

Interface Shear Behavior of Precast Prestressed Concrete Beams with Slender Webs in Beam and Slab Bridge Construction

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Declaration

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Abstract

All over the world precast prestressed concrete bridge beams are commonly used in small to medium sized highway and railway bridges, i.e. 20-50m spans. The majority of these bridges are built using I and U shaped precast prestressed concrete bridge girders with slender webs. When combined with a cast insitu deck slab they form a composite section that must be capable of resisting the horizontal shearing forces developed at the interface between the two elements. Interface shear requirements are generally more onerous than vertical shear when determining reinforcement ratios and this has become quite a contentious issue for bridge engineers in recent years with differing clauses in the design codes. This thesis presents work which examines the performance of thirteen 3.2m long reinforced concrete T-beams and seven 3.3m long beams dynamically and statically tested using a four point loading set-up with varying load and reinforcement ratios crossing the interface. The interface roughness and concrete strength does not vary as the parameters chosen are synonymous with bridge construction. The study also investigates the appropriateness for setting the cohesion factor c equal to zero for fatigue and dynamic verifications as per clause 6.2.5(105) of Eurocode 2 Part 2 (2005). The aim of the research was to propose a minimum interface shear reinforcement equation for incorporation into Eurocode 2 Part 1-1 (2004). It is recognised that a minimum percentage of reinforcement is required across the interface to prevent interfacial slip and brittle failure at loss of adhesion. Eurocode 2 Part 1-1 (2004) does not presently include a minimum requirement.

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Chapter 1

Introduction

This report presents research carried out on the horizontal shear capacity of composite concrete beams with varying amounts of reinforcement crossing the interface. Phase 1 of the research study looked at reinforced concrete composite bridge beams statically loaded and phase 2 of the research study examined prestressed concrete composite bridge beams dynamically and statically loaded. In both cases the composite beams comprised a precast beam web and a cast-in-place concrete top slab. The top surface of the beams was roughened to an average depth exceeding 3mm. The beams were manufactured with C57/70 grade concrete and the insitu top slabs were manufactured with C40/50 grade concrete 28 days later.

This chapter presents an overview of the problems facing today's bridge engineers and precast bridge beam manufacturers along with the research objectives and organisation of the report.

1.1 Problem Description and Motivation

All over the world precast prestressed concrete bridge beams are commonly used in small to medium sized highway and railway bridges, i.e. 20m-50m spans. Over the past ten years the majority of these bridges in Ireland and the United Kingdom have been built using W-beams, a U shaped precast prestressed concrete bridge beam developed by Banagher Precast Concrete in 2005. When combined with a cast insitu deck slab they form a composite section that must be able to resist the horizontal shearing forces developed at the interface between the two elements. Interface shear requirements are generally more onerous than flexural shear when determining reinforcement ratios and this has become quite a contentious issue for bridge engineers in recent years with differing clauses in the design codes. This report examines the performance of 13 reinforced concrete composite beams as well as 7 prestressed concrete composite beams dynamically and statically loaded using a four point loading test with varying concrete strengths and reinforcement ratios crossing the interface. The study compares full scale beam testing to the various European and American design code provisions along with finite element analysis using midasFEA, a computer software package. The aim of the study is to propose a minimum interface shear equation for incorporation into Eurocode 2 Part 1-1 (2004). It is recognised that a minimum percentage of reinforcement is required across the interface to prevent interfacial slip and brittle

failure at loss of adhesion. Eurocode 2 Part 1-1 (2004) does not presently include a minimum requirement. The finish on the cast against concrete surface is a very important factor in calculating the interface shear strength. Eurocode 2 Part 1-1 (2004) and the CEB-FIP Model Code 2010 (2010) classify the different applicable types of finishes into four categories, very smooth (cast against a steel or timber form), smooth (left as cast), rough (exposing the aggregate or raking the concrete) and very rough (shear key or indented concrete surface). Depending on the finish of the cast against concrete the two parameters, friction (μ) and cohesion (c) are then calculated. For a typical highway bridge spanning 20m constructed with prestressed W beams the interface shear links when designed to Eurocode 2 Part 1-1 (2004) can be at less than 100mm centres, the width of the interface can be as short as 250mm so placing concrete in and around these links is quite difficult, figure 1.1 shows the close proximity of these links pre and post beam manufacture.



(a) Interface Shear Links At Beam Ends
Prior to Concreting



(b) Interface Shear Links At Beam Ends
with Rough Surface Finish

Figure 1.1: Pre and Post Manufacture

Eurocode 2 Part 2 (2005) specifically states that the coefficient of friction should be taken as zero for fatigue or dynamic verification, this is nonsensical amongst bridge engineers as the coefficient of friction cannot be equal to zero and if it is, then the amount of interface shear reinforcement will increase considerably. Over the past ten years designers have been guestimating the coefficient of friction based on their own views as there is not enough information in the design codes specifically for precast prestressed bridge beams with cast on deck slabs. Secondly Eurocode 2 Part 1-1 (2004) does not have a minimum percentage of interface reinforcement, it is recognised that a minimum percentage of reinforcement is required across the interface to prevent interfacial slip and brittle failure at loss of adhesion. These are the issues that justify the importance of this thesis.

1.2 Research Objectives

Three major objectives have been defined for this research study: First to carry out an appraisal of present Eurocode 2 Part 1-1 (2004) requirements regarding interface shear, particularly regarding the interface roughness coefficients and comparison with other recognised design standards. Second to propose a minimum interface shear requirement

for Eurocode 2 Part 1-1 (2004) based on the testing undertaken. Eurocode 2 Part 1-1 (2004) does not presently include a minimum requirement. The proposal will involve a review of the other design standards and the database of test results along with experimental testing as part of the project. Recent bridge projects in Ireland and the United Kingdom have used the CEB-FIP Model Code 2010 (2010) recommendations for minimum interface shear.

Thirdly from the conclusions reached with the testing undertaken, to propose a value for the coefficient of cohesion based on fatigue loading.

1.3 Organisation of Thesis

This thesis is structured into five chapters. In this first chapter, it is presented like so: An introduction to the research project, the problem description and motivation for the research, the objectives and the organisation of the thesis.

Chapter 2 includes a state of the art review on horizontal interface shear from 1960 to the present day, it also includes the design provisions for horizontal interface shear when designing bridge beams in Europe and the USA with a worked example comparing the differences of the major bridge design codes.

Chapter 3 describes the first phase of testing whereby reinforced concrete beams were tested. This chapter also shows the manufacture and surface preparation of the beams and slabs.

Chapter 4 describes the second phase of testing whereby prestressed concrete beams were tested. This chapter also shows the manufacture and surface preparation of the beams and slabs.

Chapter 5 presents the results and conclusions of the two phases of testing carried out in chapters 3 and 4.

Chapter 2

Interface Shear - A State of the Art Review

2.1 The shear Friction Theory

The shear friction theory is used in all of the major bridge design codes, for example BS5400-4 (1990), Eurocode 2 Part 1-1 (2004), CEB-FIP Model Code 2010 (2010), ACI 318-14 (2014) and AASHTO LRFD-BDS (2007) for designing the reinforcement crossing the horizontal interface between concrete members cast at different times. In this case a precast prestressed concrete bridge beam with a cast insitu top slab. Figure 2.1 shows a rectangular concrete block with a cracked interface which has also slipped, this is known as the saw-tooth model and was initially developed around 1960 to predict the longitudinal shear strength of concrete interfaces with the following four parameters considered:

1. Compressive strength of the weakest concrete, f_{ck}
2. Normal stress at the interface, σ_n
3. Shear reinforcement crossing the interface, A_{vf}
4. Roughness of the interface substrate. Both cohesion, c , and friction, μ , factors are used depending on the surface roughness which varies from very smooth to indented.

Birkeland and Birkeland (1966) stated that the external shearing loads tend to produce a slippage of the member along an axis, usually at an interface. This slippage is resisted by friction resulting from the external clamping force. If reinforcement is present along the interface this reinforcement will provide an external clamping force on the concrete. This reinforcement has to be anchored on both sides of the interface for it to be effective. The reinforcement then becomes stressed in tension as a direct result of the slippage of the concrete, known as dowel action, this tension provides an external clamping force on the concrete resulting in compression across the interface, resulting in the shear stresses being transferred by shear friction.

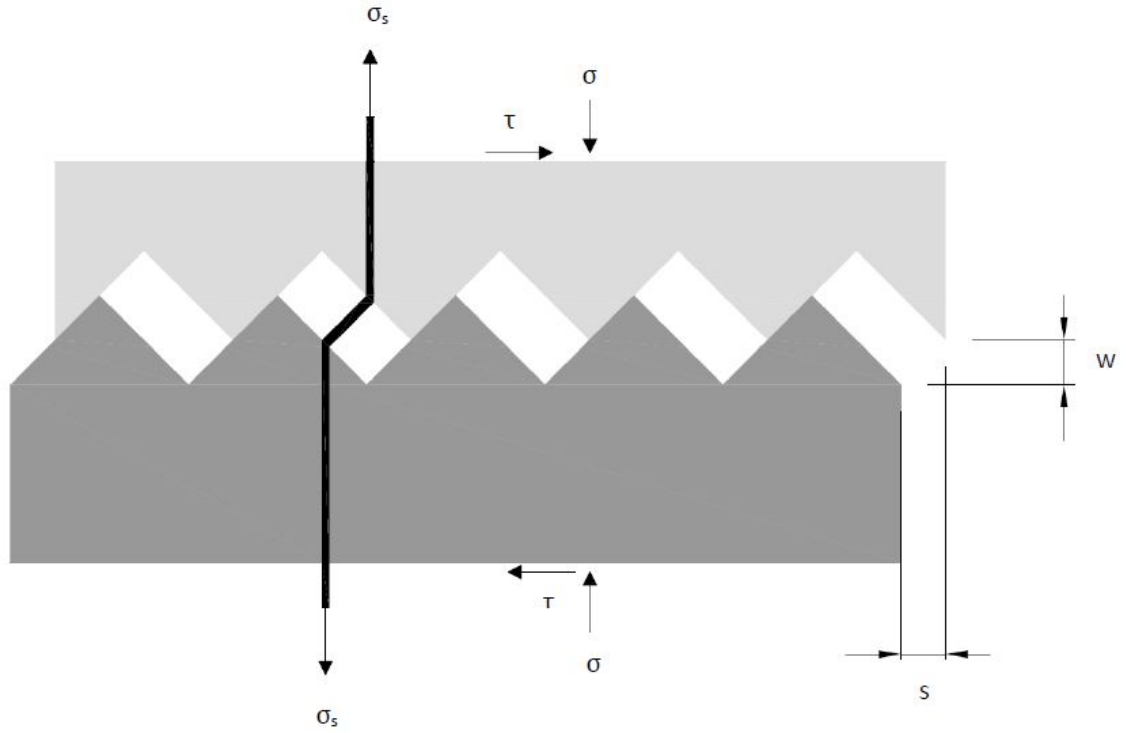


Figure 2.1: Shear Friction Theory (Santos and Julio (2014))

2.2 Horizontal Shear Stress Demand

The classical elastic method is the most common method used to calculate the horizontal shear stress at the interface of a composite beam. This method has been used since the 1960's and will be the primary equation used in this research. The elastic method for determining horizontal shear stress is

$$v_h = \frac{VQ}{Ib_v} \quad (2.1)$$

where;

v_h = horizontal shear stress at the interface (MPa)

V = interface shear force (kN)

Q = is the first moment of area of the cross section above (or below) the interface about the neutral axis, calculated as $Q = A\bar{y}$ (mm^3)

A = cross sectional area above the centroid of the composite section (mm^2)

$\bar{y}$ = distance from NA to the centroid of the area, A (mm)

I = the moment of inertia of the entire composite section (mm^4)

b_v = interface width (mm)

Figure 2.2 is the beam section used in the first phase of testing. A sample calculation for horizontal shear stress has been prepared and is shown below:

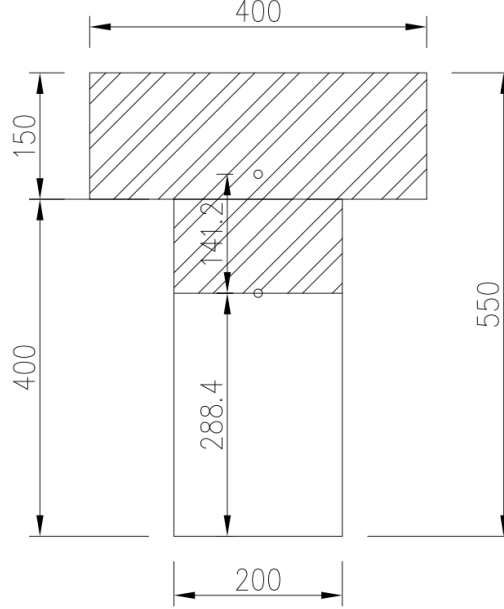


Figure 2.2: T-Beam Showing Dimensions used for Elastic Method Calculation. The shading represents the area above the centroid of the composite unit.

where;

$$A = 64410mm^2$$

$$Y_{(comp)} = 288.48mm$$

$$\bar{y} = 141.24mm$$

$$I_{(comp)} = 3.4155 \times 10^9 mm^4$$

$$Q = 9.10 \times 10^6 mm^3$$

$$V = 250kN$$

$$b_v = 200mm$$

$$v_h = 3.33MPa$$

2.3 Literature Review

The following section presents a state of the art review on horizontal interface shear from 1960 to the present day with particular emphasis on the evolution of the design formulae.

2.3.1 Anderson (1960)

Anderson (1960) tested several push-off specimens (Figure 2.3) and proposed an early design equation to determine the longitudinal shear stress at concrete interfaces. The test data confirmed that an adequate shear connection could be obtained by casting

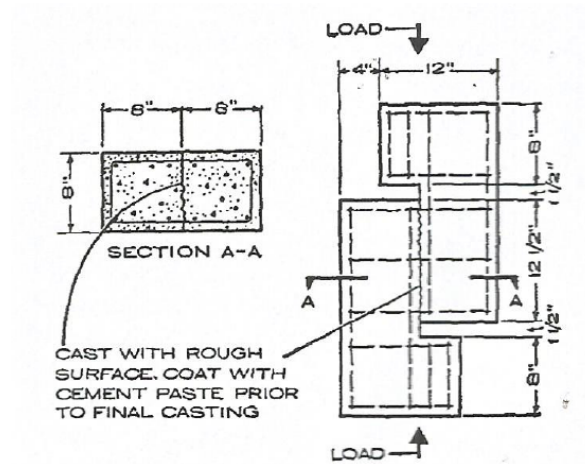


Figure 2.3: Push-Off Test Specimen (Anderson (1960))

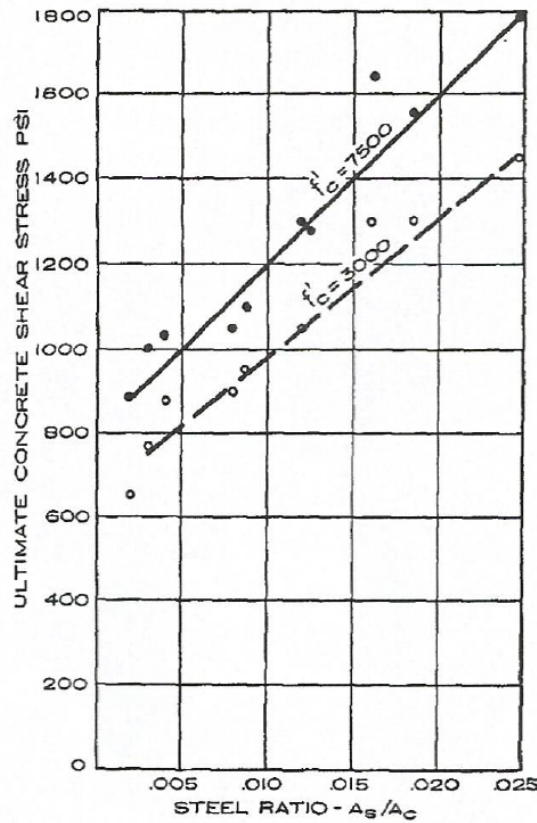


Figure 2.4: Ultimate Concrete Shear Strength v Steel Ratio (Anderson (1960))

a new concrete overlay onto an existing hardened concrete with a rough surface made up of approximately 6mm undulations and coated with a cement paste. The test data also proved that the ultimate shear strength of the joint varied with the amount of reinforced stirrups crossing the interface (Figure 2.4). The test data concluded that shear keys do not increase the shear strength of the joint if the concrete surface is rough.

Anderson (1960) came up with the following equation:

$$v_u = v_o + k\rho \quad (2.2)$$

where;

v_u = ultimate longitudinal shear stress at the interface, (MPa)

v_o = parameter experimentally obtained from push-off test specimens

k = parameter experimentally obtained from push-off test specimens

ρ = the reinforcement ratio

Anderson (1960) tested two concretes, a 20.68 MPa(3000psi) and a 51.71 MPa(7500psi). Therefore two design equations are proposed:

For 20.68 MPa Concrete:

$$v_u = 4.41 + 229\rho \quad (2.3)$$

For 51.71 MPa Concrete:

$$v_u = 5.52 + 276\rho \quad (2.4)$$

2.3.2 Hanson (1960)

Hanson (1960) was one of the first to examine the composite action between precast prestressed concrete bridge beams with cast on insitu deck slabs. He carried out tests on 62 push-off specimens (Figure 2.5) and 10 T-shaped girders (Figure 2.6) to represent bridge deck construction. Adhesive bond, roughness, indented keys and percentage of reinforcement area crossing the interface were the authors test variables. All composite test beams and cast on slabs had compressive strengths of 35 MPa and 20 MPa respectively.

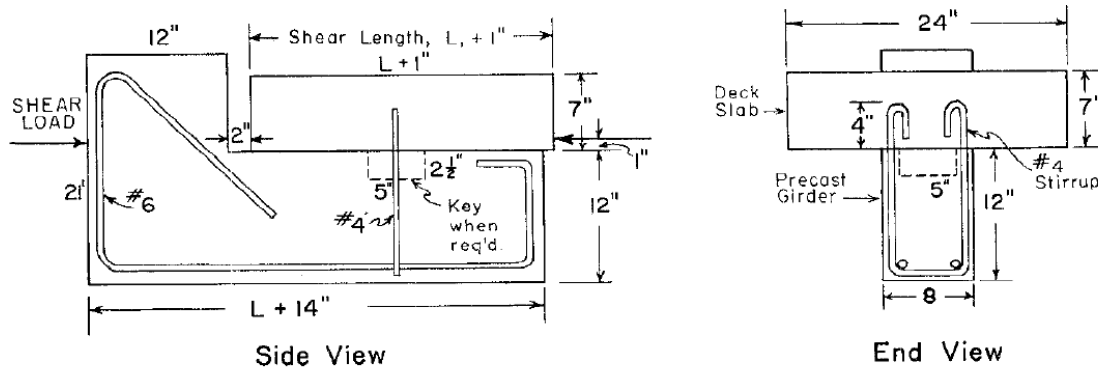


Fig. 1 — Pushoff Specimen in Casting Position.

Figure 2.5: Push-off Specimen in Casting Position (Hanson (1960))

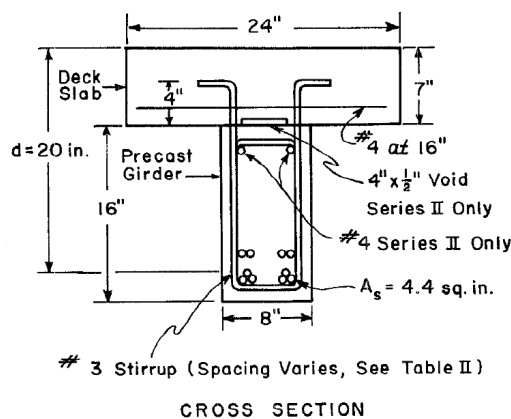


Figure 2.6: Cross Section of Girder Test Specimen (Hanson (1960))

Hanson (1960) proposed the following equation to calculate the longitudinal shear stress at the interface:

$$v_u = 3.45 + 121\rho \quad (2.5)$$

where;

v_u = ultimate longitudinal shear stress at the interface, (MPa)

ρ = the reinforcement ratio

The results of the tests carried out by Hanson (1960) showed that when the slip at the interface exceeded 0.127mm the composite action between the beam and cast on slab had ended. The author concluded that for precast prestressed bridge beams to act compositely with a cast on deck slab a combination of a rough bonded surface along with stirrups crossing the interface are required.

2.3.3 Nasser (1961)

Nasser (1961) carried out a series of tests on one precast prestressed composite bridge beam to check the beam shear behaviour under fatigue and static overload tests. One half of the beam had shear keys and the other had a rough as-cast finish. A fatigue test was carried out to check the shear stress at the interface under dynamic loading. The load used was slightly less than the design load to ensure a fatigue failure of the strand did not occur. 250 cycles per minute of load was used on the beam. The beam was dynamically loaded for 1,072,000 cycles and the results of the fatigue test showed that no slip occurred between the slab and the beam. Therefore the performance of the shear keys matched that of the ordinary rough concrete surface. Once the fatigue test was finished the beam was statically loaded to failure. The beam failed at 827kN, the design load was 205kN therefore the capacity of the beam was 300% greater than designed. During the load to failure test cracks appeared on the side of the beam with the rough as-cast finish first. Cracking did appear on the side of the beam with shear keys when the load was positioned on that side. Nasser (1961) moved the load along the beam to represent more closely bridge like conditions. The test results showed no real difference in performance between the surface with shear keys and the surface with an ordinary concrete finish. Therefore the conclusion was that there was ample interface shear reinforcement to transmit the shear stresses at the interface without the need for shear keys.

2.3.4 Birkeland and Birkeland (1966)

Birkeland and Birkeland (1966) came up with the following equation:

$$v_u = \rho f_y \tan \theta = \rho f_y \mu \quad (2.6)$$

where;

v_u = ultimate longitudinal shear stress at the interface, (MPa)

ρ = the reinforcement ratio

μ = 1.7 for monolithic construction, 1.4 for artificially roughened construction joints and 0.8 to 1.0 for ordinary construction joints and for concrete to steel interfaces

f_y = reinforcement yield strength, (MPa)

However, the above equation was limited and based on the following conditions;

$v_u \leq 5.52$ MPa

$f_c \leq 27.58$ MPa

$\rho \leq 1.5\%$

where;

f_c = compressive strength of concrete, (MPa)

Equation 2.6 was developed based on the results of Hanson (1960) push off tests, which were used to evaluate the effects of different surface finishes on the horizontal shear strength of the interface between a precast beam and a cast in situ deck slab. However, the effects of varying steel percentages were not investigated. Birkeland and Birkeland (1966) also used Mattock et al. (1960) test results which showed that with low levels of reinforcement, bond was dominant over stirrups. The authors also used data from Anderson (1960) tests, from which they noted that the reinforcement was well distributed and very well anchored.

2.3.5 Badoux and Hulsbos (1967)

Badoux and Hulsbos (1967) carried out a research project on 29 beams testing the strength of the interface joint between the precast beam and the cast in place slab. The test variables were: the amount of interface reinforcement, the surface roughness and the ratio of the shear span to effective depth of the beams. The main focus of the research was to determine the fatigue of the joint over a number of cycles of repeated load. Tests were carried out on 26 small scale reinforced concrete T-beams and 3 full scale precast prestressed bridge beams. The roughness of the surfaces of the beams varied between an intermediate finish, which was produced by applying a retarding agent to the concrete and removing it with a steel brush the following day leaving 3mm deep depressions in between the coarse aggregate and a rough finish. The rough finish resulted in 9.5mm depressions and this was produced by raking the surface with a board containing nails.

The beams were fatigue loaded with 2,000,000 cycles which took approximately 140 hours to perform. The load was applied at a rate of 250 cycles per minute. The authors defined fatigue failure as a joint slip greater than 0.254mm, (1/10 inch) for beams with

$\frac{a}{d} = 2.57$ and 3.86 respectively, where a = the distance from the support to the load and d = the effective depth to the reinforcement. The more heavily reinforced interfaces were also said to have failed when the number of joint cracks was at least equal to two-thirds the number of stirrups in the same shear span as the stirrups prevented the

joint from widening to 0.254mm even though the joint was seriously damaged and a visible loss of composite action was witnessed. Therefore joint failure occurred when the crack width spread to 0.254mm but also when substantial cracking was witnessed.

Badoux and Hulsbos (1967) stated that for beams with $\frac{a}{d} = 6.00$ and 7.71 the 0.254mm criterion was flawed and did not represent a joint failure or a loss of composite action and that the tests did not prove a definitive value for fatigue failure. The beams that did not fail during the fatigue test were subsequently statically loaded to failure. The interface cracking came as a direct result of loss of composite action which was initiated by the diagonal tension cracking of the precast beam itself. These cracks then spread horizontally along the joint towards the load point where they would form between stirrups if the stirrups centres were low.

Badoux and Hulsbos (1967) tests did prove that fatigue loading does in fact have a weakening effect on the joint interface. However, the number of tests was too limited to prove this conclusively. The test data did also prove that the results of the small scale beams gave conservative estimates for the full scale beams. Badoux and Hulsbos (1967) came up with an equation for the minimum allowable shear stress in the joint which incorporates a rough interface, the shear span and the reinforcement crossing the joint. From the tests the rough interface had significant additional shear capacity when compared to the intermediate surface finish.

$$v_u = \frac{3500}{11 + \frac{a}{d}} + 200\rho \quad (2.7)$$

where;

v_u = ultimate longitudinal shear stress at the interface, (MPa)

ρ = the reinforcement ratio

$\frac{a}{d}$ = the shear span

2.3.6 Mast (1968)

Mast (1968) compared the design of joints in precast concrete to the shear friction theory. The author stated that reinforcement was required across the interface and that the brittle tensile or shearing strength of the concrete should not be used to make the element ductile. Mast (1968) also emphasised the importance of anchoring the reinforcement both sides of the interface. He concluded that the shear friction theory provided adequate safety for the design of connections and used the equation of Birkeland and Birkeland (1966) but recommended an upper limit of $0.15f_c \tan \phi$.

$$v_u = \rho f_y \tan \phi \leq 0.15f_c \tan \phi \quad (2.8)$$

where;

v_u = ultimate longitudinal shear stress at the interface, (MPa)

$\rho = \frac{A_s}{bd}$, the reinforcement ratio

f_y = reinforcement yield strength, (MPa)

$\tan \phi$ or $\mu = 1.4$ to 1.7 for monolithic construction, 1.4 for a rough screeded interface and 0.7 to 1.0 for concrete cast against smooth or a steel interfaces respectively

f_c = concrete cylinder strength, (MPa)

2.3.7 Hofbeck et al. (1969)

Hofbeck et al. (1969) carried out a study into the shear transfer at the interface between a precast beam and a cast in place slab. 38 push-off specimens were tested, some with and some without a pre-existing crack along the shear plane. The author discussed past work by Gaston and Kriz (1964), Birkeland and Birkeland (1966) and Mast (1968). The author took a recommendation from Mast (1968) to look at already cracked concrete, e.g. placing a crack along a shear plane before the shear has been applied.

The study was to determine the influence of a pre-existing crack on the shear strength and to relate the shear transfer strength measured in push-off tests to the compressive and tensile strengths of concrete and steel.

The test data concluded that if a crack exists before the application of shear then the slip at all stages of loading will be greater than would have occurred if the crack had not been present. The presence of a crack also reduces the ultimate shear strength. The report has a summary of the effects of bar size and spacing, stirrup yield point, concrete strength, dowel action and the shear force friction theory. The test results show that dowel action of reinforcing bars crossing the interface is insignificant in initially uncracked concrete but is substantial in concrete with a pre-existing crack along the shear plane. The shear friction theory gave a reasonably conservative estimate of shear transfer strength when compared to the test results once a coefficient of friction μ , equal to 1.4 was used and providing ρf_y was less than f_c or 4.14 MPa.

2.3.8 Paulay et al. (1974)

Paulay et al. (1974) carried out tests on 36 specimens to study the contribution of dowel action, surface preparation and reinforcement content towards the shear strength of construction joints subject to monotonic and cyclic loading. Looking specifically at interface shear and in particular the factors affecting bond, dowel action and interface friction the author outlined the following:

Bond:

1. Wetting the surface of the joint before placing concrete lowers the bond
2. Cement grout or a mortar layer placed on the surface has no significant effect on bond
3. The age of concrete has little influence
4. Rough surfaces produce the greatest bond
5. The quality of bond is more important when tensile stresses are to be transferred across the joint

Dowel Action:

1. The capacity of dowel action increases with the bar diameter and concrete strength
2. Dowel strength may be derived from the following three mechanisms:

The flexure of the reinforcing bars

The shear across the bars

The kinking of the reinforcement

Out of the 36 specimens tested, 6 specimens were made identical and cast monolithically. The reinforcement content and surface preparation were the variables. The results for surface roughness showed that the units cast monolithically did not perform better than a composite construction. The results for the dowel action alone with a smooth surface were very poor resulting in only 16% of its capacity. The tests concluded that the concrete in the top layer should be of an inferior quality to that of the first pour. Several specimens were subjected to alternative static and cyclic loading of high intensity. These were insufficient to enable the collection of usable conclusions; however, they did show that even after maximum loads were attained the shear strength recorded was in excess of the design capacity in all bar one specimen, and in that one specimen it reached 90% of the design capacity.

2.3.9 Mattock et al. (1976)

Mattock et al. (1976) carried out 8 series of experimental studies on push-off test specimens with several variables considered:

1. Concrete strength
2. Surface preparation
3. Reinforcement ratio
4. Loading monotonic and cyclic

The specimens were subjected to a continuously increasing load up until failure with stoppages to mark and measure cracks. Each test took approximately fifteen minutes. Mattock et al. (1976) pre-cracked some of the specimens before the test so as to have uncracked and pre-cracked specimens with and without bond which would result in enough data to see the effectiveness of the reinforcement on the interface. The average width of this crack was 0.25mm.

Mattock et al. (1976) used the following simple equation to determine the nominal ultimate shear stress of the test specimens:

$$v_u = \frac{V_u}{A} \quad (2.9)$$

where;

V_u = Ultimate shear, (kN)

A = Area of shear plane, (mm²)

Mattock et al. (1976) found that if the interface is deliberately roughened for a composite member that this composite member behaves in a similar fashion to a monolithically constructed member with a pre-existing crack at the interface. Comparing the test results to available formulae Mattock et al. (1976) also proved that if the interface is smooth the shear is transferred across the joint mainly through the reinforcement and that where high shear stresses are expected smooth interfaces should be avoided.

Mattock et al. (1976) proposed a new Modified Shear Friction equation for calculating the shear stress of a deliberately roughened interface as follows:

$$v_u = 0.8\rho f_y + 2.76MPa \leq 0.2f_c \quad (2.10)$$

where;

v_u = ultimate longitudinal shear stress at the interface but limited to $0.2f_c$, (MPa)

ρ = the reinforcement ratio

f_y = reinforcement yield strength, (MPa)

2.3.10 Chung and Chung (1976)

Chung and Chung (1976) carried out an experimental investigation on 18 T-beam test specimens (Figure 2.7) to study the effects of repeated loading on the horizontal interface between a precast beam web and a cast in place insitu top slab. The variables were the amount of reinforcement crossing the interface and the intensity of the repeated loading. The T-beams were designed to fail in horizontal shear at the interface before their flexural capacity was exceeded. The 425mm x 50mm flange was poured approximately one month after the beams. The surface was prepared as rough. An aluminium strip was placed at the interface to ensure horizontal shear failure. The 18 beams were separated into 4 series. The first beam in each series served as the control specimen and was tested statically to failure to determine the ultimate strength. The deflection at midspan was measured through a dial gauge and the crack widths with an illuminated hand microscope. The beams were loaded with a minimum load of 10% of the ultimate load and the maximum load varied from 45-70%. The repeated loading was generated by a pulsator at 240 cycles per minute. The load was repeated until the beam failed in fatigue or for 1,000,000 cycles, whichever came first.

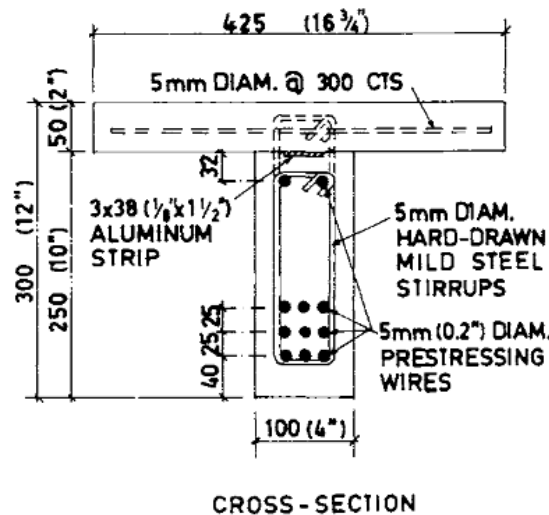


Figure 2.7: Details of Test Specimen (Chung and Chung (1976))

Of the 18 beams only 5 failed in fatigue. The maximum load on these beams was 58-65% of the ultimate. The horizontal shear stress induced at the interface was 2.78N/mm^2 to 3.15N/mm^2 . During the repeated loading test serious cracking was observed at the interface. The primary cause of failure was horizontal shear fatigue at the joint. The steel across the interface did increase the endurance of the joint as one of the beams in the series of tested beams without interface reinforcement failed after just 37,000 cycles of loading illustrating the brittle nature of the unreinforced joint. The results varied with some tested beams failing after 350,000 and 850,000 cycles even though they

contained interface shear reinforcement. Chung and Chung (1976) summarised that 18 test specimens was insufficient to arrive at an agreeable conclusion. The author did state that a repeated slip of 0.025mm may be considered as the critical value beyond which the efficiency of the joint rapidly deteriorates. This value is much smaller than the critical static slip value of 0.127mm deduced by Hanson (1960). Of the beams cyclically loaded beyond 55% of the static load strength the critical slip value of 0.025mm occurred before 100,000 cycles.

