Rehabilitation of Willaston Bridge: Utilization of Rivet Heads as Shear Connectors

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SYNOPSIS

The Willaston Bridge at Gawler, S.A. was built during 1890 using riveted wrought iron plate girders. The original timber deck was replaced with a non-composite reinforced concrete deck during 1956, which was cast on top of the girders with the top flange rivet heads embedded in the deck concrete.

A need for this bridge to carry Higher Mass Limits (HML) vehicles has arisen, but its theoretical girder capacity is too low, based on the assumption of non-composite behaviour. However, composite action has been suspected in view of the lack of evidence of damage caused by overloading.

Bridge load testing, in conjunction with laboratory tests, was used to determine the effectiveness of rivet heads acting as shear connectors. Rivet heads are quite capable of transferring the concrete/girder interface shear, but are unable to prevent separation. A laboratory push test was developed to measure the shear transferred by the rivet heads for varying interface pressures, representing the dead and live loads on the bridge. A “clamped” push test was also developed to investigate the increase in strength by clamping the concrete deck slab to the wrought iron girders.

The results of the bridge testing showed that partial composite action was occurring at the serviceability limit state, but the laboratory tests showed that its extent was unreliable at the ultimate limit state. The clamped push tests showed that clamping the deck concrete to the girders is a viable strengthening method for the ultimate limit state.

1. BACKGROUND

The Willaston Bridge at Willaston, S.A. crosses the North Para River and provides a Northern access to the Township of Gawler. It comprises three simply supported spans of 18.8m and was built during 1890 using five riveted wrought iron plate girders per span (Figures 1 and 2). The original timber deck was replaced with a non-composite reinforced concrete deck during 1956, which was cast on top of the girders with the top flange rivet heads embedded in the deck concrete.

Figure 1: Elevation of Bridge

Figure 2: General Underside of Bridge

A need for this bridge to carry Higher Mass Limits (HML) vehicles has arisen, but its theoretical girder capacity is too low, based on the assumption of non-composite behaviour. For some time it has been suspected that the bridge behaves compositely (or partially compositely), in view of its good condition and absence of overload damage. The desire to carry HML vehicles has prompted the need to either confirm this behaviour or to strengthen the bridge. Consequently it was decided to load test the bridge in conjunction with laboratory tests to determine the effectiveness of rivet heads as shear connectors.

2. BRIDGE LOAD TESTING

The objectives of the bridge load testing were:

- To determine the extent of composite action between the girders and deck slab by locating the neutral axis of the girders.
- To measure the amount of slip at the girder/ deck slab interface.
- To measure the amount of separation at the girder/ deck slab interface.
- To determine the dynamic load allowance of the bridge.
- To determine the live load distribution to the individual girders.

Van Ek Contracting was selected as the successful tenderer to perform the bridge load testing.

The neutral axis location was determined by attaching strain gauge transducers to the girders at midspan and quarter-span of span 1 and at mid-span of span 2 (the latter being used to confirm the mid-span results of span 1).

Slip and separation across the concrete/girder interface were measured at quarter span and near the girder ends using strain transducers across the interface (Figure 3).

A “legal” 43.5 tonne six axle semi-trailer with measured axle loads and spacings was used as a test vehicle. It was used to load the bridge without any other traffic present (Figure 4).

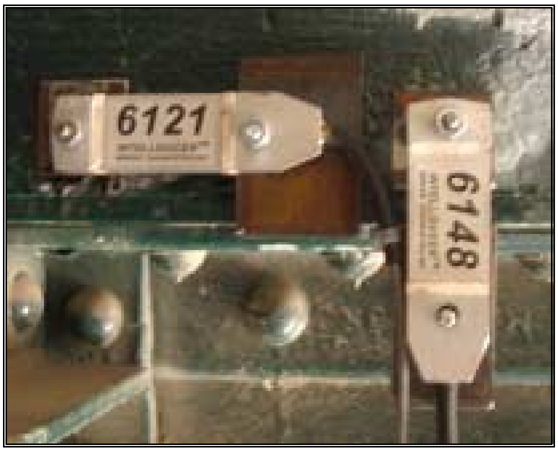

Figure 3: Transducers Measuring Slip and Separation

Figure 4: Test Vehicle on Bridge

It crossed the bridge along three different tracks: adjacent to a kerb, adjacent to the carriageway centreline and along the centreline. The superposition of the first two tracks allowed the results for two trucks on the bridge simultaneously to be determined, enabling the live load distribution to be determined. Two speeds were used – crawl speed and "normal" driving speed for the road geometry – to enable the dynamic load allowance (DLA) of the bridge to be determined.

