web} := h - 2 \cdot tf$

$b_{web} = 862.6 \cdot mm$

Web slenderness $\lambda_{e\_web} := \frac{b_{web}}{tw} \cdot \sqrt{\frac{f_y}{250MPa}}$

$\lambda_{e\_web} = 51.97$

Web yield slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ey\_web} := 115$

Web plasticity slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ep\_web} := 82$

Web slenderness ratio $\frac{\lambda_{e\_web}}{\lambda_{ey\_web}} = 0.45$

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Overall section

Web has higher slenderness ratio, therefore section slenderness properties are based on web

Section slenderness $\lambda_s := \lambda_{e\_web}$

$\lambda_s = 52$

Section plasticity limit $\lambda_{sp} := \lambda_{ep\_web}$

$\lambda_{sp} = 82$

Section yield limit $\lambda_{sy} := \lambda_{ey\_web}$

$\lambda_{sy} = 115$

As the section slenderness is less than the section plasticity limit, the section may be classed as being "compact"

Step 2e - Identify construction methods - propped or unpropped

The drawings do not provide any information on the construction methods used for the beams. Assume unpropped construction (conservative).

Step 3 - Identify critical section(s) of the main supporting members and the critical effect(s) on them

This step involves determining the maximum sagging moment in the composite beams under the evaluation load. This would involve undertaking a grillage analysis.

Typically for a simply supported bridge, maximum sagging moment would be at midspan. However as the bridge has a skew, the maximum moment will tend to be closer towards the obtuse corners. For the purposes of this example, will assume that maximum moment occurs at midspan of the beams.

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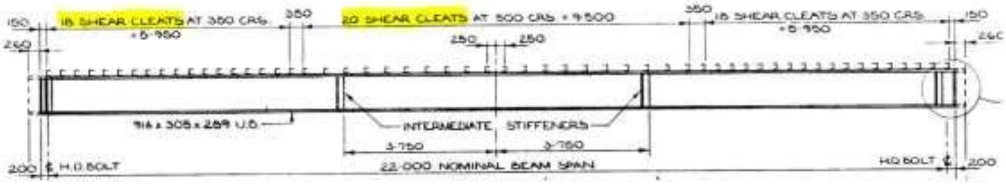
Step 3a - Determine the total number of shear connectors "n" between the locations of zero and maximum sagging moment



Number of shear connectors "n" $n_{asbuilt} := 18 + 0.5 \cdot 20$ $n_{asbuilt} = 28$

However for the purpose of this example consder a lower number of shear connectors $n := 20$

Step 3b - Determine the degree of the shear connection "η" if the shear connectors are deemed to be ductile

Determining design resistance of shear connector (using NZS3404 Eqn 13.3.2.2)...

$$P_{Rk} := 31.2 \left(t_f + 0.5 \cdot t_w\right) \cdot l_c \cdot \sqrt{\frac{f_c}{MPa}} \, MPa$$

$$P_{Rk} = 288 \cdot kN$$

$$\phi_v := 0.80$$

$$P_{Rd} := \phi_v \cdot P_{Rk}$$

$$P_{Rd} = 231 \cdot kN$$

Determining the design value of the compressive force in the concrete...

$$N_{c1} := n \cdot P_{Rd}$$

$$N_{c1} = 4611 \cdot kN$$

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Determining the design value of the compressive force in concrete slab when not limited by the shear connection...

Crossectional area of steel beam $A_s := (h - 2 \cdot tf) \cdot tw + 2 \cdot w \cdot tf$

$A_s = 36514 \cdot mm^2$

Strength reduction factor for steel $\phi := 0.9$ (AS 5100.6 Table 3.2)

Design yield strength of steel $f_{yd} := \phi \cdot f_y$

$f_{yd} = 310.5 \cdot MPa$

Strength reduction factor for concrete $\phi_c := 0.6$ (AS 5100.6 Table 3.2)

Design compressive strength of concrete $f_{cd} := \phi_c \cdot f_c$

$f_{cd} = 15 \cdot MPa$

Design compressive force in concrete with full shear connection $N_{cf} := \min\left(A_s \cdot f_{yd}, 0.85 \cdot f_{cd} \cdot b_{eff} \cdot t_{slab}\right)$

$N_{cf} = 5738 \cdot kN$

Therefore the degree of shear connection is $\eta := \dfrac{N_{c1}}{N_{cf}}$ (Equation 4.1)

$\eta = 0.8$ shear connection capacity governs as n is less than unity

Therefore $N_c := \min\left(N_{c1}, N_{cf}\right) = 4611 \cdot kN$

Step 3c - Determine the degree of the shear connection "η_{min}"

The minimum degree of shear connection n_{min} can be determined as shown below for steel sections with equal flanges.

$$\eta_{min} := \max\left[1 - \left(\frac{350MPa}{f_y}\right) \cdot \left(0.75 - 0.03 \frac{L_{ef}}{m}\right), 0.4\right]$$ (Equation 4.2 since $L_{ef} < 25m$)

$\eta_{min} = 0.91$ therefore shear connection is non-ductile as $\eta < \eta_{min}$

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Step 4 - Determine the overload capacity and/or live load capacity at each critical main member section

The beams were classed to be "compact" with ductile shear connectors. However, as $\eta<\eta_{min}$, the shear connection is deemed to be non-ductile as per note 3 of Table 4.1.

Using the non-linear method with partial shear connection, the design bending capacity is as follows.

Look at construction loading...

Construction pressure loads: Live load $LLcon := 1kPa$

Wet concrete $DLconc := \gamma_{conc} \cdot t_{slab} = 4.32 \cdot kPa$

UDL for bare steel beam $DLsteel := 289\frac{kg}{m} \cdot g = 2.83 \cdot \frac{kN}{m}$

Load tributary width $TribWidth := 2.5m$

Construction load UDL $UDLcon := (1.49LLcon + 1.35DLconc) \cdot TribWidth + 1.35DLsteel$

(Load factors are taken from Table 3.2, Bridge Manual) $UDLcon = 22.13 \cdot \frac{kN}{m}$

Construction load moment $Mcon := \frac{UDLcon \cdot L_{ef}^{\ 2}}{8}$

$Mcon = 1339 \cdot kN \cdot m$

Point D in Figure 4.1 of report (Equation 4.22) $M_{aEd} := Mcon = 1339 \cdot kN \cdot m$

For equation 4.23, need to determine the design bending moment applied to the composite section. Normally structural analysis would be done to determine this demand, however for the purposes of this example, this demand will be assumed.

Design bending moment applied to composite section (assumed for example purposes) $M_{cEd} := 1500kN \cdot m$

For equation 4.23, "k" value also has to be determined, which is governed by whether concrete reaches design compressive limit (fcd), structural steel reaches yield stress (f_{yd}), or slab reinforcement reaches yield stress (f_{sd}). Need to use the composite section properties to determine "k".

Design conc. slab compressive strength (from before) $f_{cd} = 15 \cdot MPa$

Design yield strength of steel beam (tension and compression, from before) $f_{yd} = 310.5 \cdot MPa$

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Strength reduction factor for concrete slab reinforcement $\phi_s := 0.85$ (NZS 3404 Table 13.1.2)

Design yield strength of slab reinforcement $f_{sd} := \phi_s \cdot f_s = 233.75 \cdot MPa$

Composite section properties (based on equivalent steel section)...

Modular ratio $modular\_ratio := \frac{E_s}{E_c} = 8.72$

Equivalent width of effective concrete slab $\frac{b_{eff}}{modular\_ratio} = 287 \cdot mm$

Composite section second moment of area $Ixx := 11.65 \times 10^9 mm^4$ refer transformed section properties picture on previous page

Steel beam section elastic modulus (non composite) $Zbeam := 10833000mm^3$

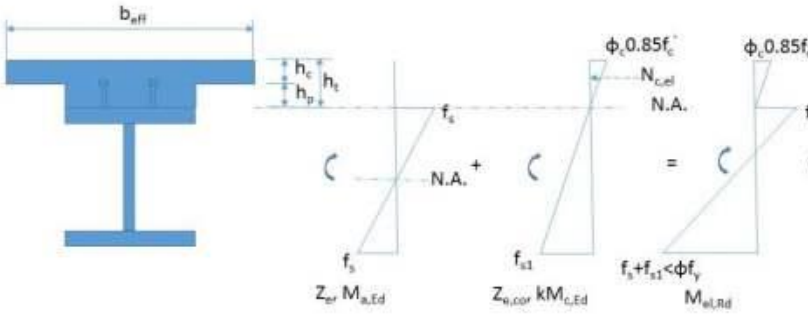
Composite section centroid height (within steel beam height) $yc := 787mm$

Height of steel beam extreme tension fibre $hst := 0mm$

Height of steel beam extreme comp. fibre $hsc := h = 927 \cdot mm$

Height of concrete extreme comp. fibre $hcc := t_{slab} + h = 1107 \cdot mm$

Height of concrete extreme comp. reinforcement bar (bar mark D1203) $hrc := hcc - 40mm - 16mm - \frac{12}{2}mm = 1045 \cdot mm$



(Figure 5.2 of report)

Stress in top and bottom of beam due to construction loading $fs := \frac{M_{aEd}}{Zbeam} = 123.6 \cdot MPa$

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Additional moments acting on composite section required to reach design limit stresses:

Steel beam tension $\quad Mst := \dfrac{(f_{yd} - fs)\cdot Ixx}{(yc - hst)} = 2767\cdot kN\cdot m$

Steel beam compression $\quad Msc := \dfrac{(f_{yd} - fs)\cdot Ixx}{(hsc - yc)} = 15598\cdot kN\cdot m$

Conc. slab compression $\quad Mcc := \dfrac{0.85 f_{cd}\cdot Ixx}{(hcc - yc)} = 464.76\cdot kN\cdot m$

Slab rebar compression. $\quad Mrc := \dfrac{f_{sd}\cdot Ixx}{(hrc - yc)} = 10571\cdot kN\cdot m$

From above, it is clear that under bending action, the concrete slab will reach its design compression limit first. Therefore "k" value will be based on this limiting stress/moment.

$$k := \frac{Mcc}{M_{cEd}} = 0.31$$

So $\quad M_{elRd} := M_{aEd} + k\cdot M_{cEd} = 1804\cdot kN\cdot m$ $\qquad$ (Equation 4.23)

For equation 4.25 or 4.26, the compressive force in the concrete slab when $M_{el,Rd}$ is applied needs to be determined.

Stress at bottom of concrete slab $\quad Stress\_btm := \dfrac{k\cdot M_{cEd}\cdot (h - yc)}{Ixx}$

$Stress\_btm = 5.57\cdot MPa$ $\quad$ (check: less than or equal to fcd -> OK)

Stress at top of concrete slab $\quad Stress\_top := \dfrac{k\cdot M_{cEd}\cdot (h + t_{slab} - yc)}{Ixx}$

$Stress\_top = 12.75\cdot MPa$ $\quad$ (check: less than or equal to fcd -> OK)

Average stress in concrete slab $\quad Stress\_average := \dfrac{Stress\_btm + Stress\_top}{2}$

$Stress\_average = 9.16\cdot MPa$

Effective concrete slab area $\quad ConcSlabArea := b_{eff}\cdot t_{slab} = 450000\cdot mm^2$

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Slab concrete force, $N_{c,EL}$

$$N_{cEl} := \text{ConcSlabArea} \cdot \text{Stress\_average}$$

$$N_{cEl} = 4122 \cdot kN$$

From before $N_c = 4611 \cdot kN$

Therefore as Nc is greater than $N_{c,el}$ need to use equation 4.25 to determine beam moment resistance

$$M_{Rd} := M_{aEd} + \left(M_{elRd} - M_{aEd}\right) \cdot \frac{N_c}{N_{cEl}} = 1859 \cdot kN \cdot m$$

This capacity is much less than the non-composite elastic capacity $M_{nc} := f_y \cdot Zbeam = 3737 \cdot kN \cdot m$

Step 5 onwards

No changes to Step 5 and onwards has been made by the report. Therefore following the end of Step 4 the engineer would follow the steps outlined in Table 7.1 of the NZTA Bridge Manual. i.e. undertake rating or posting analysis etc.