Chung and Chung (1976) concluded that fatigue failure of the interface joint is not probable if the repeated load is not more than 55% of the ultimate static load or if the horizontal shear stresses at the interface does not exceed 2.75N/mm^2 .

2.3.11 Hughes (1987)

Hughes (1987) carried out static and fatigue tests on 83 push-off specimens, 47 small reinforced concrete T-beams and 16 full scale M3 composite precast prestressed bridge beams using the formulae of BS5400-4 (1990). The study set out to define the precast surfaces in terms of measureable qualities instead of descriptive text and to investigate the effects of the surface preparation, the shear span ratio and the stirrups crossing the interface.

The full scale beam tests were carried out on 8m long M3 precast prestressed bridge beams with a 1m wide deck slab cast on top. BS5400-4 (1990) refers to the surface finishes as monolithic, composite type 1 and composite type 2. Hughes (1987) refers to these finishes as monolithic, roughened and rough as cast respectively. BS5400-4 (1990) requires a minimum percentage of interface shear links of 0.15%bh. This equates to 12mm links at 500mm centres for the M3 beam with a 300mm interface width. This level of stirrups gives an interface capacity of 1 N/mm^2 . Hughes (1987) recommends a practical minimum spacing of 100mm which equates to a percentage of interface shear links of 0.75% and a design resistance of 3 N/mm^2 .

Hughes (1987) removed the γ_m factor of 1.25 from the BS5400-4 (1990) basic design equation for the strength of the interface to give the following characteristic strength equation:

$$v_u = 1.25(v_1' + 0.7\rho f_s) \quad (2.11)$$

where;

v_u = ultimate longitudinal shear stress at the interface, (MPa)

v_1' = shear stress from Table 31 of BS5400-4 (1990) but based on actual concrete strength rather than concrete grade, (MPa)

$\phi = 0.85$, a capacity reduction factor

$$\rho = \frac{\text{stirrup area}}{\text{interface area}}$$

f_y = actual rather than characteristic reinforcement yield strength, (MPa)

The push-off tests carried out by Hughes (1987) resulted in differing minimum percentages of interface reinforcement compared to the 0.15% recommendation of BS5400-4 (1990). Simply, the amount of minimum interface stirrups should equate to 0.5 N/mm^2 , 0.85 N/mm^2 and 1.5 N/mm^2 for monolithic, roughened and rough as cast surfaces respectively. However, BS5400-4 (1990) deals with interfaces in beams and the push-off test specimens do not represent these beams accurately enough to increase the mini-

imum percentage of interface reinforcement from 0.15% (0.72 N/mm²) for the push-off test specimens.

Hughes (1987) found BS5400-4 (1990) to be conservative. The γ_m factor of 1.25 was in fact found to be correct for rough-as-cast (Type 2) surfaces but roughened (Type 1) surfaces had a safety factor of 1.6 and monolithic 2.0. These exceeded the recommended 1.25 so the code is over conservative for roughened and monolithic surfaces. The author also noted that service stresses in bridgeworks tended to be 2.0 N/mm² and that the cracking stresses of 3.0 N/mm², 2.5 N/mm² and 2.0 N/mm² for monolithic, roughened and rough as cast surfaces respectively proved that the interfaces would remain uncracked at service loads.

2.3.12 Walraven et al. (1987)

Walraven et al. (1987) carried out a large experimental study on 88 push-off test specimens to establish a new shear friction equation for determining the horizontal shear capacity at a cracked interface. This new equation allows for higher concrete strengths as 50N/mm² to 70N/mm² were now commonplace. The testing focused on the effect of static, dynamic and sustained shear loading on high strength concrete.

The test results showed that the following equation gave the most favourable shear friction capacity.

$$v_u = C_1(\rho f_y)^{C_2} \quad (2.12)$$

where;

v_u = ultimate longitudinal shear stress at the interface, (MPa)

$C_1 = 0.822 f_c^{0.406}$

ρ = the reinforcement ratio

f_y = reinforcement yield strength, (MPa)

$C_2 = 0.159 f_c^{0.303}$

f_c = concrete cylinder strength, (MPa)

Walraven et al. (1987) developed a design chart (Figure 2.8) for determining the shear friction capacity taking into account the compressive strength of the concrete. This was to overcome the complexity of using the above equation.

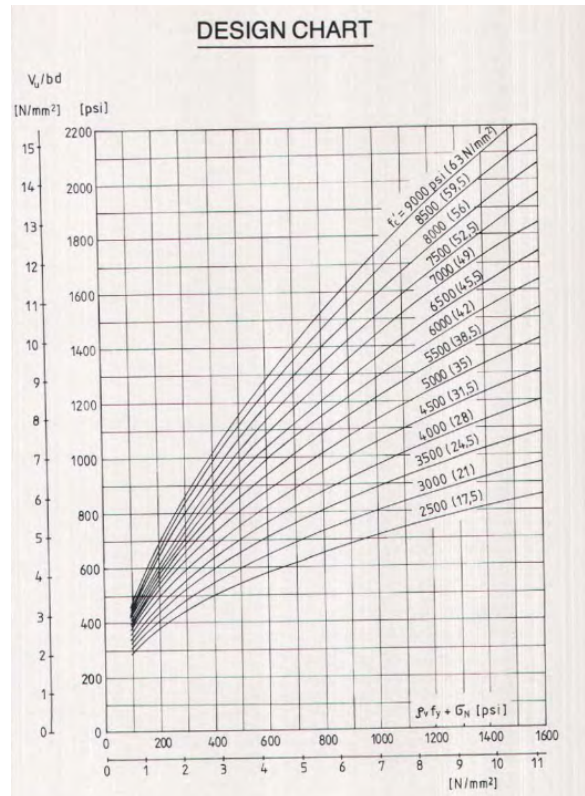


Figure 2.8: Design chart for determining the shear friction capacity by taking into account the compressive strength of the concrete (Walraven et al. (1987))

2.3.13 Bass et al. (1989)

Bass et al. (1989) carried out an experimental investigation on 33 full scale push-off specimens to test the interface shear capacities between new and old concrete interfaces. Test variables included surface preparation, amount and depth of embedment of the interface reinforcement, reinforcement properties and concrete strength. The tests showed that at low levels of slip a sandblasted surface preparation was as good as any other surface preparation. The results found that having more reinforcement crossing the interface and a heavily sand blasted surface produced maximum shear stress capacities. Specimens with a 75mm embedment depth reached peak shear stress about 30% lower than those with 150mm and 300mm embedment depths. In general the maximum shear stress and the residual capacity increased with an increase in the interface reinforcement for any dowel embedment. All specimens with 300mm dowel embedment depths exceeded the ACI 318-83 (1983) design strengths even at slip levels of 12.7mm though the ACI 318-83 (1983) calculations required an embedment depth of 450mm.

Bass et al. (1989) concluded that the deeper the embedment the better the result and with regard to the surface preparation that keeping the base block concrete strength higher than the second casting resulted in higher interface shear capacity similar to the findings of Paulay et al. (1974).

2.3.14 Loov and Patnaik (1994)

Loov and Patnaik (1994) explained that the ACI 318-83 (1983) had 5 equations that

required solving for interface shear reinforcement. The authors determined that by testing 16 beams and using the results that 4 of these equations could be replaced with a new parabolic equation modified from the one used in the PCI Design Manual (1973). This equation combines the effect of concrete strength and clamping stress. The test results indicated that an as-cast surface with the coarse aggregate left protruding could develop adequate shear resistance and simplify production. The results also proved that the stirrups were typically unstressed and ineffective until the horizontal shear stress exceeded 2.0 MPa. The roughness at the top of the precast beam, the amount of steel crossing the interface and the concrete strength are the major factors affecting the shear strength of the interface.

The beam webs were cast first with projecting steel stirrups. The flanges were cast 3 days later. The interface was left as cast with coarse aggregate protruding. Slip gauges were used to record slip and strain gauges were used to check the yielding of the reinforcement. The maximum slip for every beam which failed in horizontal shear was greater than 2mm. The test results showed that stirrups were not stressed until a horizontal shear stress of about 1.5 - 2.0 MPa was reached. This indicates the strength of an interface without stirrups. The stirrups become relatively effective when the shear stress reaches about 3.0 MPa.

The two variables for the test specimens were concrete strength and the area of reinforcement crossing the interface.

The testing is a follow-on from beams sized and tested by Patnaik (1992) in his PhD thesis. The authors stated that even at a slip of 0.13mm, a limit noted by Hanson (1960), stirrup stresses were still much lower than yield. The author also noted that stirrups near midspan and within the end block were ineffective. The test results compared favourably with the authors new equation.

$$v_u = k\lambda\sqrt{(0.1 + \rho f_y)f_c} \quad (2.13)$$

where;

v_u = ultimate longitudinal shear stress at the interface, (MPa)

k = 0.5 for composite construction and 0.6 for monolithic construction

λ = 1.0 for normal weight concrete, 0.85 for sand-lightweight concrete and 0.75 for all-lightweight concrete

ρ = reinforcement ratio

f_y = reinforcement yield strength, (MPa)

f_c = concrete compressive strength, (MPa)

The authors proposed an upper limit for horizontal shear strength to prevent local crushing of the concrete of $0.25f_c$. They also proposed an equation for composite beams without interface reinforcement as:

$$v_{no} = 0.6\sqrt{0.1f_c} \quad (2.14)$$

2.3.15 Kahn and Mitchell (2002)

50 push off specimens were tested with un-cracked, pre-cracked and cold jointed interfaces. Concrete strengths varied from 46.9 MPa to 123.4 MPa with transverse rein-

forcing ratios between 0.37% and 1.47%. The purpose of this research was to extend previous research on shear friction but with higher strength concretes. At the time of writing very little work had been done regarding concrete strengths greater than 69.0 MPa. The push off specimens were designed to be nearly identical to Hofbeck et al. (1969), Mattock et al. (1960) and Anderson (1960) in order to compare results.

In both the un-cracked and cold jointed specimens cracks were observed at loads between 50% and 75% of the peak ultimate load capacity. In the pre-cracked specimens slip between the two faces began almost immediately. The study confirmed that the residual capacity of an un-cracked specimen was approximately equal to a pre-cracked specimen with similar reinforcement. Further results were found to be much more consistent when the yield stress f_y was taken as 413.7 MPa as compared to using the measured yield stress. Kahn and Mitchell (2002) recommended that the yield strength of the reinforcement be limited to 413.7 MPa when calculating shear stress at the interface. For pre-cracked, cold jointed and un-cracked specimens shear friction strengths were stronger than predicted by Eq 2 of the ACI 318-95 (1995). Kahn and Mitchell (2002) were in agreement with Birkeland and Birkeland (1966) and Mast (1968) that using a coefficient of friction value, μ of 1.4 for a rough cold jointed interface was correct. Finally the authors proposed an upper limit on the shear stress to be 20% of the concrete strength not the 5.5 MPa as per the ACI 318-95 (1995). Their formulae for interface shear stress is below:

$$v_u = 0.05f_c + 1.4\rho_v f_y \leq 0.2f_c \quad (2.15)$$

where;

v_u = ultimate longitudinal shear stress at the interface, (MPa)

f_c = concrete compressive strength, (MPa)

ρ_v = reinforcement ratio

f_y = reinforcement yield strength, (MPa)

2.3.16 Kahn and Slapkus (2004)

Kahn and Slapkus (2004) carried out tests on 6 composite T-beams with webs containing concrete with a compressive strength of 83.6 MPa and flanges with a compressive strength of either 50.2 MPa or 77.8 MPa. The beams were 3.2m long and were based on the testing of Loov and Patnaik (1994). The flexure tests and interface failure results showed that all of the equations developed previously could be used for high strength concrete bridge sections. The aims of the study were to:

1. extend the provisions for horizontal shear strength of composite beams for use with high strength concrete T-beams
2. compare horizontal shear results from beams to shear friction push-off tests using high strength concrete
3. evaluate current composite beam design provisions for use with high-strength concrete beams.

Not only did the cast in place insitu top slab vary in concrete strength but it also varied in the area of reinforcement crossing the interface. The surface condition of

the precast web was as cast with no attempt to purposely roughen it. For the test the webs of the beams were poured, struck off level but not levelled out or finished in any way. The aggregate protruded approximately 6mm, there was no laitance or bleed water. The flanges were then placed 4 weeks after web construction. The beams were tested approximately 6 months later. Each beam was loaded monotonically to failure. Interface shear failure was defined by a slip greater than 0.25mm. In all calculations the concrete strength used was that of the insitu flange. The test results found that the equation proposed by Kahn and Mitchell (2002) gave more favourable answers and compared more accurately to the test results. The test results proved that the ACI 318-95 (1995) and AASHTO LRFD-BDS (1998) equations were far too conservative for static loads. The test results at failure closely matched the shear friction push off specimen test results using high strength concrete. The equation proposed by Loov and Patnaik (1994) most accurately predicted the interface shear strength of the composite beams. These tests proved that a clean, rough as cast surface with 6mm aggregate projection was enough to get favourable results.

2.3.17 Randl (2013)

Randl (2013) explained the background to the design approach of CEB-FIP Model Code 2010 (2010). When the author examined the test results obtained from all of the small scale testing since 1960 it turns out that small irregularities in the test set ups effect the results greatly whereas large scale test set ups reflect the real situation somewhat better.

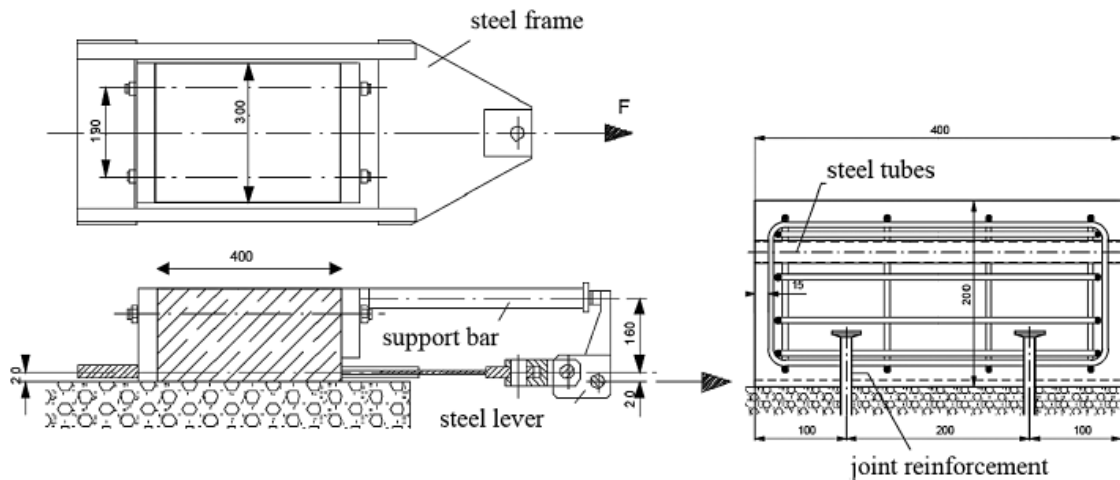


Figure 2.9: Optimised Test Set-Up With Lever Arm For Investigating Interface Shear Transfer (Source Randl (2013))

Randl (2013) carried out 83 shear tests in 5 series using the set up illustrated in Figure 2.9. The reinforcement was post installed in order to reflect an on-site situation. The reinforcing bars had mushroom heads in order to achieve better bond with the thin overlay. Tests were carried out on concrete-to-concrete bonded interfaces and concrete-to-concrete unbonded interfaces using three coats of form oil to act as a bond breaker. This was again to reflect the on-site conditions where dust and grime play an important part and surfaces might not necessarily be cleaned prior to casting of the concrete

overlay.

Randl (2013) outlined the main parameters of the shear slip phenomenon and consequently the reasoning behind their inclusion in CEB-FIP Model Code 2010 (2010):

1. Mechanical interlock and adhesive bonding: The interlocking effect is caused by the aggregate in the concrete and the roughness of the surface. Even with very low levels of reinforcement and a roughness of $R_t \geq 1.5\text{mm}$ mechanical interlocking will take place and give a considerable amount of interface shear resistance. Adhesive bonding can also contribute to the overall interface shear resistance. However, the failure slip measurements are quite low, approximately 0.05mm before the possibility of interface failure.
2. Friction: Depending on the surface roughness friction will develop as a result of compression forces perpendicular to the interface. This compression can be as a result of external actions, prestressing forces or most likely reinforcement crossing the interface causing a clamping effect.
3. Bending resistance of steel connectors, dowel action: A shear slip along an interface containing reinforcing bars leads to a lateral displacement of both ends of the reinforcing bar thus inducing bending stresses. The resistance to this bending is what is known as dowel action.
4. Interaction: Under ultimate loading conditions all of the mechanisms described above interact. After failure of the adhesive bond and with shear slip increasing the aggregate interlocking effect will also decrease when the protruding aggregates crush. However, the bending resistance of the reinforcing bars may increase with larger slips. Therefore the actual contribution of each of these mechanisms is greatly related to the roughness of the interface surface, quality of bond and amount of reinforcement crossing it.

According to Randl (2013) the shear resistance of concrete interfaces subjected to fatigue loading has not been investigated in a comprehensive way. The author states that if the shear resistance of the interface is greater than the shear stress acting on it then there is no requirement for shear reinforcement according to CEB-FIP Model Code 2010 (2010) and Eurocode 2 Part 1-1 (2004). Small scale push-off tests were carried out on cracked interfaces and the results demonstrated a greater decrease in the shear strength under cyclic loading. Therefore a simple design approach for uncracked interfaces subjected to fatigue loading is to reduce the adhesive bond to 50% and for cracked interfaces assume that the fatigue strength is 40% of the static interface shear strength.

If reinforcement is required across the interface to satisfy the design shear stress then a minimum percentage of reinforcement is required along the remainder of the interface where the interface shear stress is less than the interface resistance without interface shear reinforcement. Randl (2013) came up with the following equations after performing extensive testing and these are adopted in CEB-FIP Model Code 2010 (2010):

For Beams:

$$\rho_{min} = 0.2 \frac{f_{ctm}}{f_{yk}} \geq 0.001 \quad (2.16)$$

For Slabs:

$$\rho_{min} = 0.12 \frac{f_{ctm}}{f_{yk}} \geq 0.0005 \quad (2.17)$$

where;

ρ_{min} = minimum percentage of reinforcement

f_{ctm} = mean value of axial tensile strength of concrete, (MPa)

f_{yk} = characteristic yield strength of reinforcement, (MPa)

Randl (2013) tested smooth interfaces and rough interfaces under pure shear loading for the minimum anchorage length of the interface reinforcement.

Minimum anchorage for smooth interfaces = 6ϕ

Minimum anchorage for rough interfaces = 10ϕ

where;

ϕ = bar diameter

2.3.18 Mahmoud et al. (2014)

Mahmoud et al. (2014) carried out a study focusing on the performance of reinforced concrete flanged beams with varying shear connector distribution and beam reinforcement ratios. The authors stated that well distributed shear connectors clamp the reinforced concrete beam and cast in place slab together and their behaviour is more like that of a monolithically cast deck. The beams were tested under monotonic loading up to the appearance of the first crack at the interface along with a control beam which was tested to failure. The beams were loaded with four point loads on the top flange. Maximum slippage occurred at the beam ends, slippage was approximately zero at midspan, hence concentrating shear connectors near the supports is better than equal distribution along the beam as slippage is greater at beam ends. Using the same overall area of shear reinforcement, smaller diameter shear connectors show better ductility when compared to larger diameter connectors. Mahmoud et al. (2014) used a computer software package Response 2000 along with the strut and tie model to compare results with the experimental data.

Shear transfer failure load (kN)												
Beam ID	B02		B03		B04		B05		B06		B07	
Shear reinforcement	Ø6@ 200 mm		Ø4@ 80 mm		Ø6@ 130 mm		Ø4@ 55 mm		Ø6@ 130/285 mm		Ø4@ 55/110 mm	
Notes	Equal distribution along the beam								Concentration at supports			
Experimental results	176		178		256		257		174		175	
		$\mu(\%)$		$\mu(\%)$		$\mu(\%)$		$\mu(\%)$		$\mu(\%)$		$\mu(\%)$
Strut and tie	152	13	170	3.7	231	9.2	228	11	165	4.9	153	12.4
Compressive theory national building codes	156	11.3	170	4.4	233	8.9	248	3.6	156	10.3	170	3
Shear theory national building codes	169	4	171	4.8	186	27.6	190	27	169	3	171	3.3
Mast equation	82	53	84	52	109	57	116	54	82	53	84	51
Birkeland equation [15]	203	-15	207	-16	235	8.3	243	5.7	203	-17	207	-18
Walraven equation [16]	216	-22	219	-22	243	5	249	3	216	-24	219	-25
Loov equation [17]	191	-8.5	194	-9	220	14	227	12	191	-9.6	194	-10
Shaikh equation	177	-0.7	180	-1.3	205	20	211	18	177	-1.8	180	-2.8
Loov and Patnaik equation	198	-12	201	-13	227	11	234	9	198	-14	201	-15

= $\frac{(\text{Experimental result} - \text{Analysis result}) \times 100}{\text{Experimental result}}$

= (Experimental result - Analysis result) × 100 / Experimental result

Figure 2.10: Comparison of testing and previous research (Mahmoud et al. (2014))

Mahmoud et al. (2014) compared (Figure 2.10) his experimental results to the equations of Mast (1968), Birkeland and Birkeland (1966), Walraven et al. (1987), Shaikh (1978) and Loov and Patnaik (1994) and his own strut and tie method. Out of all of these

comparisons Shaikh (1978) and Loov and Patnaik (1994) equations gave the most favourable results but none correlated as well as the strut and tie method of the author.

2.3.19 Santos and Julio (2014)

Santos and Julio (2014) examined the shear friction theory in order to come up with a new equation that included and allowed for differential shrinkage and differential stiffness. The authors also compared the interface shear equations of the ACI 318-14 (2014), Eurocode 2 Part 1-1 (2004) and the CEB-FIP Model Code 2010 (2010).

Trying to overcome some weaknesses in the design codes Santos and Julio (2014) set about developing a non-destructive insitu method of obtaining the roughness of a concrete surface, as the roughness parameters, c and μ are still presented differently in each design code. Subsequently a large experimental study was carried out to test:

1. substrate surface preparation
2. differential shrinkage
3. differential stiffness on the bond strength of the interface between concrete layers cast at different times.

Santos and Julio (2014) carried out slant shear tests (see Figure 2.11) and splitting tests to assess the bond strength of the concrete-to-concrete interface. Left as cast, wire brushed, sand blasted, shot blasted and hand scrubbed were the finishes chosen. The castings were left 28, 56 and 84 days to maximise the effects of differential shrinkage. Two different curing conditions were used, inside and outside of the laboratory. A total of 300 specimens were tested in total. Two failure modes were observed:

1. Adhesive failure whereby the interface de-bonded
2. Cohesive failure caused by crushing of the concrete bulk.

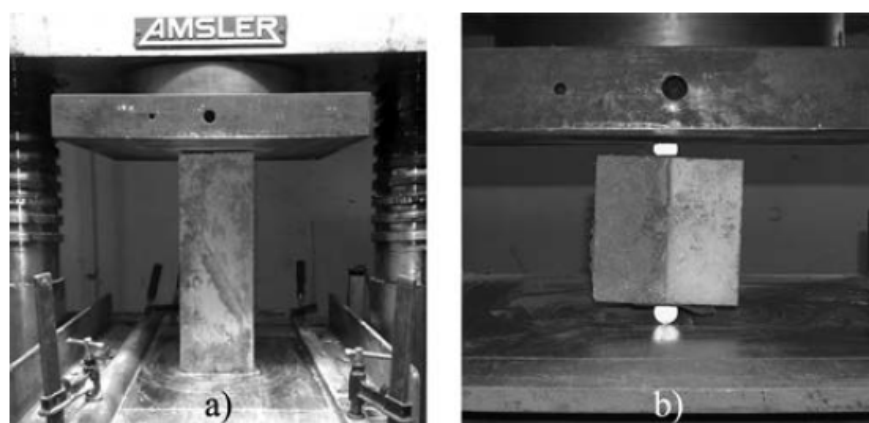


Figure 2.11: a) Slant Shear Test and b) Splitting Test (Source Santos and Julio (2014))

The test results proved that the bond strength of the interface increased with the increase in surface roughness. Santos and Julio (2014) came up with new formulae to

determine the coefficient of cohesion, c and the coefficient of friction, μ . These are based on a new technique again developed by the authors on measuring the roughness of the interface accurately.

$$c = \frac{v_u}{f_{ctm}} \quad (2.18)$$

$$\mu = \frac{v_u - cf_{ctm}}{\sigma_n} \quad (2.19)$$

where;

c = coefficient of cohesion

v_u = ultimate longitudinal shear stress at concrete to concrete interface, (MPa)

f_{ctm} = mean value of concrete tensile strength, (MPa)

μ = coefficient of friction

σ_n = normal stress acting on the interface due to external loading, (MPa)

s = spacing of horizontal shear reinforcement, (mm)

L = effective span of beam, (mm)

Santos and Julio (2014) concluded that cohesion should be reduced to zero when fatigue or dynamic loading exist on a structure and that interface reinforcement should be present under these circumstances.

2.4 Current Bridge Design Codes

The following section describes in detail a step by step approach for calculating the reinforcement required crossing the horizontal interface for each of the major bridge design codes. The section ends with a summary Table comparing the results.

2.4.1 BS5400-4 (1990)

The design approach for longitudinal shear for composite concrete beams is outlined in section 7.4.2.3 of BS5400-4 (1990).

Step 1: The applied longitudinal shear stress is calculated at the interface using the elastic method.

$$v = \frac{VA\bar{y}}{Ib} \quad (2.20)$$

where;

v = longitudinal shear stress at the interface, (MPa)

V = the vertical shear force at ultimate loads, (kN)

A = the area of the slab, (mm²)

$\bar{y}$ = the distance from the centre of the slab to the composite centroid, (mm)

I = second moment of area of the the cross section, (mm⁴)

b = width of the interface, (mm)

Step 2: The design shear resistance of the interface is calculated, where V_1 should not exceed the lesser of the following:

$$a) k_1 f_{cu} L_s \quad (2.21)$$

$$b)v_1L_s + 0.7A_ef_y \quad (2.22)$$

Step 3: This equation can be rewritten to calculate the interface stirrups required:

$$A_e = \frac{\frac{VA_y}{I} - v_1L_s}{0.7f_y} \quad (2.23)$$

where;

k_1 = a constant depending on the concrete bond across the shear plane under consideration, taken from Table 2.1

f_{cu} = concrete characteristic cube strength, (MPa)

L_s = length of the shear plane under consideration, (mm)

v_1 = the ultimate longitudinal shear stress in the concrete for the shear plane under consideration, taken from Table 2.1, (MPa)

A_e = the area of fully anchored reinforcement per unit length crossing the shear plane under consideration, (mm²)

f_y = the characteristic strength of the reinforcement, (MPa)

Table 2.1: Ultimate longitudinal shear stress, v_1 , and values of k_1 for composite members - BS5400-4 (1990) Table 31

	Concrete grade (MPa)				
Type of shear plane	20	25	30	40 or more	k1
	Longitudinal shear stress (MPa)				
Monolithic	0.90	0.90	1.25	1.25	0.15
Type 1	0.50	0.63	0.75	0.80	0.15
Type 2	0.30	0.38	0.45	0.50	0.09

Step 4: A check must be carried out to ensure the design links are greater than the minimum links of 0.15%bh spaced at a maximum of 600mm or four times the slab thickness.

where;

b = the width of the interface, (mm)

h = design length, normally 1000mm

2.4.2 Eurocode 2 Part 1-1 (2004)

Eurocode 2 Part 1-1 (2004) differs from BS5400-4 (1990) in how it categorises the surface finishes. Eurocode 2 Part 1-1 (2004) classifies the surface finishes as very smooth, smooth, rough or indented. For standard bridge beam construction the surface roughness used in both design and practice is of a rough type. This means it has 1.5mm average roughness which can be achieved by raking, exposing of aggregate or by other

means.

Similar to BS5400-4 (1990) the horizontal shear stress is worked out using the elastic formula.

Step 1: Design value of interface shear stress, Exp(6.24):

$$V_{Edi} = \frac{\beta V_{Ed}}{z b_i} \quad (2.24)$$

where;

β = the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered

V_{Ed} = the transverse shear force, (kN)

z = the lever arm of the composite section, (mm)

b_i = the width of the interface, (mm)

Step 2: Design shear resistance of the interface, Exp(6.25):

Rewriting Exp 6.25 to calculate Interface Links:

$$\rho_{min} = \frac{V_{Edi} - c f_{ctd} - \mu \sigma_n}{f_{yd}(\mu \sin \alpha + \cos \alpha)} \quad (2.25)$$

where;

c = the cohesion factor

μ = the coefficient of friction

f_{ctd} = the tensile stress of the new concrete layer, (MPa)

f_{yd} = the design yield strength of reinforcement, (MPa)

α = the angle of reinforcement to interface, (Degrees)

Step 3: Area of Links Required:

$$\rho_{min} \times b \quad (2.26)$$

Step 4: Design Shear Resistance of the Interface without interface reinforcement:

$$V_{Rdi} = c f_{ctd} + \mu \sigma_n \quad (2.27)$$

Step 5: Design shear resistance of the interface using provided reinforcement, Exp(6.25):

$$V_{Rdi} = c f_{ctd} + \mu \sigma_n + \rho f_{yd}(\mu \sin \alpha + \cos \alpha) \quad (2.28)$$

Step 6: Minimum reinforcement - Eurocode 2 Part 1-1 (2004) does not specify a minimum reinforcement limit.

Step 7: Maximum spacing - Eurocode 2 Part 1-1 (2004) does not specify a maximum spacing limit.

2.4.3 CEB-FIP Model Code 2010 (2010)

CEB-FIP Model Code 2010 (2010) is very similar in its approach to horizontal interface shear as Eurocode 2 Part 1-1 (2004).

The procedure is outlined below:

Similar to BS5400-4 (1990) and Eurocode 2 Part 1-1 (2004) the horizontal shear stress is worked out using the elastic formula.

Step 1: Design value of interface shear stress, Exp(7.3-49):

$$\tau_{Edi} = \frac{\beta V_{Ed}}{z b_i} \quad (2.29)$$

where;

β = the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered

V_{Ed} = the transverse shear force, (kN)

z = the lever arm of the composite section, (mm)

b_i = the width of the interface, (mm)

Step 2: Design shear resistance of the interface, Exp(7.3-51):

Rewriting Exp 7.3-51 to calculate Interface Links:

$$\rho_{min} = \frac{\tau_{Rdi} - c_r f_{ck}^{1/3} - \mu \sigma_n}{K_1 f_{yd} (\mu \sin \alpha + \cos \alpha) + K_2 \sqrt{f_{yd} f_{cd}}} \leq \beta_c v f_{cd} \quad (2.30)$$

where;

c_r = the cohesion for aggregate interlock effects at rough interfaces

K_1 = the interaction coefficient for tensile force activated in the reinforcement or the dowels

K_2 = the interaction coefficient for flexural resistance

μ = the coefficient of friction

ρ = the reinforcement ratio of the reinforcing steel crossing the interface

σ_n = the (lowest expected) compressive stress resulting from an eventual normal force acting on the interface, (MPa)

f_{ck} = the characteristic compressive cylinder strength at 28 days, (MPa)

f_{cd} = the design value of the new concrete layers compressive strength, (MPa)

f_{yd} = the design yield strength of reinforcement, (MPa)

α = the inclination of reinforcement crossing the interface, (Degrees)

β_c = the coefficient for the strength of the compression strut

$v = 0.55 \left(\frac{30}{f_{ck}} \right)^{1/3} \leq 0.55$

The coefficients for different surface roughnesses in interfaces reinforced with dowels or rebar are given in Table 2.2

Step 3: Area of Links Required:

$$\rho_{min} \times b \quad (2.31)$$

Table 2.2: Coefficients for different surface roughnesses(CEB-FIP Model Code 2010 (2010))

Surface roughness	c_r	K_1	K_2	β_c	$\mu(f_{ck} \geq 20)$	$\mu(f_{ck} \geq 35)$
Very rough $R_t \geq 3.0mm$	0.2	0.5	0.9	0.5	0.8	1.0
Rough $R_t \geq 1.5mm$	0.1	0.5	0.9	0.5	0.7	
Smooth	0	0.5	1.1	0.4	0.6	
Very Smooth	0	0	1.5	0.3	0.5	

Step 4: Design Shear Resistance of the Interface without interface reinforcement (7.3-50):

$$\tau_{Rdi} = c_a f_{ctd} + \mu \sigma_n \leq 0.5 v f_{cd} \quad (2.32)$$

Step 5: Design shear resistance of the interface using provided reinforcement, Exp(7.3-51):

$$\tau_{Rdi} = c_r f_{ck}^{1/3} + \mu \sigma_n + K_1 \rho f_{yd} (\mu \sin \alpha + \cos \alpha) + K_2 \rho \sqrt{f_{yd} f_{cd}} \leq \beta_c v f_{cd} \quad (2.33)$$

Step 6: Minimum reinforcement - clause 6.3.5:

$$\rho_{w,min} = 0.2 \frac{f_{ctm}}{f_{yk}} \geq 0.001 \quad (2.34)$$

Step 7: Maximum spacing - CEB-FIP Model Code 2010 (2010) does not specify a maximum spacing limit:

2.4.4 ACI 318-14 (2014)

The design approach for interface shear for composite concrete beams is outlined in section 16.4 of the ACI 318-14 (2014).

