Steel Innovations Conference 2013
Christchurch, New Zealand
21-22 February 2013

KAITUNA RIVER BRIDGE – TAURANGA EASTERN LINK – COMPOSITE STEEL BRIDGE DESIGN CASE STUDY

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Abstract

The Kaituna River Bridge is a 190m long, 27m wide, four span continuous composite steel bridge currently being constructed as part of the Tauranga Eastern Link project. This paper discusses the design challenges and innovation involved to produce economic and practical design solutions. The seismicity within the Tauranga area coupled with poor ground conditions required innovative design of the lateral load resisting system to transfer the seismic loads efficiently. Design of the superstructure to satisfy the recommended fatigue loading and the added effects on fatigue due to 40T Articulated Dump Trucks posed design challenges.

The superstructure design included: the utilisation of slender beams, which maximised the steel weight to strength ratio; friction grip steel bolted connections, designed to provide no slip at the serviceability limit state; and the high slip co-efficient of the corrosion protection system was beneficial and allowed a reduction in the number of TF bolts required in the main girder splices. During tendering and option development phase of the bridge design, various options were considered and a steel bridge was the most favourable and economic option. Fabrication and construction of the steel superstructure is currently underway and the construction methodology is presented and discussed.

Introduction

The Kaituna River Bridge is a composite steel ladder type bridge currently under construction as part of the Tauranga Eastern Link (TEL) motorway located south east of Tauranga. The TEL project is being constructed by the Fulton Hogan Heb Construction Alliance with URS New Zealand as the lead design consultant, together with Opus International Consultants, Peters and Cheung, and Bartley consultants. The Kaituna River Bridge carries 4 lanes of traffic, spans over a local road and provides an essential crossing over the Kaituna River. The bridge is the longest continuous steel ladder girder bridge in New Zealand, with an overall length of 190m. It is a unique type of ladder bridge as it consists of four main girders with three cross girders between the main girders. Typical ladder bridges consist of two main girders and a single cross girder.

This paper looks into: the design development of the Kaituna River Bridge, focusing on the design challenges and innovations; the final design; and the construction methodology of the steel superstructure.

Design Development

Specimen Design

The need for a composite steel girder bridge was realized at the specimen design stage, as a minimum span over the river of 55m was required by the Principals Requirements due to supporting foundations within the river being excluded by the resource consent, as the river is widely used by the public for recreational activities. This excluded looking at the typical precast concrete beam type bridges as they can only generally span up to a maximum of 34 meters. The specimen design for the Kaituna River Bridge consisted of a 3 span continuous composite steel girder superstructure with seven 2300mm deep uniform depth steel I-

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girders. The three span bridge spanned over the river and the two adjacent stop banks. An additional overpass structure was to be built for the local road, Bell Road, to be taken underneath the TEL on the west side of the river.

Tender Design – Option development

At the tender design stage, it was decided to re-design the specimen design into a larger 4 span bridge which spanned over the local road and the river and avoided the need for a separate overpass. The versatility of a steel bridge allowed for the 3 span continuous bridge to be lengthened into a longer 4 span bridge which spanned over the local road. The vertical height clearance for the local road was able to be achieved by changing the depth and flange width of the steel girders. Another driving factor for the 4 span bridge option was very poor ground conditions at the site, and it was found to be favorable to remove the embankment between the three span bridge and local road overpass. The outcome of removing the earth embankment, removed a pre-load soil surcharge requirement. It also resulted in less seismic movement of the surroundings soils and less seismic imposed movement of the soil onto the structure.

The depth of the steel girders can be changed near the pier to provide a haunch. The beam was chosen to be haunched at the pier regions where high hogging bending moments occur. Haunching the main girders provides a deeper beam depth and a stronger section of beam at the haunched location, allowing the beam depth over the span section to be shallower. The four span haunched structure opens up the area beneath the bridge which makes it more visually appealing for the local road users and users of the river.