The transducer outputs were fed into a lap top computer loaded with specialist software, which was able to produce plots of strain versus truck position for all transducers. Figure 5 shows a typical plot of girder top and bottom gauges at midspan of the centre girder.

Figure 5: Typical Transducer Output Showing MidspanTop and Bottom Strains vs Truck Position

3. LABORATORY TESTING

The laboratory testing was undertaken by Engtest, the consulting branch of the School of Civil and Environmental Engineering at The University of Adelaide, under the supervision of the second and third co-authors. The tests were designed to determine the behaviour of the concrete-girder interface under varying degrees of interface normal compressive stress, and to investigate strengthening alternatives should they be required.

The procedure involved the manufacture of steel rivets and plates to represent the riveted girders top flanges used in the construction of the Willaston Bridge. The rivets were machined from bright steel with a 2mm reduction to model possible corrosion of the original rivets. For each test specimen, two steel plates were drilled so that two rows of four rivets could be inserted in each plate. The test specimens were formed by casting 20MPa concrete with 20mm aggregate between the two steel plates. During testing, the concrete block was loaded while the slip and separation between the concrete and steel interface was monitored. Prior to each test the chemical bond between the concrete and steel was broken.

3.1 Partial Composite Action Using Interface Friction

Figure 6 illustrates typical strain distributions for a steel-concrete beam subject to flexure depending on the degree of interaction. The no-interaction (non-composite) strain distribution occurs when there is no shear connection and the steel and concrete components act independently. Consequently, there are two neutral axes at the centroids of the steel and concrete components. This is the type of analysis currently used in the assessment of slab-on-girder beams. A situation of full-interaction (full-composite) arises when there is no slip at the interface. In this case, there is a single continuous strain distribution for the entire section with only one neutral axis at the centroid of the composite section. This analysis is common in the design of new composite steel-concrete structures because of its simplicity and conservativeness with respect to the magnitude of the shear force resisted by the stud shear connectors. Consequently, these two distributions define the range of possible distributions so that for a steel-concrete beam with an intermediate degree of interaction (partially-composite), the strain distribution must lie within these bounds.

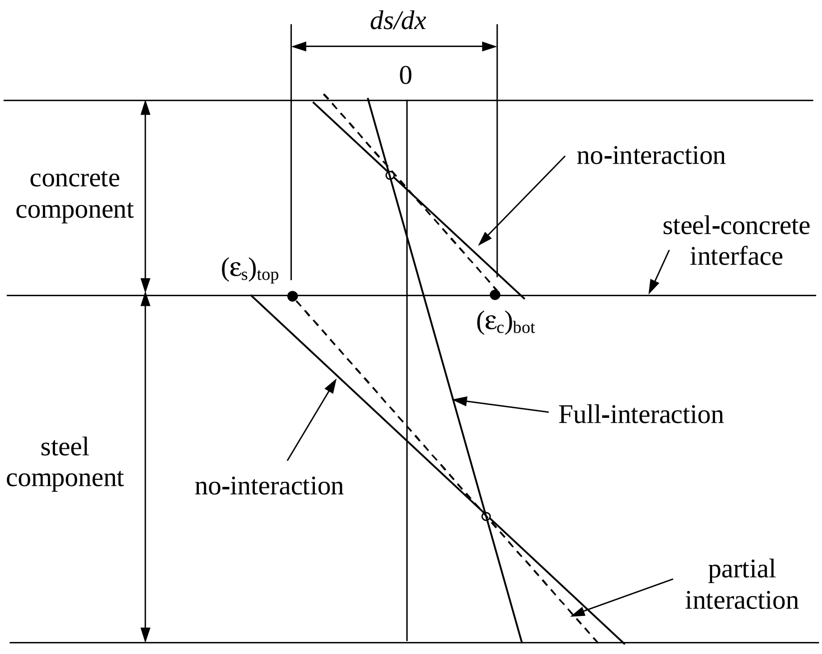

Figure 6: Steel-concrete beam strain distributions

The slip-strain, ds/dx, at the steel-concrete interface is a measure of the degree of interaction between the components. In Fig. 6, the slip-strain is identified as the difference between the strain in the concrete element and the strain in the steel element at the interface, that is $(\varepsilon_c)_{bot} - (\varepsilon_s)_{top}$. The maximum slip-strain possible is that determined from a no-interaction analysis and the integration of slip-strain along the length of the beam gives the slip.