Step 1: Design value of interface shear stress, Exp(7.3-49):

$$v_h = \frac{V_u}{b_v d} \quad (2.35)$$

where;

v_h = the horizontal shear stress at the interface, (MPa)

V_u = the vertical shear force at ultimate loads, (kN)

b_v = the width of topping to beam interface, (mm)

d = distance from the compression force for the entire composite section to the centroid

of the tension steel (Need not be taken as less than $0.8h$), (mm)

Step 2: Design shear resistance of the interface. The code specifies that the factored horizontal shear force capacity, ϕV_{nh} , must exceed the factored vertical shear force demand, V_u .

$$\phi V_{nh} \geq V_u \quad (2.36)$$

where;

$\phi = 0.75$, a strength reduction factor for shear as specified by Table 21.2.1 of the code

V_{nh} = horizontal shear capacity, (kN)

V_u = vertical shear demand, (kN)

V_u is obtained from the worst case design scenario and V_{nh} is obtained from the below interface condition.

The interface surface is clean, free of laitance and intentionally roughened.

$$V_{nh} = 80b_v d \quad (2.37)$$

where;

b_v = width of the interface, (mm)

d = the distance from the extreme compression fibre for the composite section to the centroid of the tension steel, (mm)

Step 3: Area of Links Required:

This equation can be rewritten to calculate the required area of interface shear reinforcement:

$$A_{vf} = \frac{V_u}{\phi f_y (\mu \sin \alpha + \cos \alpha)} \quad (2.38)$$

Step 4: Design Shear Resistance of the Interface without interface reinforcement:

$$v_{nh} = 80b_v d \quad (2.39)$$

where;

v_{nh} = the nominal horizontal shear strength, (MPa)

Step 5: Design shear resistance of the interface using provided reinforcement:

$$V_n = A_{vf} f_y (\mu \sin \alpha + \cos \alpha) \quad (2.40)$$

Step 6: Minimum reinforcement is calculated as per 16.4.6.1:

$$A_{vmin} = \text{Maximum} \left(0.75 \sqrt{f'_c} \frac{b_w s}{f_{yt}}, \frac{50b_w s}{f_{yt}} \right) \quad (2.41)$$

where;

A_{vmin} = minimum area of shear reinforcement, (mm^2)

f'_c = concrete compressive strength, (MPa)

Step 7: Maximum spacing:

- a) min of 4b and 4h, (mm)
- b) 600mm

2.4.5 AASHTO LRFD-BDS (2007)

Section 5.8.4 of AASHTO LRFD-BDS (2007) outlines the procedure for calculating the interface shear transfer across a joint.

Step 1: The applied horizontal shear stress is calculated at the interface using the elastic method.

$$v_h = \frac{VQ}{Ib_v} \leq \text{Minimum}(K_1f_c, K_2) \quad (2.42)$$

where;

v_h = horizontal shear stress at the interface, (MPa)

V = the vertical shear force at ultimate loads, (kN)

Q = first moment of inertia with respect to the neutral axis of the slab, calculated as $Q=A\bar{y}$; where A is the area above the centroid of the composite section and $\bar{y}$ is the distance from the area above the centroid of the composite section to the neutral axis of the composite section, (mm³)

I = second moment of area of the the cross section, (mm⁴)

b_v = width of the interface, (mm)

f_c = specified 28 day compressive strength of the weaker concrete on either side of the interface, (MPa)

K_1 = fraction of the concrete strength available to resist interface shear

K_2 = limiting interface shear resistance

Step 2: Design shear resistance of the interface. Similar to ACI 318-14 (2014), AASHTO LRFD-BDS (2007) outlines that the factored capacity, ϕV_n , must exceed the factored demand, V_h .

The nominal shear resistance, V_{ni} , of the interface plane shall be taken as:

$$V_{ni} = cA_{cv} + \mu(A_{vf}f_y + P_c) \quad (2.43)$$

where;

V_{ni} = nominal horizontal shear resistance, (MPa)

A_{cv} = area of concrete engaged in shear transfer, = $b_{vi}L_{vi}$, (mm²)

b_{vi} = interface width, (mm)

L_{vi} = Half the span for a simply supported beam, (mm)

A_{vf} = area of shear reinforcement crossing the interface, (mm²)

f_y = yield strength of reinforcement, limited to 420 MPa

P_c = permanent net compressive force normal to the shear plane (i.e. dead load acting on slab), equal to zero if force is tensile, (kN)

c = cohesion factor, see Table 2.3

μ = friction factor, see Table 2.3

λ = density factor, see Table 2.3

Step 3: Area of Links Required:

This equation can be rewritten as:

$$A_{vf_{Req}} = \frac{V_h - cA_{cv} - \mu P_c}{\mu f_y} \quad (2.44)$$

Step 4: Design Shear Resistance of the Interface without interface reinforcement:

$$V_{ni} = cA_{cv} + \mu P_c \quad (2.45)$$

Step 5: Design shear resistance of the interface using provided reinforcement:

$$V_{ni} = cA_{cv} + \mu(A_{vf}f_y + P_c) \quad (2.46)$$

Step 6: Minimum reinforcement is calculated as per 5.8.4.4:

$$(A_{vf_{min}} \geq \frac{0.35A_{cv}}{f_y} \quad (2.47)$$

Step 7: Maximum spacing - AASHTO LRFD-BDS (2007) does not specify a maximum spacing limit.

Table 2.3: Cohesion, coefficient of friction, K1 and K2 factors as per AASHTO LRFD-BDS (2007)

Type of Interface	Cohesion c (MPa)	Coefficient of friction μ	K_1	K_2 (MPa)
Normal density concrete placed against a clean concrete surface, free of laitance with surface intentionally roughened to an amplitude of 6mm	1.7	1.0	0.25	10.3

2.4.6 Comparison of Horizontal Interface Shear Capacity using the Major Bridge Design Codes

Since 2011 bridge engineers in Europe have been using the provisions of Eurocode 2 Part 1-1 (2004) to calculate the shear reinforcement crossing the horizontal interface in beam and slab type bridge construction. Previous to 2011 BS5400-4 (1990) was used and this code contained a clause for the minimum percentage of reinforcement required

($0.15bh$), Eurocode 2 Part 1-1 (2004) does not contain such a formulae. Therefore CEB-FIP Model Code 2010 (2010) is used and is based on the findings of Randl (2013). A comparison of BS5400-4 (1990) and CEB-FIP Model Code 2010 (2010) as can be seen in Table 2.4, shows the results are similar with CEB-FIP Model Code 2010 (2010) being slightly more conservative.

In the United States of America the two design codes used for calculating horizontal interface shear reinforcement areas are ACI 318-14 (2014) and AASHTO LRFD-BDS (2007) and these have both been included in Table 2.4 to widen the comparison.

The shear friction theory is common to all of the design codes chosen for this comparison. The most important factors in determining the horizontal shear stress at the interface is due to a combination of cohesion, friction and dowel action and all design codes allow for these parameters, each one somewhat different depending on the contributing researchers and their experimental findings.

The figures in Table 2.4 are based on the T-Beam section for Phase 1 of the experimental testing, which will be described in the next chapter, and can be seen in Figure 2.12. The beam is 3200mm long and has an effective span of 3000mm. The reinforced concrete beam is 200mm wide x 400mm deep and has a 400mm wide x 150mm deep reinforced concrete top slab cast on to form a composite T-beam section.

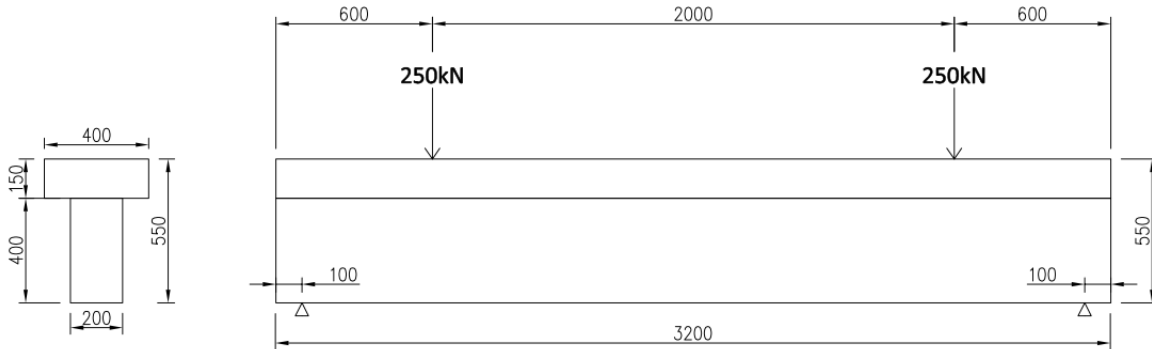


Figure 2.12: Phase 1 T-Beam Showing Loading

2.5 Discussion

2.5.1 Shear Friction Theory

The literature review summarized the shear friction theory from 1960 to the present with the key formulae shown in Table 2.5. This Table begins from the publication of the shear friction theory by Birkeland and Birkeland (1966) to the formulae of Randl (1997), who is heavily involved in the drafting of the new suite of Eurocodes due to be published in early 2023.

Table 2.4: Comparison of Horizontal Interface Shear Capacity using the Major Bridge Design Codes

Description	ACI 318-14 (2014)	AASHTO LRFD- BDS (2007)	BS5400-4 (1990)	Eurocode 2 Part 1-1 (2004)	CEB-FIP Model Code 2010 (2010)
Design value of Interface Shear Stress (MPa)	2.54	3.00	3.33	3.33	3.33
Interface Finish	Concrete placed against hardened concrete that is clean, free from laitance and inten- tionally roughened to a full amplitude of approx 6mm	Cast in place concrete slab on clean concrete girder surfaces, free of laitance with surface roughened to an amplitude of 6mm	Type 2 - Rough as Cast. The contact surface of the concrete should be cleaned to remove laitance and any loose material	Rough - A surface with at least 3mm roughness at about 40mm spacing, achieved by raking, exposing of aggregate or other means	Very Rough - A surface with at least 3.0mm average roughness, achieved by high pressure water jetting, indented or by other means
Interface Shear Resistance without Rein- forcement (MPa)	0.55	1.37	0.5	0.65	0.82
Interface Shear Resistance with 2 legs of B10@150 c/c's Stirrups (MPa)	4.47	2.95	2.33	2.25	2.29
Minimum Interface Re- inforcement Requirements (mm ² /mm)	0.19 (2B10@600)	0.17 (2B10@600)	0.3 (2B10@500)	N/A	0.34 (2B10@450)
Maximum Spacing (mm)	600	N/A ₄₃	600	N/A	N/A

Table 2.5: Shear Friction Theory Milestone Equations

Author	Equation	Note
Birkeland and Birkeland (1966)	$v_u = \mu \rho f_v$	Original Equation
Mattock and Hawkins (1972)	$v_u = 1.38 + 0.8(\rho f_y + \sigma_n)$	Contribution of Cohesion
Loov (1978)	$\frac{v_u}{f_c} = k \sqrt{\frac{\rho f_y + \sigma_n}{f_c}}$	Concrete Strength Added
Walraven et al. (1987)	$v_u = C1(\rho f_y)^{c2}$	Based on Spherical Model
Randl (1997)	$v_u = c f_c^{\frac{1}{3}} + \mu(\rho k f_y + \sigma_n) + \alpha \rho \sqrt{f_y f_c}$	Cohesion, Friction and Dowel Action all considered

2.5.2 Minimum Interface Reinforcement

Currently Eurocode 2 Part 1-1 (2004) does not have a minimum percentage of interface reinforcement requirement, it is recognised that a minimum percentage of reinforcement is required across the interface to prevent interfacial slip and brittle failure at loss of adhesion. BS5400-4 (1990) and CEB-FIP Model Code 2010 (2010) have similar requirements which the majority of bridge designers adhere to at present. The push-off tests carried out by Hughes (1987) resulted in differing minimum interface reinforcement compared to the 0.15% recommendation of BS5400-4 (1990). Simply the amount of minimum interface reinforcement should be able to resist $0.5N/mm^2$, $0.85N/mm^2$ and $1.5N/mm^2$ for monolithic, roughened and rough as cast surfaces respectively. However, BS5400-4 (1990) deals with interfaces in beams and the push-off test specimens do not represent these beams accurately enough to increase the minimum percentage of interface reinforcement from 0.15% ($0.72N/mm^2$) for the push-off test specimens. If reinforcement is required across the interface to satisfy the design shear stress then a minimum percentage of reinforcement is required along the remainder of the interface where the interface shear stress is less than the interface shear stress resistance without interface shear reinforcement (Randl (2013)). Randl (2013) proposed an equation, after performing extensive testing, which was adopted in the latest version of the CEB-FIP Model Code 2010 (2010). Table 2.6 identifies the differences in the design codes in relation to the minimum percentage of interface reinforcement. It is clear further testing is required on this topic.

2.5.3 Dynamic or Fatigue Verifications

It is the author's observation that most bridge engineers do not adhere to clause 6.2.5(105) of Eurocode 2 Part 2 (2005) for short-span highway and railway bridges. Reducing the coefficient of cohesion c to zero increases the amount of shear reinforcement crossing the interface to unmanageable levels. Figure 2.13 shows the end of a precast prestressed W beam during manufacture with 12mm links at 100mm centres.

Table 2.6: Minimum Interface Reinforcement Equations

Author	Equation	Note
Hughes (1987)	$0.5N/mm^2, 0.85N/mm^2$ and $1.5N/mm^2$	For monolithic, roughened and rough as cast surfaces
BS5400-4 (1990)	$0.15\%bh$	
Eurocode 2 Part 1-1 (2004)	N/A	
CEB-FIP Model Code 2010 (2010)	$0.2 \frac{f_{ctm}}{f_{yk}} \geq 0.001$	
Randl (2013)	$0.2 \frac{f_{ctm}}{f_{yk}} \geq 0.001$	
ACI 318-14 (2014)	Maximum of $0.062 \sqrt{f'_c} \frac{b_w s}{f_y}$ and $0.35 \frac{b_w s}{f_y}$	

These links were designed using a c value of 0.4 as per clause 6.2.5(2) of Eurocode 2 Part 1-1 (2004). This is a very high ratio of stirrups, reducing c to zero would mean 12mm links at 80mm centres, an increase of 20%. Reducing c to zero also results in the design shear resistance of the interface without stirrups being equal to zero when designing to Exp(6.25) of Eurocode 2 Part 1-1 (2004) or Exp(7.3-50) of CEB-FIP Model Code 2010 (2010), suggesting that the longitudinal shear stress capacity for concrete at the interface is neglected. According to Randl (2013) the shear resistance of concrete interfaces subjected to fatigue loading has not been investigated in a comprehensive way. There is no requirement for minimum shear reinforcement according to CEB-FIP Model Code 2010 (2010) and Eurocode 2 Part 1-1 (2004) if the shear stress does not exceed the shear capacity of the joint. Prestressed bridge beams will have vertical shear reinforcement crossing the interface between beam web and on-site insitu top slab and it is the author's observation from his own experience in designing prestressed bridge beams that this steel is adequate to cater for horizontal interface requirements. This is indeed the feeling of the technical committee (CEN/TC 250/SC 2/WG 1: TG4) elected to draft the new section on interface shear for the upcoming revised Eurocodes, a committee that the author is a member of.



Figure 2.13: W Beam being poured in a precast factory

Chung and Chung (1976) were one of the first to carry out dynamic testing in 1976. They concluded that fatigue failure of the interface joint is not probable if the repeated load is not more than 55% of the ultimate static load or if the horizontal shear stress at the interface does not exceed $2.75N/mm^2$. That is an extremely high figure considering BS5400-4 (1990) for a C40/50 grade concrete and type 1 treated surface finish is approximately 0.8 MPa. Eurocode 2 Part 1-1 (2004) for a similar concrete strength and a rough interface (1.5mm average roughness) is 0.65 MPa. CEB-FIP Model Code 2010 (2010) for a similar concrete strength and very rough interface is also 0.65 MPa. The calculated capacity of an unreinforced interface has lowered through code provisions compared with test data since 1976. What is also shown is the variety of values between the three bridge codes used by Irish and UK bridge engineers for the past twenty years. Hughes (1987) carried out small-scale push-off tests, small-scale RC T-beams and large-scale precast prestressed M-beams in 1987 using both static and cyclical loading. He concluded that service stresses in bridgeworks tended to be $2.0N/mm^2$ and that the cracking stresses of $3.0N/mm^2$, $2.5N/mm^2$ and $2.0N/mm^2$ for monolithic, roughened and rough as-cast surfaces respectively proved that the interfaces would remain uncracked at service loads. These results are higher than the capacities of the three bridge codes and closer in fact to the $2.75N/mm^2$ of Chung and Chung (1976). Loov and Patnaik (1994) carried out tests on RC T-beams in 1994 the results of which proved that the stirrups are typically unstressed and ineffective until horizontal shear stress exceeded 2.0 MPa. This indicates the strength of an interface without stirrups. They did confirm that the stirrups became relatively effective when the shear stress reached 3.0 MPa.

In summary, not enough testing has been carried out to validate the overly conservative approach of reducing c to zero for fatigue verifications. In reality all bridges are dynamically loaded as they are used by vehicular traffic which cause repeated and cyclic loads. To reduce c to zero results in an uneconomical design. Further laboratory testing should be sought to investigate clause 6.2.5(105) of Eurocode 2 Part 2 (2005).

Chapter 3

Phase 1 - Experimental Program, Analysis and Discussion

3.1 Phase 1 Introduction

From the extensive literature review the most common small to medium scale experimental testing specimen is the reinforced concrete T-Beam. This form of specimen was used by Hanson (1960), Badoux and Hulsbos (1967), Chung and Chung (1976), Hughes (1987), Loov and Patnaik (1994), Kahn and Slapkus (2004) and Mahmoud et al. (2014). With the rapid increase in concrete strengths and concrete technology it was felt that experimenting, as well as researching previous findings, was the way forward for this research project. The author also had access to a precast concrete manufacturer who was keen on experimental testing.

Therefore, the first phase of testing examined the performance of 13 3.2m long reinforced concrete (RC) T-beams split into 4 series each containing 3 beams plus a control beam. These were all statically loaded using a 4-point loading test with the loading and reinforcement crossing the interface the varying parameters. The beam length, section size, test parameters and the use of a 4-point load was chosen after the extensive literature review.

It is the author's observation from working on small to medium bridge spans (20m to 50m) that C57/70 grade precast prestressed concrete bridge beams are predominately used with C40/50 insitu top slabs. The finish on the cast against concrete surface is also a very important factor in calculating the interface shear strength. Eurocode 2 Part 1-1 (2004) and the CEB-FIP Model Code 2010 (2010) classify the different applicable types of finishes into four categories, very smooth (cast against a steel or timber form), smooth (left as cast), rough (exposing the aggregate or raking the concrete) and very rough (shear key or indented concrete surface). Depending on the finish of the cast against concrete the two parameters, friction (μ) and cohesion (c) are then calculated. For the RC T-beams a rough finish was used as this is the most common type of finish used by precasters (Sources: Banagher Precast Concrete and Shay Murtagh Precast Concrete). These parameters were confirmed by senior bridge engineers practising in Ireland and the UK (Sources: Finian Burke (ARUP Consulting Engineers Dublin) and Michael Minehane (RPS Consulting Engineers Cork)).

3.2 Design of the Reinforced Concrete (RC) T-Beam

3.2.1 Flexure and Vertical Shear

The test specimen's section size was selected after an extensive literature review of horizontal shear research (Hanson (1960), Saiidi et al. (1990) and Kahn and Slapkus (2004)). The RC T-Beam was designed to be simply supported with a two point loading arrangement symmetrical about the centre point. The reaction at each support is equal to the applied load. This results in a shear span of constant shear extending out from the support to the applied load with zero shear between the two loading points, this can be seen in Figure 3.1. A trapezoidal bending moment diagram results which can be seen in Figure 3.2.

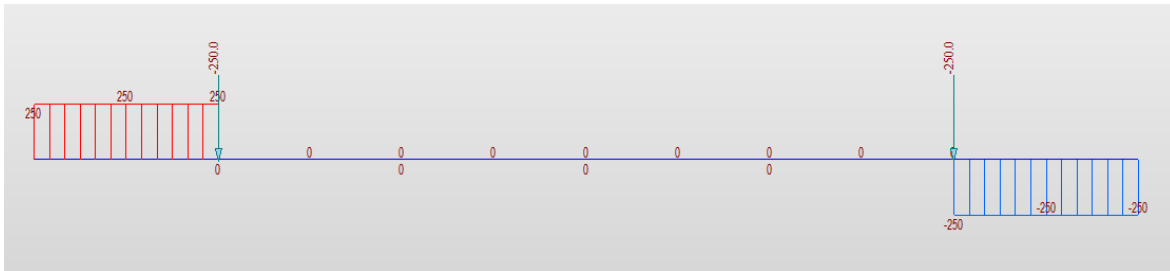


Figure 3.1: Shear Force Diagram for the T-Beams under 250 kN point loads (Self weight of Beam ignored)

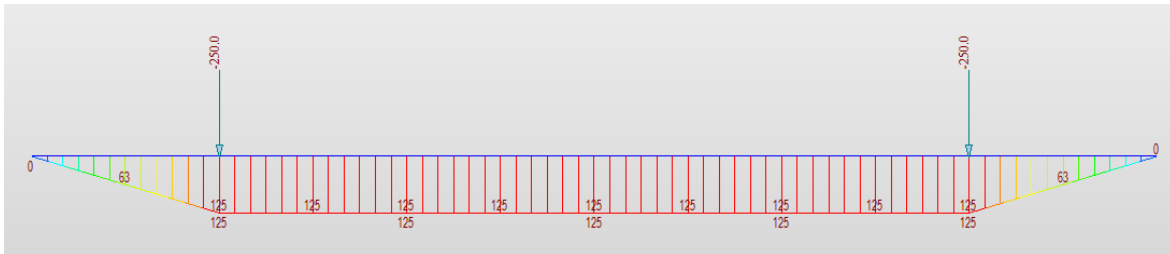


Figure 3.2: Bending Moment Diagram for the T-Beams under 250 kN point loads (Self weight of Beam ignored)

This loading arrangement results in a large area of zero shear which theoretically corresponds to an area of zero horizontal shear. Therefore this area has maximum moment and flexural failure could result. The beams are designed as per Eurocode 2 Part 1-1 (2004) and are over reinforced for flexure and vertical shear so that horizontal shear failure at the interface is achieved first. 250 kN was chosen as the optimum point load in order for high horizontal shear stresses to exist at the interface. This gave a horizontal shear stress of 3.33 MPa on the interface. The shear capacity of the interface was calculated to be 0.65 MPa, therefore it was felt that an interface failure would occur at this load.

3.2.2 Interface Shear Calculation

The following set of calculations are based on Eurocode 1 Part 2 (2002). A comparison was also carried out on all of the other major bridge design codes and is shown in Table

2.4 at the end of Chapter 2.

Design value of interface shear stress, Exp(6.24):

$$\begin{aligned} V_{Edi} &= \frac{\beta V_{Ed}}{z b_i} \\ &= \frac{1 \times 250 \times 1000}{375.44 \times 200} \\ &= 3.33 MPa \end{aligned} \quad (3.1)$$

where;

β = is the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered

V_{Ed} = is the transverse shear force, (kN)

z = is the lever arm of the composite section, (mm)

b_i = is the width of the interface, (mm)

Design shear resistance of the interface, Exp(6.25):

Rewriting Exp (6.25) to calculate Interface Links:

$$\begin{aligned} \rho_{min} &= \frac{V_{Edi} - c f_{ctd} - \mu \sigma_n}{f_{yd}(\mu \sin \alpha + \cos \alpha)} \\ &= \frac{3.35 - 0.4 \times 1.64 - 0.7 \times 0}{435(0.7 \times \sin 90 + \cos 90)} \\ &= 0.0088 \end{aligned} \quad (3.2)$$

where;

c = is the cohesion factor

μ = is the coefficient of friction

f_{ctd} = is the tensile stress of the new concrete layer, (MPa)

f_{yd} = is the design yield strength of reinforcement, (MPa)

α = is the angle of reinforcement to interface, (Degrees)

Area of Links Required:

$$\rho_{min} \times b = 0.0088 \times 200 = 1.77 mm^2/mm \quad (3.3)$$

Area of Links Provided:

$$\begin{aligned} 2B12@125mm c/c's &= 1.81 mm^2/mm \\ \rho_{Provided} &= 0.0090 \end{aligned} \quad (3.4)$$

Design Shear Resistance of the Interface without interface reinforcement:

$$V_{Rdi} = c f_{ctd} + \mu \sigma_n = 0.4 \times 1.64 + 0.7 \times 0 = 0.65 MPa \quad (3.5)$$

Design shear resistance of the interface using provided reinforcement, Exp(6.25):

$$\begin{aligned} V_{Rdi} &= c f_{ctd} + \mu \sigma_n + \rho f_{yd}(\mu \sin \alpha + \cos \alpha) \\ &= 0.4 \times 1.64 + 0.7 \times 0 + 0.0088 \times 435(0.7) \\ &= 3.41 MPa \end{aligned} \quad (3.6)$$

3.3 Beam Fabrication

All of the RC T-Beams were manufactured in Banagher Precast Concrete, Co Offaly. Banagher Precast Concrete is the leading manufacturer of precast concrete products in Ireland and the United Kingdom and kindly supplied all materials for the manufacture and testing of the beams. The mould for the beam web was constructed out of timber which can be seen in Figure 3.3.



Figure 3.3: TB1-2, TB1-3 and TB1-4 Manufacturing Stages

13 RC T-Beams were manufactured in the Spring and Summer of 2016 and tested in Trinity College Dublin and Banagher Precast Concrete in the Summer and Winter of 2016. Each beam (see Figure 3.4) had a top flange width of 400mm and depth of 150mm. The web was 400mm deep and 200mm wide. The beams were 3200mm long with an effective span of 3000mm. The beams were overly reinforced for bending with 2Nr 32mm bars running longitudinally and overly reinforced for vertical shear with 12mm links at 150mm centres. 4Nr 12mm lacer bars ran longitudinally in the beam and 8Nr 10mm lacer bars ran longitudinally in the top slab. The beams were designed to fail in interface shear only.

These beams were split into 4 series, with 3 beams in each series. A monolithically poured control beam was also manufactured and tested statically to failure in order to obtain its ultimate strength. The remaining 3 beams in each series were loaded statically to failure.

Test series 1 beams had no reinforcement crossing the interface. Test series 2 beams had 10mm stirrups at 150mm centres (0.5% bh) crossing the interface, test series 3 beams had 10mm stirrups at 300mm centres (0.25% bh) crossing the interface and test

series 4 beams had 10mm stirrups at 500mm centres (0.15% bh) crossing the interface. The varying parameters were the level of load and the ratio of reinforcing stirrups crossing the interface. The beams were poured with C57/70 concrete and the top slabs were cast 28 days later with C40/50 concrete. The beams had a rough surface finish in accordance with Eurocode 2 Part 1-1 (2004) section 6.2.5 (2) and this was tested using the sand patch procedure.

The beams were tested, as will be described later, up to the appearance of the first crack at the interface except for the monolithic control beam which was tested to failure. This is the proposed failure criteria for the testing and follows the work carried out by Hanson (1960) and Chung and Chung (1976).

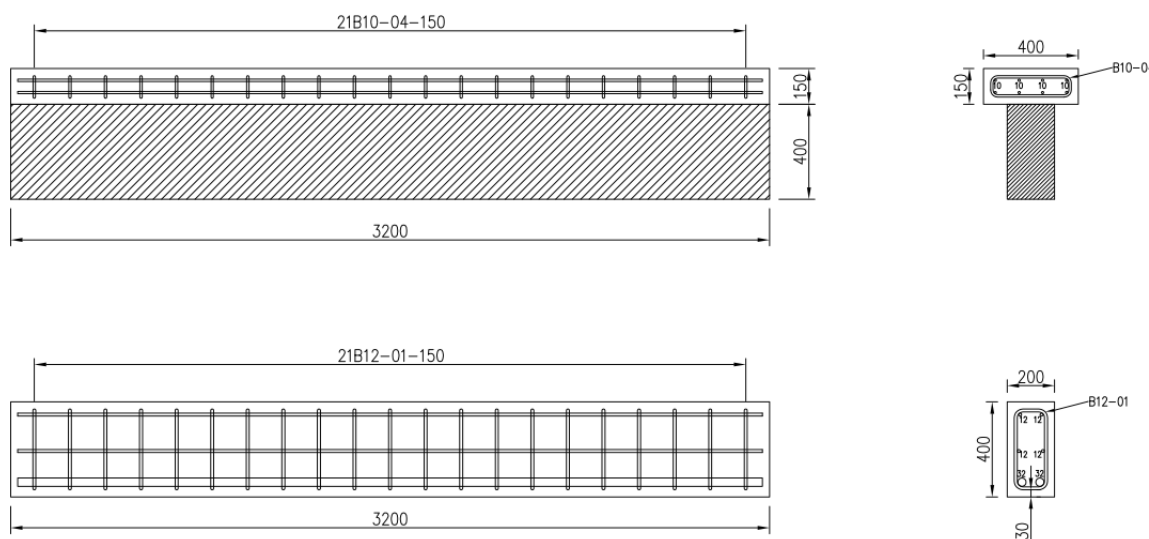


Figure 3.4: TB1-2, TB1-3 and TB1-4 Production Drawing

3.3.1 Design Life, Exposure Class, Nominal Cover and Concrete Mix Design

Bridge structures have a 100 year design life. In order to achieve this design life they must be designed for durability as well as strength. First the applicable exposure classes are calculated, then a concrete mix is chosen based on a nominal reinforcement cover depth. All of this information is crucial in order to detail the reinforced concrete T-beams accurately. Figure 3.5 shows a cross section through a prestressed concrete bridge beam and cast in place deck slab. This is a standard construction detail for a beam and slab type bridge deck on a small to medium sized (20-50m) highway or railway bridge. The exposure class and nominal cover values are shown on the diagram. The exposure to the top of the insitu slab will be XC3 as it gets protected by a waterproofing membrane on site before the bridge is open to traffic. The sides and soffit of the beams will be XD1 exposure as there will be airborne chlorides. The internal void is XC3 exposure as there will only be carbonation in this location.

The concrete mix is selected using strength and durability criteria. The beam is required to be C57/70 for strength purposes and to achieve a high transfer strength of C40/50 after 24 hours so that the prestress can be transferred into the beams and the beams can be demoulded.

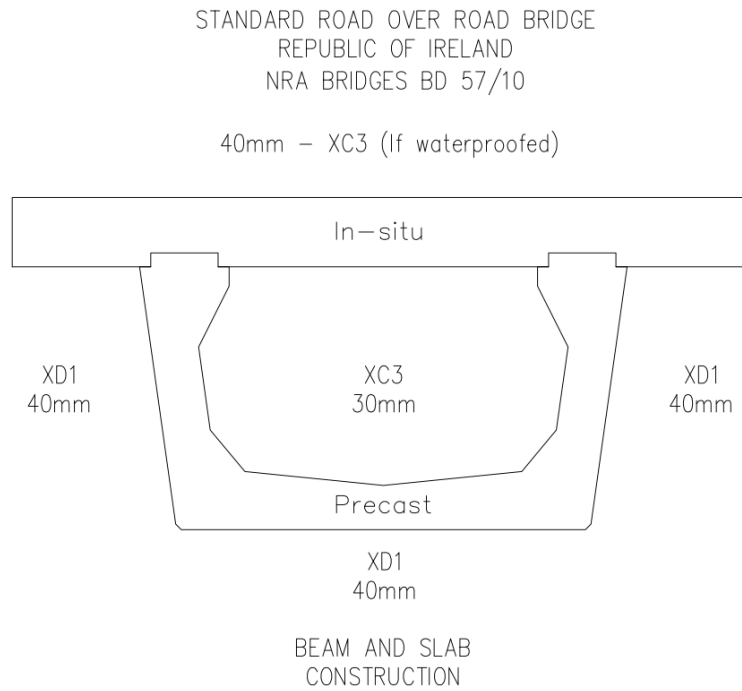


Figure 3.5: Exposure Class Diagram

Table 3.1 shows the C57/70 mix design for the prestressed concrete bridge beams. There are 475kgs per m³ of CEM1 cement in the mix to get the required strength. There is no cement replacement used in the mix as these slow the early strength gain of the concrete mix according to Banagher Precast Concrete, the precast beam manufacturer. The early strength is critical in precast operations as the prestress cannot be transferred into the beams until the transfer strength has been reached. Occasionally the transfer strength dictates the 28 day strength. The water cement ratio is 0.4, which is as low as a precaster will go to in order to achieve a quality product in terms of the aesthetic. Finally a superplasticiser, Premia 360, is used which is a high range water reducer based on modified polycarboxylate. It is recommended for concrete requiring high short and long term compressive strengths.

Table 3.2 shows the C40/50 mix design for the insitu deck slab. There are 380kgs per m³ of CEM1 cement in the mix to get the required strength. A cement replacement like GGBS or PFA could be used on site but was not chosen on this occasion. The superplasticiser was also used.