Various 4 span bridge options were considered during the tender design. The main factor considered was the superstructure steel weight per m^2 of bridge deck area, as a kg/m^2 value. From previous experience an optimum weight for the bridge of this span would be under $220kg/m^2$. The options below were sufficiently designed to enable a superstructure steel weight to be estimated:

7 or 8 girders option

A multi-girder (7-8 girders) option was considered with main girders spaced at approximately 3.5m. No cross girders were required as the deck slab can span up to 4m between the main girders. This option was the heaviest in steel weight and raised difficulties in providing maintenance and inspection access to every girder, which was a Principal Requirement.

2 girder ladder deck option

A typical twin girder ladder deck option was considered with a 21m long cross girder. The weight of steel was $215kg/m^2$, and had advantages in terms of less pieces of steel to splice together and potentially less physical construction time. However these advantages were not enough to outweigh the disadvantages, namely: the size of the flange plates required for the main girders was 140mm thick for the pier sections; and the weight of the sections required would have been physically challenging to handle and assemble onsite, large cranes would be required to deal with this.

4 girder ladder deck option

A four girder ladder deck option with three cross girders spanning in-between the main girders was also considered. The main girders were spaced at approximately 8m. This option had a steel weight of $200kg/m^2$ and the pieces of steel and plate thicknesses were within typical fabrication capabilities. This was the most economical superstructure layout, and hence was submitted in the tender package.

A concrete box girder option was considered but this had a high initial cost that was not recoverable from the benefits offered by a concrete box girder.

Final Structure Details

Post tender award, the 4 girder ladder deck option was confirmed to be the final layout for the steel superstructure. The 4 girder ladder deck varies from the typical ladder girder bridge, which consists of two main girders with cross girders in-between. The Kaituna River Bridge consists of 4 main girders with three cross girders in-between each main girder. This arrangement gave the least steel superstructure weight when compared to the other options mentioned in the section above. In the detailed design period the span lengths were altered and positions of piers and abutments moved, along with the realignment of the local road to come up with the most economical and complete solution. The final center to center spans lengths are 39.1m, 52.7m, 55m and 41.5m. The deck slab cantilevers 1m at each end of the bridge to support the bridge expansion joints, giving a continuous steel beam length of 190m. The bridge is skewed at piers C, D and abutment E to accommodate the natural flow direction of the river and stop-bank alignment on the eastern side. Figure 1 below shows the final general arrangement as a plan and elevation view of the bridge.