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Part 2 : COMPUTATION OF SAGGING MOMENT CAPACITY USING AS 5100.6

Steel section classification - compact or non-compact

As per AS5100.6 section 5.1

Top Flange

Flange outstand $b_{flange} = 144.1\cdot mm$

Flange slenderness $\lambda_{e\_flange} = 5.29$

Flange yield slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ey\_flange} = 16$

Flange plasticity slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ep\_flange} = 9$

Flange slenderness ratio $\dfrac{\lambda_{e\_flange}}{\lambda_{ey\_flange}} = 0.33$

Web

Web outstand $b_{web} = 862.6\cdot mm$

Web slenderness $\lambda_{e\_web} = 51.97$

Web yield slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ey\_web} = 115$

Web plasticity slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ep\_web} = 82$

Web slenderness ratio $\dfrac{\lambda_{e\_web}}{\lambda_{ey\_web}} = 0.45$

Overall section

Web has higher slenderness ratio, therefore section slenderness properties are based on web

Section slenderness $\lambda_s = 51.97$

Section plasticity limit $\lambda_{sp} = 82$

Section yield limit $\lambda_{sy} = 115$

As the section slenderness is less than the section plasticity limit, the section may be classed as being "compact"

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Effective width of concrete flange

As per AS5100.6 section 4.4.1

For girders which have composite flanges on both sides of the web, effective width shall be the least of

one-fifth of the effective span length of the girder $b_1 := \frac{L_{ef}}{5} = 4.4 m$

the distance centre-centre of girders $b_2 := 2.5 m$

twelve times the least thickness of the slab $b_3 := 12 \cdot t_{slab} = 2160 \cdot mm$

Effective width of concrete flange $b_{eff\_AS} := \min(b_1, b_2, b_3) = 2160 \cdot mm$

Degree of shear connectivity

Based on Equation 5.1 of the NZTA research report, the characteristic resistance of shear connectors

$P_{Rk} = 288 \cdot kN$

Considering a capacity reduction factor (Table 3.2) $\phi_s = 0.85$

The design value of the shear connector $N_{c1\_AS} := n \cdot \phi_s \cdot P_{Rk} = 4899 \cdot kN$

Determining the design value of the compressive force in concrete deck slab with full shear connection

Cross sectional area of steel beam $A_s = 36514 \cdot mm^2$

Design strength of structural steel $f_{yd} = 310.5 \cdot MPa$

Design compressive strength of concrete $f_{cd} = 15 \cdot MPa$

Design compressive force in concrete with full shear connection

$$N_{cf\_AS} := \min(A_s \cdot f_{yd}, 0.85 \cdot f_{cd} \cdot b_{eff\_AS} \cdot t_{slab})$$

$$N_{cf\_AS} = 4957 \cdot kN$$

Degree of shear connector $\eta_{AS} := \frac{N_{c1\_AS}}{N_{cf\_AS}} = 0.99$

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Bending capacity of composite beam

As per AS5100.6 section 6.3 ->
For compact beam, section capacity (Ms) = plastic moment capacity (Mp)

As per AS5100.6 section 6.4 ->
For beams continuously restrained by deck at compression flange level,
beam moment capacity (Mb) = section capacity (Ms)

Determining the plastic moment capacity (Ms).
As per Clause 6.3.3, simple plastic theory shall be used to determine Ms.

Thickness of composite concrete slab $t_{slab} = 180 \cdot mm$

Effective width of concrete slab $b_{eff\_AS} = 2.16 \cdot m$

Maximum possible compressive force in concrete slab $F_{slab\_AS} := 0.85 \cdot f_c \cdot b_{eff\_AS} \cdot t_{slab} = 8262 \cdot kN$

Force transferred to concrete slab via shear connectors $R_{cc\_AS} := \eta_{AS} \cdot F_{slab\_AS} = 8165 \cdot kN$

Depth of equivalent rectangular stress block $a_{c\_AS} := \dfrac{R_{cc\_AS}}{0.85 \cdot f_c \cdot b_{eff\_AS}} = 177.88 \cdot mm$

Area of steel beam cross-section $A_s = 36514 \cdot mm^2$

Maximum possible tensile force in steel beam $F_{beam} := A_s \cdot f_y = 12597 \cdot kN$

Compressive force in steel beam
(As $F_{beam} > F_{slab}$) $R_{sc\_AS} := \dfrac{A_s \cdot f_y - R_{cc\_AS}}{2} = 2216 \cdot kN$

Area of top flange $A_{flange} := w \cdot tf = 9846 \cdot mm^2$

Thickness of flange in compression
(assuming web is not in compression) $t_{fcomp\_AS} := \dfrac{R_{sc\_AS}}{f_y \cdot w} = 20.88 \cdot mm$

Less than flange thickness (32mm)
so assumption valid

Thickness of top flange in tension $t_{ften\_AS} := tf - t_{fcomp\_AS} = 11.12 \cdot mm$

Area of top flange in tension $A_{flten\_AS} := t_{ften\_AS} \cdot w = 3423 \cdot mm^2$

$A_{flten\_AS} = 3423 \cdot mm^2$

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Depth of web $d_{web} := h - 2 \cdot tf = 862.6 \cdot mm$

Area of web of steel beam $A_{web} := d_{web} \cdot tw = 16821 \cdot mm^2$

Height of centroid of steel beam in tension

$$e_{1\_AS} := \frac{A_{flange} \cdot \frac{tf}{2} + A_{web} \cdot \left(tf + \frac{d_{web}}{2}\right) + A_{flten\_AS} \cdot \left(tf + d_{web} + \frac{t_{flten\_AS}}{2}\right)}{A_{flange} + A_{web} + A_{flten\_AS}} = 367 \cdot mm$$

Height of centroid of steel beam in compression $e_{2\_AS} := h - t_{fcomp\_AS} \cdot 0.5 = 916 \cdot mm$

Height of centroid of compression block in slab $e_{3\_AS} := h + t_{slab} - 0.5 \cdot a_{c\_AS} = 1018 \cdot mm$

Lever arm between steel area in tension and steel area in compression $L_{1\_AS} := e_{2\_AS} - e_{1\_AS} = 550 \cdot mm$

Lever arm between steel area in tension and compression block in slab $L_{2\_AS} := e_{3\_AS} - e_{1\_AS} = 651 \cdot mm$

Nominal moment resistance $M_{s\_AS} := R_{sc\_AS} \cdot L_{1\_AS} + R_{cc\_AS} \cdot L_{2\_AS}$

$M_{s\_AS} = 6533 \cdot kN \cdot m$

Strength reduction factor (Table 3.2) $\phi = 0.9$

Design moment resistance $M_{Rd\_AS} := \phi \cdot M_{s\_AS} = 5880 \cdot kN \cdot m$

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Part 3 : COMPARISON OF FLEXURAL CAPACITIES COMPUTED ABOVE

Design flexural capacity using the NZTA research report $M_{Rd\_NZTA} := M_{Rd} = 1859 \cdot kN \cdot m$

Design flexural capacity using AS 5100.6 $M_{Rd\_AS} = 5880 \cdot kN \cdot m$

Ratio of capacities between AS5100.6 and NZTA research report $\frac{M_{Rd\_AS}}{M_{Rd\_NZTA}} = 3.16$

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Example 3: 85% SHEAR CONNECTION ($n_{min} < n < 1.0$)

Part 1 : COMPUTATION OF SAGGING MOMENT CAPACITY USING THE NZTA RESEARCH REPORT

The steps outlined below follow the procedures outlined in Section 5.2 of the report, which are largely based on Table 7.1 of the NZTA Bridge Manual.

Step 1 - Carry out site inspection

This would be undertaken as per NZTA Bridge Manual guidelines.

Step 2 - Determine appropriate material strengths

The design material strengths for the bridge components were obtained from the bridge drawings. No deterioration in the material strengths was assumed.

Structural steel yield strength (Grade 50C) $f_y := 345\text{MPa}$

Structural steel ultimate tensile strength (Grade 50C) $f_u := 450\text{MPa}$

Concrete compressive strength $f_c := 25\text{MPa}$

Yield strength of slab reinforcement $f_s := 275\text{MPa}$

Density of steel $\gamma_s := 7850\dfrac{\text{kg}}{\text{m}^3}$

Unit weight of concrete $\gamma_{conc} := 24\dfrac{\text{kN}}{\text{m}^3}$

Steel young's modulus $E_s := 205\text{GPa}$

Concrete young's modulus (based on NZS3101)

$$E_c := 3320\cdot\sqrt{\frac{f_c}{\text{MPa}}\cdot 10^{-6}}\,\text{GPa} + 6900\text{MPa}$$

$$E_c = 23.5\cdot\text{GPa}$$

Step 2a - Identify types of the shear connectors and determine their ductility

For the bridge beams, 152x76x18 channel shear connectors are used with the following dimensions:

Height of channel $h_c := 152\text{mm}$

Width of channel $w_c := 76\text{mm}$

Flange thickness of channel $t_f := 9.14\text{mm}$

Web thickness of channel $t_w := 6.35\text{mm}$

Channel thickness considered $t_c := t_w$ $\quad t_c = 6.35\cdot\text{mm}$

Length of channel $l_c := 150\text{mm}$

Channel connectors are deemed to be ductile if $H/t_w/L > 0.124$ (Equation 5.2)

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$\text{DuctilityTest} := \left(\frac{h_c}{t_c}\right)\cdot\left(\frac{1}{l_c}\right)$ $\text{DuctilityTest} = 0.16\cdot\frac{1}{\text{mm}}$ which is greater than 0.124

Therefore, channel connectors are considered to be ductile.

Step 2b - Identify dimensions of concrete slab, steel sections and beam effective span for calculation of effective width of concrete flange

Effective bridge span (equal to simply supported span between bearings) $L_{ef} := 22\text{m}$

Concrete slab tributary width (consider inner beams) $b_{tributary} := 2.5\text{m}$

Concrete slab thickness $t_{slab} := 180\text{mm}$

Steel beam section height $h := 926.6\text{mm}$

Steel beam section width $w := 307.7\text{mm}$

Steel beam web thickness $tw := 19.5\text{mm}$

Steel beam flange thickness $tf := 32.0\text{mm}$

Step 2c - Calculate effective width of the concrete flange

Distance between outstand shear connectors (or length of channel connector) $b_0 := l_c = 150\cdot\text{mm}$

Geometric width of concrete flange on each side of the web $b_i := \frac{b_{tributary} - b_0}{2} = 1.18\times10^3\cdot\text{mm}$

Effective width of concrete flange on each side of the web (limited by geometric width) $\frac{L_{ef}}{8} = 2.75\,\text{m}$

$$b_{ei} := \min\left(\frac{L_{ef}}{8}, b_i\right)$$

$b_{ei} = 1175\cdot\text{mm}$ Note: in our example, this is the same for each side of the inner beam web

Total effective width $b_{eff} := b_0 + 2\cdot b_{ei}$ (Equation 5.3 which determines effective width at beam midspan)

$b_{eff} = 2.5\,\text{m}$

Note that equation 5.3 was used for two reasons (i) critical bending demand for a simply supported beam will be at midspan (ii) using equation 5.4 would yield a lower effective width value, a lower maximum potential compressive force in the concrete slab, and therefore potentially a higher degree of shear connectivity - therefore it is conservative to use equation 5.3

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Step 2d - Determine steel section classifications

In this step, using Section 5.1 of AS5100.6 (2004), the section is classified as either compact or non-compact. Note that universal beams are hot-rolled sections.

Top Flange

Flange outstand $b_{flange} := \frac{w - tw}{2}$

$b_{flange} = 144.1 \cdot mm$

Flange slenderness $\lambda_{e\_flange} := \frac{b_{flange}}{tf} \cdot \sqrt{\frac{f_y}{250MPa}}$

$\lambda_{e\_flange} = 5.29$

Flange yield slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ey\_flange} := 16$

Flange plasticity slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ep\_flange} := 9$

Flange slenderness ratio $\frac{\lambda_{e\_flange}}{\lambda_{ey\_flange}} = 0.33$

Web

Web outstand $b_{web} := h - 2 \cdot tf$

$b_{web} = 862.6 \cdot mm$

Web slenderness $\lambda_{e\_web} := \frac{b_{web}}{tw} \cdot \sqrt{\frac{f_y}{250MPa}}$

$\lambda_{e\_web} = 51.97$

Web yield slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ey\_web} := 115$

Web plasticity slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ep\_web} := 82$

Web slenderness ratio $\frac{\lambda_{e\_web}}{\lambda_{ey\_web}} = 0.45$

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Overall section

Web has higher slenderness ratio, therefore section slenderness properties are based on web

Section slenderness $\lambda_s := \lambda_{e\_web}$

$\lambda_s = 52$

Section plasticity limit $\lambda_{sp} := \lambda_{ep\_web}$

$\lambda_{sp} = 82$

Section yield limit $\lambda_{sy} := \lambda_{ey\_web}$

$\lambda_{sy} = 115$

As the section slenderness is less than the section plasticity limit, the section may be classed as being "compact"

Step 2e - Identify construction methods - propped or unpropped

The drawings do not provide any information on the construction methods used for the beams. Assume unpropped construction (conservative).