3.3.2 Cube Results

Three Standard 100mm x 100mm cubes were taken for all beams and slabs that were cast in order to obtain an average 28 day strength. Table 3.3 displays all of these results. The table contains the date that each beam and slab were poured in the factory. The concrete grade, test date, age at time of test and the average 28 day result. The C57/70 grade concrete cubes were all over 70 N/mm² at 28 days as were the C40/50 grade concrete cubes. The C40/50 grade concrete cubes were much higher than the design figure of 50 N/mm² at 28days and this is purely down to the precaster. The C40/50 grade concrete used in Banagher Precast Concrete is one of the more popular mix designs. In a precast concrete factory repetition is key and moulds are

Table 3.1: C57/70 Prestressed Beam Mix Design

Material Description	Mass per m ³	Units	Material Supplier
Portland Cement CEM I	475	kg's	Irish Cement
20mm Aggregate	830	kg's	Banagher Precast Concrete
Sand	370	kg's	Banagher Precast Concrete
Crushed Limestone Fines	560	kg's	Banagher Precast Concrete
Max Free Water	180	Ltrs	Banagher Precast Concrete
Water Cement Ratio	0.4		
Admixture Description			
Premia 360 - superplasticiser	4	Ltrs	Chryso

Table 3.2: C40/50 Deck Slab Mix Design

Material Description	Mass per m ³	Units	Material Supplier
Portland Cement CEM I	380	kg's	Irish Cement
14mm Aggregate	600	kg's	Banagher Precast Concrete
8mm Aggregate	320	kg's	Banagher Precast Concrete
Sand	855	kg's	Banagher Precast Concrete
Max Free Water	152	Ltrs	Banagher Precast Concrete
Water Cement Ratio	0.40		
Admixture Description			
Premia 360 - superplasticiser	2	Ltrs	Chryso

used every day. The turnover of a mould in 24 hours is key and concrete is required to be a certain strength (normally C28/35) in order to demould large precast units safely without damaging them.

Table 3.3: Concrete Cube Compressive Strength Results Phase 1

Pour Date	Element	Concrete Grade	Test Date	Age (Days)	Cube Size (mm)	Ave 28 Day Result (Days)
24/04/16	TB1-2(B)	C57/70	23/05/16	28	100 x 100	84.84
23/05/16	TB1-2(S)	C40/50	20/06/16	28	100 x 100	74.60
04/05/16	TB1-3(B)	C57/70	01/06/16	28	100 x 100	73.67
01/06/16	TB1-3(S)	C40/50	29/06/16	28	100 x 100	72.77
27/04/16	TB1-4(B)	C57/70	25/05/16	28	100 x 100	78.72
25/05/16	TB1-4(S)	C40/50	22/06/16	28	100 x 100	76.30
16/05/16	TB2-1(B)	C57/70	13/06/16	28	100 x 100	72.26
16/05/16	TB2-1(S)	C40/50	13/06/16	28	100 x 100	72.15
28/04/16	TB2-2(B)	C57/70	26/05/16	28	100 x 100	65.24
26/05/16	TB2-2(S)	C40/50	23/06/16	28	100 x 100	78.01
29/04/16	TB2-3(B)	C57/70	27/05/16	28	100 x 100	64.04
30/05/16	TB2-3(S)	C40/50	27/06/16	28	100 x 100	71.18
03/05/16	TB2-4(B)	C57/70	31/05/16	28	100 x 100	66.02
31/05/16	TB2-4(S)	C40/50	28/06/16	28	100 x 100	70.00
26/04/16	TB3-2(B)	C57/70	24/05/16	28	100 x 100	79.43
24/05/16	TB3-2(S)	C40/50	21/06/16	28	100 x 100	77.52
06/05/16	TB3-3(B)	C57/70	03/06/16	28	100 x 100	72.07
07/06/16	TB3-3(S)	C40/50	05/07/16	28	100 x 100	62.53
11/05/16	TB3-4(B)	C57/70	08/06/16	28	100 x 100	79.13
10/06/16	TB3-4(S)	C40/50	08/07/16	28	100 x 100	63.56
10/05/16	TB4-2(B)	C57/70	07/06/16	28	100 x 100	71.91
09/06/16	TB4-2(S)	C40/50	07/07/16	28	100 x 100	64.76
09/05/16	TB4-3(B)	C57/70	07/06/16	29	100 x 100	72.28
08/06/16	TB4-3(S)	C40/50	06/07/16	28	100 x 100	68.66
12/05/16	TB4-4(B)	C57/70	09/06/16	28	100 x 100	69.81
13/06/16	TB4-4(S)	C40/50	11/07/16	28	100 x 100	68.76



Figure 3.6: Phase 1 28-day compressive strength

3.4 Sand Patch Test

The sand patch test is still the simplest way of obtaining the surface roughness of the interface. The test involves spreading a known quantity of fine calibrated sand on the surface, spreading it out in a circular fashion and measuring the area of the circle once the sand has spread out and finished off flush with the surface. Using the known volume and dividing this by the now calculated area results in an average thickness. Equation 3.7 is used to calculate the average thickness of the interface.

$$Thickness = \frac{V}{\pi r^2} \quad (3.7)$$

where;

V = Volume of sand

r = average radius of the measuring circle

The sand patch test is inexpensive and easy to perform both on site and in the laboratory. However, narrow surfaces with high ratios of reinforcement crossing the interface make the test difficult. Table 3.4 summarises the results of the sand patch tests carried out on the RC T-beams tests. 4 sample precast surfaces were manufactured and tested in order to produce a template for the required surface finish, see S1, S2, S3 and S4 in Table 3.4. All beams apart from TB1-4 had rough surface finishes all greater than the limit of 3mm. The affects of the lower surface finish on the interface shear capacity is discussed later in the chapter.

Table 3.4: Results of Sand Patch Test on Test Beams

Sample	No. of tests	Ave. dia. (mm)	Area (m ²)	Vol. of sand used (ml)	Thickness (mm)	Thickness Req. (mm)	Pass / Fail
S1	2	230	0.042	95	2.29	3	Fail
S2	2	250	0.049	95	1.94	3	Fail
S3	2	180	0.025	95	3.73	3	Pass
S4	2	175	0.024	95	3.95	3	Pass
TB1-2	2	190	0.028	95	3.35	3	Pass
TB1-3	2	170	0.023	95	4.19	3	Pass
TB1-4	2	157.5	0.019	47.5	2.44	3	Fail
TB2-2	2	135	0.014	47.5	3.32	3	Pass
TB2-3	2	135	0.014	47.5	3.32	3	Pass
TB2-4	2	140	0.015	47.5	3.09	3	Pass
TB3-2	2	180	0.025	95	3.73	3	Pass
TB3-3	2	182.5	0.026	95	3.63	3	Pass
TB3-4	2	177.5	0.025	95	3.84	3	Pass
TB4-2	2	170	0.023	95	4.19	3	Pass
TB4-3	2	180	0.025	95	3.73	3	Pass
TB4-4	2	177.5	0.025	95	3.84	3	Pass

3.5 Phase 1 Experimental Study

3.5.1 Scope

The beam size was chosen after an extensive literature research on past studies, see Figures 3.7, 3.8 and 3.9. This enabled similar research to be carried out with a view to comparing results with past research. The small reinforced concrete composite T-beam represents a short bridge beam with a composite top slab. The thin web and cast insitu top slab is synonymous with bridge works and effectively represents the beam and slab type construction of so many bridges throughout Europe.

The variables investigated were:

- The area of reinforcement crossing the interface. The stirrup diameter was constant but the spacing varied
- Loading

The constants were:

- Concrete strength. The beams were all C57/70 and the slabs C40/50.
- Interface roughness. The beams were all roughened to an average depth of 3mm minimum.

These parameters are quite typical of precast bridge beams and cast in place insitu top slabs.

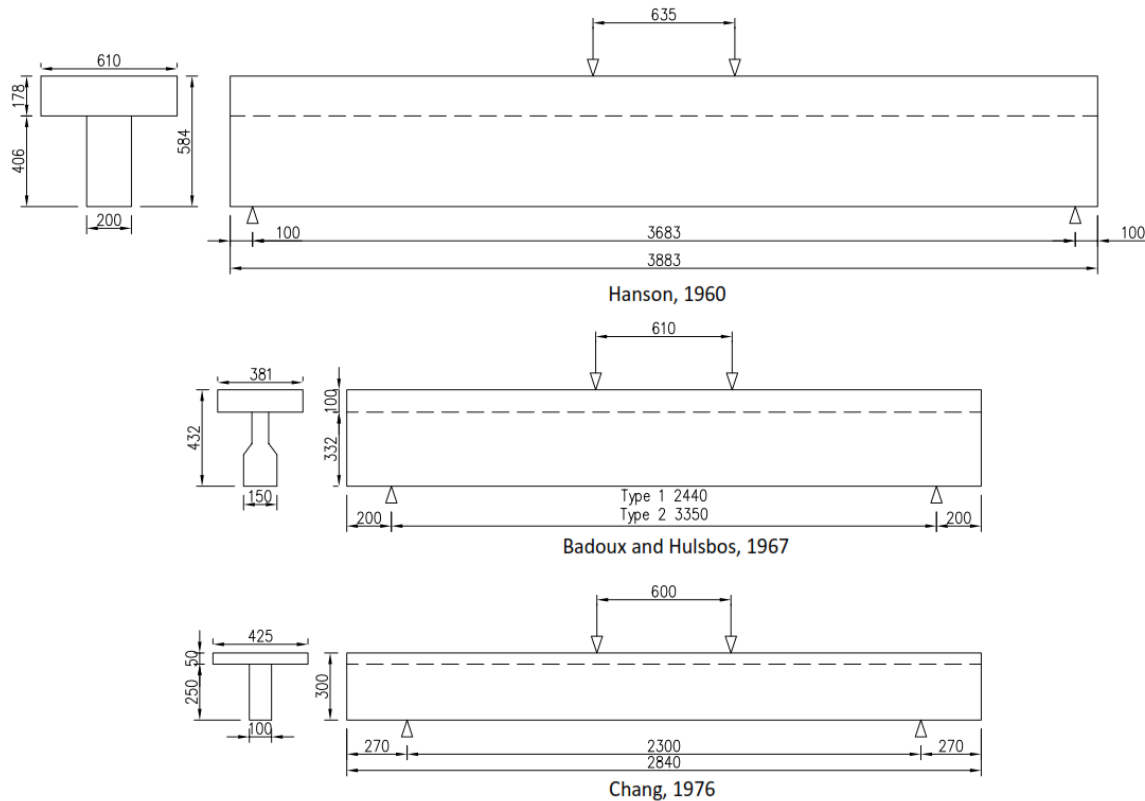


Figure 3.7: Previous Testing 1960 - 1980

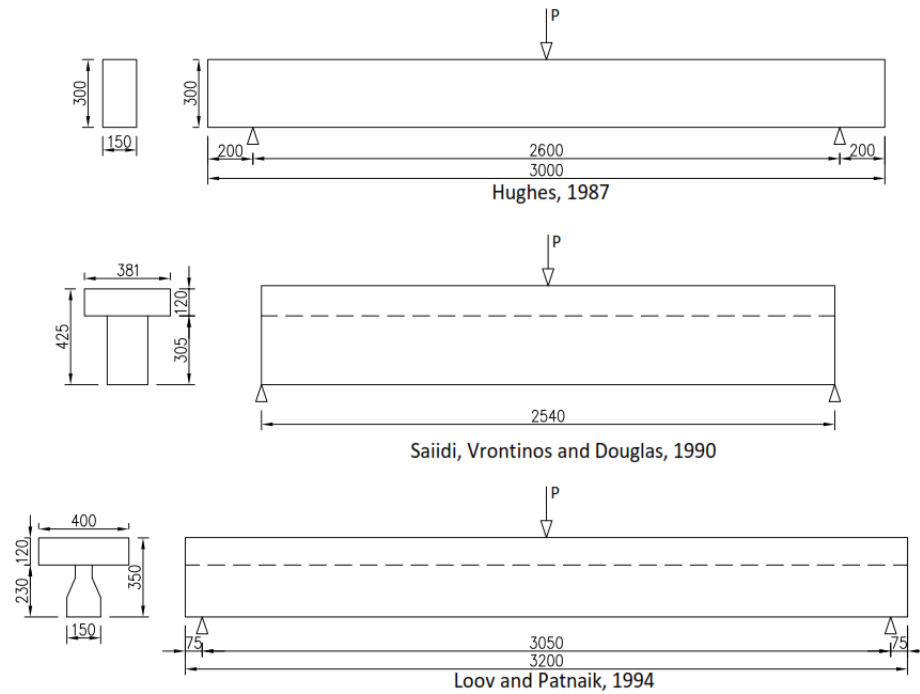


Figure 3.8: Previous Testing 1980 - 2000

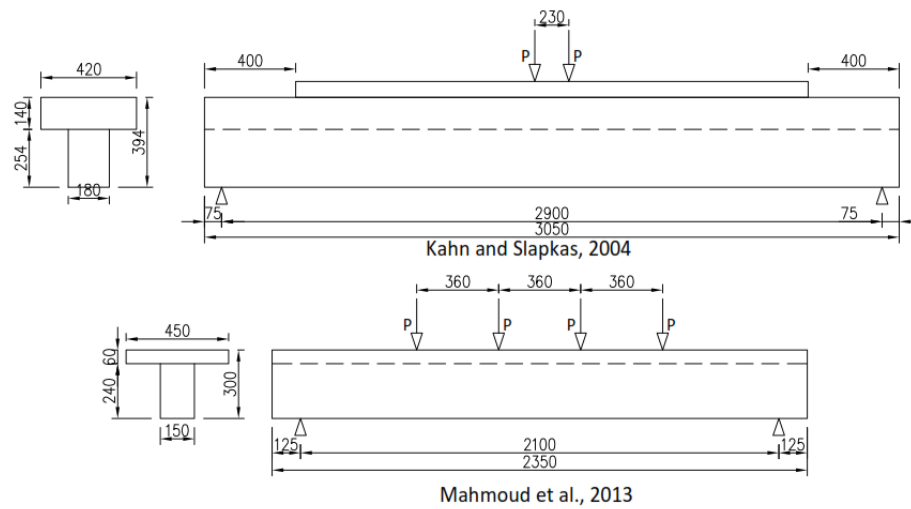


Figure 3.9: Previous Testing 2000 - 2015

3.5.2 Specimens

A total of 13 reinforced concrete T-Beams were manufactured and tested. These were split into 4 groups, each group had 3 beams and finally a monolithic control beam to test the capacity of the loading rig and gain an ultimate load criterion. The test specimens of series 1 were called TB1-2, TB1-3 and TB1-4. These beams had no stirrups crossing the interface and had a calculated interface shear resistance of 0.65 MPa, 20% of the applied loading stress. The test specimens of series 2 were called TB2-2, TB2-3 and TB2-4. These beams had 10mm diameter stirrups at 150mm c/c's (0.5% bh) crossing the interface and had a calculated interface shear resistance of 2.25 MPa, 67% of the applied loading stress. The test specimens of series 3 were called TB3-2, TB3-3 and TB3-4. These beams had 10mm diameter stirrups at 300mm c/c's (0.25% bh) crossing the interface and had a calculated interface shear resistance of 1.45 MPa, 44% of the applied loading stress. Finally the test specimens of series 4 were called TB4-2, TB4-3 and TB4-4. These beams had 10mm diameter stirrups at 500mm c/c's (0.15% bh) crossing the interface and had a calculated interface shear resistance of 1.13 MPa, 34% of the applied loading stress. The applied loading stress was based on a 250 kN point load. The production drawing for test beam series 2 is shown in Figure 3.10. The lower part of the drawing shows the reinforcement that is cast into the beam and the top part of the drawing shows the reinforcement that is in the insitu top slab.

The monolithic control beam had 10mm diameter stirrups at 150mm c/c's which was the strongest beam in the range and gave the ultimate loading capacity from which to test the remaining composite specimens. The production drawing for the control beam is shown in Figure 3.11.

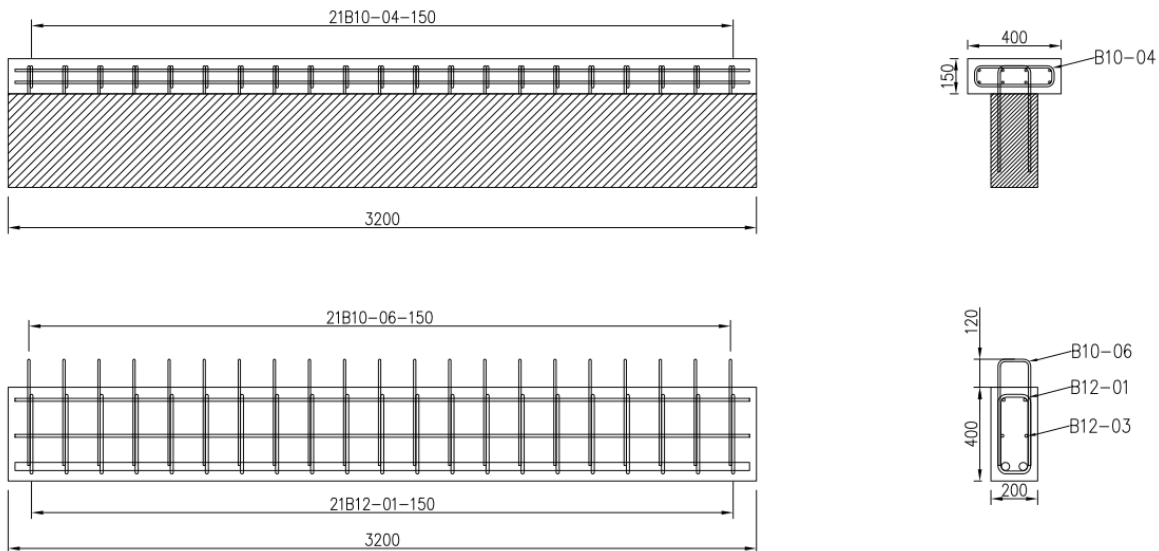


Figure 3.10: TB2 Production Drawing

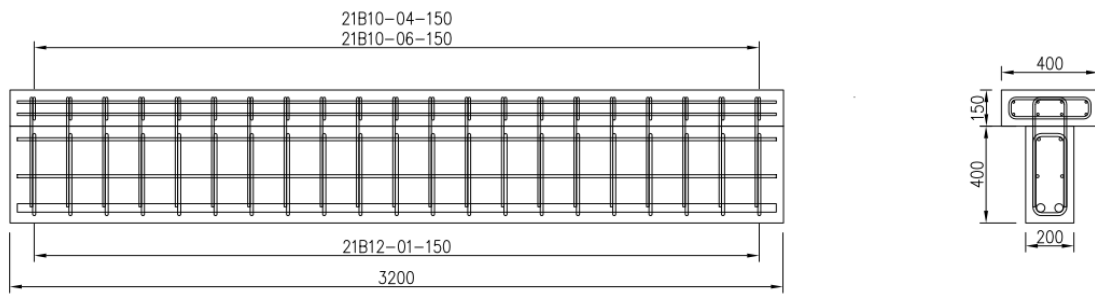


Figure 3.11: Control Beam Production Drawing

3.5.3 Surface preparation

Before the test beams were manufactured, 4 samples (see Figure 3.13), sized 450mm x 450mm were cast to develop a procedure to achieve a surface with 3mm of roughness as per section 6.2.5 (2) of Eurocode 2 Part 1-1 (2004). The first two samples were poured and left to set, after approximately 15 minutes 8mm aggregate was dashed on top of the still wet concrete and left to dry. The following day the top surface of the concrete was power washed and any loose material blown off the surface. This method gave an average roughness of approximately 2mm. Samples 3 and 4 were poured and left to set, after approximately 15 minutes 8mm aggregate was dashed on top of the still wet concrete. After a further 30 minutes a roughening tool was used on the surface, this roughening tool was manufactured out of timber with projecting wood screws and can be seen in Figure 3.12. The device was 120mm wide so as to be able to fit between the projecting interface links cast into the beams at 150mm centres. Samples 3 and 4 gave roughnesses of approximately 4mm and were used as templates for the finish to the tops of the beams. Table 3.4 shows the individual results of the surface finish to each test beam. All beams bar TB1-4 (2.44mm) had an average roughness greater than 3mm.



Figure 3.12: Surface Roughening Tool

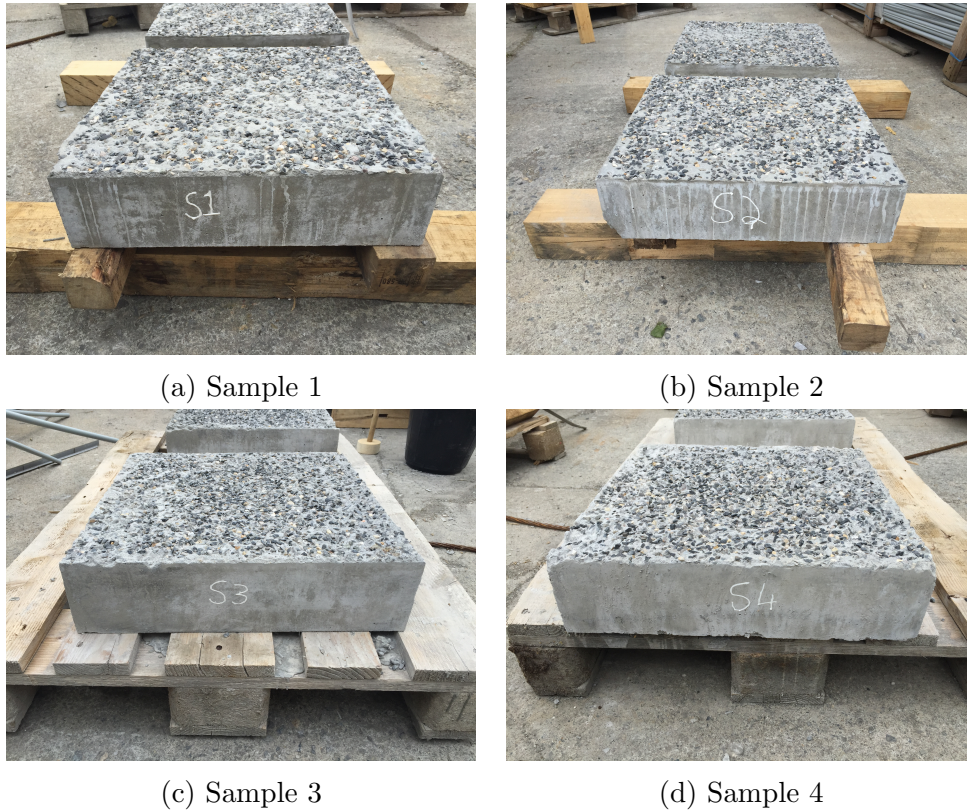


Figure 3.13: Surface roughness preparation samples

3.5.4 Testing instrumentation

(a) Crack Microscope

The crack microscope (Figure 3.14a) is designed for measuring crack widths in concrete members. The apparatus incorporates an adjustable lamp and the image is focused by turning a knob. The eyepiece scale can be turned through 360 degrees to align with the direction of the crack or pitch under examination. There is a measurement scale which you hold over the crack, this scale picks up cracks from 0.02mm.

(b) Movement of the Slab relative to the Beam

Linear Voltage Displacement Transducers (LVDTs) are a popular and accurate piece of equipment for measuring the relative displacement between two points. LVDTs are measuring devices based on the linear differential transformer principal which produces an electrical output signal precisely proportional to the mechanical displacement of the sensing probe. An LVDT was positioned on the test beams at midspan to measure the vertical movement of the slab relative to the beam which can be seen in Figure 3.14b



(a) Crack Microscope Phase 1



(b) LVDT to measure the displacement of the slab relative to the beam

Figure 3.14: Testing instrumentation 1 of 2

(c) Vertical Slip Gauges

Displacement transducers were also used to measure the vertical displacement of the slab relative to the beam, four per beam positioned under the loading points each side of the beam, see Figure 3.15a. They are based on 10kOhm precision potentiometers which have their threaded screws detached from their usual anchor point. This facilitates movement of the screw in a piston motion which results in a change in resistance linearly proportional to displacement. The head of the screw is anchored independently of the potentiometer body so that the movement of one part relative to the other can be measured. An excitation voltage of 2.5Vdc is applied to the potentiometer and the signal output is taken from the wiper pin.

(d) Strain Gauges

Strain gauges (see Figure 3.15b) were positioned at the bottom of the interface link projecting into the top slab just below where the top slab link joins with the interface link. The methodology behind this was that once the interface link began to strain the surface interface was no longer resisting the load and had failed. If the strain gauge was higher on the interface link some additional resistance may have come from the top slab link, which would have distorted the results. TB4-2, TB4-3 and TB4-4 all had 2 sensors installed, one at each end approximately an effective depth past the support. TB2-2, TB2-3, TB2-4, TB3-2, TB3-3 and TB3-4 all had 4 sensors installed as they contained higher ratios of interface shear reinforcement.



(a) View of Potentiometers positioned stuck to slab with screw touching the underside of the slab



(b) Strain gauge positioned on stirrup of TB4-3 just above the interface and protected with black silicon

Figure 3.15: Testing instrumentation 2 of 2

3.5.5 Test procedure

Testing took place in Trinity College Dublin and Banagher Precast Concrete. The beams were placed on 50mm solid round bars to act as point supports 100mm from the end of the 3.2m long beam giving an effective span of 3.0m. The two point loads were applied 500mm from each end support, again using a spreader beam with 50mm solid round bars to apply the point loads. The set-up is shown in Figure 3.16.

All of the beams were subject to static tests. The first beam that was tested was TB2-1, the monolithic beam to calculate the ultimate breaking load capacity. The test rig in Trinity had a capacity of a single 1000 kN point load at midspan and when positioned onto the spreader beam it would disperse into two 500 kN loads. The capacity of the interface with 2 legs of 10mm diameter stirrups at 150mm c/c's was 2.25 MPa. A 500 kN point load resulted in a shear stress on the interface of 6.70 MPa, triple the capacity. Therefore it was thought that the rig capacity would not be reached before interface failure.

TB2-1 was tested on the 4th of August 2016. The data logger was set up and recorded load, displacement and strain on one system. The load was steadily placed and stopped at 50 kN intervals (25 kN at each end) to measure cracks and observe the beam. The first hairline crack was witnessed at 400 kN well above the 350 kN capacity of the interface. The vertical shear crack was measured using the crack microscope and found to be less than 0.05mm. The beam was behaving as a composite beam and the crack was a standard vertical shear crack running from the load point to the support point. The load was increased and again stopped at 50 kN intervals. The test rig capacity peaked at 1150 kN, which placed 7.70 MPa of shear stress on the interface with no

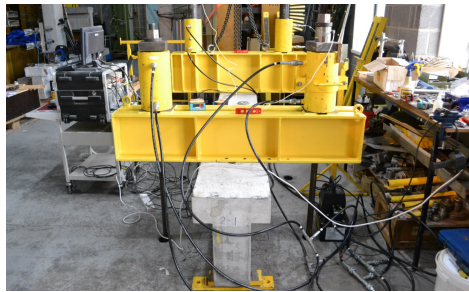
signs of any cracks. There were several vertical shear and flexural cracks present as can be seen in Figure 3.16b. The beam was designed to be able to resist two 250 kN point loads not two 575 kN point loads. The largest of these cracks was 0.3mm. Slight concrete spalling at the supports and at the load points was also noticed which would have resulted in a failure. The test took approximately 45 minutes to complete.



(a) TB2-1 Post Test Picture



(b) TB2-1 Post Test Picture 2



(c) TB2-1 Post Test Picture 3



(d) TB2-1 Post Test Picture 4

Figure 3.16: Selection of photo's showing TB2-1 after the test was carried out

3.5.6 Phase 1 Testing - First Series

The beams were designed for flexure and shear based on a load of 500 kN. The load was placed at midspan and onto a spreader beam as per Figure 3.17. This resulted in an interface shear stress of 3.35 MPa under the 2 load points. 3 composite reinforced concrete T-beams were tested under static loading. The beams had no stirrups crossing the interface. According to Eurocode 2 Part 1-1 (2004) the capacity of the interface without shear reinforcement is 0.65 MPa.

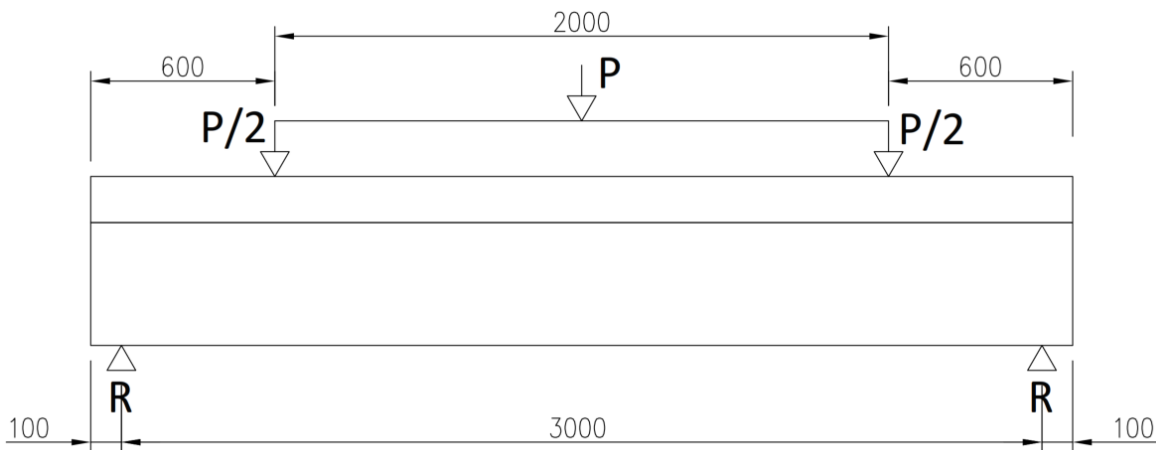


Figure 3.17: TB1 Load Arrangement

TB1-2

TB1-2 was tested on the 15th of August 2016 and was 57 days old. The average roughness of the interface was 3.35mm. The first hairline cracks were witnessed at 400 kN. The load was being increased in 50 kN intervals and there were no cracks at 350 kN. The cracks were measured using the crack microscope and found to be less than 0.05mm. The beam was behaving like the monolithic control beam and the worst crack was a vertical shear crack running from the load point to the support point. There were also much smaller flexural cracks towards the midspan of the beam. At 550 kN some minor crushing of the concrete was witnessed under the load points. The cracks that opened at 400 kN were still much the same size. At 650 kN (4.35 MPa, over 6 times the design capacity of the interface) additional and longer vertical shear cracks were observed and these were of a size that measured 0.1mm. The cracks ran from the support to the junction of the slab and along the interface horizontally and then vertically up the slab to the location of the loading point. Unfortunately for this test the small potentiometers did not read and so the width of the crack could not be identified. From manual measurement with the crack microscope a value of 0.05mm maximum was identified however the interface did not slip and the beam behaved very much like a fully monolithic beam. The load was increased and the test stopped at 1180 kN when the vertical shear cracks exceeded 0.3mm - the maximum allowable crack width for reinforced beams. The capacity of the testing frame in Trinity College restricted further load increments and so the test was stopped. When the test finished the shear stress on the interface was 7.90 MPa, over 12 times the design capacity of the interface. At this stress there were no signs of interface slippage or failure. The interface crack was a maximum of 100mm long and travelled from load point to support point. It did not travel along the underside of the slab at any location apart from this. Figure 3.18 shows the interface crack pattern and Figure 3.19 shows a close up view looking from the underside of the slab. The crack is travelling up diagonally on the right hand side, it meets the underside of the slab (the interface) and travels along it for approximately 100mm, it then travels up the slab to meet the loading point.

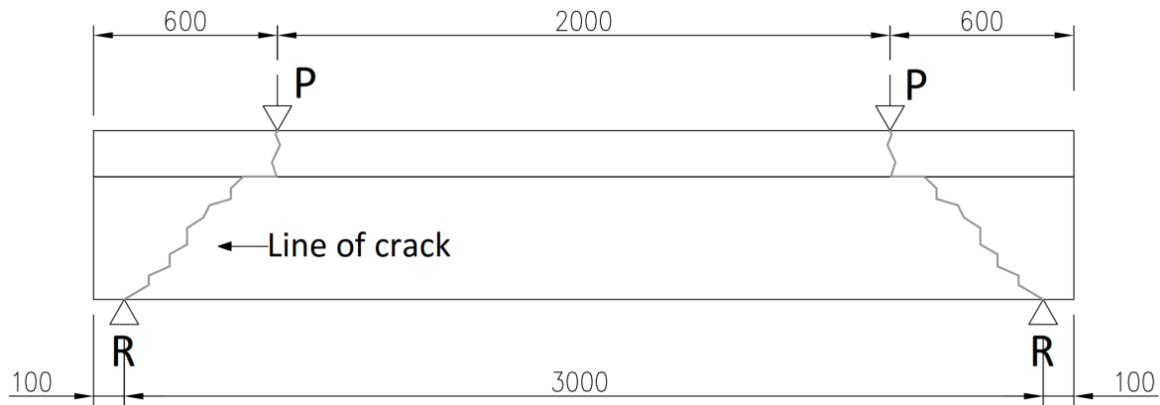


Figure 3.18: TB1-2 Failure Crack



Figure 3.19: Close up of Interface Crack

TB1-3

TB1-3 was tested on the 17th of August 2016 and was 50 days old. The average roughness of the interface was 4.19mm. The first hairline cracks were again witnessed at 400 kN, almost identical to TB1-2. At 800 kN (5.36 MPa, over 8 times the design capacity of the interface) and similar to the pattern of TB1-2 the first interface crack was witnessed, note the higher capacity of the interface which had a higher average surface roughness. The small potentiometers read 0.0015mm of a beam to slab displacement and proved delamination of the interface. These gauges did not measure slip just displacement. The beam and slab had deflected 4mm at this stage which was in line with the anticipated values from the computer software programme. From initial observations it was felt that these beams were far too strong and stiff to get a significant interface failure. The load was increased and the test stopped at 1100 kN when the vertical shear cracks were at a maximum of 0.32mm. When the test finished the shear stress on the interface was 7.37 MPa, over 11 times the design capacity of the interface. At this stress there was no sign of interface slippage or failure. The interface crack was identical to TB1-2 in its profile. The maximum displacement that was read from the gauges was 0.04mm. The cracks only ran for a short distance as per Figure 3.18 and Figure 3.19.