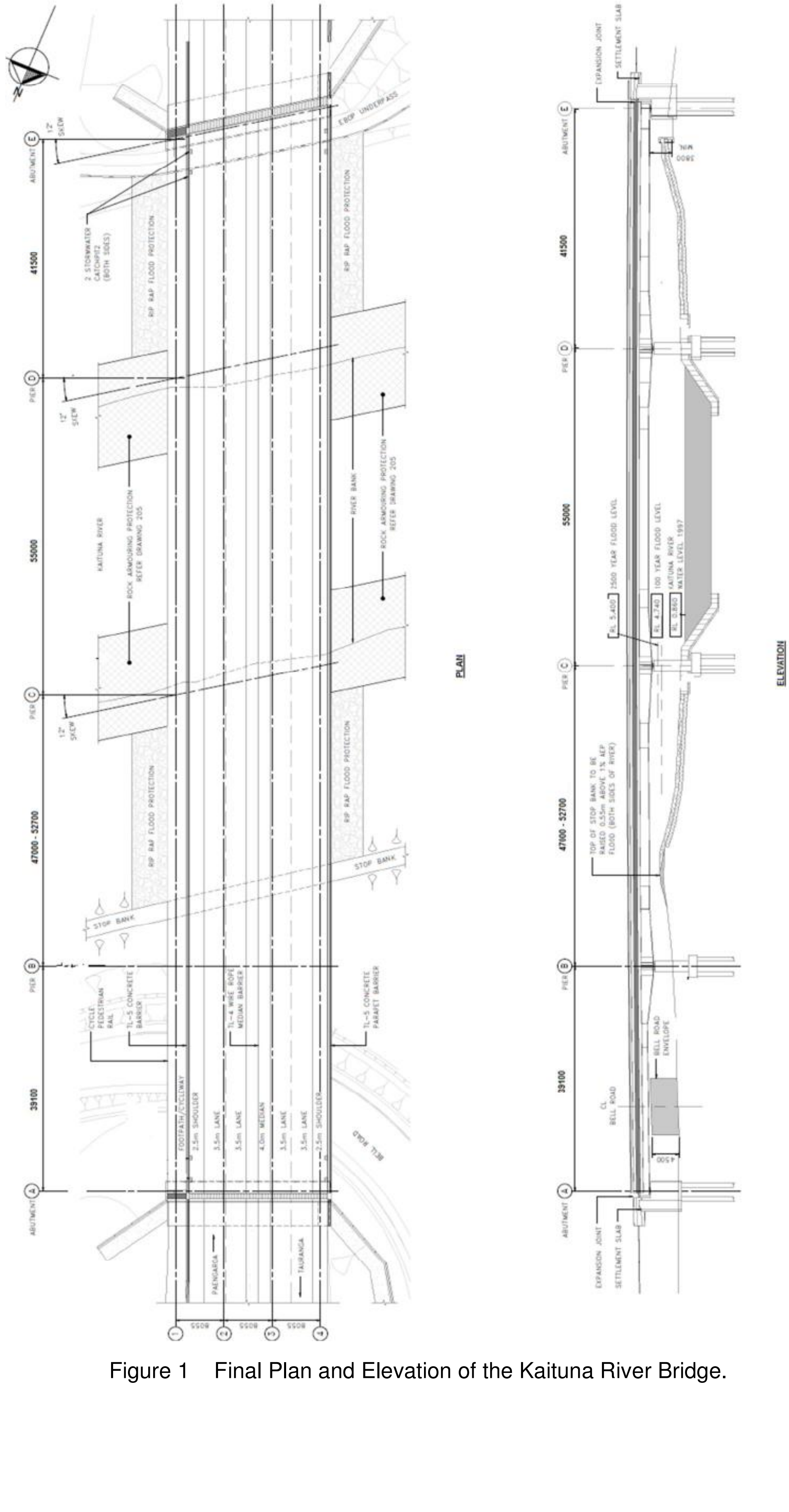

Figure 1 Final Plan and Elevation of the Kaituna River Bridge.

The local road which passes underneath the span between Abutment A and Pier B requires a minimum vertical clearance envelope of 4.50m. The main girder depth is approximately 1550mm deep over the road, compared to a deeper slender beam depth of 2050mm over the river and stop bank spans. The haunched main girder depth is approximately 2600mm deep at the pier sections.

Detailed Design

General

The load path for vertical and horizontal loading comprises a steel superstructure transferring the loads through pot and elastomeric bearings into the substructure. The substructure comprises reinforced concrete columns and pile caps at the piers, and reinforced concrete abutment beams which are separated from the earth embankment. The pier pile caps and abutment beams are founded on 50m long top driven concrete filled steel tubes, 710mm diameter, 12mm wall thickness. 68 No. piles in total were required for the whole bridge.

Both vertical and horizontal load combinations for the design of the bridge were in accordance with the load combinations defined in the NZTA Bridge Manual (NZTA 2003). The steel plate grade used for the fabrication of the steel girders was grade 350L0 in accordance with Steel Structures Standard (NZS 3404.1:2009). The steel grade is required to be ‘L0’ due to conditions of the site and based on the in-service temperatures that the steel is subjected to throughout the design life.

Main Girder Design

Design of the main girders and cross girders was in accordance with AS5100.6 which provides design methods for slender beam webs and flanges, outside the range of those covered by NZS 3404.(NZS 3404:1997) Slender beams can provide more efficient steel weight to strength ratios for typical bridge beams. The steel superstructure is designed to remain elastic in all load cases.