The presence of frictional forces at the interface of a slab-on-girder beam, as a result of the interface normal compressive stresses, provides the mechanism for partial-interaction between the steel and concrete components. Therefore, the stresses for a given applied bending moment will be less than those predicted by no-interaction theory. As the degree of interaction due to interfacial friction alone is relatively small, the partial-interaction distribution will generally tend towards that of the no-interaction bound as illustrated by the dashed line in Fig. 6. The behaviour exhibited by slab-on-girder beams due to the effect of friction at the steel-concrete interface can be examined as a special case of classical linear-elastic partial-interaction theory developed by Newmark et al (1). Furthermore, research on the partial-interaction behaviour of composite beams by Seracino et al (2) and Oehlers and Sved (3) proved to be invaluable in this investigation.

The stress resultants acting at an analysis section (that is, the position of maximum moment) allowing for interface friction in a slab-on-girder beam is shown in Fig. 7 where the axial forces in the components F_{fric} are acting through the respective centroids. The force F_{fric} is the longitudinal frictional resistance along the interface, which is analogous to the resistance of mechanical shear connectors, assumed to be rigid plastic for low degrees of shear connection where failure is governed by fracture of the connectors, as in the mixed analysis approach developed by Oehlers and Sved (3). Hence, the total applied moment M_{app} is resisted by three components given by the following equilibrium equation as given by Seracino et al (4)

$$M_{app} = M_s + M_c + F_{fric}(h_s + h_c) \qquad 1$$

where M_c and M_s are the moments in the concrete slab and steel girder respectively, and h_c and h_s are the distances between the steel-concrete interface and the centroid of the concrete slab and steel girder respectively. The last term in the right hand side of Eq. 1 is referred to as the composite moment $M_{comp} = F_{fric}(h_s + h_c)$ because it is a result of the composite action between the concrete and steel components. It is M_{comp} in slab-on-girder beams, due to interface friction, that results in the observed composite action in bridge beams without mechanical forms of shear connection.

Hence, the aim of the experimental program was to determine the relationship between the longitudinal shear force and normal compressive stress of the concrete-girder interface of the Willaston Bridge. The experimental program was divided into two series of push tests. The first series was used to simulate the actual condition of the interface where the normal compressive stress was induced using springs as described in the following section. The second series of tests investigated the use of external brackets to clamp the interface and prevent separation, effectively increasing the normal compressive stress. This test series is described in Section 3.3.