TB1-4

TB1-4 was tested on the 7th of October 2016 and was 108 days old. The average roughness of the interface was 2.44mm which failed the 3mm criteria set out in the tests. The first hairline cracks were witnessed at 350 kN, similar but slightly lower than TB1-2 and TB1-3 - possibly down to the lower surface roughness. At 600 kN (4.00 MPa, over 6 times the design capacity of the interface) and similar to the pattern of TB1-2 the first interface crack was witnessed. This was 50 kN lower than TB1-2 and 200 kN lower than TB1-3. Possibly as a result of the lower surface roughness of 2.44mm. The small potentiometers read 0.04mm of a beam to slab displacement. The beam and slab had deflected 3.5mm at this stage. The load was increased but the test was stopped at a load of 850 kN as the steel spreader beam in the Trinity laboratory was starting to buckle. When the test finished the shear stress on the interface was 5.69 MPa, over 8 times the design capacity of the interface. At this stress there were no signs of interface slippage or failure. The interface crack was similar to TB1-2 and TB1-3 in its profile. The maximum displacement that was read from the gauges was 0.045mm. The cracks only ran for a short distance as per Figure 3.18 and Figure 3.19.

3.5.7 Phase 1 Testing - Second Series

Similar to series 1 3 composite reinforced concrete T-beams were tested under static loading. The beams had 2 legs of B10 stirrups at 150mm centres crossing the interface. According to Eurocode 2 Part 1-1 (2004) the capacity of the interface with shear reinforcement equating to the above is 2.25 MPa compared to the 0.65 MPa capacity of TB1-2 beams. The calculated horizontal shear stress was calculated to be 3.35 MPa. The expectation before testing started was that this beam should be the strongest and should not fail in horizontal shear under the load capabilities of the test rig in Trinity College.

TB2-2

TB2-2 was tested on the 14th of October 2016 and was 115 days old. The average roughness of the interface was 3.32mm. The first hairline cracks were witnessed at 400 kN. These were measured using the crack microscope and found to be less than 0.05mm. The beam was behaving similarly to test series one beams. At 1150 kN the cracks that opened at 400 kN had increased in number and size to 0.15mm but still no interface crack. The test rig had been strengthened and the load was continued up to 1500 kN (10.04 MPa, over 4 times the design capacity of the interface). The 1500 kN value was far higher than the 600 kN and 800 kN values of test series 1 outlining the importance of interface links. At this point and without an interface crack the test was stopped as the rig was starting to show signs of failure. The widest vertical shear crack was 0.3mm. Once the load was taken off and the beam recovered 90% of the cracks closed and were not visible. The small potentiometers read a beam to slab displacement of 0.08mm so delamination occurred but was not visible to the observers. The small potentiometers started recording a displacement from approximately 300 kN starting at 0.01mm and continued to rise to 0.08mm at 1500 kN. At maximum load the beam had deflected 8.5mm.

TB2-3

TB2-3 was tested on the 19th of October 2016 and similar to TB2-2 was 115 days old. The average roughness of the interface was also similar to TB2-2 at 3.32mm. The first hairline cracks were witnessed at 350 kN. At 550 kN the cracks that opened at 350 kN had increased in number and size but still none greater than 0.1mm and no interface crack. At 1200 kN the widest cracks were 0.3mm but still no interface crack. There was the usual crushing of concrete at both load and support locations. The test was stopped at 1300 kN (8.71 MPa, approximately 4 times the design capacity of the interface). Once the load was taken off most cracks closed as per TB2-2. The small potentiometers read a beam to slab displacement of 0.04mm so delamination occurred but was not visible to the observers. The small potentiometers started recording a displacement from approximately 300 kN starting at 0.01mm and continued to rise to 0.04mm at 1300 kN. At maximum load the beam had deflected 8.0mm. TB2-3 was identical to TB2-2 in all parameters apart from the strength of the insitu top slab. TB2-2 had a 7.0 MPa additional strength to TB2-3 at 28 days. This could answer the question of how TB2-2 started to show signs of cracking at a load 50 kN greater than TB2-3.

TB2-4

TB2-4 was tested on the 21st of October 2016 and was 116 days old. The average roughness of the interface was 3.09mm. This was less than both TB2-2 and TB2-3 but still greater than the limit of 3mm. The first hairline cracks were witnessed at 400 kN. At 750 kN the cracks that opened at 400 kN had increased in number and size but still none greater than 0.1mm and no interface crack. The test was stopped at 1375 kN (9.21 MPa, approximately 4 times the design capacity of the interface). Once the load was taken off most cracks closed as per TB2-2 and TB2-3. The small potentiometers read a beam to slab displacement of 0.06mm so delamination occurred but was not visible to the observers. The small potentiometers started recording a displacement from approximately 200 kN starting at 0.01mm and continued to rise to 0.06mm at 1375 kN. At maximum load the beam had deflected 7.0mm.

3.5.8 Phase 1 Testing - Third Series

The third series of test beams were tested in Banagher Precast Concrete. A steel frame capable of 2000 kN was designed, manufactured and installed at the facility to try and get interface failure. The frame can be seen in Figure 3.20.



Figure 3.20: Beam Test Specimen prior to loading in Banagher Precast Concrete

As per series 1 and series 2 3 composite reinforced concrete T-beams were tested under static loading. The TB3 beams had 2 legs of B10 stirrups at 300mm centres crossing the interface. According to Eurocode 2 Part 1-1 (2004) the capacity of the interface with shear reinforcement equating to the above is 1.45 MPa compared to the 0.65 MPa capacity of the TB1 beams and the 2.25 MPa capacity of the TB2 beams. The calculated horizontal shear stress was calculated to be 3.35 MPa. The expectation before testing started was that this beam should be weaker than the TB2 series beams and with the higher capacity of the frame the beam should fail in horizontal shear.

TB3-2

TB3-2 was tested on the 9th of December 2016 and was 172 days old. The average roughness of the interface was 3.73mm. The first hairline cracks were witnessed at 350 kN. These were measured using the crack microscope and found to be all less than 0.05mm. The cracking pattern was similar to series 1 and 2. At 700 kN some minor crushing of the concrete was witnessed under the support points. The cracks that opened at 350 kN were still of a similar size. At 1000 kN (6.70 MPa, over 4 times the design capacity of the interface) additional and longer vertical shear cracks were observed, with the largest being 0.1mm. The cracks ran from the support to the junction of the slab and along the interface horizontally and then vertically up the slab to the location of the loading point. Only one of the four small potentiometers worked for this test. It read 0.01mm at 1000 kN and finished at 0.014mm at 1400 kN. The load was increased and at 1450 kN the beam failed (see Figure 3.21) when the concrete at the supports crushed and spalled.

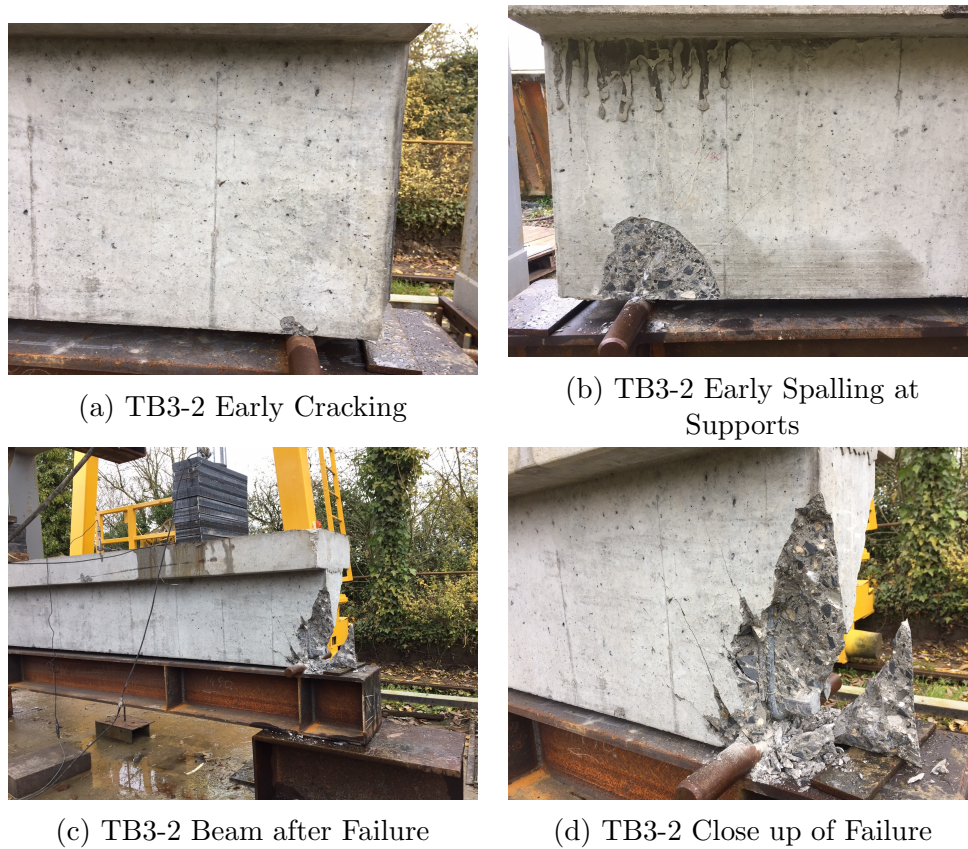


Figure 3.21: Selection of photo's showing TB3-2 during and after the test was carried out

TB3-3

TB3-3 was tested on the 9th of December 2016 and was 158 days old. On examination the top slab cube strength for TB3-3 was 62.5 MPa at 28days and the top slab cube strength for TB3-2 was 77.5 MPa. The average roughness of the interface was 3.63mm similar to TB3-2. The cracking pattern and local crushing were similar to TB3-2. When the test was stopped at 1000 kN (6.70 MPa) the first interface crack was observed. The loading was increased in increments of 50 kN unless stopped manually to inspect and at 950 kN there was no interface crack. Specimens TB3-2 and TB3-3 compared favourably in all aspects of the test. The vertical shear crack ran from the support to the junction of the slab and along the interface horizontally and then vertically up the slab to the location of the loading point. The load was increased and at 1450 kN the beam failed when the concrete at the supports crushed and burst away similar to TB3-2. All four small potentiometers were working for this test. One end read 0.05mm and the other 0.02mm at 1000 kN and finished at 0.06mm and 0.03 respectively at 1415 kN. These indicated a distinct but minor delamination of the interface. Figure 3.22 shows a selection of photo's from the test. Figure 3.22a shows a crack running from load point to support along the interface with the vertical displacement potentiometer in view. Figure 3.22b is a side view of the local crushing at the support. Figure 3.22c displays a rear picture of the reinforced T-beam post test where the end concrete exploded under the load at the support and Figure 3.22d is a rear view of the test frame.

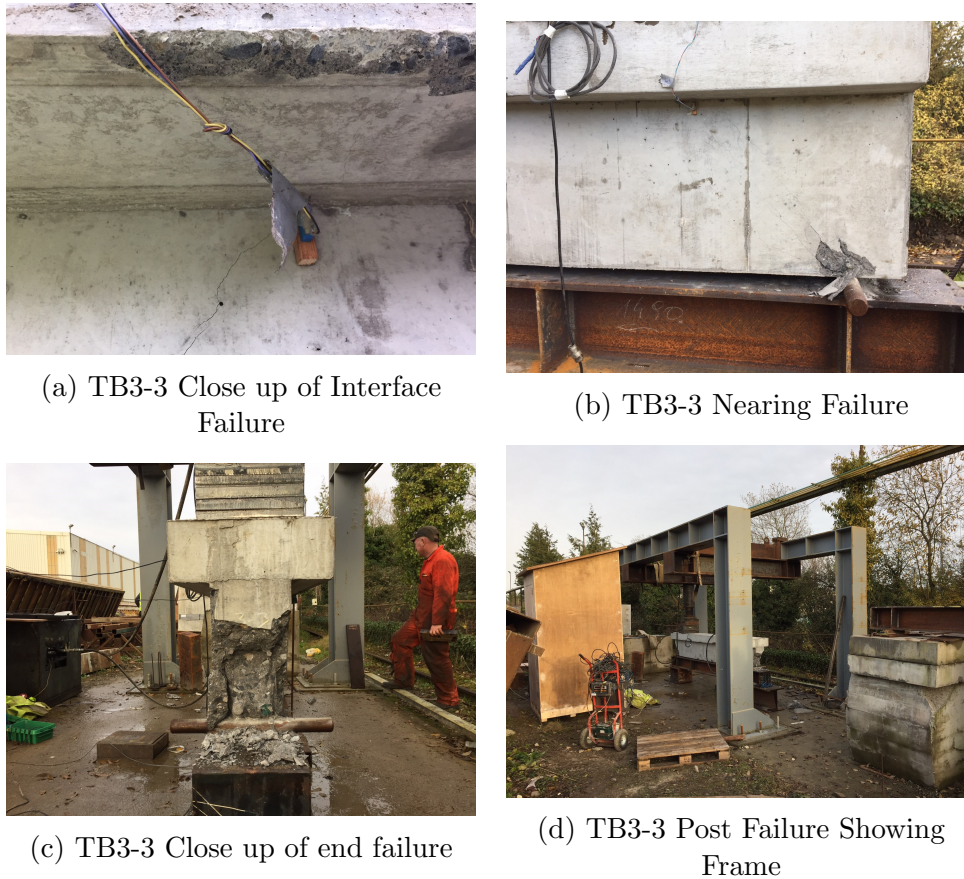


Figure 3.22: Selection of photo's showing TB3-3 during and after the test was carried out

TB3-4

TB3-4 was tested on the 21st of December 2016 and was 167 days old. The top slab cube strength for TB3-4 was 63.5 MPa at 28days, similar to that of TB3-3. The average roughness of the interface was 3.84mm. The cracking pattern and local crushing were similar to TB3-2 and TB3-3. At 778 kN (5.21 MPa, over 3 times the design capacity of the interface) the first interface crack was observed. (This cannot be explained as the concrete strength and average surface roughness was identical to TB3-3 as was the test rig and test set-up.) The vertical shear crack ran from the support to the junction of the slab and along the interface horizontally and then vertically up the slab to the location of the loading point as per all of the other specimens with interface cracks. The load was increased and at 1430 kN the beam failed when the concrete at the supports crushed and burst away. This was an identical failure to TB3-2 and TB3-3. The potentiometers read 0.006mm and 0.011mm at 789 kN and 0.023mm and 0.017 respectively at 1430 kN. Figure 3.23 shows a graph of the four small potentiometers. P1 and P2 are at one end of the beam, one at each side of the interface. P3 and P4 are at the other end of the beam, one at each side of the interface. P3 and P4 are very similar as would be expected and seem to be working reasonably well with a maximum beam to slab displacement of 0.023mm which is extremely low at 1430 kN (9.58 MPa - well over 6 times the capacity of the interface).



Figure 3.23: TB3-4 Graph for Beam to Slab Displacement

3.5.9 Phase 1 Testing - Fourth Series

The fourth series of test beams were also tested in Banagher Precast Concrete. 3 composite reinforced concrete T-beams were tested under static loading. The beams had 2 legs of B10 stirrups at 500mm centres crossing the interface. According to Eurocode 2 Part 1-1 (2004) the capacity of the interface with shear reinforcement equating to the above is 1.13 MPa. This equates to the minimum amount of interface links according to BS5400-4 (1990) of $0.15\%bh$.

TB4-2

TB4-2 was tested on the 22nd of December 2016 and was 169 days old. The concrete cube strengths for all of the TB4 series beams were very similar so any disparities in results could not be placed to concrete strength. The average roughness of the interface was 4.19mm. The first hairline cracks were witnessed at 350 kN. These were measured using the crack microscope and found to be all less than 0.05mm. The cracking pattern was similar to series 1,2 and 3. At 516 kN some minor spalling of the concrete was witnessed under the support points. At 648 kN some flexural cracks were noted along the beam near midspan, these were measured at less than 0.05mm. When the test was stopped at 778 kN (5.21 MPa, over 3 times the design capacity of the interface) the first interface crack was observed. This was identical to TB3-4. The lesser number of shear links did not seem to have any difference on the first interface crack. However, the surface roughness was 4.19mm and on TB3-4 was 3.84mm so that would have had an impact and may explain the result. The load was further increased and at 1038 kN the beam failed when the concrete at the supports crushed and burst away. This was much lower than series 3 beams and can only be put down to the fact that

the links were spaced further apart in this series of beams. However, test series 1 beams had no links crossing the interface and these did not fail in this fashion. At 750 kN one of the potentiometers read 0.01mm, at 778 kN it read 0.125mm and at 1038 kN (Failure) it read 0.237mm. When the beam failed and recovered the reading dropped back to 0.129mm. This was the largest displacement recorded and proved a substantial interface delamination compared to the other beams tested to date. The other 3 potentiometers did not register and were omitted from the test data. Figure 3.24 shows a graph of shear stress versus beam deflection. Also included on the graph are horizontal lines representing the stress when the interface cracked and horizontal lines for all of the major bridge codes and their capacities with 2 legs of 10mm diameter stirrups crossing the interface at 500mm centres. The yellow line represents the stress when the first interface crack was witnessed and equates to 5.21 MPa. The Eurocode 2 Part 1-1 (2004) capacity calculated at 1.13 MPa represents a factor of safety of 4.6 proving that the minimum interface reinforcement requirements of BS5400-4 (1990) could still be used. Figure 3.25 shows a graph of the only potentiometer working, P1. Examining the graph it is clear to see that the interface is uncracked up until 778 kN approximately and then a brittle delamination which is only stopped by the inclusion of the minimum horizontal shear links. This shows the brittle nature of the joint and the necessity of links.

Figure 3.26 shows a graph of micro strain versus load. The strain gauges did not compute for series 2 and 3 but seemed to be working satisfactorily for series 4. The strain in the links picked up at just over 200 kN which equates to an interface shear stress of 1.34 MPa. Examining Figure 3.25 P1 starts to register at around the same value which would strengthen the cause for inclusion of the P1 data even though it was the only potentiometer working.

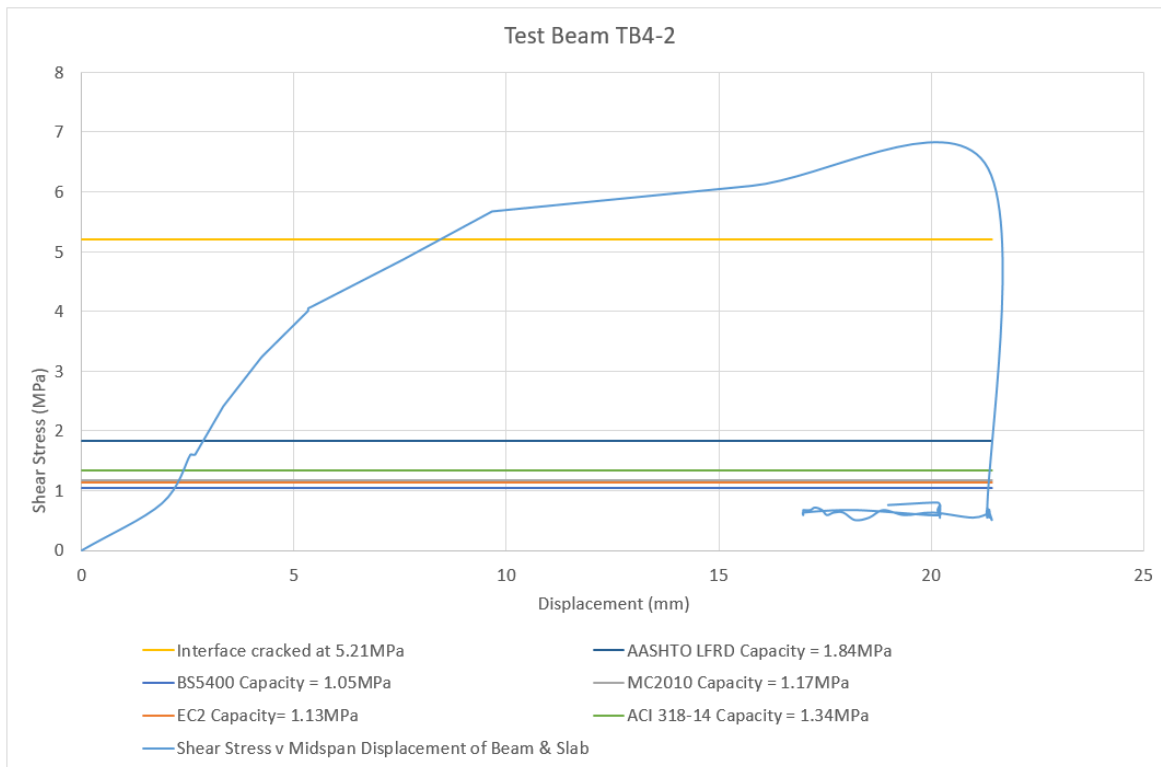


Figure 3.24: TB4-2 Graph for Stress v Deflection

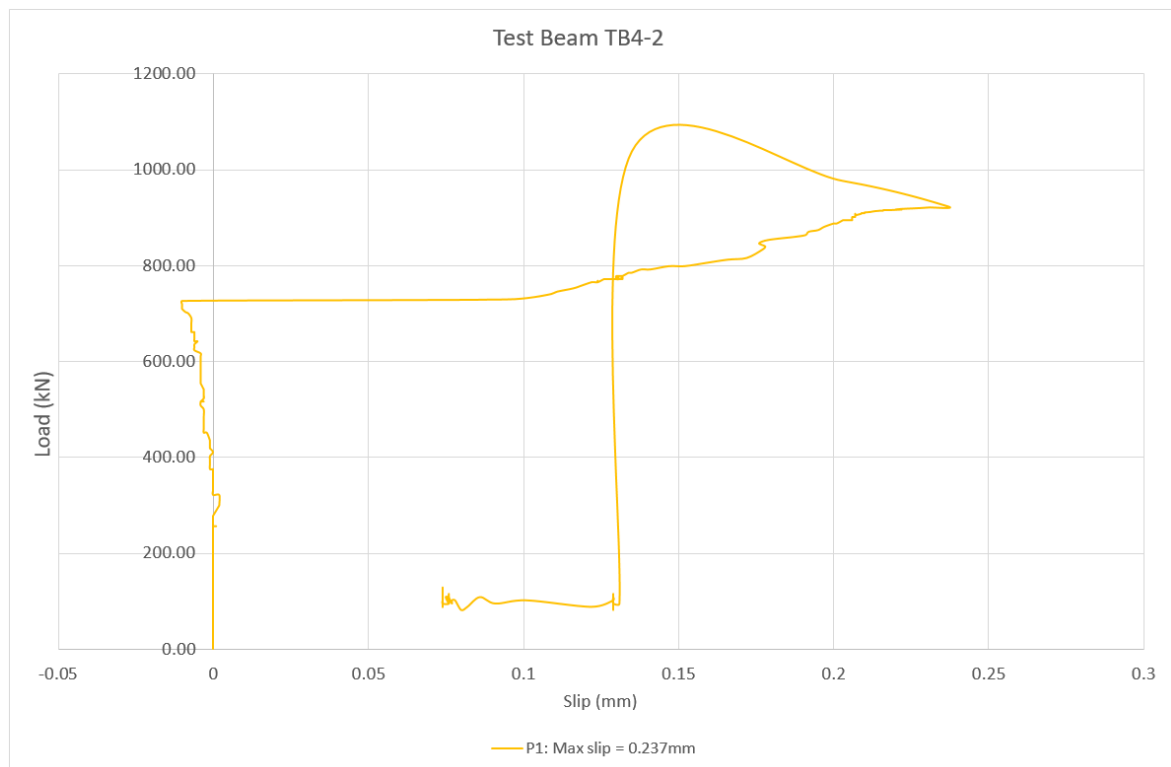


Figure 3.25: TB4-2 Graph for Beam to Slab Displacement

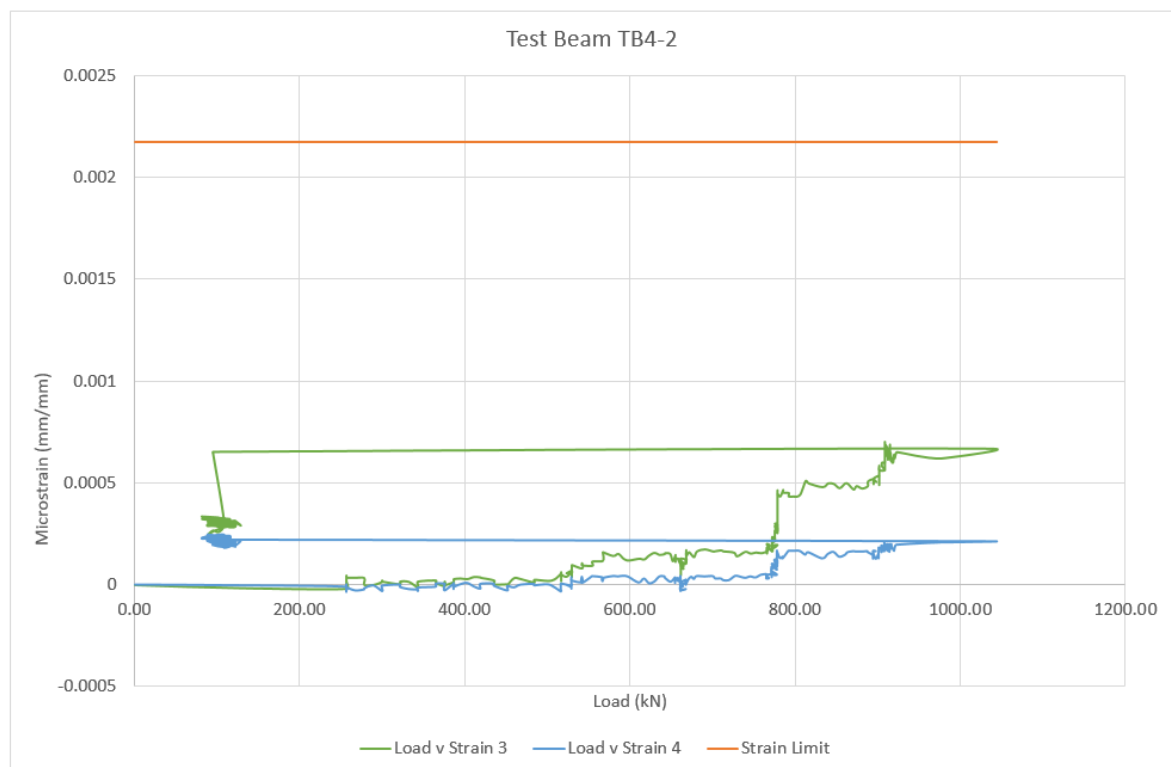


Figure 3.26: TB4-2 Graph for Microstrain v Load

TB4-3

TB4-3 was tested on the 22nd of December 2016 and was 168 days old. The average roughness of the interface was 3.73mm. Both the slab strength, age and surface roughness were near identical for TB4-3 and TB4-2 so similar results were expected before this test started. The cracking pattern was similar to TB4-2. As per TB4-2 and TB3-4 when the test was stopped at 778 kN (5.21 MPa) the first interface crack was observed. The load was increased and at 1176 kN the beam failed when the concrete at the supports crushed and burst away. This was much lower than series 3 beams but higher than TB4-2. At 675 kN one of the potentiometers read -0.05mm, at 750 kN it read 0.129mm and just before 1170 kN (Failure) it read 0.192mm. When the beam failed and recovered the reading dropped back to 0.1mm. This was very similar in its behaviour to TB4-2. The strain in the links started to register just below 200 kN (1.34 MPa). The potentiometer P3 started to register at around the same value. Considering that the strain in the links did not start to record until 1.34 MPa it can be assumed that the concrete alone is resisting horizontal shear until this point.

TB4-4

TB4-4 was tested on the 22nd of December 2016 and was 165 days old. The average roughness of the interface was 3.84mm. The cracking pattern was similar to TB4-2 and TB4-3. When the test was stopped at 648 kN (4.34 MPa) the first interface crack was observed. This was a lot lower than TB4-2 and TB4-3 even though the surface roughness, concrete cube strength, age and loading were all similar. It proves that testing only 3 beams per series is not enough to get accurate and credible findings. The load was increased and at 1170 kN the beam failed when the concrete at the supports crushed and burst away in a similar fashion to TB4-2 and TB4-3. Both beam supports failed equally at the same time. From examining the potentiometers the displacements never reached the values of TB4-2 and TB4-3. This beam seemed to be clamped much better and did not delaminate. The maximum displacement was 0.032mm compared to TB4-2 (0.237mm) and TB4-3 (0.192mm) Unfortunately the strain gauges did not compute for this beam.

Figure 3.27 shows a graph of shear stress versus beam deflection. The yellow line represents the stress when the first interface crack was witnessed and equates to 4.34 MPa. The Eurocode 2 Part 1-1 (2004) capacity calculated at 1.13 MPa represents a factor of safety of 3.85. Examining Figure 3.28 P1 and P2 start to register at the same reading (684 kN) to that visually observed during the test. P1 and P2 are positioned at the end of the beam with P1 on the left hand side and P2 on the right hand side of the beam. They both showed delamination but with differing values.

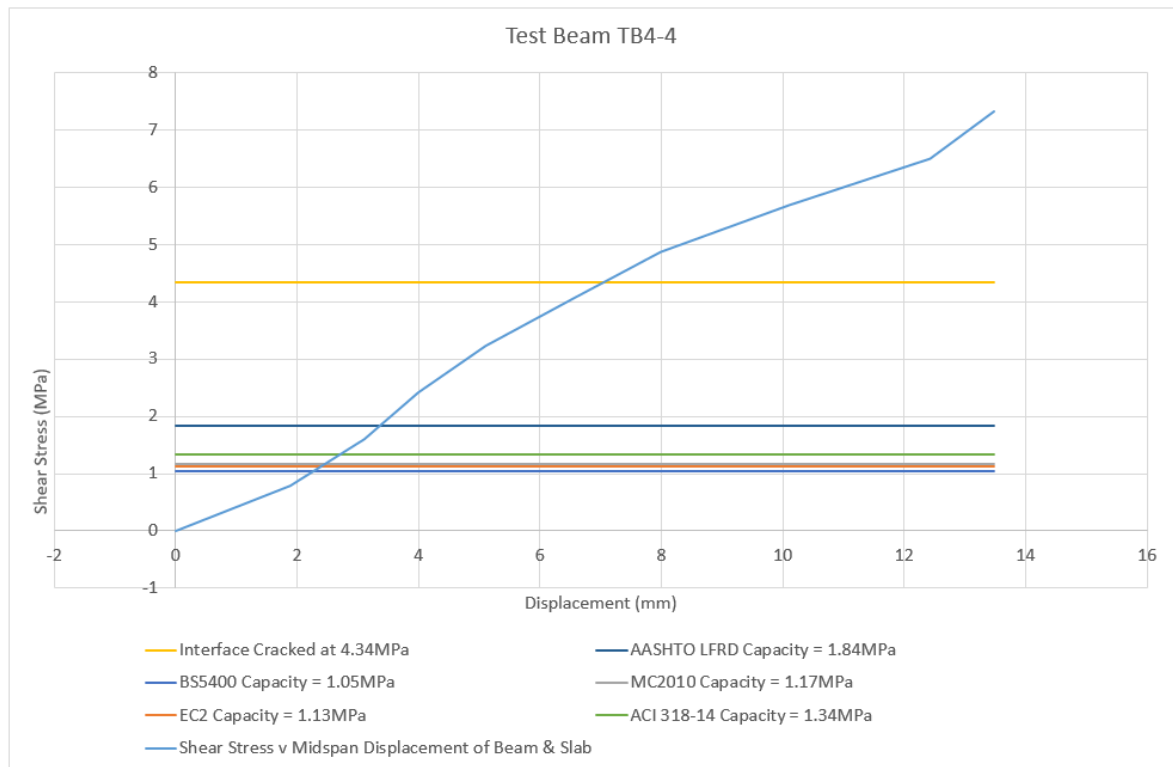


Figure 3.27: TB4-4 Graph for Stress v Deflection

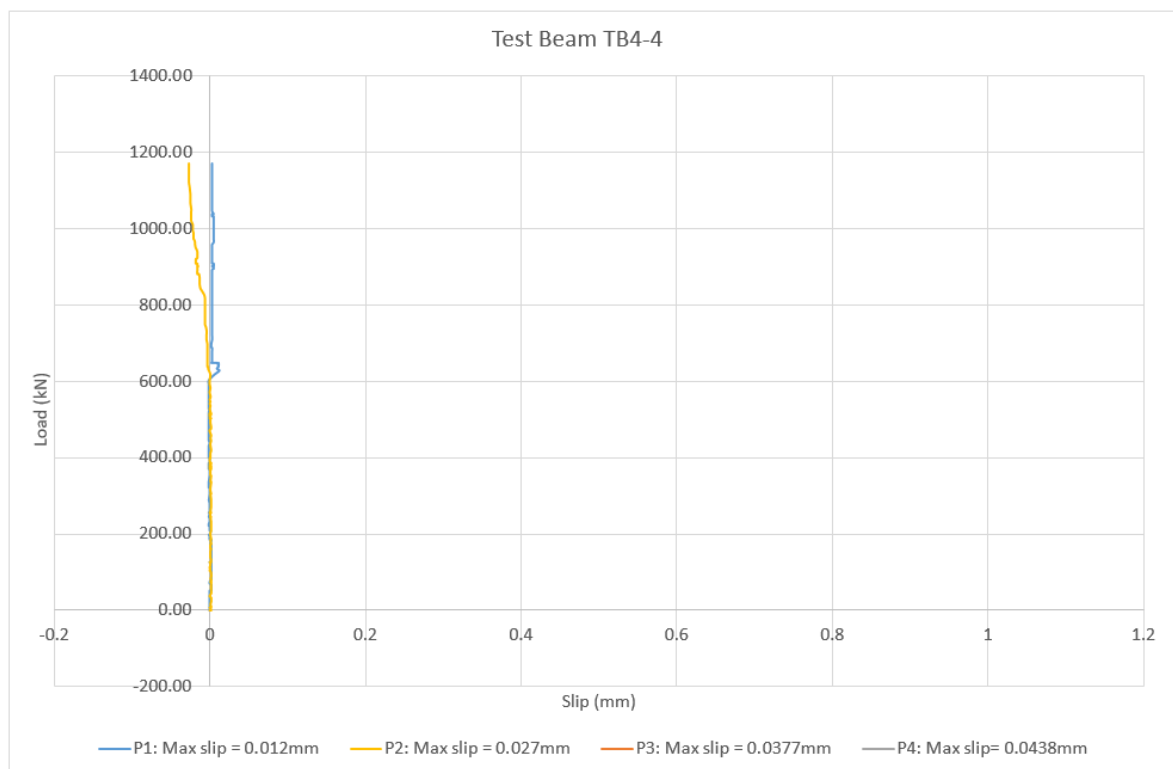


Figure 3.28: TB4-4 Graph for Beam to Slab Displacement

3.6 Testing - Phase 1 - Discussion

(a)Code Provisions:

The test results show that the current bridge design codes for Ireland, the UK, Europe and the US are conservative. The provisions of the design codes for horizontal shear are safe for the concrete strengths used in the tests. It may be possible that the codes are not safe for lower concrete strengths.