An example of the slender beam design is for the main girder mid-span section. The section consisted of a of 2000mm deep web, with a thickness of 20mm, and a top & bottom flange width of 750mm, with a thickness of 25mm. Equations 1 and 2 below from AS5100.6 give the slenderness values for the web and flange of this particular beam.

$$Web\ slenderness\ = \frac{d}{t_w} \times \sqrt{\frac{f_y}{250}} = \frac{2000}{20} \times \sqrt{\frac{350}{250}} = 118 \quad (1)$$

$$Flange\ slenderness\ = \frac{d}{t_w} \times \sqrt{\frac{f_y}{250}} = \frac{375}{25} \times \sqrt{\frac{350}{250}} = 17.7 \quad (2)$$

The web and flanges slenderness values for the example beam above are greater than the yield slenderness limit specified in AS5100.6, Table 5.1, which categorises them in the non-compact range. (AS5100.6 2004)

Figure 2 below is taken from Figure 5.1.4 from AS5100.6 (AS5100.6 2004) and shows the area which is not considered to be effective when the beam section is determined to be non-compact. AS5100.6 provides the method on how to determine the effective section properties for a non-compact beam. The method from AS5100.6 reduces the effective area of cross section to be considered in the strength calculation of the beam by omitting an area of steel in the beam cross section. This produces an efficient design when the beam depths and plate thicknesses can be adjusted, allowing the optimum weight to strength ratio to be found.

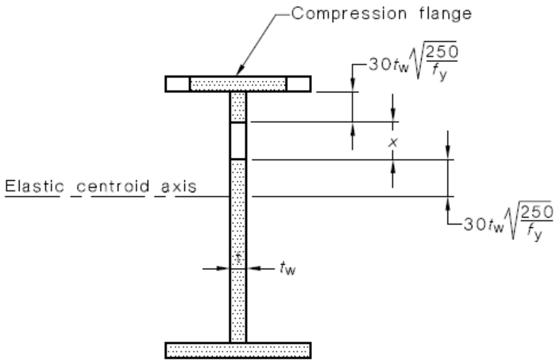

Figure 2 AS5100.6:2004 - Figure 5.1.4, Reproduced with permission from SAI Global Ltd under Licence1302-c055.

U-frame Action

Mobilising U-frame action between the main girders and cross girder provides buckling stability to the main girders without the need for cross bracing. AS5100.6 gives design provisions for U-frame action when a cross-girder (transom) is connected to two girders with full length stiffeners. This allows for a smaller effective length to be used in the design. The nature of U-frame action provides greater rotational restraint to the girders and suppresses certain types of buckling modes from occurring. (AS5100.6 2004)

There are three typical load states to be considered for design of a composite steel-concrete beam. These are:

- Steel only load case (steel alone resists all loads)
- Long term loading (composite section resists long term loads)
- Short term loading (composite section resists short term loads, e.g. live loading)

The u-frame action provided by the cross girders and stiffeners arrangement reduces the effective length, L_e, used for determining the bending strength of the beam, of the main girder in the sagging regions (mid spans) for the steel only load case. The u-frame action also reduces the effective length of the main girder in the hogging regions (piers) for all three load states. Once the concrete deck is cast, it restrains the main girder from buckling and provides a rigid diaphragm like restraint to the top flange of all the steel girders.

Corrosion Protection and High Strength Friction Grip Bolts

Corrosion protection of the steel superstructure is critical for the bridge to achieve its required design life. The atmospheric corrosivity category for the Kaituna River Bridge is determined to be 'Class D' in accordance with the New Zealand Steelwork Corrosion and Coatings Guide (El Serraf and Clifton 2011). The Principal's Requirement is for '25+ years to first major maintenance'. An Inorganic Zinc Silicate (IZS) coating system was chosen, as it was the most cost effective and met the requirements of both AS/NZS2312 (AS/NZS 2312:2002) and the New Zealand Steelwork Corrosion and Coatings Guide. Both documents give a 25+ years design life for an IZS3 coating with a DFT of 125µm.

The main girder splices on the bridge are high strength friction grip bolted connections. This type of connection is designed to ensure no slip between the splice plates for serviceability limit state loading on the bridge. The type of coating applied to the steel will greatly alter the friction coefficient or slip factor (μ_s). A typical friction slip coefficient of 0.35 is normally specified for clean "as-rolled" surfaces when designing to NZS 3404.1 (NZS 3404.1 1997).