Step 3 - Identify critical section(s) of the main supporting members and the critical effect(s) on them

This step involves determining the maximum sagging moment in the composite beams under the evaluation load. This would involve undertaking a grillage analysis.

Typically for a simply supported bridge, maximum sagging moment would be at midspan. However as the bridge has a skew, the maximum moment will tend to be closer towards the obtuse corners. For the purposes of this example, will assume that maximum moment occurs at midspan of the beams.

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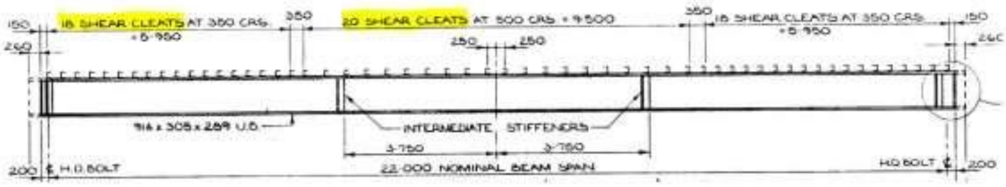
Step 3a - Determine the total number of shear connectors "n" between the locations of zero and maximum sagging moment



Number of shear connectors "n" $\quad n_{asbuilt} := 18 + 0.5 \cdot 20 \quad n_{asbuilt} = 28$

However for the purposes of this example consider a lower number of shear connectors $\quad n := 24$

(to examine case with $n_{min} < n < 1.0$)

Step 3b - Determine the degree of the shear connection "η" if the shear connectors are deemed to be ductile

Determining design resistance of shear connector (using NZS3404 Eqn 13.3.2.2)...

$$P_{Rk} := 31.2\left(t_f + 0.5 \cdot t_w\right) \cdot l_c \cdot \sqrt{\frac{f_c}{MPa}} \, MPa$$

$$P_{Rk} = 288 \cdot kN$$

$$\phi_v := 0.80$$

$$P_{Rd} := \phi_v \cdot P_{Rk}$$

$$P_{Rd} = 231 \cdot kN$$

Determining the design value of the compressive force in the concrete...

$$N_{c1} := n \cdot P_{Rd}$$

$$N_{c1} = 5533 \cdot kN$$

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Determining the design value of the compressive force in concrete slab when not limited by the shear connection...

Crossectional area of steel beam $A_s := (h - 2 \cdot tf) \cdot tw + 2 \cdot w \cdot tf$

$A_s = 36514 \cdot mm^2$

Strength reduction factor for steel $\phi := 0.9$ (AS 5100.6 Table 3.2)

Design yield strength of steel $f_{yd} := \phi \cdot f_y$

$f_{yd} = 310.5 \cdot MPa$

Strength reduction factor for concrete $\phi_c := 0.6$ (AS 5100.6 Table 3.2)

Design compressive strength of concrete $f_{cd} := \phi_c \cdot f_c$

$f_{cd} = 15 \cdot MPa$

Design compressive force in concrete with full shear connection $N_{cf} := \min\left(A_s \cdot f_{yd}, 0.85 \cdot f_{cd} \cdot b_{eff} \cdot t_{slab}\right)$

$N_{cf} = 5738 \cdot kN$

Therefore the degree of shear connection is $\eta := \dfrac{N_{c1}}{N_{cf}}$ (Equation 4.1)

$\eta = 0.96$ shear connection capacity governs as n is less than unity

Therefore $N_c := \min\left(N_{c1}, N_{cf}\right) = 5533 \cdot kN$

Step 3c - Determine the degree of the shear connection "η_{min}"

The minimum degree of shear connection n_{min} can be determined as shown below for steel sections with equal flanges.

$$\eta_{min} := \max\left[1 - \left(\frac{350MPa}{f_y}\right) \cdot \left(0.75 - 0.03\frac{L_{ef}}{m}\right), 0.4\right]$$ (Equation 4.2 since $L_{ef} < 25m$)

$\eta_{min} = 0.91$ therefore shear connection is ductile as $\eta > \eta_{min}$

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Step 4 - Determine the overload capacity and/or live load capacity at each critical main member section

The beams were classed to be "compact" with ductile shear connectors. As $\eta > \eta_{min}$, the shear connection is deemed to be ductile as per note 3 of Table 4.1. Also, as $\eta < 1$, partial shear connection can be achieved, so equations in section 4.3.3 of the report apply. Which equations to use depends on the location of the plastic neutral axis.

Depth of concrete slab $h_{cs} := t_{slab} = 180 \cdot mm$

$h_t := t_{slab} = 180 \cdot mm$ ignoring "pops"

The height of "pops" varies along the girder length from 50mm at support to 25mm at mid span. Inclusion of "pops" will marginally increase the bending capacity.

Depth of structural steel section $h_a := h = 926.6 \cdot mm$

Design value of plastic resistance of steel section to normal force $N_{pla} := A_s \cdot f_{yd} = 11337 \cdot kN$

Design compression resistance of the concrete flange $N_{plc} := 0.85 \cdot f_{cd} \cdot b_{eff} \cdot h_{cs} = 5738 \cdot kN$

Design plastic resistance of steel section flange $N_{plf} := w \cdot tf \cdot f_{yd} = 3057 \cdot kN$

Design plastic resistance of steel section web $N_{plw} := N_{pla} - 2 \cdot N_{plf} = 5223 \cdot kN$

Design compression resistance of the concrete flange considering degree of shear connection $\eta \cdot N_{cf} = 5533 kN$

As $\eta N_{cf} > N_{plw}$, plastic neutral axis in steel flange, equation 4.20 applies

Beam bending resistance

$$M_{plRd} := N_{pla} \cdot \frac{h_a}{2} + \eta \cdot N_{cf} \cdot \left(h_t - \frac{\eta \cdot N_{cf}}{N_{plc}} \cdot \frac{h_{cs}}{2} \right) - \frac{\left(N_{pla} - \eta \cdot N_{cf} \right)^2}{N_{plf}} \cdot \frac{tf}{4}$$

$$M_{plRd} = 5680 \cdot kN \cdot m$$

Step 5 onwards

No changes to Step 5 and onwards has been made by the report. Therefore following the end of Step 4 the engineer would follow the steps outlined in Table 7.1 of the NZTA Bridge Manual. i.e. undertake rating or posting analysis etc.

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Part 2 : COMPUTATION OF SAGGING MOMENT CAPACITY USING AS 5100.6

Steel section classification - compact or non-compact

As per AS5100.6 section 5.1

Top Flange

Flange outstand $b_{flange} = 144.1 \cdot mm$

Flange slenderness $\lambda_{e\_flange} = 5.29$

Flange yield slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ey\_flange} = 16$

Flange plasticity slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ep\_flange} = 9$

Flange slenderness ratio $\frac{\lambda_{e\_flange}}{\lambda_{ey\_flange}} = 0.33$

Web

Web outstand $b_{web} = 862.6 \cdot mm$

Web slenderness $\lambda_{e\_web} = 51.97$

Web yield slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ey\_web} = 115$

Web plasticity slenderness limit (From Table 5.1 AS5100.6) $\lambda_{ep\_web} = 82$

Web slenderness ratio $\frac{\lambda_{e\_web}}{\lambda_{ey\_web}} = 0.45$

Overall section

Web has higher slenderness ratio, therefore section slenderness properties are based on web

Section slenderness $\lambda_s = 51.97$

Section plasticity limit $\lambda_{sp} = 82$

Section yield limit $\lambda_{sy} = 115$

As the section slenderness is less than the section plasticity limit, the section may be classed as being "compact"

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Effective width of concrete flange

As per AS5100.6 section 4.4.1

For girders which have composite flanges on both sides of the web, effective width shall be the least of

one-fifth of the effective span length of the girder $b_1 := \frac{L_{ef}}{5} = 4.4\text{m}$

the distance centre-centre of girders $b_2 := 2.5\text{m}$

twelve times the least thickness of the slab $b_3 := 12 \cdot t_{slab} = 2160 \cdot \text{mm}$

Effective width of concrete flange $b_{eff\_AS} := \min(b_1, b_2, b_3) = 2160 \cdot \text{mm}$

Degree of shear connectivity

Based on Equation 5.1 of the NZTA research report, the characteristic resistance of shear connectors

$$P_{Rk} = 288 \cdot \text{kN}$$

Considering a capacity reduction factor (Table 3.2) $\phi_s := 0.85$

The design value of the shear connector $N_{c1\_AS} := n \cdot \phi_s \cdot P_{Rk} = 5879 \cdot \text{kN}$

Determining the design value of the compressive force in concrete deck slab with full shear connection

Cross sectional area of steel beam $A_s = 36514 \cdot \text{mm}^2$

Design strength of structural steel $f_{yd} = 310.5 \cdot \text{MPa}$

Design compressive strength of concrete $f_{cd} = 15 \cdot \text{MPa}$

Design compressive force in concrete with full shear connection

$$N_{cf\_AS} := \min(A_s \cdot f_{yd}, 0.85 \cdot f_{cd} \cdot b_{eff\_AS} \cdot t_{slab})$$

$$N_{cf\_AS} = 4957 \cdot \text{kN}$$

Degree of shear connector $\eta_{AS} := \frac{N_{c1\_AS}}{N_{cf\_AS}} = 1.19$

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Bending capacity of composite beam

As per AS5100.6 section 6.3 ->
For compact beam, section capacity (Ms) = plastic moment capacity (Mp)

As per AS5100.6 section 6.4 ->
For beams continuously restrained by deck at compression flange level, beam moment capacity (Mb) = section capacity (Ms)

Determining the plastic moment capacity (Ms).
As per Clause 6.3.3, simple plastic theory shall be used to determine Ms.

Thickness of composite concrete slab $t_{slab} = 180 \cdot mm$

Effective width of concrete slab $b_{eff\_AS} = 2.16 \cdot m$

Maximum possible compressive force in concrete slab $F_{slab\_AS} := 0.85 \cdot f_c \cdot b_{eff\_AS} \cdot t_{slab} = 8262 \cdot kN$

Force transferred to concrete slab via shear connectors $R_{cc\_AS} := \eta_{AS} \cdot F_{slab\_AS} = 9798 \cdot kN$

Depth of equivalent rectangular stress block $a_{c\_AS} := \dfrac{R_{cc\_AS}}{0.85 \cdot f_c \cdot b_{eff\_AS}} = 213.46 \cdot mm$

Area of steel beam cross-section $A_s = 36514 \cdot mm^2$

Maximum possible tensile force in steel beam $F_{beam} := A_s \cdot f_y = 12597 \cdot kN$

Compressive force in steel beam
(As $F_{beam} > F_{slab}$) $R_{sc\_AS} := \dfrac{A_s \cdot f_y - R_{cc\_AS}}{2} = 1400 \cdot kN$

Area of top flange $A_{flange} := w \cdot tf = 9846 \cdot mm^2$

Thickness of flange in compression
(assuming web is not in compression) $t_{fcomp\_AS} := \dfrac{R_{sc\_AS}}{f_y \cdot w} = 13.18 \cdot mm$

Less than flange thickness (32mm)
so assumption valid

Thickness of top flange in tension $t_{ften\_AS} := tf - t_{fcomp\_AS} = 18.82 \cdot mm$

Area of top flange in tension $A_{flten\_AS} := t_{ften\_AS} \cdot w = 5789 \cdot mm^2$

$A_{flten\_AS} = 5789 \cdot mm^2$

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Depth of web $d_{web} := h - 2 \cdot tf = 862.6 \cdot mm$

Area of web of steel beam $A_{web} := d_{web} \cdot tw = 16821 \cdot mm^2$

Height of centroid of steel beam in tension

$$e_{1\_AS} := \frac{A_{flange} \cdot \frac{tf}{2} + A_{web} \cdot \left(tf + \frac{d_{web}}{2}\right) + A_{flten\_AS} \cdot \left(tf + d_{web} + \frac{t_{flten\_AS}}{2}\right)}{A_{flange} + A_{web} + A_{flten\_AS}} = 406 \cdot mm$$