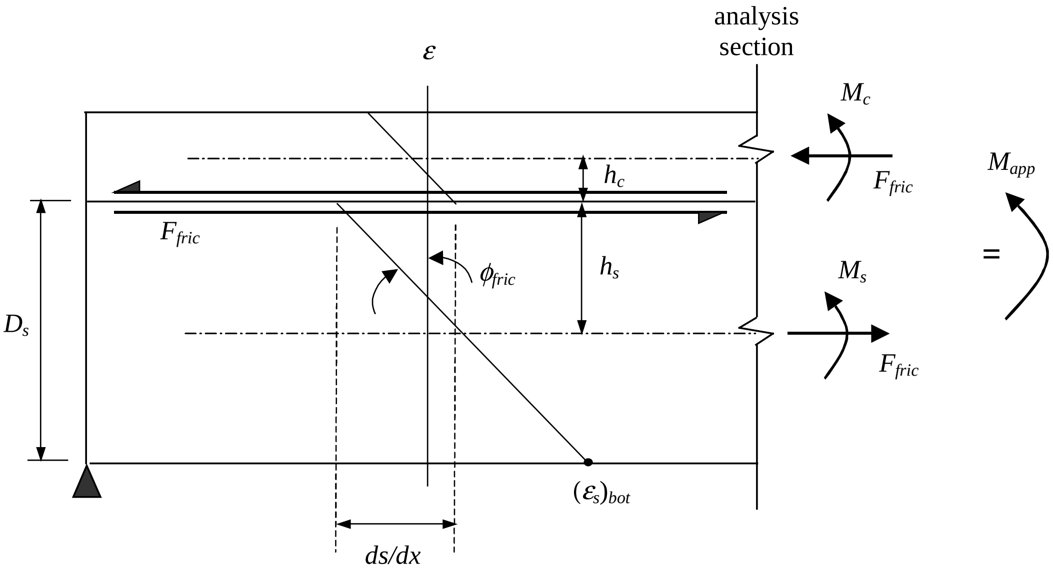

Figure 7: Stress resultants in a slab-on-girder beam

3.2 Spring Loaded Tests

Six tests were undertaken in this series where the interface normal compressive stress was induced by springs as shown in the photograph of the test set-up in Fig. 8. This test series was used to investigate the behaviour of the interface under a range of normal compressive stresses simulating the actual condition of the bridge.

Figure 8: Spring Loaded Test Setup

Figure 9 shows the experimental results of shear force/rivet head against average slip. The labels on the curves represent the applied interface normal compressive stress in units of MPa, where 0.08 MPa (or 80 kPa) is approximately the maximum stress expected in the Willaston Bridge due to dead and live loads. This is equivalent to a normal force/rivet head of 1.4 kN and results in a maximum shear force/rivet head of approximately 4.4 kN. It can be seen that the initial response is very stiff with the maximum longitudinal shear resistance being attained at a slip of less than 0.5 mm. As expected, the maximum longitudinal shear resistance increases as the interface normal compressive stress increases. Photographs of the plate with rivet heads and concrete after failure are shown in Fig. 10.

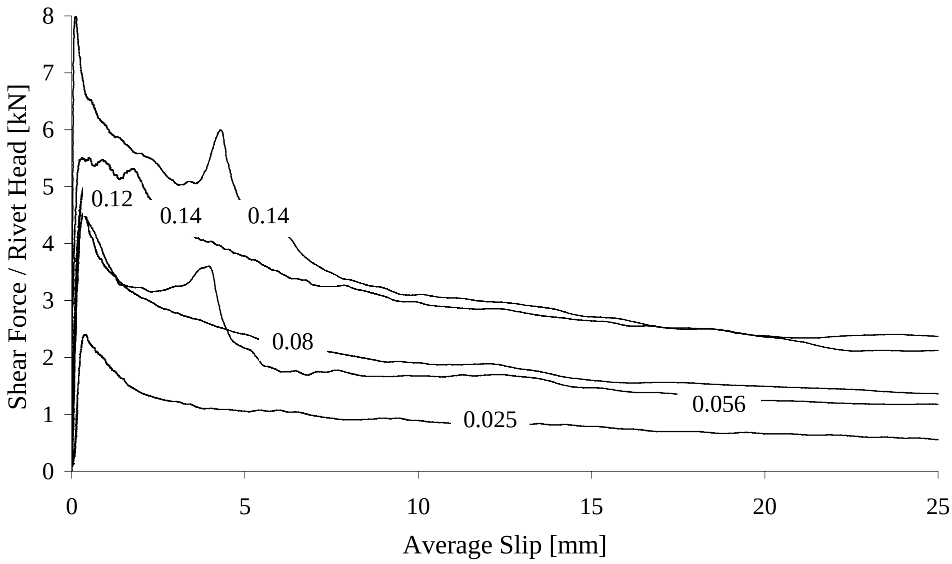

Figure 9: Interface Response for Spring Loaded Tests

(a) Plate with Rivet Heads

(b) Concrete

Figure 10: Typical Failure of Spring Loaded Tests

3.3 Bracket Tests

An external clamping arrangement consisting of bolts and a steel angle bracket was proposed to increase the maximum shear force/rivet head. An additional six tests were undertaken using various bolt and bracket arrangements in order to determine the best solution for the strengthening of the longitudinal shear connection of the Willaston Bridge.

The brackets were manufactured from 100mm long and 12mm thick steel angles. As the inside corner of the angle would need to machined to a 90° angle to ensure a snug fit with the steel and concrete at the interface, a 16mm thick 100mm x 65mm steel spacer was used between the girder flange and the bracket to simplify the preparation of the bracket. The spacer did not have an adverse effect on the strength of the proposed strengthening system. A photograph of the bracket test setup is shown in Fig. 11.