When a coating is applied to steel, NZS 3404 (NZS 3404.1 1997) states that the "slip factor shall be based up test evidence". Zinc silicate primers typically produce slip factors in the range of 0.38 to 0.65 (Mandeno 2005). The paint manufacturer supplied test evidence that a minimum slip factor of 0.50 was able to be achieved. As a result of higher than normal slip factor, the number of bolts used for the friction grip bolted connections was considerably less than if the 0.35 slip factor value was used in the design. A reduction in the number of bolts was able to be made because of the higher slip factor, which provided time and cost savings. As a result of less bolts, the splice plates were also smaller in size.

All bolts used for main girder splices were Hot Dip Galvanized (HDG) M30 Grade 8.8 bolts, the HDG coating is not sufficient for a 25+ year to first major maintenance design life and hence the bolts require an additional coating system over top of the galvanizing to give the required life to first maintenance. A simple comparative test was undertaken on the overcoat system to be used for the bolts. The test looked at: the surface preparation required for the overcoat; ease of onsite application, comparing brushing and spraying; and surface finish. The final chosen paint product of inorganic zinc silicate (IZS3SB) was the most time and cost efficient when sprayed on and a brush finish was deemed as to be a less effective means of application, with 'mud-cracking' more likely to occur. Two other zinc primer products, which were assessed to meet the life to first maintenance requirements, were compared with the chosen paint product, but were found to be poorer overcoat systems to apply to the bolts once installed (El Sarraf and Mandeno 2012).

Articualted Dump Truck Loading and Fatigue Design

The Kaituna River Bridge is on the critical path for construction of the TEL, and hence early construction completion allows for Articulated Dump Trucks (ADTs) to cross the Kaituna River Bridge, transporting material from one side of the river to the other saving a 45min trip via other roads. 40 Tonne ADTs were considered for the design, which have axle weights as below, and a gross weight of 648kN, or 66 Tonne.

- Front axle – 190kN empty, 190kN loaded
- Rear front – 67kN empty, 229kN loaded
- Rear back – 67kN empty, 229kN loaded

The ADTs were considered as an Overload Case when considering load combinations in accordance with the TNZBM.

Fatigue Design

The superstructure steelwork is required to cope with the effects of fatigue in accordance with AS5100.6 as modified by HERA document "Recommended Draft Fatigue Design Criteria for Bridges Version 3" dated 26th July 2007 (Clifton 2007) The average daily traffic volume for the Kaituna River Bridge is 17920 vehicles (includes both directions) with 16% HCV, based on an estimated traffic flow for 2016.

The stress in the steel is designed to a specific stress limit in accordance with NZS3404 (NZS 3404:1997), which is based on the number of cycles determined from the "Recommended Draft Fatigue Design Criteria" document as above. If the stress caused by the fatigue loading is lower than the limit required, then the fatigue check is considered to be satisfactory. However, fatigue damage caused by the ADTs is cumulative to the number of cycles specified in the HERA document. The method below was used to evaluate the effect of the ADTs on the fatigue design.

The effect of fatigue due to the articulated dump trucks running across the bridge is calculated based on the number of trucks crossing per day over the period it will be used as a haul road. The number of fatigue cycles assumed is 548,000, which is a conservative estimate based on the number of trips across the bridge. To determine the effect of the ADT trucks on the bridge fatigue life, the following steps were taken;

- Divide the 'number of actual cycles from the ADT truck' with the 'allowable number of cycles correlating to the stress in the element considered'.
- Divide the 'number of stress cycles for the 100year design (standard fatigue loading)' with the 'number of allowable cycles at that stress range'.
- The sum of both fractions must be equal to or less than 1.

This method was accepted by the external Peer Reviewer and Client.

Seismic Design

The structure is relatively stiff as the supporting columns and abutments are all less than 6m high. The short natural period of the bridge coupled with the poor ground conditions meant that quite a high coefficient of acceleration of 0.80 was designed for in the 1 in 2500 year design seismic event.