Height of centroid of steel beam in compression $e_{2\_AS} := h - t_{fcomp\_AS} \cdot 0.5 = 920 \cdot mm$

Height of centroid of compression block in slab $e_{3\_AS} := h + t_{slab} - 0.5 \cdot a_{c\_AS} = 1000 \cdot mm$

Lever arm between steel area in tension and steel area in compression $L_{1\_AS} := e_{2\_AS} - e_{1\_AS} = 514 \cdot mm$

Lever arm between steel area in tension and compression block in slab $L_{2\_AS} := e_{3\_AS} - e_{1\_AS} = 594 \cdot mm$

Nominal moment resistance $M_{s\_AS} := R_{sc\_AS} \cdot L_{1\_AS} + R_{cc\_AS} \cdot L_{2\_AS}$

$M_{s\_AS} = 6536 \cdot kN \cdot m$

Strength reduction factor (Table 3.2) $\phi = 0.9$

Design moment resistance $M_{Rd\_AS} := \phi \cdot M_{s\_AS} = 5882 \cdot kN \cdot m$

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Part 3 : COMPARISON OF FLEXURAL CAPACITIES COMPUTED ABOVE

Design flexural capacity using the NZTA research report $M_{Rd\_NZTA} := M_{plRd} = 5680 \cdot kN \cdot m$

Design flexural capacity using AS 5100.6 $M_{Rd\_AS} = 5882 \cdot kN \cdot m$

Ratio of capacities between AS5100.6 and NZTA research report $\frac{M_{Rd\_AS}}{M_{Rd\_NZTA}} = 1.04$

Appendix G: Historic steel mechanical properties

G1 New Zealand Standards 1989–1997 references

Table G.1 List of steel standards referenced in NZS 3404 from 1989 to 1997

<table>
<tr><th>Year</th><th>Standard</th><th>Structural steel shall comply with:</th><th>Structural steel - weather resistant</th><th>Structural steel</th><th>Rolled steel for general structures</th><th>Weldable structure steels</th><th>Hot-rolled flat products</th><th>Hot-rolled products for non-alloy structural steel and their technical delivery conditions</th></tr>
<tr><td rowspan="3">1989</td><td rowspan="9">NZS 3404</td><td>NZ and Australian steel</td><td>AS 1205 – refer to table G.13</td><td>AS 1204 – refer to tables G.7 & 8</td><td colspan="2"></td><td rowspan="2">-</td><td rowspan="4">-</td></tr>
<tr><td>British steels</td><td rowspan="2">-</td><td>BS 4</td><td>-</td><td>BS 4360 – refer to table G.3</td></tr>
<tr><td>Japanese steels</td><td rowspan="2">-</td><td>JIS G 3101</td><td>JIS G 3106</td><td>JIS G 3193</td></tr>
<tr><td rowspan="3">1992</td><td>NZ and Australian Steel</td><td>AS 3678 – refer to table G.9</td><td colspan="2" rowspan="2">-</td><td>AS 1594 - refer to table G-10 & 11</td></tr>
<tr><td>British steels</td><td rowspan="2"></td><td>BS 4</td><td></td><td>BS EN 10 025 – refer to table G.6</td></tr>
<tr><td>Japanese steels</td><td rowspan="2">-</td><td>JIS G 3101</td><td>JIS G 3106</td><td>JIS G 3193</td><td rowspan="2">-</td></tr>
<tr><td rowspan="3">1997</td><td>NZ and Australian Steel</td><td>AS 3678 – refer to table G.9</td><td colspan="2" rowspan="2">-</td><td>AS 1594 - refer to tables G.10 & 11</td></tr>
<tr><td>British steels</td><td rowspan="2"></td><td>BS 4</td><td></td><td>BS EN 10 025 – refer to table G.6</td></tr>
<tr><td>Japanese steels</td><td>-</td><td>JIS G 3101</td><td>JIS G 3106</td><td>JIS G 3193</td><td></td></tr>
</table>

G2 British Standards 1906–2004

Table G.2 Historic British steel standards' mechanical properties up to 1979

Year	Standard	Thickness	Yield stress f_y	Tensile strength f_u	
			N/mm²	N/mm²	N/mm²
		mm	min	min	max
1906	BS 15			386.12	441.28
1912		–		386.12	455.07
1930	BS 15			386.12	455.07
1936	BS 15	–		386.12	455.07
1941	CF(15)7376	–		386.12	455.07
1948–1961	BS 15	≤19	247.04	386.12	455.07
		38	231.6	386.12	455.07
		>38	227.74	386.12	455.07
1934–1942	BS 548	≤31.75	355.12	510.23	592.97
	Bridge	44.45	339.68	510.23	592.97
	w/d 1965	57.15	324.24	510.23	592.97
		69.85	308.8	510.23	592.97
		>69.85	293.36	510.23	592.97
1941	BS 968	Mechanical properties the same as BS 548			
1941	Bridge	Same as BS 548			
1943	BS 968	≤19.05	324.24	482.65	565.39
		>19.05	293.36	455.07	537.81
1962		≤15.88	355.12	441.28	537.81
		31.75	347.4	441.28	537.81
		50.80	339.68	441.28	537.81

Year	Standard	Thickness	Yield stress f_y	Tensile strength f_u	
			N/mm^2	N/mm^2	N/mm^2
		mm	min	min	max
1968	BS 4360	This standard supersedes BS 15, BS 968, BS 2762, BS 3706. Four different ultimate tensile groups of steel were included.			
		-		358.54	-
				386.12	
				441.28	
				496.44	
1969		Metric units were issued without any technical alteration			
1972		The scope of specification was extended to include weathering steels			
1979		Weldable structure steel is currently under review			

CF(15) 7376: 1941 – War emergency revision to BS 15

Table G.3 Historic steel mechanical properties to BS 4360 from 1968 to 1990

Year	Grade	Min yield (N/mm^2)					Min tensile (N/mm^2)	Max tensile (N/mm^2)	Min elongation – on a gauge length mm		
		Thickness (mm)									
		≤16	16<x≤40	40<x≤63	63<x≤100	100<x≤150			80mm[(e)]	200mm[(f)]	$5.65\sqrt{S_0}$
1968	40A[3]	–				–			–	22%	25%
	40B	231.7	223.9	220.1	208.5		401.6	478.8		22%	25%
	40C	231.7	223.9[3]	220.1	208.5		401.6	478.8		22%	25%
	40D	262.6	247.1	239.4	223.9		401.6	478.8		22%	25%
	40E	262.6	247.1	239.4	223.9		401.6	478.8		22%	25%
	43A1[(c)]	–					432.4	509.7		20%	22%
	43A	247.1	239.44	231.7	216.2		432.4	509.7		20%	22%
	43B	247.1	239.44	231.7	216.2		432.4	509.7		20%	22%
	43C	247.1	239.44	231.7	216.2		432.4	509.7		20%	22%
	43D	278	270.3	254.8	239.4		432.4	509.7		20%	22%

Year	Grade	Min yield (N/mm^2)					Min tensile (N/mm^2)	Max tensile (N/mm^2)	Min elongation – on a gauge length mm		
		Thickness (mm)									
		≤16	16<x≤40	40<x≤63	63<x≤100	100<x≤150			80mm[e]	200mm[f]	$5.65\sqrt{S_0}$
	43E	278	270.3	254.8	239.4		432.4	509.7		20%	22%
	50A	-					494.2	617.8		18%	20%
	50B	355.2	347.5	339.8	324.3		494.2	617.8[a]		18%	20%
	50C	355.2	347.5	339.8	324.3		494.2	617.8[a]		18%	20%
	50D	355.2	347.5	339.8	–		494.2	617.8		18%	20%
1969	Metric units were issued without any technical alteration										
1972	The scope of specification was extended to include weathering steels										
1979	There were no alterations to mechanical properties of plates										
1986	40A	235	225	215	205	185	340	500	25%	22%	25%
	40B	235	225	215	205	185	340	500	25%	22%	25%
	40C	235	225[3]	215	210	185	340	500	25%	22%	25%
	40D	235	225	215	215	205	340	500	25%	22%	25%
	40EE	260	245	240	225	205	340	500	25%	22%	25%
	43A	275	265	255	245	225	430[g]	580[g]	23%	20%	22%
	43B	275	265	255	245	225	430[g]	580[g]	23%	20%	22%
	43C	275	265	255	245	225	430[g]	580[g]	23%	20%	22%
	43D	275	265	255	245	225	430[g]	580[g]	23%	20%	22%
	43EE	275	265	255	245	225	430[g]	580[g]	23%	20%	22%
	50A	355	345	340	325	305	490[h][i]	640[h][i]	20%	18%	20%
	50B	355	345	340	325	305	490[h][i]	640[h][i]	20%	18%	20%
	50C	355	345	340	325	305	490[h][i]	640[h][i]	20%	18%	20%
	50D	355	345	340	325	305	490[h][i]	640[h][i]	20%	18%	20%
	50DD	355	345	340	325	305	490[i][j]	640[i][j]	20%	18%	20%
	50EE	355	345	340	325	305	490	640	20%	18%	20%

Year	Grade	Min yield (N/mm^2)					Min tensile (N/mm^2)	Max tensile (N/mm^2)	Min elongation – on a gauge length mm		
		Thickness (mm)									
		≤16	16<x≤40	40<x≤63	63<x≤100	100<x≤150			80mm[e]	200mm[f]	$5.65\sqrt{S_0}$
	50F	390	390	-			490	640	20%	18%	20%
1990	40EE	260	245	240	225	205	340	500	25%	22%	25%
	43EE	275	265	255	245	225	430[g]	580[g]	23%	20%	22%
	50EE	355	345	340	325	305	490[h][i]	640[h][i]	20%	18%	20%
	50F	390	390	–			490	640	20%	18%	20%

[a] Min tensile strength 478.8N/mm² for material over 63.5mm

[b] Min yield stress values for material over 63.5mm thick to be agreed between the manufacturer and the purchaser

[c] Min yield stress 230N/mm² for material up to and including 19mm thick

[d] Min yield stress 247.1N/mm² for material up to and including 19.1mm thick

[e] Up to and including 9mm thick, 17% for grades 40A-43EE and 16% for grades 50A-50EE

[f] Up to and including 9mm thick, 16% for grades 40A-43EE and 15% for grades 50A-50EE

[g] Min tensile strength 410N/mm² for material over 100mm thickness

[h] Min tensile strength 480N/mm² for material between 63mm and including 100mm thickness

[i] Min tensile strength 460N/mm² for material over 100mm thickness

[j] Min tensile strength 480N/mm² for material between 63mm and including 100mm thickness

Table G.4 Historic steel mechanical properties to BS EN 10113-3:1993

Grade		Min yield (N/mm²)			Min tensile (N/mm²)	Max tensile (N/mm²)	Min elongation - on a gauge length mm
		Thickness (mm)					
According EN 10027-1 and ECISS IC 10	According EN 10027-2	≤16	16<x≤40	40<x≤63			$5.65\sqrt{S_0}$
S275M	1.8818	275	265	255	360	510	24%
S275ML	1.8819						
S355M	1.8823	355	345	335	450	610	22%
S355ML	1.8834						
S420M	1.8825	420	400	390	500	660	19%
S420ML	1.8836						
S460M	1.8827	460	440	430	530	720	17%
S460ML	1.8838						
40E	-	260	-		400	480	-
43E		275			430	240	
50E		355			490	620	
50F		390			490	620	
55C		450			550	700	
55E		450			550	700	
55F		450			550	700	

Table G.5 Historic steel mechanical properties to BS 7668: 1994

Grade	Min yield (N/mm²) Thickness (mm)			Min tensile N/mm2	Min elongation – on a gauge length mm
	≤12	12<x≤25[(a)]	25<x≤40		$5.65\sqrt{S_0}$
S345J0WPH	345	325	325	480	21%
S345J0WH	345	345	345	480	21%
S345GWH	345	345	345	480	21%

[(a)] Only circular hollow sections are available in thicknesses over 20mm

Table G.6 Historic steel mechanical properties to BS EN 10025-4: 2004

Grade		Min yield[a] (N/mm²)[b]						Tensile strength (N/mm²)[b]					Min elongation – on a gauge length mm
		Thickness (mm)						Thickness (mm)					
According EN 10027-1 and CR 10260	According EN 10027-2	≤16	16<x≤40	40<x≤63	63<x≤80	80<x≤100	100<x≤120[d]	≤40	40<x≤63	63<x≤80	80<x≤100	100<x≤120	5.65√S0[c]
S275M	1.8818	275	265	255	245	245	240	370 to 530	360 to 520	350 to 510	350 to 510	350 to 510	24%
S275ML	1.8819												
S355M	1.8823	355	345	335	325	325	320	470 to 630	450 to 610	440 to 600	440 to 600	430 to 590	22%
S355ML	1.8834												
S420M	1.8825	420	400	390	380	370	365	520 to 680	500 to 660	480 to 640	470 to 630	460 to 820	19%
S420ML	1.8836												
S460M	1.8827	460	440	430	410	400	385	540 to 720	530 to 710	510 to 690	500 to 680	490 to 660	17%
S460ML	1.8838												

[a] For plate, strip and wide flats with widths ≥ 600mm the direction transverse (t) to the rolling direction applies. For all other products the values apply for the direction parallel (l) to the rolling direction.