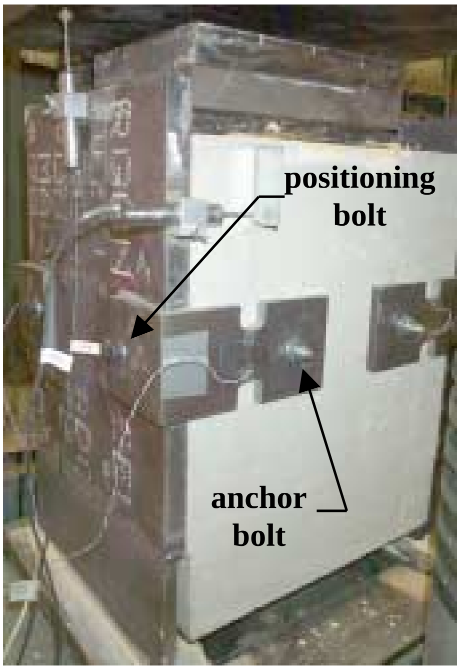

Figure 11: Bracket Test Setup

A 10mm diameter threaded positioning bolt was used to locate the bracket on the steel flange of the girder as shown in Fig. 11. The anchor bolts used to attach the bracket to the side of the concrete haunch, also shown in Fig. 11, were 12mm diameter *Hilti HAS-E Threaded Anchor Rod* with *Hilti HVU Adhesive Capsule* and *Hilti Dynamic Set Washers*. The brackets were machined for the experimental tests to produce a "dog-bone" shape shown in Fig. 11, so that the force in the bracket could be accurately determined. The edge distance of the anchor bolt to the concrete-girder interface was found to influence the failure mode of the clamping system as illustrated in Fig. 12.

(a) concrete splitting (100 mm edge distance) (b) anchor bolt shear (125 mm edge distance)

Figure 12: Failure Modes of Bracket Tests

The purpose of the proposed external bracket is to increase the normal force/rivet head which, as can be deduced from Fig. 9 for the spring loaded tests, is an effective way of increasing the maximum shear force/rivet head. The separation between the girder flange and the concrete as the shear force increases induces a tensile force in the external bracket effectively clamping the girder flange to the concrete. The mechanical anchorage provided by the anchor bolt permits the tensile force in the bracket to develop. The anchor bolts are loaded predominately in shear. It is important in this system to limit the amount of separation required to develop the tensile force in the bracket. This is achieved in two ways: first, the positioning bolt ensures that the steel angle bracket, spacer and girder flange are in contact; and second, the slip between the anchor bolt and bracket is minimised by using the Hilti Dynamic Set Washers. The results of the tests for the proposed bracket strengthening system where failure was by anchor bolt shear are shown in Fig. 13. The normal force/rivet head was increased to approximately 8 kN and the *minimum* maximum shear force/rivet head was increased to 12.4 kN. Furthermore, the proposed bracket strengthening system fails in a ductile manner, with a slip at maximum shear force of approximately 6 mm, which is similar to the slip capacity of typical stud shear connectors as given by Oehlers and Bradford (5).