Examining the test results (see Table 3.5) the first series of beams had an average shear stress at the first interface crack of 4.78 MPa. When compared to the horizontal interface shear resistance without stirrups of 0.65 MPa as per Eurocode 2 Part 1-1 (2004) results in a factor of safety of 7.36. The second series of beams had stirrups at 150mm centres (2.25 MPa capacity) and did not exhibit an interface crack even at stresses averaging 9.32 MPa - the tests had to be stopped as the test rig was unsafe. The third series of beams had stirrups at 300mm centres (1.45 MPa capacity) with an average shear stress at the first interface crack of 6.20 MPa, this resulted in a factor of safety of 4.28. Finally the fourth series of beams had stirrups at 500mm centres (1.13 MPa capacity) with an average shear stress at the first interface crack of 4.92 MPa, resulting in a factor of safety of 4.35. In summary there is a minimum factor of safety of 4.0 between test results and code calculations. Table 3.6 contains a summary of the results of the first phase of testing. The table outlines the beam type, beam age and slab age. The 28 day strength of both the beam and slab are also shown. Finally in relation to design capacities the links crossing the interface, the roughness of the interface and the capacity of the joint as per Eurocode 2 Part 1-1 (2004) are recorded.

(b)Surface Finish:

Precast concrete manufacturers cast bridge beams internally and externally during all seasons. Trying to achieve a surface finish with high strength flowable concrete is extremely difficult. The roughening method used in this experimental study is practical and has been incorporated by Banagher Precast Concrete. In summary the beams are poured and left as cast for approximately 30 minutes. The finishing foremen dash the surface with 8mm aggregate and tamp it into the fresh concrete with a roughening tool. The beams are then covered and cured. When the beams are removed from the manufacturing line they are power washed and any loose aggregate is cleaned away. This method achieves an average 3mm of roughness when measured using the sand patch test. Table 3.6 displays the surface roughness results from the first phase of testing using the methods for roughening as outlined above.

General workmanship standards as regards consistency of the finish along a beam are extremely important and are easier to maintain in a quality controlled factory environment.

(c)Minimum Links:

At present there is no minimum interface links provision in Eurocode 2 Part 1-1 (2004). Engineers are using the minimum provisions of BS5400-4 (1990) and CEB-FIP Model Code 2010 (2010) for bridge beam designs, both of these design codes specify links in the range of $0.15\%bh$. The beams in phase one were overly robust and did not experience

Table 3.5: Testing - First Series - Average Factor of Safety

Beam Type	Design Capacity (MPa)	Shear Stress @ 1st Interface Crack (MPa)	Factor of Safety
TB1-2	0.65	4.65	7.15
TB1-3	0.65	5.36	8.25
TB1-4	0.65	4.35	6.69
TB2-2	2.25	No Interface crack(10.04)	NA
TB2-3	2.25	No Interface crack(8.71)	NA
TB2-4	2.25	No Interface crack(9.21)	NA
TB3-2	1.45	6.7	4.63
TB3-3	1.45	6.7	4.63
TB3-4	1.45	5.21	3.59
TB4-2	1.13	5.21	4.61
TB4-3	1.13	5.21	4.61
TB4-4	1.13	4.34	3.84

interface failures at the loads the testing rig could produce. Without interface failures the varied area of reinforcement crossing the interface was not demonstrated sufficiently and further research and testing is required.

Table 3.6 displays the amount of interface links for all beam types as well as the design capacity of the joint as per the provisions of Eurocode 2 Part 1-1 (2004). Table 3.5 compares these figures to the actual experimental data and gives the factor of safety achieved.

Table 3.6: Testing - First Series - Summary of Results

Beam Type	Beam Age (Days)	Slab Age (Days)	Beam Str. (MPa)	Slab Str. (MPa)	Int. Links	Design Cap. (MPa)	Shear Stress @ 1st Crack (MPa)	SR (mm)
TB1-2	112	84	84.84	74.60	None	0.65	4.65	3.35
TB1-3	105	77	73.67	72.77	None	0.65	5.36	4.19
TB1-4	163	135	78.72	76.30	None	0.65	4.35	2.44
TB2-2	169	141	65.24	78.01	2B10-150	2.25	No Int. Crack	3.32
TB2-3	173	142	64.04	71.18	2B10-150	2.25	No Int. Crack	3.32
TB2-4	171	143	66.02	70.00	2B10-150	2.25	No Int. Crack	3.04
TB3-2	227	199	79.43	77.52	2B10-300	1.45	6.7	3.73
TB3-3	217	185	72.07	62.53	2B10-300	1.45	6.7	3.68
TB3-4	224	195	79.13	63.56	2B10-300	1.45	5.21	3.84
TB4-2	226	196	71.91	64.76	2B10-500	1.13	5.21	4.19
TB4-3	227	197	72.28	68.66	2B10-500	1.13	5.21	3.73
TB4-4	224	192	69.81	68.76	2B10-500	1.13	4.34	3.84
Note: Str. = Strength, Int. = Interface, Cap. = Capacity, SR = Surface roughness								

3.7 Phase 1 Conclusions

The following are the conclusions made by the author based on the data obtained for the first phase of testing:

1. More time should be spent on analytical modelling in order to specify a section size that can be brought to interface failure with a specified testing rig capacity.
2. The roughening of the interface surface using aggregate provided a sufficient rough surface and can be used in practice once the joint is power washed and any loose materials washed away.
3. The interface joint cracking was as a direct result of diagonal shear cracking running from the load point to the support point.
4. Minimum horizontal interface shear links based on the formula of BS5400-4 (1990) can still be used in design. The test data averaged a shear stress capacity of 4.92 MPa compared to a design resistance of 1.13 MPa as per Eurocode 2 Part 1-1 (2004) resulting in a factor of safety of 4.35.
5. A non reinforced interface has substantial shear resisting capacity. The test data averaged a shear stress capacity of 4.78 MPa compared to a design resistance of 0.65 MPa as per Eurocode 2 Part 1-1 (2004) resulting in a factor of safety of 7.4.
6. As both the beam and slab concrete cube strengths were similar it was difficult to predict if they had any impact on the results. The cube strengths were all above 60.0 MPa and no beams or slabs with low cube strengths were compared or tested. Further research should be carried out in this area.
7. Interface surface roughness was consistent between 3mm and 4mm. Lower and higher surface roughness's could be researched further to see their impact.

3.8 Phase 1 Recommendations

1. For precast concrete prestressed bridge beams the use of a rough as cast surface finished with 8mm aggregate will suffice for a rough surface as per the requirements of Eurocode 2 Part 1-1 (2004).
2. The minimum interface stirrup requirements as per BS5400-4 (1990) should be used by bridge engineers and should be incorporated in the new version of Eurocode 2 Part 1-1 (2004).

Chapter 4

Phase 2 - Experimental Program, Analysis and Discussion

4.1 Phase 2 Introduction

The second phase of the research project examined the performance of 7 3.3m long prestressed concrete T-beams cyclically and statically loaded using a four point loading test similar to the research carried out by Kovach (2008). Experimental phase 1 testing highlighted a number of key issues that needed to be resolved in phase 2. The main issue was the strength of the reinforced concrete interface and the lack of an interface failure in the testing carried out. Therefore the literature had to be researched further in order to manufacture and test a specimen and achieve an interface failure. Testing carried out by Kovach (2008) on unreinforced interfaces had similar shaped T-beams which were prestressed and had interface failures at approximately 300 kN, which was the limit of the hydraulic testing rig in Trinity College. Phase 1 testing provided data on minimum interface shear links but in order to be able to propose a new value for the coefficient of cohesion, a fatigue test was required.

The surface finish was to be rough and be limited to 3mm minimum with the concrete strength as per phase 1. These beams were reinforced with prestressing strands, as the majority of bridge beams are prestressed. 3 beams had one 8mm (0.15%bh - minimum interface links as per BS5400-4 (1990)) stirrup crossing the interface and 3 had no stirrups crossing the interface. There was also a control beam manufactured that would be tested statically to failure. The reasoning behind 3 beams per series was to keep with the methodology of phase 1.

4.1.1 Fatigue Loading Procedure

The beams were loaded with 16,800 cycles of load, the load itself varied per beam depending on the ultimate strength of the monolithically tested control beam. The monolithically tested control beams ultimate strength represented Load Model 1(LM1) loading from Eurocode 1 Part 2 (2002). As per clause 4.6.2 of this code Fatigue Load Model 1 (FLM1) is equal to 70% of LM1, this was represented in the testing and can be seen in Table 4.1. The number of cycles was taken from Eurocode 1 Part 2 (2002) and is shown in Table 4.2. From this table it can be shown that two million commercial vehicles cross the slow lane of a motorway bridge per year which equates to

two hundred and forty million commercial vehicles during its lifespan. LM1 represents a vehicle weighing 600 kN with four 150 kN point loads dispersed through 400mm wide contact lengths. The test beams have 550mm wide top slabs and therefore can take half of a LM1 vehicle. From Table NA5 of Eurocode 1 Part 2 (2002) the heaviest vehicle to use the bridge crosses it ten times per million crossings. The closest vehicle in weight to a LM1 vehicle crosses it seventy times per million crossings or 16,800 times during the life of a bridge. Based on a cycle taking 3 seconds, 16,800 cycles would take approximately 14 hours to perform. Clause 6.8.7 of Eurocode 2 Part 2 (2005) states that Miners rule should be used to calculate the total number of cycles required. Using a minimum stress of 0.5 MPa and a maximum stress of 4.0 MPa the number of cycles is approximately 500,000 which is not practical for a laboratory test as it would take over 400 hours to perform.

Table 4.1: Phase 2 Test Specimen Parameters

Test Specimen	Repeated Load		Reinforcing Stirrups			ISF
	Min	Max	No.	Size	Centres	
Control Beam	To Failure		0	0	0	R
PTB1-1	40%	70%	0	0	0	R
PTB1-2	40%	70%	0	0	0	R
PTB1-3	40%	70%	0	0	0	R
PTB2-1	40%	70%	1	8	400	R
PTB2-2	40%	70%	1	8	400	R
PTB2-3	40%	70%	1	8	400	R
Note: ISF = Interface Surface Finish, M = Monolithic, R = Rough						

Table 4.2: Indicative number of heavy goods vehicles expected per year and per lane in Ireland (Source Table NA4 of IS EN 1991-2)

Traffic Categories			N _{obs} per lane (millions/yr)	
Type (see TD9/07 Table 4)	Carriageway layout	No. of lanes per carriageway	Each slow lane	Each fast lane
Type 3 dual	Dual	3	2.0	1.5

4.2 Design of the Prestressed T-Beam

4.2.1 Prestressed Concrete T-Beams

The test specimens section size was selected after the results of the first phase of testing proved that to get interface failure the load needed to be excessive when compared to

the capacity of the section. The RC T-Beam of Phase 1 failed predominantly in vertical shear when the vertical cracks exceeded 0.3mm. Therefore when considering the section size for phase 2, a full interface failure was deemed extremely important. The cyclic load rig in Trinity College had a maximum capacity of 300 kN. This was the total load and could be spread through a spreader beam as per phase 1 in order to obtain a 4 point test. The slab did not cover the entire length of beam, this was based on previous research by Loov and Patnaik (1994) but was missed when the test specimens of phase 1 were being designed and detailed. They cited an issue with cracks forming between the support of the beam and the interface at a distance equal to the effective depth of the web section. The slab at the end of the beam provides restraint against interface slip and to correct this the authors suggested reducing the slab length to allow for the horizontal shear failure. Kovach (2008) also tested similar beams statically with no links crossing the interface. The author followed the approach of Loov and Patnaik (1994) and Kovach (2008).

4.2.2 Prestress and Vertical Shear

The test specimen's section size was selected after an extensive literature review of horizontal shear research (Chung and Chung (1976), Hughes (1987) and Kovach (2008)). The beams were designed as per Eurocode 2 Part 1-1 (2004) and were overly reinforced for flexure and vertical shear so that horizontal shear failure at the interface was achieved first. 150 kN was chosen as the optimum point load in order for high horizontal shear stresses to exist at the interface. This corresponds to Load Model 1 as per Eurocode 1 Part 2 (2002)

The prestressed T-Beam was designed similar to phase 1 i.e. to be simply supported with a two point loading arrangement symmetrical about the centre point. The reaction at each support is equal to the applied load. This results in a shear span of constant shear extending out from the support to the applied load with zero shear between the two loading points, see Figure 4.1. A trapezoidal bending moment diagram results which can be seen in Figure 4.2. The prestress is designed as per Eurocode 2 Part 1-1 (2004).

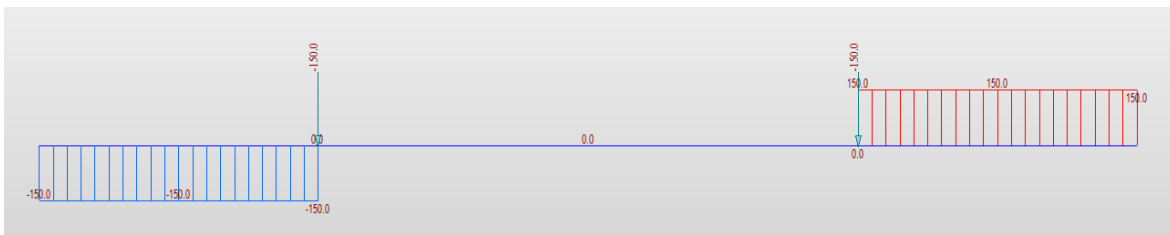


Figure 4.1: Shear Force Diagram for the Prestressed T-Beams under 150 kN point loads (Self weight excluded)

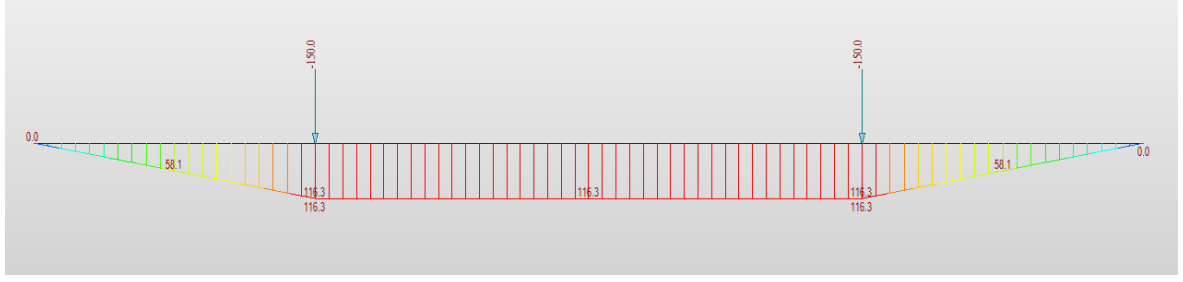


Figure 4.2: Bending Moment Diagram for the Prestressed T-Beams under 150 kN point loads (Self weight excluded)

4.2.3 Interface Shear

The following set of calculations are based on Eurocode 1 Part 2 (2002). A comparison was also carried out on all of the other major bridge design codes and is shown in Table 4.3.

Design value of interface shear stress, Exp(6.24):

$$\begin{aligned} V_{Edi} &= \frac{\beta V_{Ed}}{z b_i} \\ &= \frac{1 \times 150 \times 1000}{272.51 \times 75} \\ &= 7.34 MPa \end{aligned} \quad (4.1)$$

where;

β = the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered

V_{Ed} = the transverse shear force (kN)

z = the lever arm of the composite section (mm)

b_i = the width of the interface (mm)

Design shear resistance of the interface, Exp(6.25):

Rewriting Exp 6.25 to calculate Interface Links:

$$\begin{aligned} \rho_{min} &= \frac{V_{Edi} - c f_{ctd} - \mu \sigma_n}{f_{yd} (\mu \sin \alpha + \cos \alpha)} \\ &= \frac{7.34 - 0.4 \times 1.64 - 0.7 \times 0}{435 (0.7 \times \sin 90 + \cos 90)} \\ &= 0.0220 \end{aligned} \quad (4.2)$$

where;

c = the cohesion factor

μ = the coefficient of friction

f_{ctd} = the tensile stress of the new concrete layer (MPa)

f_{yd} = the design yield strength of reinforcement (MPa)

α = the angle of reinforcement to interface (Degrees)

Area of Links Required:

$$\rho_{min} \times b = 0.0220 \times 75 = 1.65mm^2/mm \quad (4.3)$$

Area of Links Provided:

$$\begin{aligned} 1B16@120mmc/c's &= 1.68mm^2/mm \\ \rho_{Provided} &= 0.0223 \end{aligned} \quad (4.4)$$

Design Shear Resistance of the Interface without interface reinforcement:

$$V_{Rdi} = cf_{ctd} + \mu\sigma_n = 0.4 \times 1.64 + 0.7 \times 0 = 0.65MPa \quad (4.5)$$

Design shear resistance of the interface using provided reinforcement, Exp(6.25):

$$\begin{aligned} V_{Rdi} &= cf_{ctd} + \mu\sigma_n + \rho f_{yd}(\mu \sin\alpha + \cos\alpha) \\ &= 0.4 \times 1.64 + 0.7 \times 0 + 0.0223 \times 435(0.7) \\ &= 7.46MPa \end{aligned} \quad (4.6)$$

Table 4.3: Phase 2 Interface Shear Requirements Design Codes Comparison Summary

Design Code	Design Value of Interface Shear Stress (MPa)	Shear resistance without reinforcement (MPa)	Shear resistance with links of 1B16 @ 100mm c/c's	Minimum links required A_{sv}/s_v (mm^2/mm)	Maximum longitudinal link spacing (mm)
BS5400-4 (1990)	7.34	0.5	8.32	0.11	400
Eurocode 2 Part 1-1 (2004)	7.34	0.65	7.46	N/A	N/A
CEB-FIP Model Code 2010 (2010)	7.34	0.82	7.54	0.13	N/A
ACI 318-14 (2014)	6.25	0.57	29.31	0.07	600
AASHTO LRFD-BDS (2007)	7.67	2.19	12.98	0.06	N/A
Eurocode 2 Part 1-1 (2017)	7.34	0.65	7.46	N/A	N/A
Note: Eurocode 2 Part 1-1 (2017) is a not a working draft					

Table 4.3 is a summary output table comparing all of the major bridge design codes in terms of horizontal interface shear provisions. The major outputs like interface shear stress, shear resistance for an unreinforced interface, shear resistance of a reinforced interface, minimum links requirements and finally maximum interface link spacing. The bridge design code pre Eurocode 2 Part 1-1 (2004) was BS5400-4 (1990) and this code was used to design all bridge structures in Ireland and the United Kingdom up until 2011. It is still being referenced by UK design engineers on a daily basis.

Discussing the summary table the following points are noted:

1. The calculation of interface shear stress on the joint is similar for all codes of practice
2. The capacity of an unreinforced joint is similar across all codes apart from AASHTO LRFD-BDS (2007)

3. 1 B16 link at 100mm centres would be required to satisfy the shear stress at the interface.
4. Whereas the minimum shear links equate to 1 B8 link at 400mm centres.
5. The current Eurocode 2 Part 1-1 (2004) does not specify a minimum requirement nor does the proposed generation 2 Eurocode 2 as of May 2018. The generation 2 Eurocodes are out for public consultation and are due to be published in 2023.

4.3 Beam Fabrication

4.3.1 Prestressed Concrete T-Beams

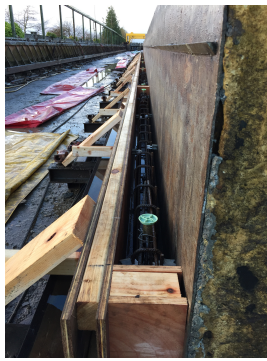
7 Prestressed concrete T-Beams were manufactured in Banagher Precast Concrete in late December 2016 and tested in Trinity College Dublin in the Spring of 2017. As discussed the beams were based on similar research carried out by Kovach (2008). Each beam had a top flange width of 550mm and depth of 100mm. The web was 300mm deep and 150mm wide. The beams were 3300mm long with an effective span of 3050mm. The beams were overly reinforced for bending with 4 12.9mm prestressing strands and overly reinforced for vertical shear with 10mm links at 200mm centres. 3 10mm lacer bars ran longitudinally in the top slab. The beams were designed to fail in interface shear only. Full details can be seen in Figures 4.3 and 4.4.

These beams were split into two series of three beams each. A control beam was also manufactured and tested statically to failure in order to obtain its ultimate strength. The three beams in each series were loaded dynamically for 16,800 cycles or to failure, whichever came first.

Test series 1 beams had no reinforcement crossing the interface. Test series 2 beams had an 8mm stirrup at 400mm centres ($0.15\% b h$ = minimum steel as per BS5400-4 (1990)) crossing the interface to represent minimum interface reinforcement.

The varying parameters were the level of load and the ratio of reinforcing stirrups crossing the interface. The beams were poured with C57/70 concrete, which is synonymous with precast prestressed bridge beams and the top slabs cast twenty eight days later with C40/50 concrete, which is used as standard for on-site bridge deck construction. The beams had a rough surface finish in accordance with Eurocode 2 Part 1-1 (2004) Section 6.2.5 (2) and this was tested using the sand patch procedure.

through the stopend at one end of the prestressing line and pulled towards the anchor at the other end of the line. The strands were then stressed using a prestressing jack to 75% of the breaking load of 279 kN which equates to 209.25 kN. Once the strands had been stressed and the links and spacers placed the beams were ready to be poured. All beams were poured together and it took approximately one hour to pour and finish the beams. The beams were left to set for approximately 30 minutes and were roughened using the home-made roughening tool as per Figure 4.5d. The next step was to apply the 8mm aggregate to the tops of the beams to give the required rough surface finish. This was done by dashing 8mm aggregate at the top surface whilst it was still in its fresh state. Trial panels of the mix were cast and this procedure perfected before application to the beams. Once the beams had cured and prior to the casting of the top slab the beams were power washed and any loose aggregate removed. The 8mm aggregate finish can be seen in Figure 4.5c.



(a) PTB 1 Beams



(b) PTB 2 Beams



(c) Finished Beam



(d) Surface Roughening

Figure 4.5: PTB1 and PTB2 Manufacturing Stages

4.3.2 Design Life, Exposure Class, Nominal Cover and Concrete Mix Design

This was discussed in detail in phase 1 and all of these parameters are the same for phase 2.

Table 4.4: Concrete Cube Compressive Strength Results

Pour Date	Element	Concrete Grade	Test Date	Age (days)	Cube Size (mm)	Ave 28 Day Result
21/12/16	PTB1-1(B)	C57/70	17/05/17	148	100 x 100	77.02MPa
19/01/17	PTB1-1(S)	C40/50	17/05/17	119	100 x 100	77.22MPa
21/12/16	PTB1-2(B)	C57/70	06/06/17	168	100 x 100	77.02MPa
19/01/17	PTB1-2(S)	C40/50	06/06/17	139	100 x 100	77.22MPa
21/12/16	PTB1-3(B)	C57/70	12/06/17	174	100 x 100	77.02MPa
19/01/17	PTB1-3(S)	C40/50	12/06/17	145	100 x 100	77.22MPa
21/12/16	PTB2-1(B)	C57/70	26/06/17	188	100 x 100	77.02MPa
19/01/17	PTB2-1(S)	C40/50	26/06/17	159	100 x 100	77.22MPa
21/12/16	PTB2-2(B)	C57/70	29/05/17	160	100 x 100	77.02MPa
19/01/17	PTB2-2(S)	C40/50	29/05/17	131	100 x 100	77.22MPa
21/12/16	PTB2-3(B)	C57/70	16/06/17	178	100 x 100	77.02MPa
19/01/17	PTB2-3(S)	C40/50	16/06/17	149	100 x 100	77.22MPa

4.3.3 Cube Results

3 standard 100mm x 100mm cubes were taken for all beams and slabs that were cast to obtain average 28 day results. Table 4.6 displays all of these results. The table contains the date that each beam and slab were poured in the factory. The concrete grade, test date, age at time of test and the average 28 day result. The C57/70 grade concrete cubes were all over 70 N/mm² at 28days as were the C40/50 grade concrete cubes.

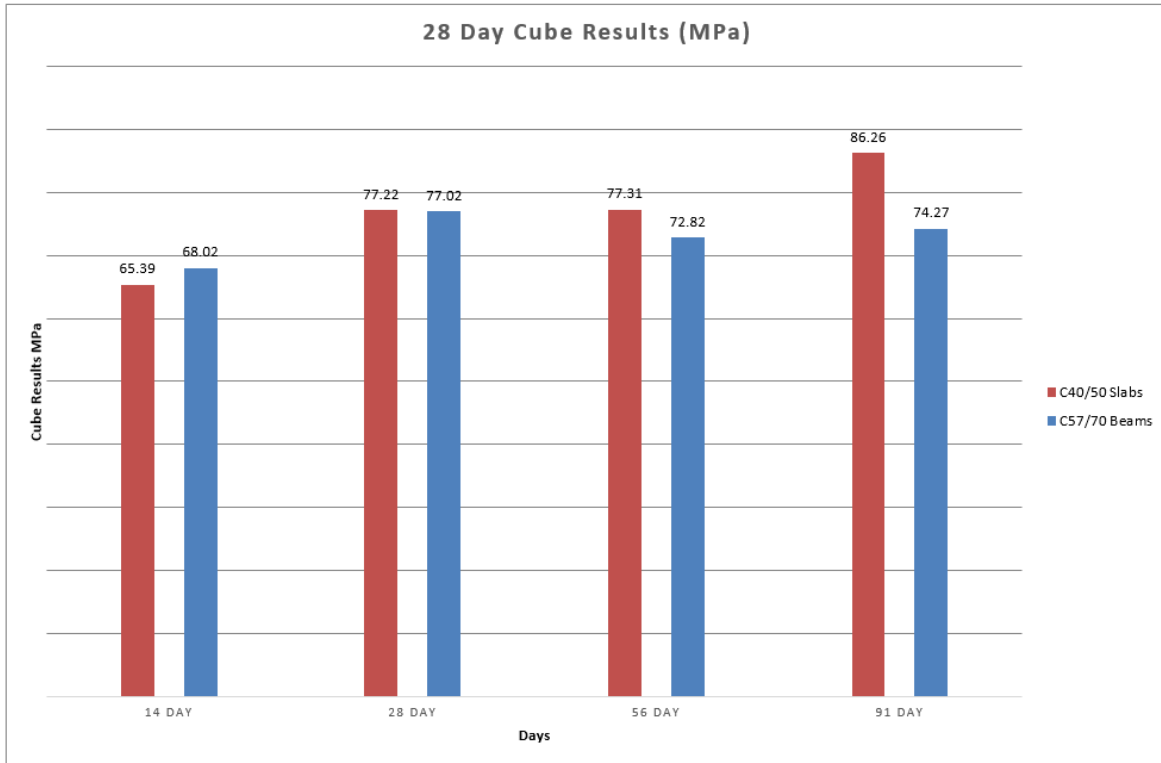


Figure 4.6: Phase 2 28day compressive strength

4.4 Sand Patch Test

The sand patch test was carried out as per the methodology used in phase 1. The rough interface surface limit of 3mm as per Eurocode 2 Part 1-1 (2004) was used. All interface surfaces passed this criteria with results presented in Table 4.5.

Table 4.5: Results of Sand Patch Test on Phase 2 Prestressed T Beams

Sample	No. of tests	Ave. dia. (mm)	Area (m ²)	Vol. of sand used (ml)	Thickness (mm)	Thickness Req. (mm)	Pass / Fail
Control Beam	2	136.67	0.015	47.5	3.24	3	Pass
PTB1-1	2	130.00	0.013	47.5	3.58	3	Pass
PTB1-2	2	136.67	0.015	47.5	3.24	3	Pass
PTB1-3	2	135.00	0.014	47.5	3.32	3	Pass
PTB2-1	2	131.67	0.014	47.5	3.49	3	Pass
PTB2-2	2	134.17	0.014	47.5	3.36	3	Pass
PTB2-3	2	137.50	0.015	47.5	3.20	3	Pass

4.5 Phase 2 Experimental Study

4.5.1 Scope

The scope of phase 2 was to identify a weaker concrete section with a similar span to phase 1 that could be tested to failure with the limiting criteria of a 300 kN maximum capacity Trinity College hydraulic test rig. Lessons were learned from phase 1 and the section size was decreased in order to obtain interface failure. Research carried out by Chung and Chung (1976), Hughes (1987) and Kovach (2008) was fundamental in choosing the section for phase 2. The beams were prestressed and the loading was cyclic in order to achieve data that could produce enough evidence to propose a new coefficient of cohesion for calculating interface shear provisions to Eurocode 2 Part 1-1 (2004).

The variables investigated were:

- a) Stirrups crossing the interface
- b) Loading, both cyclic and static

The constants were:

- a) Concrete grade. The beams were C57/70 and the slabs C40/50.
- b) Interface roughness. The beams were all roughened to a minimum depth of 3mm.

These parameters are quite typical of precast bridge beams and cast in place insitu top slabs.

4.5.2 Test Specimens

The test specimens of series 1 were called PTB1-1, PTB1-2 and PTB1-3. These beams had no stirrups crossing the interface. The test specimens of series 2 were called PTB2-1, PTB2-2 and PTB2-3. These beams had 8mm stirrups at 400mm c/c's (0.15% bh) crossing the interface. The production details for test beam series 2 are shown in Figures 4.7 and 4.8.

Each beam had a top flange width of 550mm and depth of 100mm. The web was 300mm deep and 150mm wide. The beams were 3300mm long with an effective span of 3050mm. The beams were overly reinforced for bending with 4Nr 12.9mm diameter prestressing strands running longitudinally in the bottom of the beam and overly reinforced for vertical shear with 10mm links at 200mm centres. 3Nr 12mm lacer bars ran longitudinally and 12mm lacer bars ran transversely in the top slab. The beams were designed to fail in interface shear first.

Figure 4.7: Prestressed TB2 Section

Figure 4.8: Prestressed TB2 Production Drawing

4.5.3 Testing instrumentation

Once the beams were positioned in the testing frame the instrumentation was installed. The instrumentation consisted of a hydraulic load cell and pump, two linear variable displacement transducers (LVDT's), four strain gauges attached to the outside face of the slab (SG1, 2, 3 & 4), two horizontal (SG8 & 9) and two vertical slip gauges (SG6 & 7). The instrumentation can be seen in Figures 4.9 and 4.11 and the layout can be seen in Figure 4.10. The test set up is based on the work carried out by Kovach (2008).