[b] 1MPa = N/mm2.

[c] For product thickness < 3mm for which test pieces with a gauge length of L_0 = 80mm shall be tested, the values shall be agreed at the time of the enquiry and order.

[d] For long products of thickness ≤ 150mm apply.

G3 Australian Standards 1972–1996

Table G.7 Historic steel mechanical properties to AS 1204:1972

Year	Standard	Grade	Min yield (N/mm²)						Min tensile (N/mm²)[b]	Max tensile (N/mm²)	Min elongation–- on a gauge length mm	
			Thickness (mm)[a]									
			≤8	8<x≤12	12<x≤20	20<x≤40	40<x≤50	50<x≤150			200mm[c]	$5.65\sqrt{S_0}$
1972	AS 1204	250 250 L0 and L15	280	260	250	230	230	230	410	-	20%	22%
		300 300 L0 300 L0 and L15	310	300	280	280	280	280	450		19%	21%
		350 350 L0 350 L0 and L15	360	360	340	340	340	330	480		18%	20%
		400 400 L0 400 L0 and L15	410	410	-				520		16%	18%
		500 500 L0 500 L0 and L15	480	-					550		14%	16%

[a] Tensile test requirements for plate over 150mm thick in grades 250, 300 and 350, over 12mm thick in grade 400 and over 10mm thick in grade 500 are subject to negotiation between the purchaser and manufacturer.

[b] The minimum tensile strength requirement does not apply to plates under 6mm thick for grade 250.

[c] The minimum elongation on 200mm gauge length for thicknesses up to and including 8mm is 16% for grades 250 and 300 and 15% for grades 350 and 400.

Table G.8 Historic steel mechanical properties to AS 1204:1980

Year	Standard	Grade	Min yield (N/mm²) Thickness (mm)(a)						Min tensile (N/mm²)(a)	Min elongation – on a gauge length mm
			≤8	8<x≤12	12<x≤20	20<x≤40	40<x≤80	80<x≤180		$5.65\sqrt{S_0}$
1980	AS 1204	200	200	200	–				300	24%
		250	280	260	250	240	240	230	410	22%
		250 L0 and L15	280	260	250	250	240	240	410	22%
		350	360	360	350	340	340	330	480	20%
		350 L0 and L15	360	360	350	340	340	330	480	20%

(a) The minimum tensile strength requirement does not apply to plates under 6mm thick for grade 250.

Table G.9 Historic steel mechanical properties to AS 3678 from 1990 to 1996

Year	Standard	Grade	Min yield (N/mm²) Thickness (mm)							Min tensile (N/mm²)(a)	Min elongation – on a gauge length mm
			≤8	8<x≤12	12<x≤20	20<x≤32	32<x≤50	50<x≤80	80<x≤180		$5.65\sqrt{S_0}$
1990	AS 3678	200	200	200	-					300	24%
		250(c)	280	260	250	250		240	230	410	22%
		250 L15	280	260	250	250		240	240	410	22%
		300 300 L15	320	310	300	280		280	280	430	21%
		350 350 L15	360	360	350	340		340	330	450	20%
		400 400 L15	400	400	380	360		-		480	18%
1996		200	200	200	-					300	24%

Year	Standard	Grade	Min yield (N/mm²) Thickness (mm)							Min tensile (N/mm²)[a]	Min elongation – on a gauge length mm
			≤8	8<x≤12	12<x≤20	20<x≤32	32<x≤50	50<x≤80	80<x≤180		$5.65\sqrt{S_0}$
		250[c]	280	260	250	250	250	240	230	410	22%
		250 L15	280	260	250	250	250	240	240	410	22%
		300 300 L15	320	310	300	280	280	270	260	430	21%
		350 350 L15	360	360	350	340	340	340	330	450	20%
		400 400 L15	400	400	380	360	360	360	-	480	18%
		450 450 L15	450	450	450	420	400	-		520	16%

[a] So is the cross sectional area of the test piece before testing

[b] Elongation need not be determined for floorplate

[c] For grade 250, the minimum tensile strength requirement does not apply to material under 6mm thick

Table G.10 Historic mechanical properties for extra formability, structural and weather-resistant grades for AS 1594

Year	Standard	Grade	Nominal thickness	Minimum upper yield stress[(a)]	Minimum tensile strength	Elongation, % min[(b)]		
						Gauge length (*Lo*)		
						mm		
			mm	MPa	MPa	50	80	200
1992	AS 1594	XF300	≥3	300	440	28	26	20
			>3			31	29	23
		XF400	≥3.5	380	460	25	23	18
			>3.5	360	440	25	23	18
		XF500	≤13	480	570	18	16	14
1997		XF300	≥3	300	440	28	26	20
			>3	300	440	31	29	23
		XF400	≤8	380	460	25	23	18
		XF500	≤8	480	570	18	16	14
2002		Same as AS 1594: 1997						

[(a)] If a product does not exhibit a well-defined yield point, the 0.2% proof stress should be determined (see AS 1391).

[(b)] L_o = Original gauge length of the test piece.

Table G.11 Historic mechanical properties for formability, structural and weather-resistant grades for AS 1594

Year	Standard	Grade[d]	Min yield (N/mm²)	Min tensile (N/mm²)	Min elongation (% as a proportion of gauge length)[a][b]					
					Thickness (mm)					
					≤3			>3		
					L_o=50	L_o=80	L_o=200	L_o=50	L_o=80	L_o=200
1992	AS 1594	HD1	(see note [c])	(see note [c])	-					
		HD2	(see note [c])	(see note [c])	30	28	20	34	32	22
		HD3	(see note [c])	(see note [c])	34	32	22	38	36	24
		HD4	200	320	36	34	24	40	38	26
		HD200	200	300	24	22	17	28	26	19
		HD250	250	350	22	20	16	26	24	17
		HD300	300	400	20	18	15	24	22	16
		HD300/1	300	430	20	18	15	24	22	16
		HD350	350	430	18	16	14	22	20	15
		HW350	340	450	-		15	-		15
		HD400	400	460	16	14	13	20	18	14
1997		HA 1	(see note [c]) 200	(see note [c]) 300	—					
		HA 3	200	300	34	32	22	36	34	24
		HA 4N	170	280	36	34	24	38	36	26
		HA 200	200	300	24	22	17	28	26	19
		HA 250, HU 250	250	350	22	20	16	26	24	17
		HA 300, HU 300	300	400	20	18	15	24	22	16

Year	Standard	Grade[d]	Min yield (N/mm²)	Min tensile (N/mm²)	Min elongation (% as a proportion of gauge length)[a][b]					
					Thickness (mm)					
					≤3			>3		
					L_o=50	L_o=80	L_o=200	L_o=50	L_o=80	L_o=200
		HA 300/1	300	430	20	18	15	24	22	16
		HA 350	350	430	18	16	14	22	20	15
		HW 350	340	450	-	-	15	-	-	15
		HA 400	380	460	16	14	13	20	18	14
2002		Same as AS 1594: 1997								

[a] L_o = Original gauge length of test piece.

[b] Elongation testing is not required for floorplate.

[c] For design purposes, yield and tensile strengths approximate those of structural grade HA200. For specific information contact the supplier.

[d] The letter 'D' indicates deoxidisation practice, which may be U, R or A

.

G4 Historical mechanical properties for grade 55 plates

Table G.12 Historic mechanical properties for grade 55 plates for BS 4360 1968–1990

Year	Standard	Grade	Min yield (N/mm²) Thickness (mm)				Min tensile (N/mm²)	Max tensile (N/mm²)	Min elongation - on a gauge length mm		
			≤16	16<x≤25	25<x≤40	40<x≤63			80mm	200mm	$5.65\sqrt{S_0}$
1968	BS 4360	55C	447.9	432.4	417	-	556.0	695.0	-	17%[a]	19%
		55E	447.9	432.4	417	401.6	556.0	695.0		17%[a]	19%
1972		55C	450	430	415	-	-	-		17% 2	119%
		55E	450	430	415	400				17% 3	219%
1979		55C	450	430	415	-				17%	19%
		55E	450	430	415	400				17%	19%
		55F	450	430	415	-				17%	19%
1986		55C	450	430	-	-	550	700	19%[b]	17%[c]	19%
		55EE	450	430	415	400	550	700	19%[b]	17%[c]	19%
		55F	450	430	415	-	550	700	19%[b]	17%[c]	19%
1990		55C	450	430	-	-	550	700	19%	17%	19%
		55EE	450	430	415	400	550	700	19%	17%	19%
		55F	450	430	415	-	550	700	19%	17%	19%

[a] Under 19.1mm thick, 16% for grades 40 and 43 and 15% for grades 50 and 55

[b] Up to and including 9mm thick, 17% for grades 40A–43EE and 16% for grades 50A–50EE

[c] Up to and including 9mm thick, 16% for grades 40A–43EE and 15% for grades 50A–50EE

G5 Mechanical properties for weather resistant steel plates

Table G.13 Historic steel mechanical properties for weather resistant steel plates BS 4360 from 1972 to 1994

<table>
<tr><th rowspan="3">Year</th><th rowspan="3">Standard</th><th rowspan="3">Grade</th><th colspan="5">Min yield (N/mm^2)</th><th rowspan="3">Min tensile (N/mm^2)</th><th colspan="2">Min elongation – on a gauge length mm</th></tr>
<tr><th colspan="5">Thickness (mm)[b]</th><th rowspan="2">200mm[a]</th><th rowspan="2">$5.65\sqrt{S_0}$</th></tr>
<tr><th>≤12</th><th>12<x≤25</th><th>25<x≤40</th><th>40<x≤63</th><th>63<x≤100</th></tr>
<tr><td rowspan="5">1972</td><td rowspan="17">BS 4360</td><td>WR50A1</td><td>340</td><td>325</td><td>325</td><td rowspan="2">-</td><td rowspan="2">-</td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td>WR50A</td><td>340</td><td>-</td><td>-</td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td>WR50B1</td><td>345</td><td>345</td><td>345</td><td>340</td><td>By agreement</td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td>WR50B</td><td>345</td><td>345</td><td>-</td><td rowspan="4">-</td><td rowspan="14">-</td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td>WR50C</td><td>345</td><td>345</td><td>345</td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td rowspan="6">1979</td><td>WR50A1</td><td>345</td><td>325</td><td>325</td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td>WR50A</td><td>345</td><td>-</td><td>-</td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td>WR50B1</td><td>345</td><td>345</td><td>345</td><td>340</td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td>WR50B</td><td>345</td><td>345</td><td>345</td><td>340</td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td>WR50C1</td><td>345</td><td>345</td><td>345</td><td>340</td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td>WR50C</td><td>345</td><td>345</td><td>345</td><td>340</td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td rowspan="3">1986</td><td>WR50A</td><td>345</td><td>325</td><td>325</td><td></td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td>WR50B</td><td>345</td><td>345</td><td>345</td><td>340</td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td>WR50C</td><td>345</td><td>345</td><td>345</td><td>340[b]</td><td>480</td><td>19%</td><td>21%</td></tr>
<tr><td rowspan="3">1994</td><td>S345J0WPH</td><td>345</td><td>325[c]</td><td>325</td><td rowspan="3">-</td><td>480</td><td></td><td>21%</td></tr>
<tr><td>S345J0WH</td><td>345</td><td>325[c]</td><td>345</td><td>480</td><td></td><td>21%</td></tr>
<tr><td>S345GWH</td><td>345</td><td>325[c]</td><td>345</td><td>480</td><td></td><td>21%</td></tr>
</table>