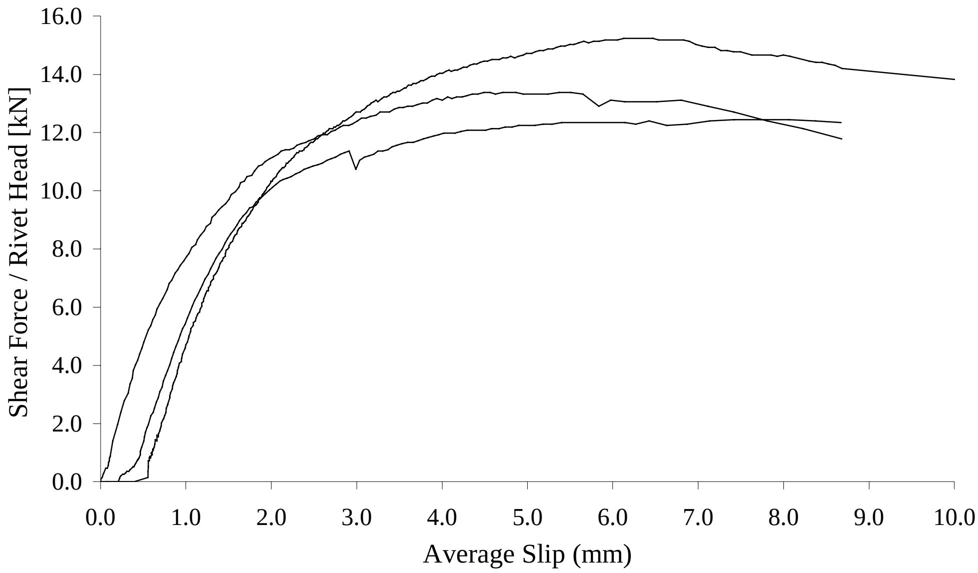

Figure 13: Interface Response for Bracket Tests

4. DISCUSSION OF RESULTS

4.1 Bridge Load Tests

The neutral axis locations in the girders were found to be above the girder mid-depth in all cases, but generally lower than expected for full composite action, indicating that a variable degree of composite action was occurring. Girders that were directly loaded by a wheel line appeared to behave more compositely than those distant from a wheel line. Similarly, outer girders, which support more dead load than inner girders (due to kerbs, barriers and a footpath), appeared to behave more compositely than inner girders. The most plausible explanation for this is that these girders were subjected to a higher interface normal force, enabling the rivet heads to transmit higher interface shear flows. However, the fact that the degree of composite action was variable indicated that it could not be relied upon when estimating the ultimate limit state girder moment capacities.

Extremely low values of slip and separation were measured – the maximum slip was 0.02 mm while the maximum separation was 0.01 mm. This seems to conflict with the finding of partial composite action, where larger values were expected. Full composite action would be expected to produce values of this very low magnitude.

By comparing the crawl speed and "normal" speed traverse results of the test vehicle, it was possible to determine the dynamic load allowance (DLA) caused by the test vehicle for one lane loaded. The maximum DLA was calculated by this method to be 0.265, which compares favourably with a "rule of thumb" method to estimate the first mode natural frequency of the bridge as 120/span (m) leading to a DLA of 0.276 (6). The maximum DLA occurred in girders remote from the load application, while the most heavily loaded girder had a DLA of only 0.05. The load test did not measure the interaction of the DLA effects of two lanes loaded, which can lead to an overall reduction in the net DLA.

Van Ek Contracting created a computer model which was calibrated against the results of the load test so that the live load distribution and cross sectional strain profiles matched the test results as closely as possible (to within 3.4%). This was an iterative process of modifying flexural rigidities (EI) and boundary conditions. The modelling was complicated by the fact that the bridge was partially continuous at one pier where the webs of abutting beams were connected by riveted web angles, but the flanges were discontinuous. The final calibrated model resulted in beam section properties that were higher for the outer girders than the inner girders, were higher than the theoretical non-composite section properties, but still well below the theoretical fully composite section properties. This showed that the bridge was behaving partially compositely at the elastic serviceability condition, but there was no evidence to confidently show that the same degree of composite behaviour could be relied upon at the ultimate limit state.

Laboratory testing was used to verify the serviceability behaviour, to determine the ultimate limit state capacity and to conceive and test a strengthening method if required.

4.2 Laboratory Load Tests

The spring loaded tests confirmed the presence of longitudinal shear forces along the concrete-girder interface that results in the composite action observed in the load tests of the Willaston Bridge. However, the tests also demonstrated that shear resistance decreases rapidly once the peak shear force is attained at a very small slip. Hence, a second series of tests was developed to investigate strengthening alternatives to increase the longitudinal shear strength and improve the ductility of the interface so that a reliable moment capacity could be determined using standard partial interaction theory. A steel angle bracket bolted to the interface was shown to be effective in limiting the separation of the interface resulting in an increased longitudinal shear capacity in addition to improving the ductility of the interface. Since the preferred failure mode is anchor bolt shear rather than concrete splitting (Figure 12), a bolt edge clearance in excess of 125 mm was recommended for the strengthening design.