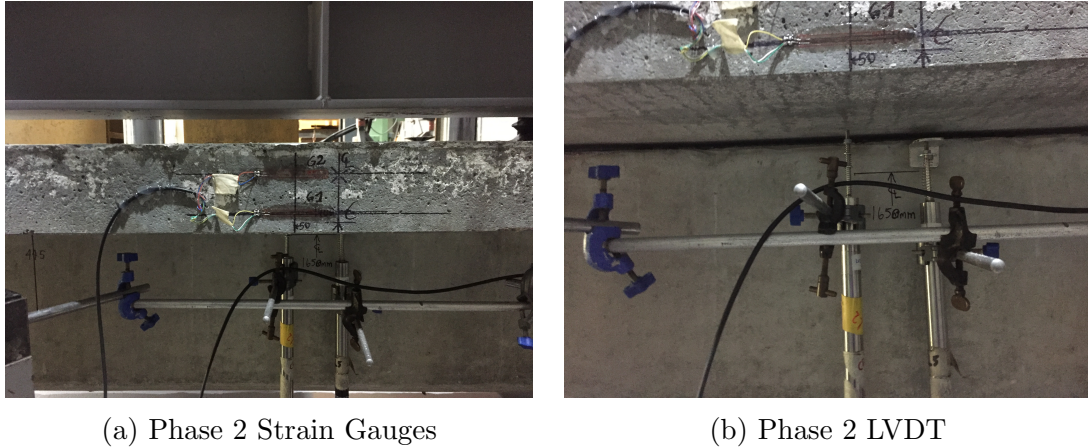


Figure 4.9: Phase 2 Testing instrumentation 1 of 2

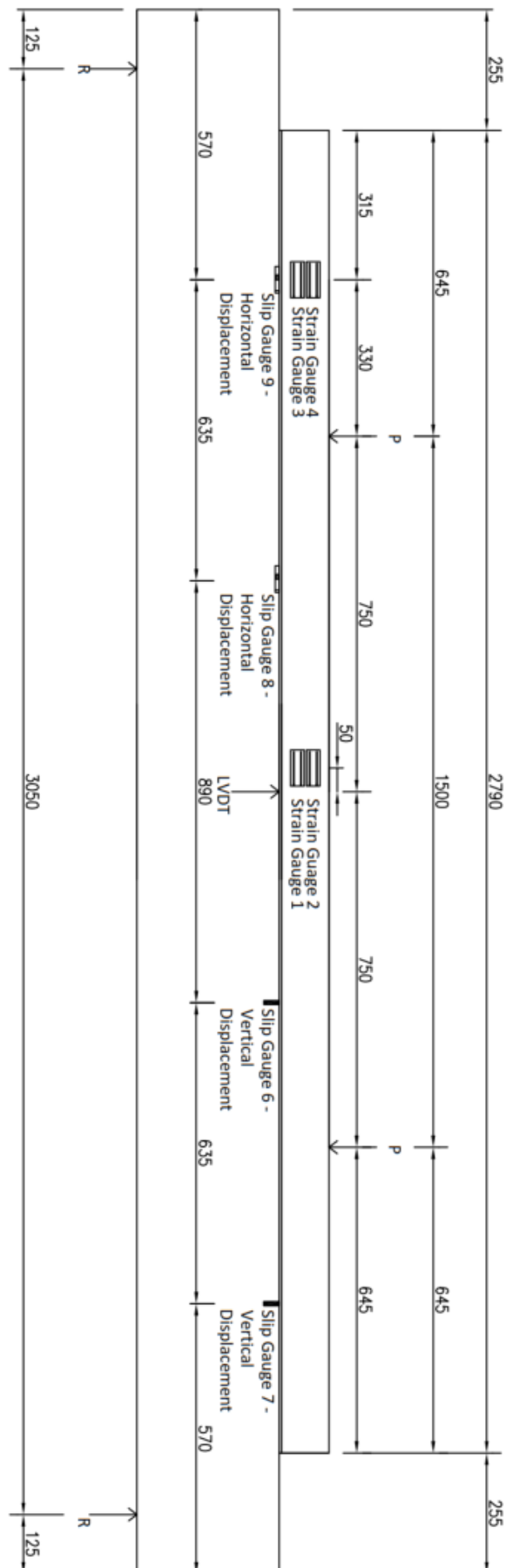


Figure 4.10: Phase 2 Instrumentation

(a) Strain Gauges

Two strain gauges were attached to the outside face of the slab near the beam end and close to the midspan. The strain gauges were used to calculate the horizontal shear stress at the interface. The strain gauges were positioned as per Figure 4.10 and can be seen close up in Figure 4.9a.

(b) Movement of the Slab relative to the Beam

Linear Voltage Displacement Transducers (LVDTs) were used at midspan to measure the vertical movement of the slab relative to the beam as per Phase 1, these can be seen in Figure 4.9b

(c) Horizontal Slip Gauges

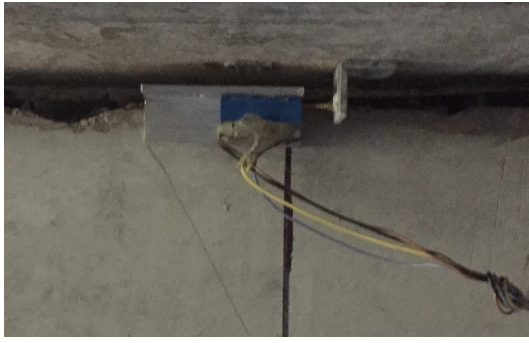
Displacement transducers as described in Phase 1 were used along the interface to measure the relative horizontal movement of the beam and the slab. Two per beam positioned as per Figure 4.10 and can be seen close up in Figure 4.11a. These gauges were fixed to a small steel angle which was positioned onto the beam web. Another small steel angle was placed at the underside of the slab. The spring loaded gauge was zero'd at the start of the test and once the slab started to slip the pin would contract or expand to measure the displacement.

(d) Crack Microscope

The crack microscope as described in Phase 1 was used to determine the crack spacing. The crack microscope can be seen in Figure 4.11b.

(e) Vertical Slip Gauges

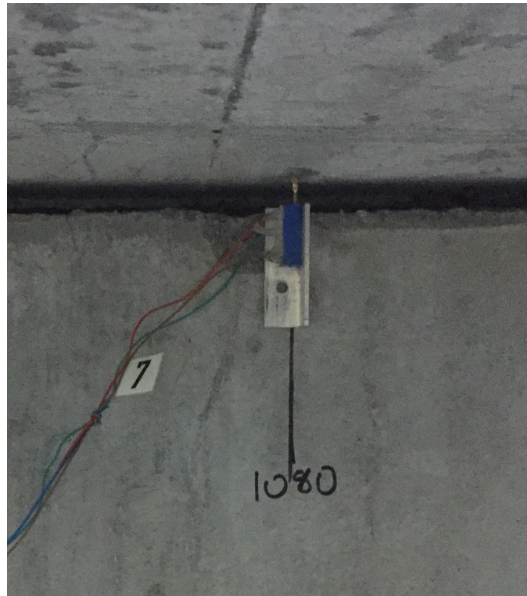
Displacement transducers as described in Phase 1 were also used along the interface to measure the relative vertical movement of the beam and the slab. Two per beam positioned as per Figure 4.10 and can be seen close up in Figure 4.11c. These gauges were adhesively fixed to a small steel angle which was adhesively fixed to the beam web. The end of the projecting pin was adhesively fixed to the underside of the slab. The spring loaded gauge was zero'd at the start of the test and once the slab started to move vertically the pin would contract or expand to measure the displacement.



(a) Phase 2 Horizontal Slip Gauge



(b) Crack Microscope



(c) Phase 2 Vertical Slip Gauge

Figure 4.11: Phase 2 Testing instrumentation 2 of 2

4.5.4 Test Procedure

The prestressed T-beams were tested in Trinity College Dublin during the Summer of 2017. There were two series of test beams with three beams per series. The first series had a control beam statically loaded to failure to obtain the cracking load. Once defined the other beams were cyclically loaded to 70% of this value as discussed earlier. If after 16,800 cycles the beams had not failed they were loaded statically to failure.

The concrete beams were placed on a structural steel spreader beam which had 150mm wide x 250mm long x 50mm thick solid steel bearing pads to act as supports at the end of the 3.3m long concrete beam giving an effective span of 3.050m. The two point loads were applied using a steel spreader beam again fixed with 150mm wide x 250mm long x 50mm thick solid steel pads 900mm from the end of the concrete beam. The test setup can be seen in Figure 4.12, Figure 4.13 and Figure 4.14.

All of the beams were subject to dynamic and static tests. The first beam that was tested was the control beam, PTB1-4 to calculate the ultimate breaking load capacity. This load was to be equivalent to Load Model 1 as per Eurocode 2 Part 2 (2005). For

fatigue verifications as per Eurocode 2 Part 2 (2005) the fatigue load model is 70% of this value and that is the load the beams would be subjected to. The rig in Trinity College was limited to 300 kN of a point load at midspan and when positioned onto the spreader beam dispersed into 2 150 kN loads. The capacity of the interface with no stirrups was calculated to be 0.65 MPa as per the provisions of Eurocode 2 Part 1-1 (2004). A 150 kN point load resulted in a shear stress on the interface of 7.34 MPa, over ten times the capacity of the joint.

The control beam was tested on the 15th of May 2017 and was 117days old. The data logger was set up and recorded load, vertical and horizontal displacement and concrete strain on one system. The load was steadily placed and stopped at 50 kN intervals (25 kN at each end) to measure cracks and observe the beam. Firstly the beam was subjected to 1000 cycles of load between 150 kN and 250 kN to test the hydraulic jack, pump and to make sure the system was live and recording. The test was a success but no damage was done to the beam. All of the displacement gauges read near zero, the largest value was a vertical displacement of 0.11mm. The beam was then loaded statically to failure. Horizontal slip gauge 9 was positioned near the end of the beam and this was the gauge that gave the largest reading, a value of 1.52mm to show a delamination of the interface at a total load of 276 kN (6.75 MPa). This was noted as the interface failure criterion.

The proposed test methodology was to cycle the beams for 16,800 cycles at a load range of 150 kN to 250 kN. If interface failure did not occur a static test to failure would be carried out.



Figure 4.12: Phase 2 Test Set up Photo

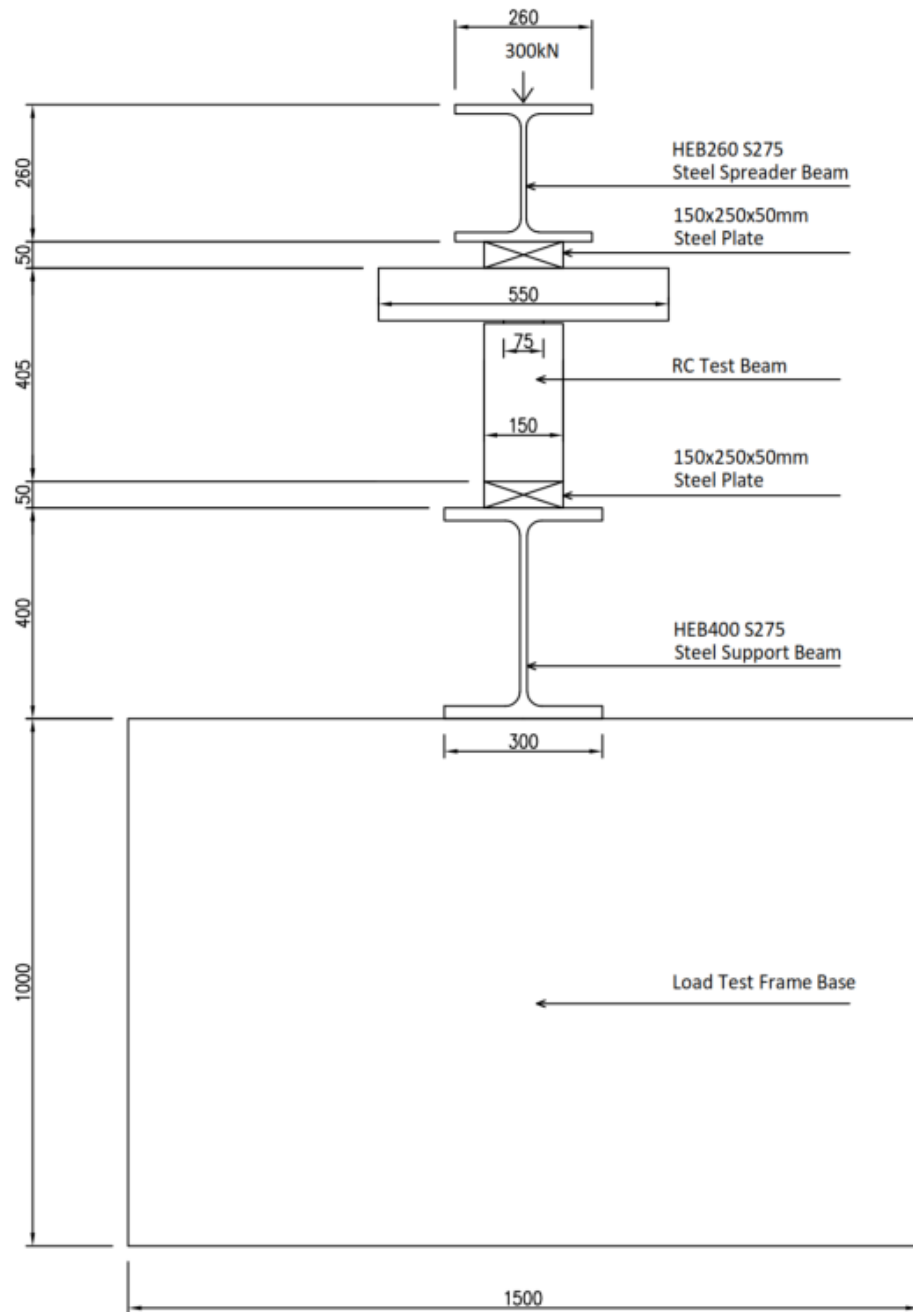


Figure 4.13: Phase 2 Test Set Up Side View

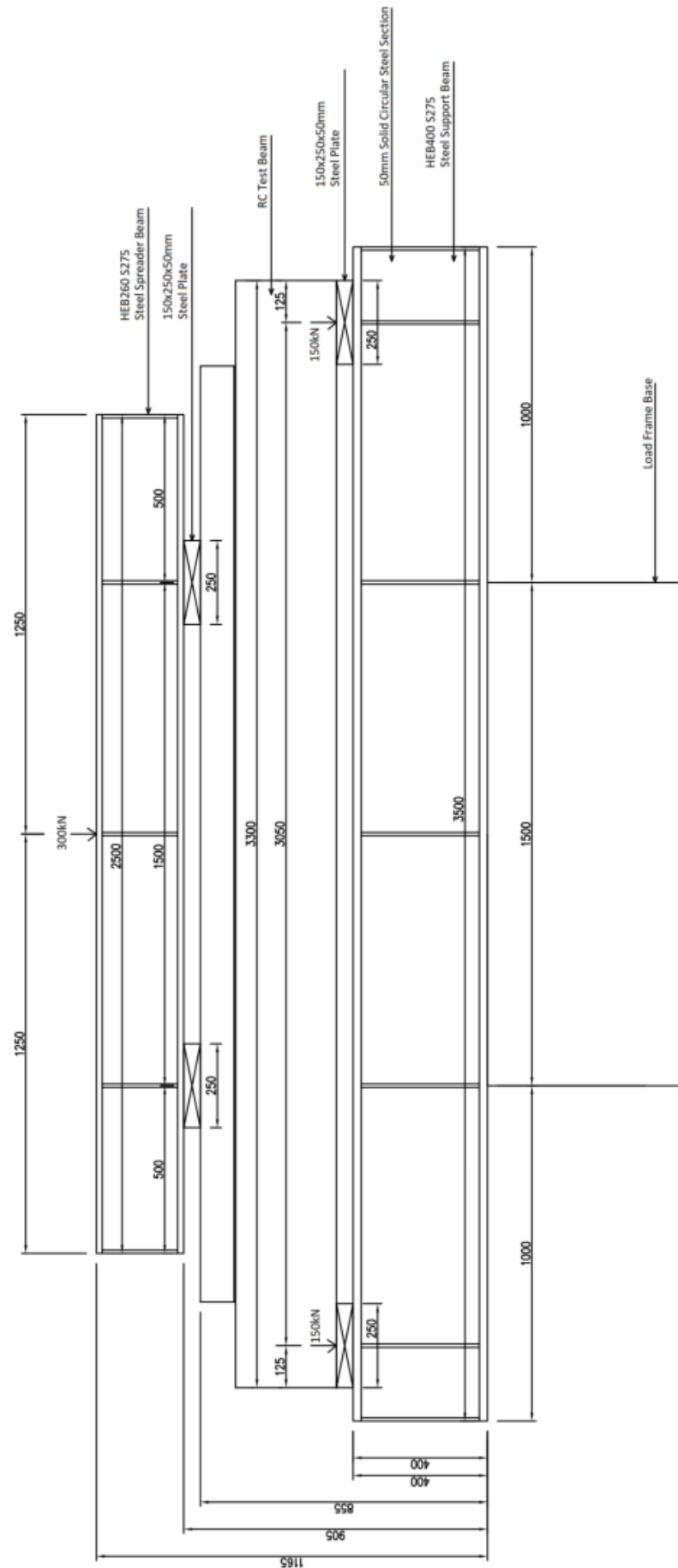


Figure 4.14: Phase 2 Test Set Up Front View

4.5.5 Phase 2 Testing - Series 1 Beams

The beams were designed for flexure and shear based on a load of 300 kN. The load was placed at midspan and dispersed onto a spreader beam as per Figure 4.13 and 4.14. This resulted in an interface shear stress of 7.34 MPa under the 2 load points. 3 composite prestressed concrete T-beams were tested under cyclic and static loading. The beams had no stirrups crossing the interface. According to Eurocode 2 Part 1-1 (2004) the capacity of the interface without shear reinforcement is 0.65 MPa. The moment capacity of the beam is 160 kNm and the vertical shear capacity is 250 kN.

PTB1-1

PTB1-1 was statically tested on the 17th of May 2017 and was 119 days old. The cyclic test was carried out over 3 days prior to the static test. The cyclic test was carried out between 10am and 5pm and stopped when the lab was unoccupied over night. From a visual inspection the beam was structurally sound, there were no signs of vertical shear or flexural cracks. The interface was uncracked even though gauge 9 read -0.71mm at maximum load during the cyclic test. This was put down to a faulty gauge as gauge 8 read -0.04mm and the vertical displacement gauges read 0.09mm and 0.01mm. There was no explanation for how a reading of -0.71mm was obtained apart from a faulty gauge. Therefore the gauges were all reset to zero for the static test. As per phase 1 the beam was loaded in increments of 50 kN with the test paused for inspection. There was no movement on any sensors until a load of 100 kN (2.45 MPa) was recorded. Horizontal slip gauge 8, which was positioned near the midspan was the first sensor to activate.

After 200 kN (4.89 MPa) sensors 6, 7 and 8 were all recording. Slip gauge 9 did not record and was omitted from the test. After 300 kN (7.34 MPa) all sensors were now displaying movement with sensor 7 the highest at 0.11mm. The load was held and the interface started to delaminate. The load was held for a further 5 minutes with sensors 8 and 9 increasing gradually. The load was increased and at 343 kN (8.39 MPa) delamination of the interface occurred. The horizontal slip gauges went from -0.09mm and -0.2mm to -1.09mm and -1.76mm respectively. The stress that caused a delamination was 8.39 MPa, which is 13 times greater than the design capacity of 0.65 MPa.

PTB1-2

PTB1-2 was statically tested on the 6th of June 2017 and was 139 days old. 17209 cycles of load between 150 kN (3.67 MPa) and 250 kN (6.12 MPa) were carried out at 0.5 Hertz over 3 days prior to the static test. From a visual inspection of the beam and of the recorded data the interface was uncracked after the cyclic test with all 4 gauges reading values between 0.05mm and -0.11mm. Therefore all gauges were reset to zero for the static test.

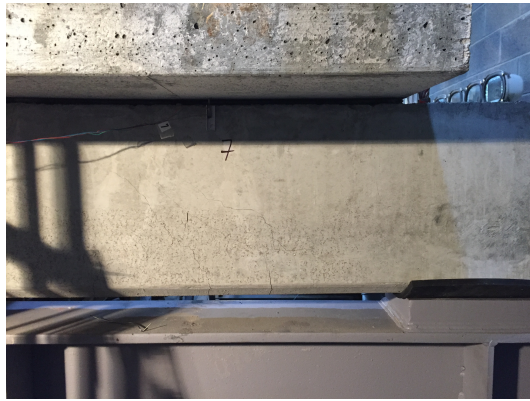
Similar to PTB1-1 there was no movement on any sensors until a load of 100 kN (2.45 MPa) was recorded. Vertical displacement gauge 6 showed gradual movement with increase in load. At 150 kN (3.67 MPa) all sensors started to signal movement albeit very slight. As the loading increased horizontal slip gauge 9, which was positioned near the beam end, started to increase. All sensors were still reading under 0.1mm of movement at this stage with some early stage cracking, which can be seen in Figures

4.15a and 4.15b. At 300 kN (7.34 MPa) vertical gauge 6 and horizontal gauge 9 both read 0.1mm and were steadily increasing. There were several vertical shear cracks clearly visible, see Figure 4.15c. These were the standard vertical shear cracks running from load point to support. All of these cracks were on the right hand side of the beam where gauge 6 was located.

At 317 kN (7.76 MPa) there were quite a lot of vertical shear cracks, with some in excess of 0.25mm. Gauge 6 was reading a vertical displacement of 0.9mm which proved delamination had occurred, see Figure 4.15d. The beam was now carrying all of the load and the vertical shear cracks were excessive. All of the other gauges never read values greater than 0.1mm. This test showed the brittle nature of an unreinforced interface where local delamination can materialise under high loads. The test was stopped at this stage as the loading was over the capacity of the test rig and delamination had occurred.

Figure 4.16 shows a graph for strain versus load on gauges 3 and 4, which are positioned at the ends of the beam where the interface crack appeared. Both gauges are reading similar values and are linear up until the test is stopped. The spike in results is as a result of the test being held in position at 317 kN to inspect the beam and finalise the recording of data.

Figure 4.17 shows a graph for load versus horizontal slip on gauges 8 and 9. Gauge 9 was positioned at the beam end where the interface crack appeared and gauge 8 was positioned more towards the midspan of the beam. Gauge 9 displayed gradual movement of the joint in relation to the increase in load, which was to be expected, and showed the steady increase in slip at the 50 kN load increments.



(a) PTB1-2 Early Cracking



(b) PTB1-2 Close up of Cracking



(c) PTB1-2 Cracking at 300kN



(d) PTB1-2 Delamination of Interface

Figure 4.15: Phase 2 PTB1-2 Test Photo's

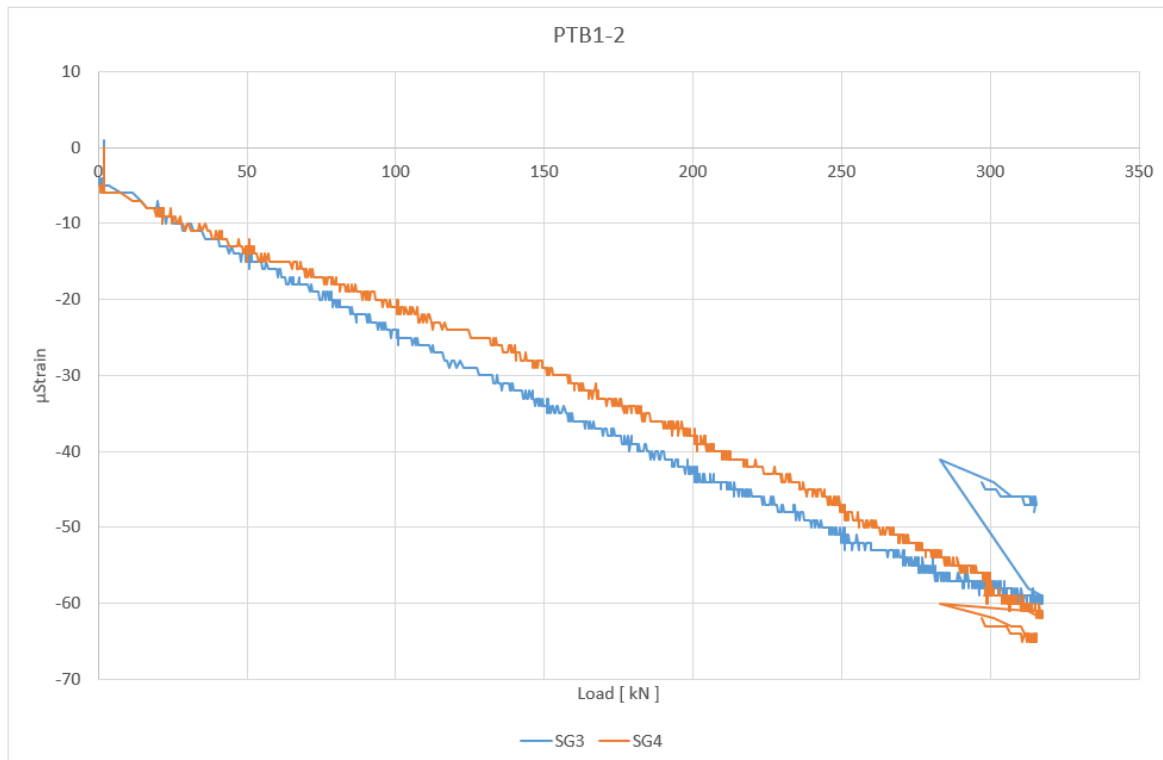


Figure 4.16: PTB1-2 Strain v Load

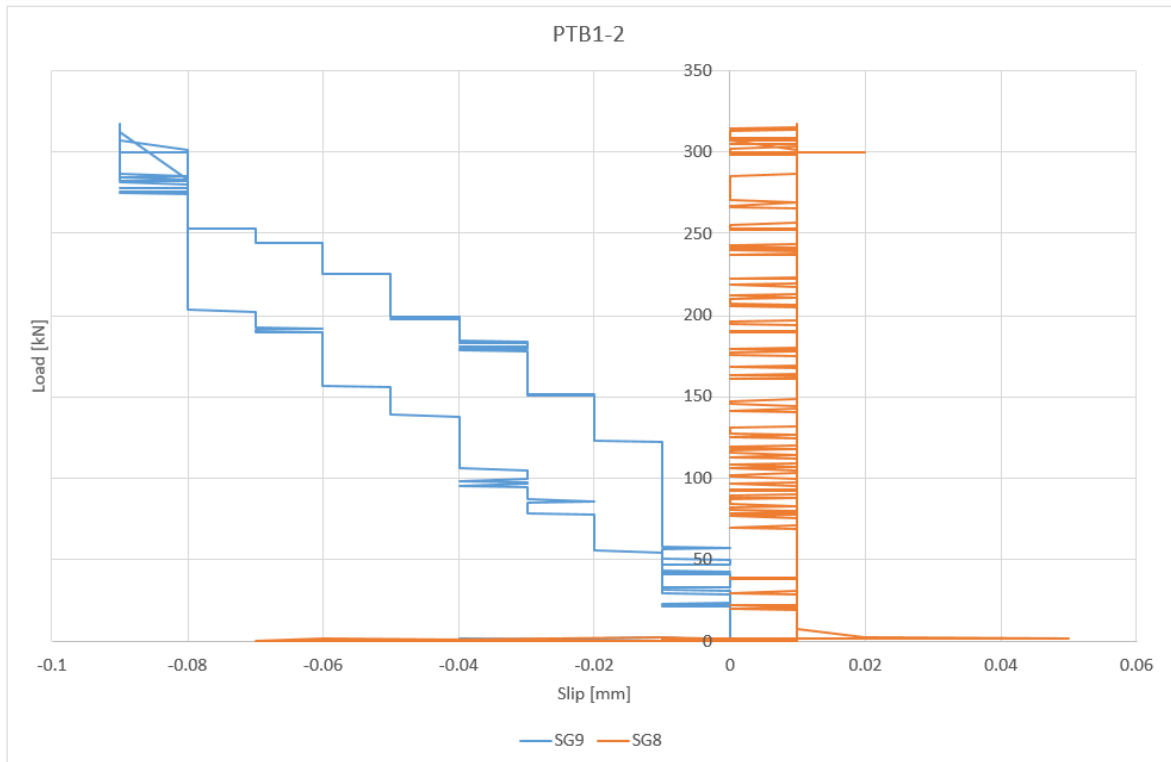


Figure 4.17: PTB1-2 Load v Slip

PTB1-3

PTB1-3 was statically tested on the 12th of June 2017 and was 145 days old. 16878 cycles of load between 150 kN (3.67 MPa) and 250 kN (6.12 MPa) were carried out using the test rig shown in Figure 4.18a. From a visual inspection the interface was again uncracked after the cyclic test with all 4 gauges reading values between 0.17mm and -0.09mm. Therefore the gauges were all reset to zero for the static test.

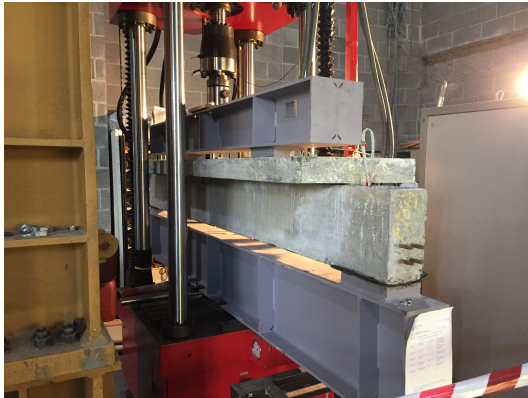
The sensors started recording data when the load was increased to 100 kN (2.45 MPa). Vertical displacement gauges 6 and 7 along with horizontal slip gauge 9 were all recording movement. At 150 kN (3.67 MPa) vertical gauge 7 had reached 0.1mm, at 200 kN (4.89 MPa) it had reached 0.15mm and at 300 kN (7.34 MPa) it had reached 0.2mm. Early cracking can be seen in Figure 4.18b.

At 325 kN (7.95 MPa) gauge 7 started to steadily increase from 0.2mm upwards to 0.4mm whilst the load was held. Gauge 6 was static at 0.4mm and as the load was held it increased rapidly to 0.69mm, gauge 8 the horizontal slip gauge close to midspan increased from 0.02mm to 0.82mm, see graph on Figure 4.19.

The loading was withdrawn and the test stopped. Once delamination occurred (see Figure 4.18d) at 325 kN (7.95 MPa) the beam acted alone and took the weight of the slab plus the live load. Vertical shear cracks were now dominant on the face of the beam and were well in excess of 0.3mm and can be seen in Figure 4.18c.

PTB1-1 failed at 343 kN (8.39 MPa), PTB1-2 at 317 kN (7.76 MPa) and PTB1-3 at 325 kN (7.95 MPa). All failures were similar and consistent with each other, which was very positive considering only 3 beams were tested. Examining Figure 4.19 in more detail shows gauge 9 gradually increasing with load, whereas gauge 8 displays no movement right up until delamination occurs at 325 kN (7.95 MPa) showing the brittle

nature of the joint.



(a) PTB1-3 Test Set-up



(b) PTB1-3 Cracking



(c) PTB1-3 Cracking at 325 kN



(d) PTB1-3 Close up of Interface
Delamination

Figure 4.18: Phase 2 PTB1-3 Test Photo's

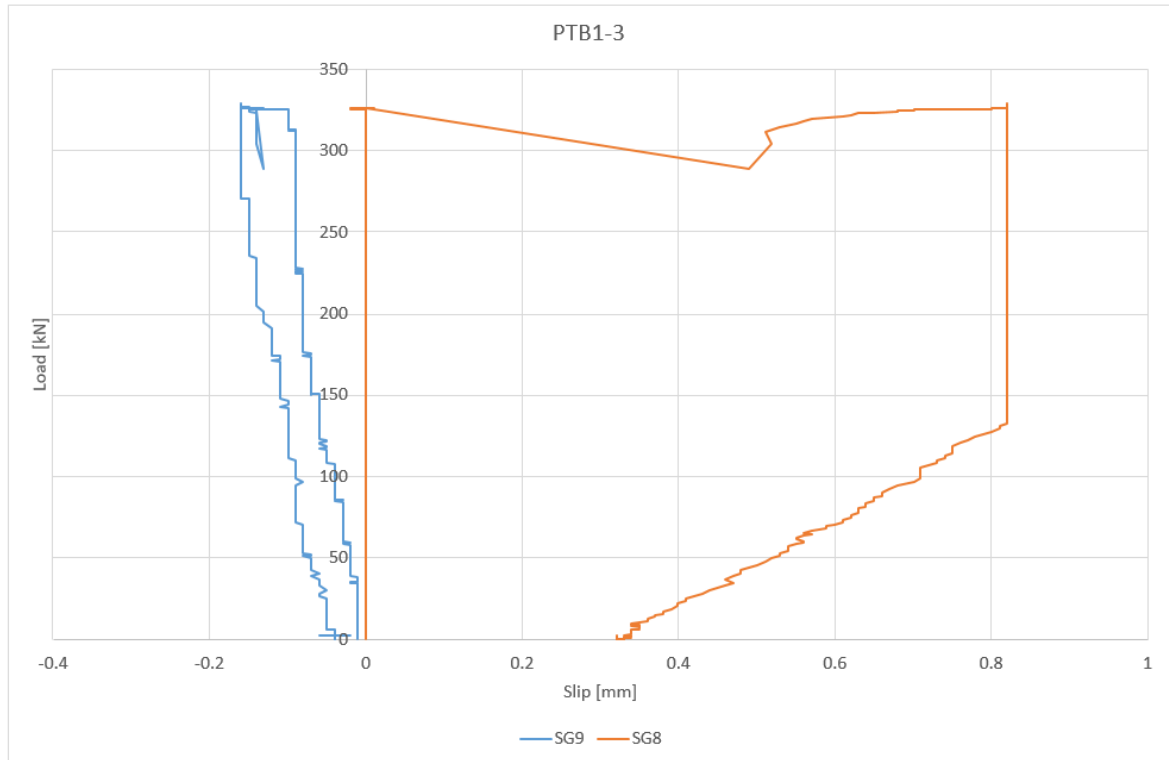


Figure 4.19: PTB1-3 Load v Slip

4.5.6 Phase 2 Testing - Series 2 Beams

The beams of test series 2 had 8mm diameter stirrups at 400mm centres crossing the interface. This was equivalent to $0.15\%bh$, which is the minimum area of stirrups required by BS5400-4 (1990). According to Eurocode 2 Part 1-1 (2004) the capacity of the interface with this amount of shear reinforcement is 1.17 MPa

PTB2-1

PTB2-1 was statically tested on the 26th of June 2017 and was 159 days old. 16890 cycles of load between 150 kN (3.67 MPa) and 250 kN (6.12 MPa) were carried out at 0.5 Hertz over 3 days prior to the static test. The beam and recorded data was inspected post cyclic test and the interface was found to be uncracked with all 4 gauges reading values between 0.02mm and -0.08mm, a lot lower than the first series of test beams with no interface stirrups. Therefore the gauges were all reset to zero for the static test.