[a] Min elongation of 17% for material under 9mm

[b] Up to and including 63mm

[c] Only circular hollow sections are available in thicknesses over 20mm

Table G.14 Historic steel mechanical properties for weather resistant steel plates to AS 3678 from 1990 to 1996

<table>
<tr><th rowspan="3">Year</th><th rowspan="3">Standard</th><th rowspan="3">Grade</th><th colspan="5">Min yield (N/mm²)</th><th rowspan="2">Min tensile (N/mm²)</th><th rowspan="2">Min elongation – on a gauge length mm</th></tr>
<tr><th colspan="5">Thickness (mm)</th></tr>
<tr><th>≤8</th><th>8<x≤12</th><th>12<x≤20</th><th>20<x≤32</th><th>32<x≤50</th><th></th><th>$5.65\sqrt{S_0}$</th></tr>
<tr><td rowspan="2">1990</td><td rowspan="4">AS 3678</td><td>WR350/1</td><td rowspan="2">340</td><td rowspan="2">340</td><td rowspan="2">340</td><td rowspan="2" colspan="2">340</td><td rowspan="2">450</td><td rowspan="2">20%</td></tr>
<tr><td>WR350/ 1L0</td></tr>
<tr><td rowspan="2">1996</td><td>WR350/1</td><td rowspan="2">340</td><td rowspan="2">340</td><td rowspan="2">340</td><td rowspan="2">340</td><td rowspan="2">340</td><td rowspan="2">450</td><td rowspan="2">20%</td></tr>
<tr><td>WR350/ 1L0</td></tr>
</table>

Table G.15 Historic steel mechanical properties for weather resistant steel plates to AS 1205 from 1972 to 1980

<table>
<tr><th rowspan="3">Year</th><th rowspan="3">Standard</th><th rowspan="3">GRADE</th><th colspan="4">Yield stress N/mm²</th><th rowspan="3">Min tensile (N/mm²)</th><th colspan="2" rowspan="2">Min elongation – on a gauge length mm</th></tr>
<tr><th colspan="4">Thickness of material (t) mm</th></tr>
<tr><th>10</th><th>12</th><th>20</th><th>50</th><th>200mm</th><th>$5.65\sqrt{S_0}$</th></tr>
<tr><td rowspan="15">1972</td><td rowspan="16">AS 1205</td><td>WR350/1</td><td rowspan="10">-</td><td rowspan="2">-</td><td>340</td><td rowspan="2">-</td><td rowspan="5">480</td><td rowspan="5">18%</td><td rowspan="5">20%</td></tr>
<tr><td>WR350/1L0</td><td>340</td></tr>
<tr><td>WR350/2</td><td>340</td><td rowspan="2">-</td><td>340</td></tr>
<tr><td>WR350/2L0</td><td rowspan="2">-</td><td>340</td></tr>
<tr><td>WR350/2L15</td><td>340</td><td rowspan="11">-</td></tr>
<tr><td>WR400/1</td><td>410</td><td rowspan="10">-</td><td rowspan="5">520</td><td rowspan="5">16%</td><td rowspan="5">18%</td></tr>
<tr><td>WR400/1L0</td><td>410</td></tr>
<tr><td>WR400/2</td><td>410</td></tr>
<tr><td>WR400/2L0</td><td>410</td></tr>
<tr><td>WR400/2L15</td><td>410</td></tr>
<tr><td>WR500/1</td><td>480</td><td rowspan="5">-</td><td rowspan="5">550</td><td rowspan="5">14%</td><td rowspan="5">16%</td></tr>
<tr><td>WR500/1L0</td><td>480</td></tr>
<tr><td>WR500/2</td><td>480</td></tr>
<tr><td>WR500/2L0</td><td>480</td></tr>
<tr><td>WR500/2L15</td><td>480</td></tr>
<tr><td>1980</td><td colspan="8">Same as 1972 version</td></tr>
</table>

Appendix H: Summary of assessment procedure for bending capacity of existing New Zealand composite bridges with channel shear connectors

This appendix provides a stand-alone assessment procedure for design engineers without need to cross-reference other parts of this report.

Step 1: Carry out site inspection

Identify structural deterioration of the bridge structure, including, among others, the condition of the shear connectors.

Step 2: Determine appropriate material strengths

Refer section 3.4 and 3.5, appendix G of this report and section 7.3 of the *Bridge manual,* 3rd edition.

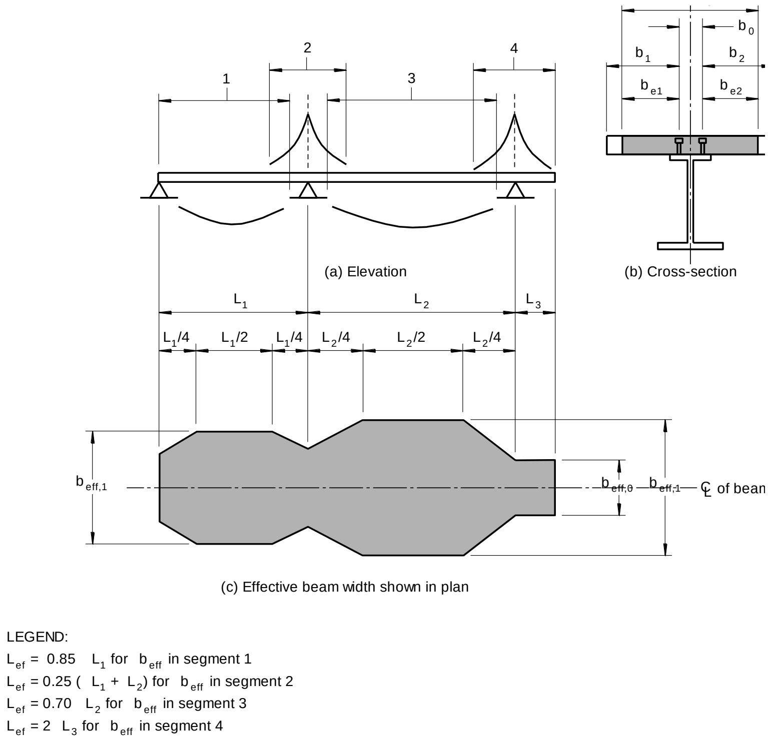
Step 2a: Identify types of the shear connectors and determine the ductility of the shear connectors

- Headed shear studs may be considered ductile when the overall height after welding is not less than four times the shank diameter d_{bs} and $16mm < d_{bs} \leq 25mm$.
- Channel shear connectors may be considered ductile if the channel unit web slenderness $H/t_w/L > 0.124$ (where H, t_w and L are the height, web thickness and length of the channel shear connectors, respectively).

Step 2b: Identify dimensions of the concrete slabs and steel sections and the bridge effective span for the calculation of the effective width of concrete flange

Use figure H.1 to calculate the effective bridge span L_{ef} for calculating the effective width of concrete slab. For a simply supported bridge, the effective span equals the effective bridge span.

Figure H.1 Equivalent spans L_{ef} for calculation of the effective width of concrete flange



Step 2c: Calculate effective breadth of the concrete flange

At mid-span or an internal support, the total effective width b_{eff} may be determined as:

$$b_{eff} = b_0 + \Sigma b_{ei} \qquad \text{(Equation H.1)}$$

Where:

b_0 = distance between the centres of the outstand headed stud connectors, or the width of the shear connector (when channel shear connectors are used).

b_{ei} = value of the effective width of the concrete flange on each side of the web and taken as $L_{ef}/8$ (but not greater than the geometric width b_i) The value b_i defines the distance from the outstand shear connector to a point mid-way between adjacent webs, measured at mid-depth of the concrete flange, except that at a free edge b_i is the distance to the free edge. L_{ef} is taken from step 2b.

The effective width b_{eff} at an end support is determined as:

$$b_{eff} = b_0 + \Sigma \beta_i b_{ei} \qquad \text{(Equation H.2)}$$

with

$$\beta_i = (0.55 + 0.025\ L_{ef}\ /\ b_{ei})$$

Where:

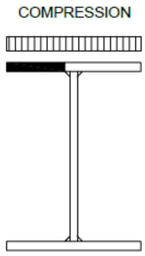
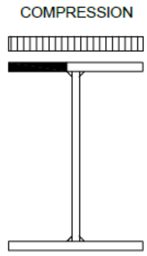
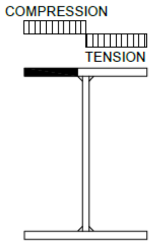
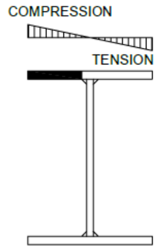
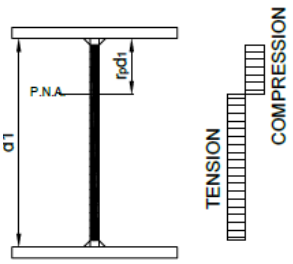
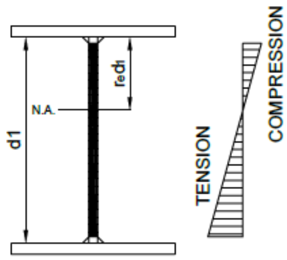
b_{ei} = effective width and L_{ef} is taken from step 2b

Step 2d: Determine steel section classifications

Any section damage observed from step 1 shall be considered. Use table 5.1 of AS 5100.6:2004 to determine the section classifications. The section is deemed to be compact if section slenderness $\lambda_e < \lambda_{ep}$ otherwise non-compact section should be considered.

The table is replicated below.

Table H.1 Values of plate element slenderness limits (table 5.1 of AS 5100.6:2004)

Plate element type	Longitudinal edges supported	Residual stress (a)	Plasticity limit, λep	Stress distribution	Yield limit, λey	Stress distribution
Flat	One	SR HR LW, CF HW	10 9 8 8		16 16 15 14	
(Uniform compression)						
Flat	One	SR HR LW, CF HW	10 9 8 8		25 25 22 22	
(Maximum compression at unsupported edge, zero stress or tension at supported edge)						
Flat	Both	Any	For 1.0 ≥ rp ≥ 0.5 $\frac{111}{4.7r_p - 1}$ For rp < 0.5 $\frac{41}{r_p}$ See note (b)		For 1.0 ≥ re ≥ 0 $\frac{60}{r_e}$ See note (c)	
(Web of beam with neutral axis not at mid height)						

SR = stress relieved; HR = hot-rolled or hot-finished; CF = cold-formed; LW = lightly welded longitudinally; HW = heavily welded longitudinally

Notes:

(a) Welded members whose compressive residual stress are less than 40MPa may be considered to be lightly welded.

(b) r_p is the ratio of the distance from the plastic neutral axis to the compression edge of the web to the depth of the web.

(c) r_e is the ratio of the distance from the elastic neutral axis to the compression edge of the web to the depth of the web.

Step 2e: Identify construction methods – propped or unpropped

Construction method may be identified from the as-built drawings. If the information is not available, it is reasonable to conservatively assume that unpropped construction was used.

Step 3: Identify critical section(s) of the main supporting members and the critical effect(s) on them

Incorporating information from steps 2b, 2c and 2d, establish and run global grillage analysis to obtain the location of the maximum sagging bending moment in the composite beams under evaluation load (eg 50MAX and HPMV) and the bending moment under critical vehicle loading located closer to the support.

Step 3a: Determine the total number of shear connectors *n* between the locations of zero and maximum sagging moment in the bending moment envelope

For simply supported bridges, locations of zero moment are at supports. For continuous beams, zero moment is located at either end supports (or points of contraflexture).

For continuous beams, the lesser number of the shear connectors from both the locations of zero moment (ie points of contraflexure or at supports, when simply supported) to the location of the maximum sagging moment should be used.

The location of the maximum sagging moment for both simply supported and continuous bridges may be determined from the bending moment envelope.