5. BRIDGE STRENGTHENING

A repair method was conceived, the principle of which was to prevent separation between the concrete deck and the wrought iron girders. This would enable the top flange rivet heads to transfer interface shear flow, realising their full potential as shear connectors. The method involved the attachment of "passive" clamps to the concrete deck haunch sides, consisting of mild steel flats bent to an angle of 90^0 so that they hook under the top flanges (Figure 14). The clamps do not take load until the interface attempts to separate, generating sufficient normal force so that the interface friction force exceeds the theoretical shear flow. In some respects this effect is analogous to the concept of "shear friction" (7).

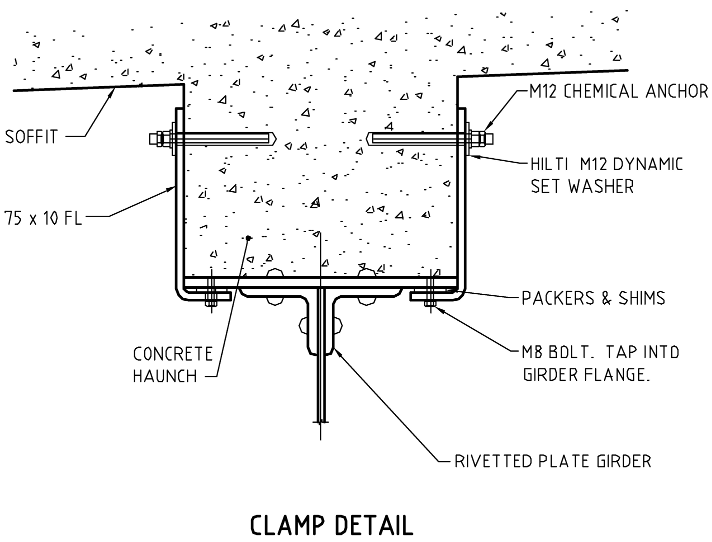

Figure 14: Bridge Strengthening

Slip between the concrete anchor bolt and the clamp body is prevented by using *Hilti Dynamic-Set* washers, which enable the annular space between the bolt shank and the plate to be filled with epoxy. If slip were permitted to occur, then the clamp effectiveness would be compromised. The foot of the clamp is attached to the top flange by M8 bolts to prevent the foot tending to move outwards as the clamp takes load.

The spacing of the anchors was designed as follows: The clamps were designed for the ultimate limit state (ULS). The required ULS moment capacity of the worst loaded girder was determined. The required interface frictional force required to develop this moment capacity was determined (5). This force was converted to an equivalent shear force per rivet by dividing it by the number of rivets between the beam end and the critical section (midspan). The clamp spacing was then determined by proportion with the test results, knowing that the test specimens represented a clamp spacing of 406 mm. The assumption in this approach of load sharing between all the active rivets is valid because the clamped tests showed that large slips were measured while the rivets were still able to sustain the interface shear force – i.e. a long ductile plateau was exhibited. The actual clamp spacing adopted was chosen to fit into the panel lengths between vertical web stiffeners.

The strengthening works can be performed entirely under the bridge without disruption to traffic. There was no concern for the effect of traffic vibrations on the setting epoxy in the dynamic set washers because, at service loads, the bridge load tests showed slip and separations that were barely measurable. The clamps do not become active until the ultimate limit state is approached.

6. **CONCLUSIONS**

Bridge load tests have shown that a wrought iron girder and reinforced concrete deck bridge behaves with partial composite action at service loads by virtue of the top flange rivet heads embedded in the deck concrete. The rivet heads are able to carry interface shear flow, but the degree is variable and load dependent. The service load test results cannot be extrapolated to the ultimate limit state, so that the bridge strength cannot be estimated with confidence.

The laboratory tests on representative specimens that were spring loaded and unclamped have confirmed this behaviour, showing that slip and separation at the concrete-girder interface, when mobilised, causes a rapid loss of strength with increasing slip. Additional tests on clamped bracket specimens have shown that brackets reduce slip and separation so that the load-slip characteristics of the interface exhibit a ductile plateau similar to that of a typical stud shear connector. This means a reliable ultimate strength can be achieved, much higher than without brackets. The addition of brackets enables a controllable degree of partial interaction to be engineered so that a reliable ultimate moment capacity can be achieved.

A method of clamping the deck slab and wrought iron girders together was designed and tested in the laboratory to determine its effectiveness. The results of the testing allowed a clamping design to be prepared for the Willaston Bridge. The strengthening works can be entirely carried out beneath the bridge without disruption to traffic.

7. REFERENCES

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