As per the beams of series 1 there was no data on any sensors until a load of 100 kN (2.45 MPa) was recorded. Vertical displacement gauge 7 along with horizontal slip gauge 9 started recording movement between 100 kN (2.45 MPa) and 150 kN (3.67 MPa). At 250 kN (6.12 MPa) vertical gauge 7 read 0.02mm and horizontal gauge 9 -0.03mm. The effect of the stirrups was noticeable in the readings.

At 300 kN (7.34 MPa) gauge 9 read -0.06mm, all of the other gauges were stable and there were no signs of cracks. At 325 kN (7.95 MPa) the load was held, slip gauge 9 read -0.13mm and vertical shear cracks could be seen on the face of the beam, see Figures 4.20a and 4.20b. These cracks measured 0.1mm. Strain gauges 3 and 4 showed a slight spike (see Figure 4.21) which represented the cracks occurring. However as

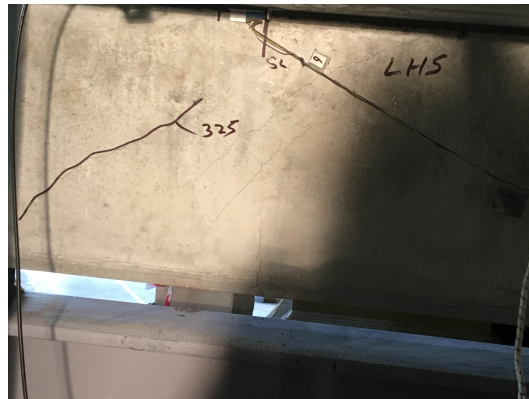
the load was held the interface did not slip any more and the effect of the stirrups was quite visible.

The load was increased to 350 kN (8.56 MPa) which was not possible with series 1 beams. The load was held and the interface sensors were stable apart from gauge 9 which read -0.23mm along with some additional vertical shear cracks, see Figure 4.20c. The load was held and flexural cracks started to appear. The cracks were now well in excess of 0.25mm and travelled under the beam and across the soffit as can be seen in Figure 4.20d. The load was increased to 364 kN (8.90 MPa) and held for 5 minutes. This was the maximum load that the test rig was able to produce. Gauge 9 was reading -0.28mm and holding under the load. The load was withdrawn slowly and gauge 9 reverted back to -0.09mm. The effect of the stirrups was quite evident in this test. Beam failure was recorded as a vertical shear and flexural failure not an interface failure.

Comparing Figure 4.22 with 4.19 the effect of the interface links can be seen. The beam interface of PTB1-3 slips to 0.89mm at a load of 325 kN whereas the beam interface of PTB2-1 holds at this load and at a maximum load of 364 kN. This can only be put down to the presence of interface links as the concrete strength and surface roughness are similar.



(a) PTB2-1 Close up of Crack



(b) PTB2-1 Cracks at 325 kN



(c) PTB2-1 Cracks at 350 kN



(d) PTB2-1 Cracks Underneath the
Beam

Figure 4.20: Phase 2 PTB2-1 Test Photo's

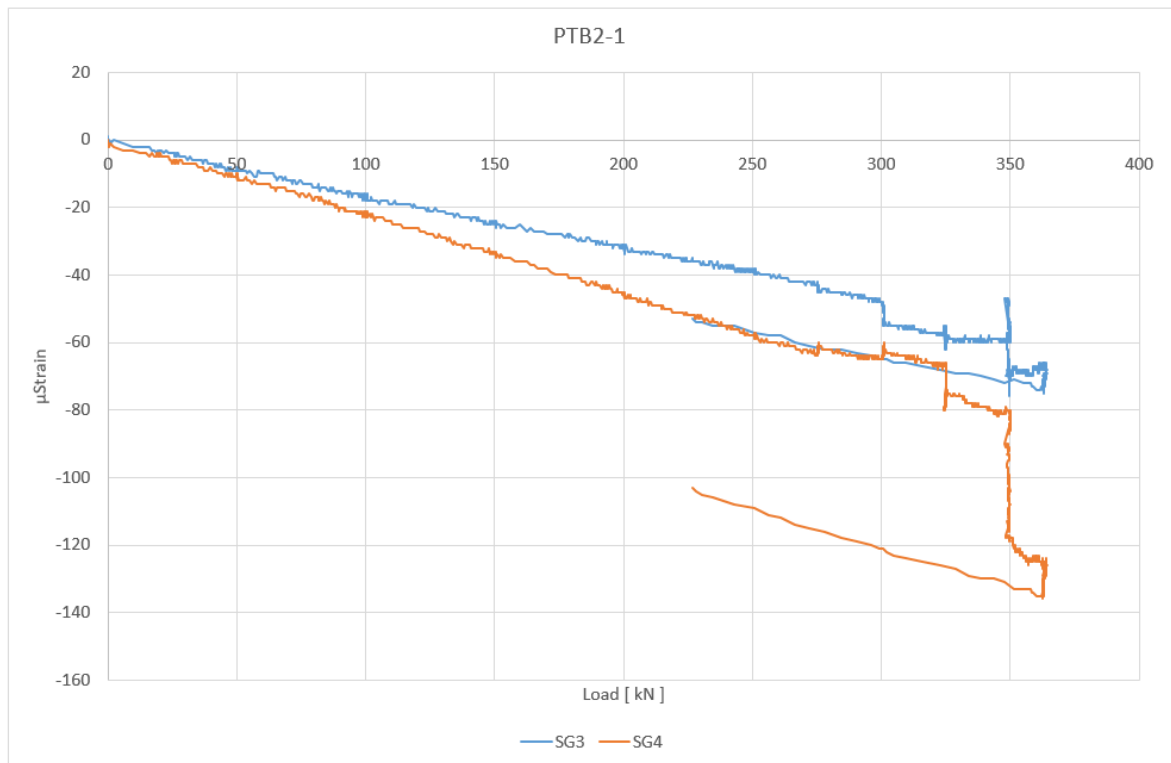


Figure 4.21: PTB2-1 End Strain

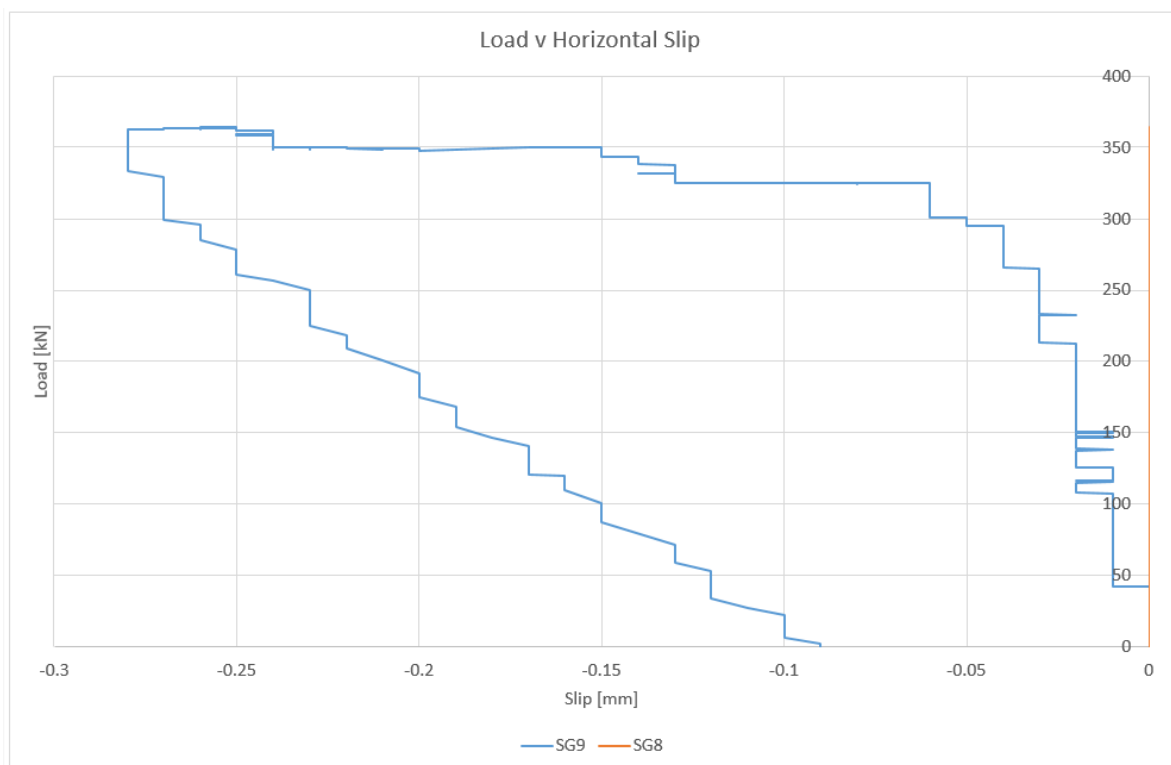


Figure 4.22: PTB2-1 Load v Slip

PTB2-2

PTB2-2 was statically tested on the 29th of May 2017 and was 131 days old. 17300 cycles of load were carried out on the beam. From a visual inspection the beam was structurally sound, there were no signs of vertical shear or flexural cracks. The interface was uncracked and all 4 gauges read values between 0.02mm and -0.07mm.

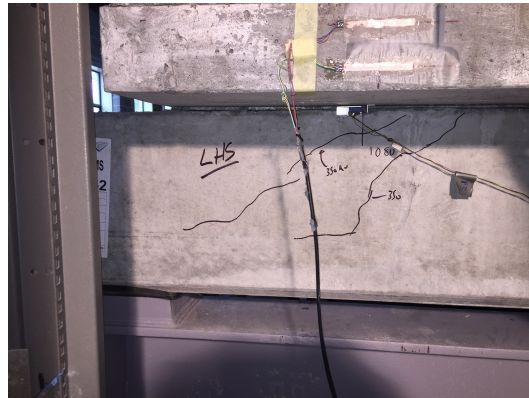
Vertical displacement gauge 7 started recording movement between 100 kN (2.45 MPa) and 200 kN (4.89 MPa). At 250 kN (6.12 MPa) vertical gauge 7 read 0.05mm. The effect of the stirrups could be seen in the gauge readings. At 300 kN (7.34 MPa) gauges 6,7 and 9 were all recording. No cracks visible up to this point. At 340 kN (8.32 MPa) the load was held and the beam examined, slip gauge 7 had moved approximately 0.1mm from the start of the test. Vertical shear cracks appeared on the face of the beam. These cracks measured in excess of 0.1mm.

The load was increased to 350 kN (8.56 MPa) with gauges 7 and 9 gradually increasing. Additional vertical and flexural cracks were appearing. The load was increased to 365 kN (8.93 MPa) and held. Gauge 7 had moved 0.2mm and gauge 9 0.3mm since the start of the test so a delamination was present but was being held by the stirrups. The load was held for 5 minutes. The vertical shear and flexural cracks were now well in excess of 0.25mm.

Finally the load was withdrawn slowly and gauge 9 reverted back to 0.06mm from -0.17mm, whilst gauge 7 reverted back to 0.02mm from 0.17mm. The beam and slab remained composite and deflected up to 10mm as can be seen in Figure 4.24. Again beam failure was recorded as a vertical shear and flexural one not an interface failure.



(a) PTB2-2 Cracking LHS



(b) PTB2-2 Cracking RHS

Figure 4.23: Phase 2 PTB2-2 Test Photo's

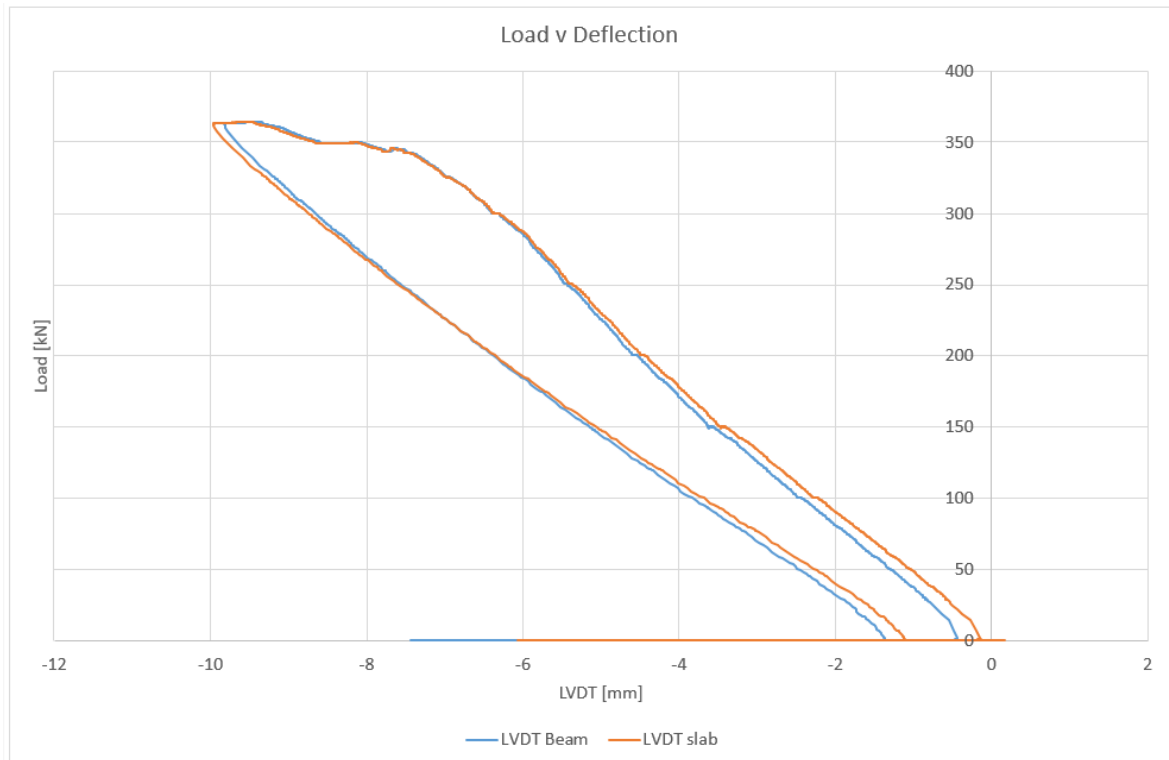


Figure 4.24: PTB2-2 Load v Deflection

PTB2-3

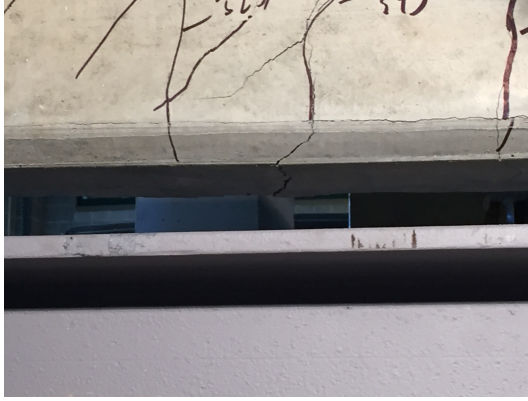
PTB2-3 was statically tested on the 16th of June 2017 and was 149 days old. 17708 cycles of load were carried out on the beam. The interface was uncracked and all 4 gauges read values between 0.13mm and -0.11mm, slightly higher values than PTB2-1 and PTB2-2. All gauges were reset to zero for the static test.

Vertical displacement gauge 7 started recording movement between 100 kN (2.45 MPa) and 200 kN (4.89 MPa). At 200 kN (4.89 MPa) vertical gauge 7 read 0.05mm very similar to PTB2-2. At 250 kN (6.12 MPa) gauge 7 read 0.11mm. All other gauges were stable at this point. At 300 kN (7.34 MPa) there were no cracks on the face of the beam.

At 325 kN (7.95 MPa) the load was held and further visual inspections took place. Slip gauge 7 read 0.17mm and vertical shear cracks were present on the face of the beam, which can be seen with the spike on the graph in Figure 4.26. The beam cracked on the left hand side only, there were no cracks on the right hand side of the beam which displayed an uneven distribution of load. The flexural cracks near the midspan were up to 0.1mm in size and the vertical shear cracks at the beam end were 0.2mm in size. The load was increased to 345 kN (8.44 MPa), this resulted in additional cracking. These cracks measured in excess of 0.25mm. Still no cracking on the right hand side of the beam. Gauge 7 read 0.2mm and gauge 9 read -0.04mm.

The load was increased to 350 kN (8.56 MPa) and held with no further movement of the interface on any gauges. The load was increased to 365 kN (8.93 MPa) with gauge 7 now recording 0.22mm. The vertical cracking was very excessive and spread down and under the web of the beam, see Figure 4.25a. The crack was measured at approximately 2mm, see Figure 4.25b. A prestressed beam failure was not favourable so the test was stopped with vertical shear and flexural failure cited.

Once the load was taken off all gauges reverted to near zero with the interface intact. The vertical shear cracks did not recover and remained open at approximately 1mm in size when the load was removed.



(a) PTB2-3 Severe Cracking



(b) PTB2-3 Cracking Measured

Figure 4.25: Phase 2 PTB2-3 Test Photo's

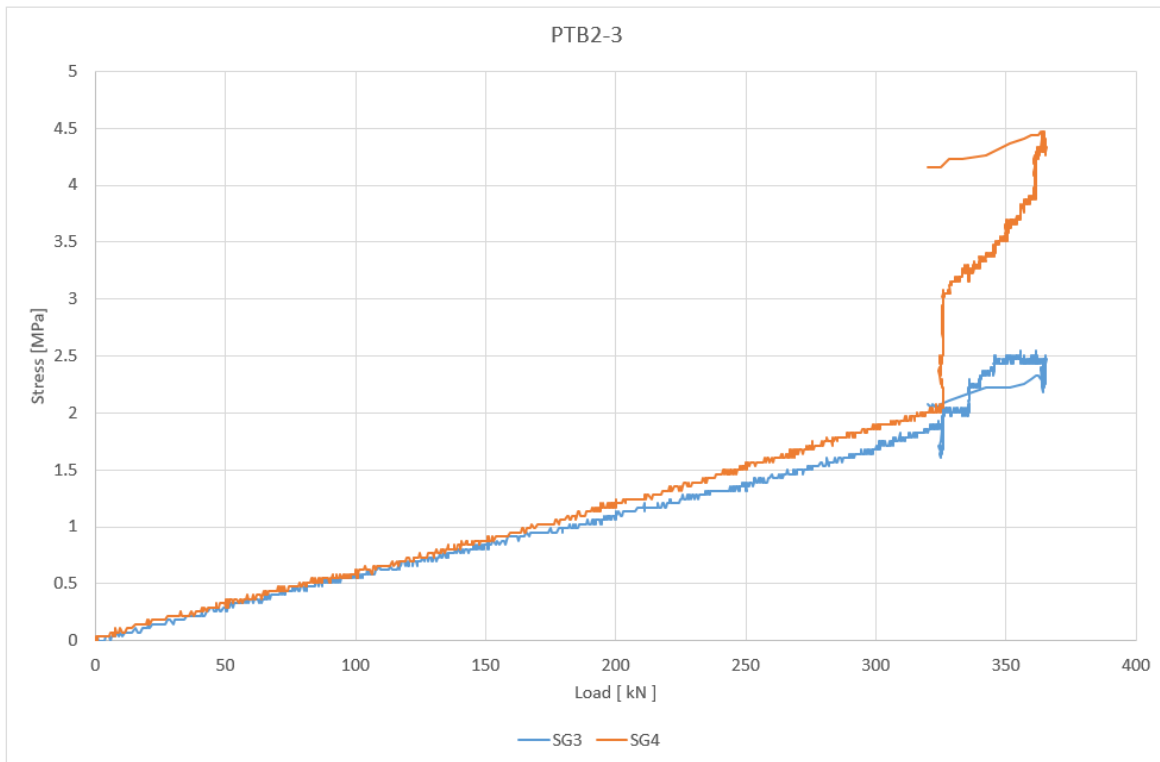


Figure 4.26: PTB2-3 Load v Stress

4.6 Testing - Phase 2 - Discussion

(a)Code Provisions:

Table 4.6: Testing - Second Series - Average Factor of Safety

Beam Type	Design Capacity (MPa)	Shear Stress @ 1st Interface Crack (MPa)	Factor of Safety
PTB1-1	0.65	8.39	12.91
PTB1-2	0.65	7.76	11.93
PTB1-3	0.65	7.95	12.23
PTB2-1	1.17	7.95	6.80
PTB2-2	1.17	8.32	7.11
PTB2-3	1.17	7.95	6.80

The test results show that the current bridge design codes for Ireland, the UK, Europe and the US are conservative. The provisions of the design codes for horizontal shear are safe for the concrete strengths used in the tests. It may be possible that the codes are not safe for lower concrete strengths. The first series of beams had an average shear stress at the first interface crack of 8.03 MPa, see Table 4.6. When compared to the horizontal interface shear resistance without stirrups of 0.65 MPa as per the provisions of Eurocode 2 Part 1-1 (2004) results in a factor of safety of 12.36. The second series of beams had an average shear stress at the first interface crack of 8.07 MPa, see Table 4.6. When compared to the horizontal interface shear resistance with 8mm stirrups at 400mm centres of 1.17 MPa as per the provisions of Eurocode 2 Part 1-1 (2004) results in a factor of safety of 6.9. The difference between the two failures was series one a brittle quick slip of the interface that continued to move as the load was increased. Series 2 beams with stirrups failed at approximately the same load but the stirrups held and clamped the interface together. The slip was much slower to occur and did not travel as long under load increase. When the load was taken off the horizontal slip recovered to almost zero.

(b)Minimum Links:

The testing carried out proves that the provisions of BS5400-4 (1990) and CEB-FIP Model Code 2010 (2010) are safe and can be used in practice. From the tests the interfaces with no links and with minimum links both experienced cracks at approximately the same stress, on average 4.85 MPa. Hughes (1987) states that for bridgeworks maximum stresses of 3.0 MPa would be normal and are well below the test results for beams with no links. As the majority of prestressed bridge beams are used in bridge construction there will always be links crossing the interface for vertical shear requirements. Once the surface is rough and links are present no additional interface links need be provided once the minimum amount of vertical links equates to $0.15\%bh$. This is also the view taken by the Technical Committee preparing the next version of the Eurocodes

Table 4.7: Testing - Second Series - Summary of Results

Beam Type	Beam Age (Days)	Slab Age (Days)	Beam Str. (MPa)	Slab Str. (MPa)	Int. Links	Design Cap. (MPa)	Shear Stress @ 1st Crack (MPa)	SR (mm)
PTB1-1	148	119	77.02	77.22	None	0.65	8.39	3.58
PTB1-2	168	139	77.02	77.22	None	0.65	7.76	3.24
PTB1-3	174	145	77.02	77.22	None	0.65	7.95	3.32
PTB2-1	188	159	77.02	77.22	1B8-400	1.17	7.95	3.49
PTB2-2	160	131	77.02	77.22	1B8-400	1.17	8.32	3.36
PTB2-3	178	149	77.02	77.22	1B8-400	1.17	7.95	3.20
Note: Str. = Strength, Int. = Interface, Cap. = Capacity, SR = Surface roughness								

4.7 Phase 2 Conclusions

The following are the conclusions made by the author based on the data obtained for the second phase of testing:

1. The roughening of the interface surface using aggregate provided a sufficient rough surface and can be used in practice once the joint is power washed and any loose materials washed away.
2. The interface joint cracking was as a direct result of diagonal shear cracking running from the load point to the support point as per phase 1.
3. The interface joint is undisturbed until loading stresses reach 2.45 MPa.
4. The joint strength in fatigue was greatly improved with the use of minimum stirrups as can be seen from the readings post cyclic testing.
5. The horizontal slip readings were the most accurate form of data and confirmed whether the joint had delaminated or not. These were not used on phase 1.
6. Minimum horizontal interface shear links based on the formula of BS5400-4 (1990) can still be used in design. The test data averaged a shear stress capacity of 8.07 MPa compared to a design resistance of 1.17 MPa as per Eurocode 2 Part 1-1 (2004) resulting in a factor of safety of 6.9.

7. A non reinforced interface has substantial shear resisting capacity. The test data averaged a shear stress capacity of 8.03 MPa compared to a design resistance of 0.65 MPa as per Eurocode 2 Part 1-1 (2004) resulting in a factor of safety of 12.35.
8. A reinforced interface with minimum stirrups ($0.15\%bh$) has a clamping affect that can withstand a horizontal interface slippage even at loads well in excess of design capacity.
9. Beam and slab concrete cube strengths did not seem to have an impact on the design. The beam cube strengths were identical as all beams were poured together, similarly the slabs. Therefore the importance of concrete strength is inconclusive and further testing should be carried out with varying concrete grades.
10. Interface surface roughness was consistent between 3mm and 3.5mm. Lower and higher surface roughness's could be researched further to see their impact.
11. The extensive literature review for a section size that would give an interface failure based on the capacity of the rig was of paramount importance to this research project in terms of the quality of results.

4.8 Phase 2 Recommendations

1. For precast concrete prestressed bridge beams the use of a rough as cast surface finished with 8mm aggregate will suffice for a rough surface as per the requirements of Eurocode 2 Part 1-1 (2004).
2. Interfaces trafficked by vehicles i.e. bridges and car parks should have a minimum percentage of reinforcement crossing the interface to prevent a brittle failure of the joint.
3. Test data and previous research find that the coefficient of cohesion for use in section 6.2.5 of Eurocode 2 Part 1-1 (2004) should still be taken as 0.2 for smooth surfaces and 0.4 for rough surfaces and not be decreased once there is a minimum area of stirrups crossing the interface equating to $0.15\%bh$.
4. Fatigue failure is not probable within 16,800 cycles if the repeated load is below 70%. No significant horizontal slip will develop at the interface under such loading.

Chapter 5

Conclusions

5.1 Comparison with Previous Research and Bridge Design Codes

Hanson (1960) concluded after testing similar beams to the phase two specimens that for precast prestressed bridge beams to act compositely with a cast on deck slab a combination of a rough bonded surface along with stirrups crossing the interface are required. The author also stated that when the interface slipped more than 0.127mm the composite action between the beam and the slab had ended. Out of the 6 beams tested in phase 2, 4 slipped greater than this value.

The results of the prestressed T beams showed high factors of safety. Nasser (1961) tested a precast prestressed bridge beam cyclically with just over 1,000,000 cycles of load. He used a load slightly lower than the design load. The beam did not fail during the cyclic test and when tested statically to failure the beam failed at a load four times the design capacity which equates well to the test data obtained.

Badoux and Hulsbos (1967) tested prestressed T beams dynamically for 2,000,000 cycles of load. They defined fatigue failure as a joint slip greater than 0.254mm. Joint failure only occurred when the beams were statically tested to failure as failure did not occur during the dynamic tests. This was very similar to the phase 2 testing. The largest slip in PTB1 series beams was 1.76mm and 0.28mm for PTB2 series beams. The effect of the stirrups greatly reducing the slip.

Mast (1968) stated that stirrups are required across the interface to prevent the brittle failure of the concrete bond. Phase 2 beam results compared favourably with this statement and recommends a minimum area of stirrups crossing the interface to prevent brittle slip.

Hofbeck et al. (1969) reported that the dowel action of stirrups crossing the interface becomes significant when the shear plane has cracked. This strengthens the findings of phase 2 testing.

Chung and Chung (1976) carried out cyclic tests on precast prestressed T beams. The authors found that stirrups crossing the joint greatly increased the endurance of the joint as one of the beams with no stirrups failed at 37000 cycles of loading whereas all of the beams with stirrups lasted far longer. The authors did state that a slip of 0.025mm may be considered as the critical value beyond which the efficiency of the joint rapidly deteriorates. All of the phase 2 test specimens experienced slips greater than this value. The authors concluded that fatigue failure would not occur if the repeated loading value was less than 55% of the ultimate load. This concurs with the

findings of phase 2 beam testing where the cyclic loading was set at 75%.

Hughes (1987) carried out cyclic and static tests on M3 prestressed bridge beams. He concluded that the 0.15%bh for minimum interface stirrups of BS5400-4 (1990) should not be altered. He did find the provisions of BS5400-4 (1990) to be conservative for roughened and monolithic surfaces but satisfactory for rough as cast surfaces. Phase 2 beam testing used the minimum requirements of BS5400-4 (1990) as per the recommendations of Hughes (1987).

The phase 2 beams had 8mm stirrups @ 400mm centres embedded 270mm into the beam. Bass et al. (1989) carried out tests to prove that the deeper the embedment of the stirrup the greater the capacity of the interface. The draft version of the Eurocode 2 Part 1-1 (2004) currently has a provision for embedment of $0.5f_yd$ which is 217.5mm. Therefore the phase 2 testing correlated with the new proposed provisions.

The maximum slip for every beam that failed in horizontal shear in tests carried out by Loov and Patnaik (1994) was 2mm. The maximum slip from the phase 2 beams was 1.76mm. Therefore according to Loov and Patnaik (1994) there was still adequate capacity in the interface to resist additional shear stress. The author also stated that the interface stirrups did not become effective until a shear stress of 2.0 MPa. This indicates the strength of the unreinforced interface. The unreinforced beams in phase 2 had adequate capacity even at stresses of 8.0 MPa.

Kahn and Slapkus (2004) defined interface shear failure as a slip greater than 0.25mm. Only 3 of the phase 2 beams had slips greater than this value.

Randl (2013) stated that if there is no reinforcement crossing the interface you will have low levels of slip failure, approximately 0.05mm. Well reinforced interfaces behave more in a ductile fashion with higher levels of slip failure, approximately 0.5mm to 1.5mm. This correlates well with the test results of phase 2.

Randl (2013) presented work on the fatigue of interface shear joints. He noted that if the shear capacity of the joint exceeds the shear stress on the joint then there is no need for interface stirrups. The 3 beams cyclically tested in phase 2 with no interface stirrups compared favourably to the conclusions of Randl (2013). Considering that there was no interface crack until an average shear stress of 8.03 MPa when the interface resistance was calculated to be 0.65 MPa using the provisions of Eurocode 2 Part 1-1 (2004). Randl (2013) also proposed provisions for minimum interface reinforcement which are now being used in CEB-FIP Model Code 2010 (2010) which gives an identical answer to the 0.15%bh provision of BS5400-4 (1990) which as discussed previously is favourable to the results of the phase 2 beam tests. Finally Randl (2013) presented a provision for minimum anchorage based on a rough interface finish which he noted to be $\geq 1.5\text{mm}$ of $10 \times$ bar diameter which is a lot lower than the provisions of the draft Eurocode 2 Part 1-1 (2004).

5.2 Objectives

In chapter 1 the objectives of this thesis were set out as follows:

1. To carry out an appraisal of present Eurocode 2 Part 1-1 (2004) requirements regarding interface shear, particularly regarding the interface roughness coefficients and comparison with other recognised design standards.
2. To propose a minimum interface shear requirement for Eurocode 2 Part 1-1 (2004) based on the testing undertaken.

3. To propose a value for the coefficient of cohesion based on fatigue loading.

5.3 Conclusions

1. A full and comprehensive detailed calculation following the provisions of section 6.2.5 of Eurocode 2 Part 1-1 (2004) was carried out and compared with BS5400-4 (1990), CEB-FIP Model Code 2010 (2010), ACI 318-14 (2014) and AASHTO LRFD-BDS (2007) and is shown in Table 2.4 for the phase 1 beam and in Table 4.3 for the phase 2 beam.
2. After the extensive beam testing phase 1 and phase 2 the author has come to the conclusion that for safety and to prevent a brittle slip of the interface the provisions of BS5400-4 (1990) ($0.15\%bh$) should be used for calculating the minimum amount of horizontal interface shear links.
3. Phase 2 tested 7 precast prestressed concrete beams with 16,800 cycles of load and none of the beams failed in interface shear. All beams were statically tested to failure after the dynamic tests. Clause 6.2.5 of Eurocode 2 Part 2 (2005) setting the coefficient of cohesion to zero is not applicable to precast quality controlled beams.

5.4 Recommendations

1. Bridge Engineers in Ireland and the UK design using the provisions of Eurocode 2 Part 1-1 (2004) when designing concrete beams. The American codes, AASHTO LRFD-BDS (2007) and ACI 318-14 (2014) as well as the CEB-FIP Model Code 2010 (2010) should be used for comparison as all 3 contain a commentary which explains the clauses in more detail and references the source behind the section.
2. Minimum links should be provided in horizontal interfaces and the provisions of BS5400-4 (1990) ($0.15\%bh$) should be used as these have proved to be safe.
3. Test data and previous research find that the coefficient of cohesion for use in section 6.2.5 of Eurocode 2 Part 1-1 (2004) should still be taken as 0.2 for smooth surfaces and 0.4 for rough surfaces and not be decreased once there is a minimum area of stirrups crossing the interface equating to $0.15\%bh$ and good quality control is assured on site and in the precast factory.

5.5 Future Work

Additional research is required to answer several questions that arose during the course of this project, namely:

1. The procedure for measuring the roughness of a surface is still the sand patch method, which although a simple technique, is time consuming. With the advancements in technology i.e. 3D printing and 3D scanning, a new way of measuring interface roughness must be researched and implemented.

2. The new draft Eurocode 2 Part 1-1 (2004) calculates minimum anchorage for horizontal interface links. Further testing and research is required on this topic in order to examine the effect of concrete strength and mix design on anchorage. The addition of overlays to strengthen bridges is common practice and a depth for interface dowels is a major design criterion.
3. Further testing of the prestressed T-beams of phase 2 should be carried out with varying concrete strengths. The interface shear capacity provisions of Eurocode 2 Part 1-1 (2004) depend largely on the strength of the concrete slab and more data would be extremely beneficial to the committee drafting the new version of Eurocode 2 Part 1-1 (2004). Ultra high performance concrete is more readily available and its benefit on interfaces should be researched.
4. Finally further research on the use of a surface bonding agent should be carried out. In bridgeworks, especially during on site operations the top surface of a bridge deck may not always be as rough as necessary. Roughening procedures are expensive when compared to cleaning the surface and applying a bonding agent.

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