Step 3b: Determine the degree of the shear connection η at maximum sagging moment location if shear connectors are deemed to be ductile from step 2a.

$$\eta = N_c / N_{c,f} \qquad \text{(Equation H.3)}$$

Where:

N_c is the design value of the compressive force in the concrete given as nP_{Rd},
$N_{c,f}$ is the design value of the compressive force in the concrete with full shear connection (which is the lesser of $A_a f_{yd}$ and $0.85 f_{cd} b_{eff} h_t$)

n is the number of shear connectors from step 3a

A_a is the cross-sectional area of the steel beam

f_{yd} is the design yield strength of the steel ($f_{yd} = \phi f_y$), f_{cd} is the design compressive strength of the concrete ($f_{cd} = \phi_c f_c^{'}$). P_{Rd} is the design resistance of a shear connector ($P_{Rd} = \phi_V P_{Rk}$). ϕ=0.9, ϕ_C=0.6, ϕ_V=0.85 should be used.

For channel shear connectors:

$$P_{Rd} = \phi_V \cdot 31.2(t_{fsc} + 0.5 t_{wsc}) L_{sc} \sqrt{f_c^{'}} \qquad \text{(Equation H.4)}$$

Where ϕ_V=0.85:

t_{fsc} is the average flange thickness of the channel

t_{wsc} is the web thickness of the channel

L_{sc} is the length of the channel connector\ $f_c^{'}$ is the concrete cylinder strength.

Step 3c: Determine the minimum degree of shear connection $\eta_{\min}$ at maximum sagging moment location for shear connection ductility check if step 3b applies.

The minimum degree of shear connection η_{min} can be obtained from the following expression:

a) For steel sections with equal flanges:

$$L_{ef} \le 25m : \eta_{\min} = \max\left\{1 - \left(\frac{350}{f_y}\right)(0.75 - 0.03L_{ef}); 0.4\right\} \quad \text{(Equation H.5)}$$

$$L_{ef} > 25m : \eta_{\min} = 1.0$$

b) For steel section shaving a bottom flange with an area equal to three times the area of the top flange:

$$L_{ef} \le 20m : \eta_{\min} = \max\left\{1 - \left(\frac{350}{f_y}\right)(0.30 - 0.015L_{ef}); 0.4\right\} \quad \text{(Equation H.6)}$$

$$L_{ef} > 20m : \eta_{\min} = 1.0$$

Where:

L_{ef}, in metres, is obtained from step 2b.

For steel sections having a bottom flange with an area exceeding the area of the top flange (but less than three times that area), η_{min} may be determined by linear interpolation from equations H.5 and H.6.

Note that if η determined in step 3b is found to be lower than η_{min}**,** ductile shear connection identified from step 2a is deemed non-ductile in choosing the evaluation equations in table H.1.

Step 3d: Determine the total number of shear connectors *n* between the locations of zero and critical sections when the vehicle is closer to the supports

Repeat step 3a, but calculate the total number of shear connectors between zero moment and critical sections.

Step 3e: Determine the degree of the shear connections η by repeating step 3b but with the number of shear connectors *n* obtained from step 3d for the critical sections

Repeat steps 2b and 2c to derive the effective concrete area of the critical sections in order to use equation H.3.

Step 4: Determine the overload capacity and/or the live load capacity at each critical main member sections

Step 4a: Choose appropriate design curves

The options for design curves are sketched in figure H.2.

With the ductility of the shear connectors determined in step 2a, steel section classification determined in step 2d, degree of the shear connection obtained in step 3b, minimum degree of the shear connection determined η_{min} in step 3c and the construction methods identified in step 2e, the engineer can decide which assessment option in table H.1 to pursue. Relevant expressions of the equations are presented following the table.

Figure H.2 Design curves for composite beam bending capacity

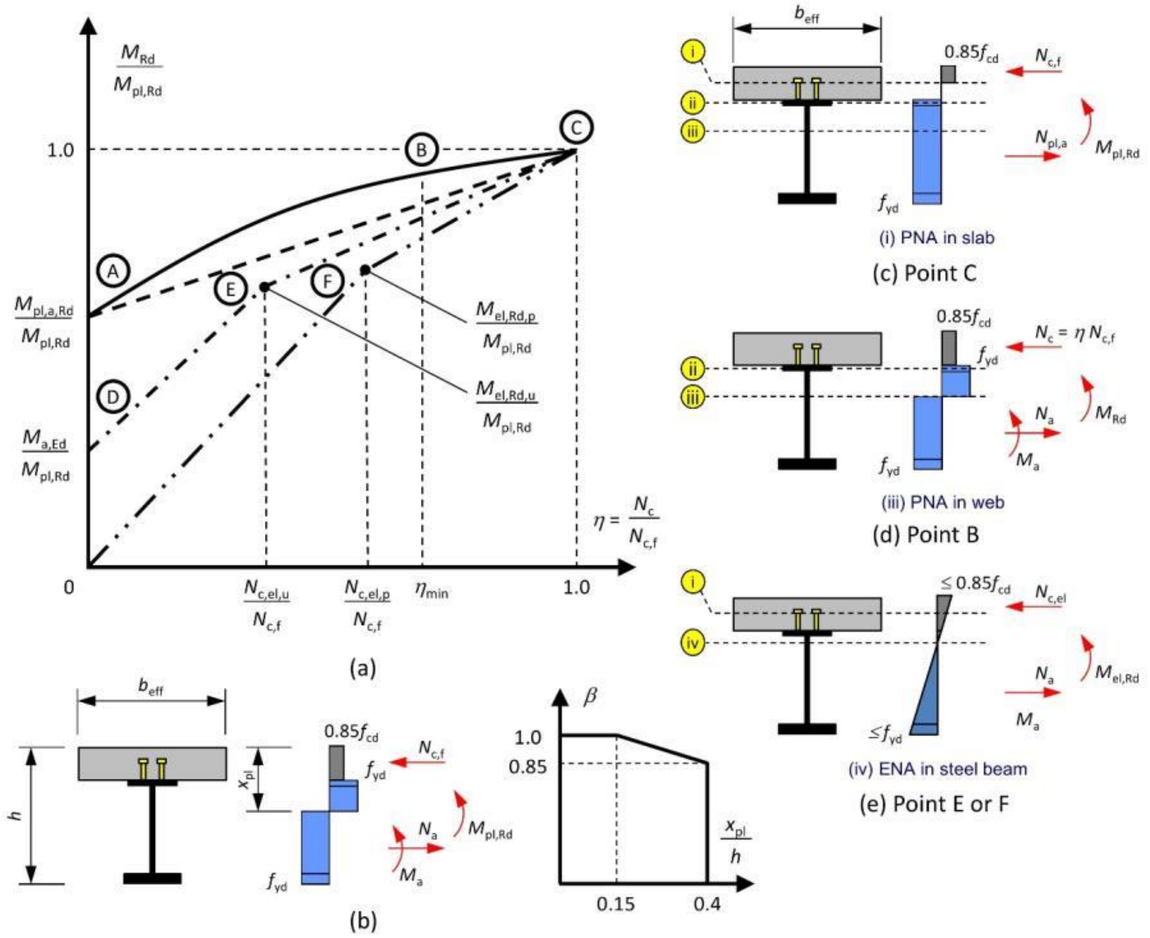
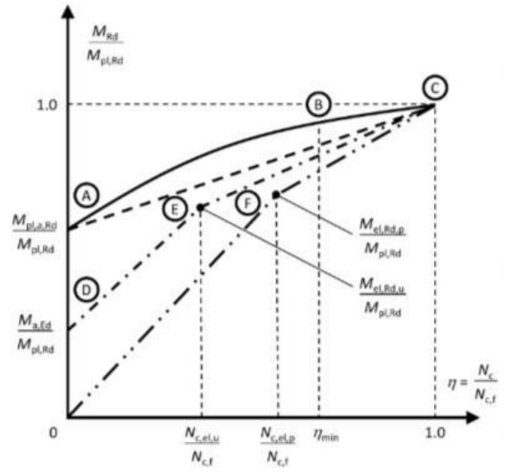


Table H.2 Assessment options for bending capacity of composite beams according to draft AS/NZS 2327



		Ductility of shear connectors			
		Ductile		**Non-ductile**	
		Options[b]	Equation no.	Options	Equation no.
Steel section classification	Compact[a]	A[d]	H.4	N/A	N/A
		C[d]	H.5 to H.16	C	H.5 to H.16
		ABC[c]	H.18 to H.20	DEC	H.25 to H.26
		AC[c]	H.21		
		DEC	H.25 to H.26		
		0FC	H.27 to H.28	0FC	H.27 to H.28
		DE	H.29	DE	H.29
		0F	H.30	0F	H.30
	Not-compact	C	H.5 to H.16	C	H.5 to H.16
		DEC	H.25 to H.26	DEC	H.25 to H.26
		0FC	H.27 to H.28	0FC	H.27 to H.28
		DE	H.29	DE	H.29
		0F	H.30	0F	H.30

Notes:

[a] The not-compact web may be treated as compact if the web section is reduced according to Cl.5.1.4 of AS 5100.6:2004.

[b] Assessment options are referred to in figure H.2.

- -Curve ABC – equilibrium method (rigid-plastic theory)
- -Line AC – simple interpolation method
- -Lines DEC/0FC – non-linear method (unpropped/propped construction)
- -Lines DE/0F – elastic method (unpropped/propped construction)

[c] These two options, i.e. Line ABC and Line AC but excluding point C, only apply when degree of shear connection η obtained in step 3b is larger than the minimum value, ie $\eta \geq \eta_{min}$ and when the steel section is compact. The remaining options in the table apply regardless of this limitation. The shear connection is deemed to be non-ductile if $\eta < \eta_{min}$ even if the individual connectors meet certain criteria and are considered ductile. When the steel beam is non-compact, similar rules to those for non-ductile shear connection apply.

[d] When points A and C are calculated, alternative equation H.4 and equations H.5~H.16 can be used respectively.

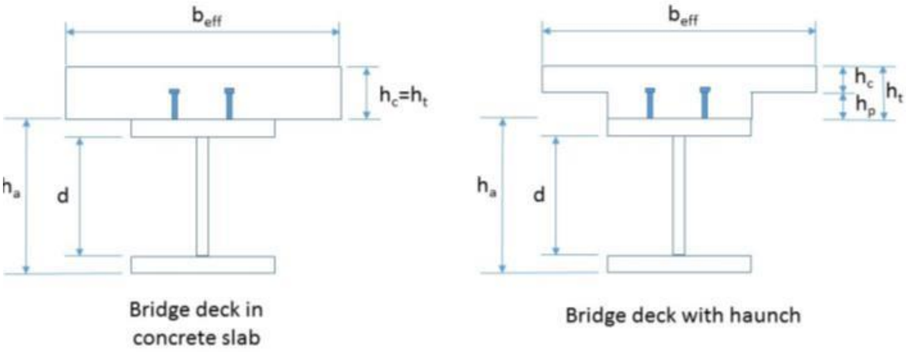
The equations in table H.2 are shown below together with the following supplementary definitions:

$N_{pl,a} = A_a f_{yd}$ Design value of the plastic resistance of the structural steel section to normal force.

$N_{pl,c} = 0.85 f_{cd} b_{eff} h_c$	Design compression resistance of the concrete flange.
$N_{pl,d} = t_w d f_{yd}$	Design value of the plastic resistance of the clear depth of the steel web to normal force.
$N_{pl,f} = b t_f f_{yd}$	Design value of the plastic resistance of the steel flange to normal force.
$N_{pl,n} = N_{pl,a} - N_{pl,d} + N_{pl,o}$	Design value of the plastic resistance of structural steel section with a class 3 web to normal force.
$N_{pl,o} = 60 \varepsilon t_w^2 f_{yd}$	Design value of the plastic resistance of the remaining portion of the compression part of a not-compact steel web to normal force. $\varepsilon = \sqrt{\frac{250}{f_y}}$
$N_{pl,w} = N_{pl,a} - 2N_{pl,f}$	Design value of the plastic resistance of the steel web to normal force.
$N_s = A_s f_{sd}$	Design value of the plastic resistance of the longitudinal steel reinforcement to normal force.
N_c	Compression force in the concrete determined by the capacity of the shear connectors nP_{Rd}
N_{cf}	Design value of the compressive normal force in the concrete flange with full shear connection.
$N_{c,el}$	Compressive force in the concrete flange corresponding to moment $M_{el,Rd}$.
b_{eff}	Effective breadth of the concrete flange
d	Clear depth of the steel web
f_{yd}	Design yield strength of steel ($f_{yd}=\phi f_y$)
f_{cd}	Design compressive strength of concrete ($f_{cd}=\phi_c f_c'$)
f_{sd}	Design yield strength of steel reinforcement ($f_{sd}=\phi f_{sy}$)
h_a	Depth of structural steel section
h_c	For profiled sheetings, the height of concrete slab above crests, ie hc=ht-hp. When solid slab is used, hc=ht.
h_p	The overall depth of the sheet excluding embossments; hp=0 when solid slab is used.
h_s	Distance from the top flange of the steel beam to the centroid of the longitudinal reinforcement in tension.
h_t	Overall depth of the concrete slab.

The geometric variables using the above definitions are presented graphically in figure H.3.

Figure H.3 Composite beam with basic variables



Non-composite beam design

***$M_{pl,a,Rd}$*, point A in figure H.2**

$$M_{pl,a,Rd} = Z_{ep} f_{yd} \qquad \text{(Equation H.7)}$$

Where:

Z_{ep} is the plastic modulus of the steel section alone or determined as per Cl.5.1.3 of AS 5100.6:2004.

Composite beams design with full shear connection

$M_{pl,Rd}$, point C in figure H.2

Sagging bending

Case 1: $N_{pl,c} < N_{pl,w}$ (plastic neutral axis in web – iii in figure H.2(c))

a) Cross-section is compact:

The depth of the plastic neutral axis, measured down from the extreme fibre of the concrete flange in compression, is given as (which may be used to classify the web according to table 4.1 of AS 5100.6:2004):

$$x_{pl} = h_t + \frac{h_a}{2} - \frac{N_{pl,c}}{N_{pl,d}}\frac{d}{2} \qquad \text{(Equation H.8)}$$

$$M_{pl,Rd} = M_{pl,a,Rd} + N_{pl,c}\frac{(h_a + h_t + h_p)}{2} - \frac{N_{pl,c}^{\ 2}}{N_{pl,d}}\frac{d}{4} \qquad \text{(Equation H.9)}$$

b) Cross-section with compact flanges, but with not-compact web that is further reduced to an effective cross-section into compact section with an effective web according to Cl.5.1.4 of AS 5100.6:2004:

$$M_{pl,Rd} = M_{pl,a,Rd} + N_{pl,c}\frac{(h_a + h_t + h_p)}{2} - \frac{\left(N_{pl,c}^{\ 2} + (N_{pl,d} - N_{pl,c})(N_{pl,d} - N_{pl,c} - 2N_{pl,o})\right)}{N_{pl,d}}\frac{d}{4} \qquad \text{(Equation H.10)}$$

Case 2: $N_{pl,c} \geq N_{pl,w}$ (plastic neutral axis in flange

a) $N_{pl,a} > N_{pl,c}$ (plastic neutral axis in steel flange – ii in figure H.2(c))

$$M_{pl,Rd} = N_{pl,a}\frac{h_a}{2} + N_{pl,c}\left(h_t - \frac{h_c}{2}\right) - \frac{(N_{pl,a} - N_{pl,c})^2}{N_{pl,f}}\frac{t_f}{4} \qquad \text{(Equation H.11)}$$

b) $N_{pl,a} \leq N_{pl,c}$ (plastic neutral axis in concrete flange – i in figure H.2(c)).

$$M_{pl,Rd} = N_{pl,a}\left(\frac{h_a}{2} + h_t - \frac{N_{pl,a}}{N_{pl,c}}\frac{h_c}{2}\right)$$ (Equation H.12)

Hogging bending

Case 1: Plastic neutral axis in web

a) Cross-section is compact, $N_s < N_{pl,w}$:

The depth of the plastic neutral axis, measured down from the extreme fibre of the concrete flange in tension, is given as (which may be used to classify the web according to table 5.1 of AS 5100.6:2004):

$$x_{pl} = h_t + \frac{h_a}{2} - \frac{N_s}{N_{pl,d}}\frac{d}{2}$$ (Equation H.13)

$$M_{pl,Rd} = M_{pl,a,Rd} + N_s\left(\frac{h_a}{2} + h_s\right) - \frac{{N_s}^2}{N_{pl,d}}\frac{d}{4}$$ (Equation H.14)

b) Cross-section with compact flanges, but with non-compact web which is further reduced to a compact cross-section with an effective web according to Cl.5.1.4 of AS 5100.6:2004: $N_s < N_{pl,o}$:

$$M_{pl,Rd} = M_{pl,a,Rd} + N_s\left(\frac{h_a}{2} + h_s\right) - \frac{\left({N_s}^2 + \left(N_{pl,d} + N_s\right)\left(N_{pl,d} + N_s - 2N_{pl,o}\right)\right)}{N_{pl,d}}\frac{d}{4}$$ (Equation H.15)

Where:

$M_{pl,a,Rd}$ is obtained from equation H.7.

Case 2: Plastic neutral axis in flange

a Cross-section is compact $N_s \geq N_{pl,w}$

i) Plastic neutral axis in steel flange $N_{pl,a} > N_s$:

$$M_{pl,Rd} = N_{pl,a}\frac{h_a}{2} + N_s h_s - \frac{\left(N_{pl,a} - N_s\right)^2}{N_{pl,f}}\frac{t_f}{4}$$ (Equation H.16)

ii) $N_{pl,a} \leq N_s$ (plastic neutral axis outside steel beam).

$$M_{pl,Rd} = N_{pl,a}\left(\frac{h_a}{2} + h_s\right)$$ (Equation H.17)

b) Cross-section with compact flanges, but with non-compact web, which is further reduced to a compact cross-section with an effective web according to Cl.5.1.4 of AS 5100.6:2004 $N_s \geq N_{pl,o}$:

i) Plastic neutral axis in steel flange $N_{pl,n} > N_s$:

$$M_{pl,Rd} = N_{pl,n}\frac{h_a}{2} + N_s h_s - \frac{\left(N_{pl,n} - N_s\right)^2}{N_{pl,f}}\frac{t_f}{4}$$ (Equation H.18)

ii) Plastic neutral axis outside steel beam $N_{pl,n} \leq N_s$

$$M_{pl,Rd} = N_{pl,n}\left(\frac{h_a}{2} + h_s\right)$$ (Equation H.19)

Composite beam with partial shear connection

M_{Rd} of any point along curve ABC in figure H.2 – equilibrium method (rigid-plastic theory)

For sagging bending

Case 1: $\eta N_{c,f} < N_{pl,w}$ (plastic neutral axis in web)

a) Cross-section is compact:

The depth of the plastic neutral axis, measured down from the extreme fibre of the concrete flange in compression, is given as (which may be used to classify the web according to table 5.1 of AS 5100.6:2004):

$$x_{pl} = h_t + \frac{h_a}{2} - \frac{\eta N_{c,f}}{N_{pl,d}}\frac{d}{2}$$ (Equation H.20)

$$M_{Rd} = M_{pl,a,Rd} + \eta N_{c,f}\left(\frac{h_a}{2} + h_t - \frac{\eta N_{c,f}}{N_{pl,c}}\frac{h_c}{2}\right) - \frac{(\eta N_{c,f})^2}{N_{pl,d}}\frac{d}{4}$$ (Equation H.21)

b) Cross-section with compact flanges, but with web being not-compact, which is further reduced to a compact cross-section with an effective web according to Cl.5.1.4 of AS 5100.6:2004:

$$M_{Rd} = M_{pl,a,Rd} + \eta N_{c,f}\left(\frac{h_a}{2} + h_t - \frac{\eta N_{c,f}}{N_{pl,c}}\frac{h_c}{2}\right) - \frac{\left((\eta N_{c,f})^2 + (N_{pl,d} - \eta N_{c,f})(N_{pl,d} - \eta N_{c,f} - 2N_{pl,o})\right)}{N_{pl,d}}\frac{d}{4}$$ (Equation H.22)

Where:

$M_{pl,a,Rd}$ is obtained in equation H.7.

Case 2: $\eta N_{c,f} \geq N_{pl,w}$ (plastic neutral axis in steel flange)

$$M_{Rd} = N_{pl,a}\frac{h_a}{2} + \eta N_{c,f}\left(h_t - \frac{\eta N_{c,f}}{N_{pl,c}}\frac{h_c}{2}\right) - \frac{(N_{pl,a} - \eta N_{c,f})^2}{N_{pl,f}}\frac{t_f}{4}$$ (Equation H.23)

M_{Rd} of any point along line AC in figure H.2 – simple interpolation method

$$M_{Rd} = M_{pl,a,Rd} + \eta(M_{pl,Rd} - M_{pl,a,Rd})$$ (Equation H.24)

Where:

$M_{pl,a,Rd}$ is obtained from equation H.7

$M_{pl,Rd}$ is obtained from equations H.8 to H.19.

Composite beam with non-linear design

$M_{a,Ed}$, point D in figure H.2 for unpropped construction method

$$M_{a,Ed} = Z_e f_s \quad \text{(Equation H.25)}$$

Where:

Z_e is the steel effective elastic section modulus

f_s is the stress at extreme fibre of the steel section under construction load.

$M_{el,Rd}$, point E in figure H.2 for unpropped construction method

$$M_{el,Rd} = M_{a,Ed} + kM_{c,Ed} \quad \text{(Equation H.26)}$$

$M_{el,Rd}$, point F in figure H.2 for propped construction method

$$M_{el,Rd} = kM_{c,Ed} \quad \text{(Equation H.27)}$$

Where:

$M_{a,Ed}$ is obtained from equation H.25

$M_{c,Ed}$ is the part of the design bending moment applied to the composite section.

k is the lowest factor such that any of the design stress limit f_{cd}, f_{yd}, f_{sd} can be reached for concrete in compression, structural steel in tension or compression and in reinforcement in tension or compression respectively.

M_{Rd} of any point along line DEC in figure H.2 for unpropped construction method – non-linear method

$$M_{Rd} = M_{a,Ed} + \left(M_{el,Rd} - M_{a,Ed}\right)\frac{N_c}{N_{c,el}} \text{ for } N_c \le N_{c,el} \quad \text{(Equation H.28)}$$

$$M_{Rd} = M_{el,Rd} + \left(M_{pl,Rd} - M_{el,Rd}\right)\frac{N_c - N_{c,el}}{N_{c,f} - N_{c,el}} \text{ for } N_{c,el} \le N_c \le N_{c,f} \quad \text{(Equation H.29)}$$

Where:

$M_{a,Ed}$ is obtained from equation H.25

$M_{el,Rd}$ is obtained from equation H.26.

$M_{pl,Rd}$ is derived from equations H.8 to H.18.

M_{Rd} of any point along line 0FC in figure H.2 for propped construction method – non-linear method

$$M_{Rd} = M_{el,Rd}\frac{N_c}{N_{c,el}} \text{ for } N_c \le N_{c,el} \quad \text{(Equation H.30)}$$

$$M_{Rd} = M_{el,Rd} + \left(M_{pl,Rd} - M_{el,Ed}\right)\frac{N_c - N_{c,el}}{N_{c,f} - N_{c,el}} \text{ for } N_{c,el} \le N_c \le N_{c,f} \quad \text{(Equation H.31)}$$

Where:

$M_{el,Rd}$ is obtained from equation H.27.

$M_{pl,Rd}$ is derived from equations H.8 to H.18.

M_{Rd} of any point along line DE in figure H.2 for unpropped construction method – elastic method

$$M_{Rd} = M_{a,Ed} + \left(M_{el,Rd} - M_{a,Ed}\right)\frac{N_c}{N_{c,el}} \text{ for } N_c \le N_{c,el} \qquad \text{(Equation H.32)}$$

M_{Rd} of any point along line 0F in figure H.21 for propped construction method – elastic method

$$M_{Rd} = M_{el,Rd}\frac{N_c}{N_{c,el}} \text{ for } N_c \le N_{c,el} \qquad \text{(Equation H.33)}$$

Step 4b: Determine load capacity at critical sections

With assessment options determined in step 4a, use the section properties such as Z_e, $M_{pl,Rd}$, $N_{c,f}$ etc of each individual critical section in determining the section capacity. In particular, degree of shear connection η (obtained from step 3b and step 3d), should be used respectively for the sections at maximum bending moment and critical sections under vehicle loading.

To determine the design capacity, strength reduction factors have to be properly assigned to each material component in the design equations, where the strength reduction factors φ=0.9, φ_c=0.6, φ_v=0.85 for steel, concrete and shear connectors, respectively as per table 3.2 of AS 5100.6:2004.

Steps 5 to 11

No amendments are made to these steps in the *Bridge manual* and are therefore omitted here.