Liquefaction induced settlement of the surrounding ground is estimated to be between 200-400mm in a 1 in 2500 year seismic event. A continuous bridge superstructure cannot accommodate this amount of vertical settlement without distress and hence the substructure is required to be supported on piles, founded 50m below the existing ground level where a dense sand material lies.

The steel girders are seated on Laminated Elastomeric Bearings at the Pier locations and free float pot bearings at the abutments. In the longitudinal direction the seismic induced loads are resisted by the three piers by flexural actions of column and substructure. The bearings at the abutments allow the superstructure to slide backwards and forwards resulting in no significant load in the longitudinal direction being taken by the abutments. The abutment pot bearings are designed to accommodate a superstructure displacement of up to +/-400mm. In the transverse direction the abutments also contribute to resisting the lateral loads as well the piers.

The cross girders along the length of the bridge are spaced at approximately 4m centers, however, the supports at piers C and D are at a 12degree skew relative to the bridge superstructure. This caused offsets between the lateral load resisting steel frame system and the supports. Figure 3 below indicates the 850mm offset of the center of the support and the center of the steel frame diaphragm, which utilises a cross girder and angle section to create the frame system. The offset lateral load resisting system was designed to be offset to the supports, to enable the cross girders to remain at even spacing's. 8 cross girders were able to be removed from the steel layout by offsetting the frame system relative to the support system, and by maximising the spacing of the cross-girders. The bottom flanges of the steel girders are designed to transfer the lateral loads from the steel frame system (superstructure) to the 250diameter solid steel shear keys through local bending of the bottom flange plates. The load in the main girders due to the eccentricity in load path is combined with the other concurrent loads in a seismic event and the steel girder was designed for combined bi-axial bending.

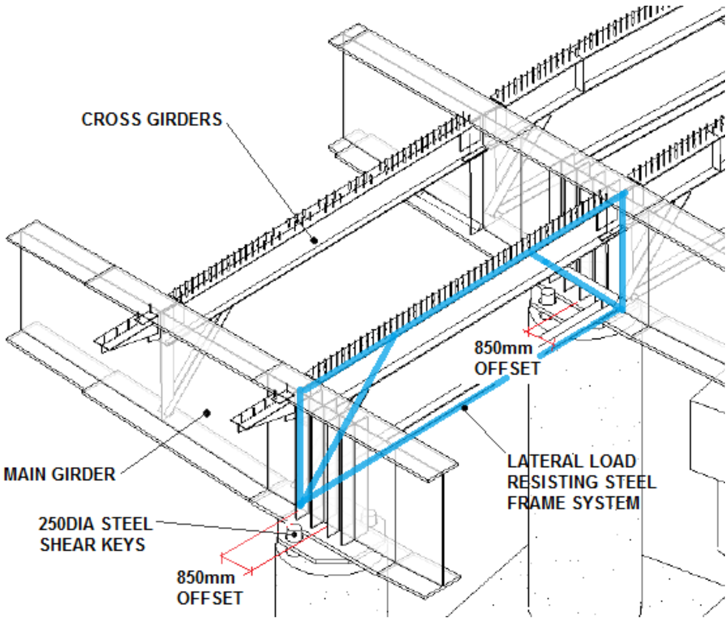

Figure 3 Pier diaphragm detail.

The seismic shears at the abutments are too high for a pot bearing to resist effectively, a special detail has been designed to allow for longitudinal sliding of the superstructure at the abutments, while at the same time transferring lateral seismic loads into the reinforced concrete abutment beam via concrete shear keys on the inner two main girders. The special detail is shown below in Figure 4. It consists of an arrangement of welded plates designed to take its attributed portion of the Ultimate Limit State (ULS) seismic load from the steel girders and transfer it through a thin elastomeric bearing, which slides back and forth on a stainless steel plate. The benefit of this connection is that it allows for simple replacement of the sliding bearing pad in case they are damaged in a greater than ULS earthquake.

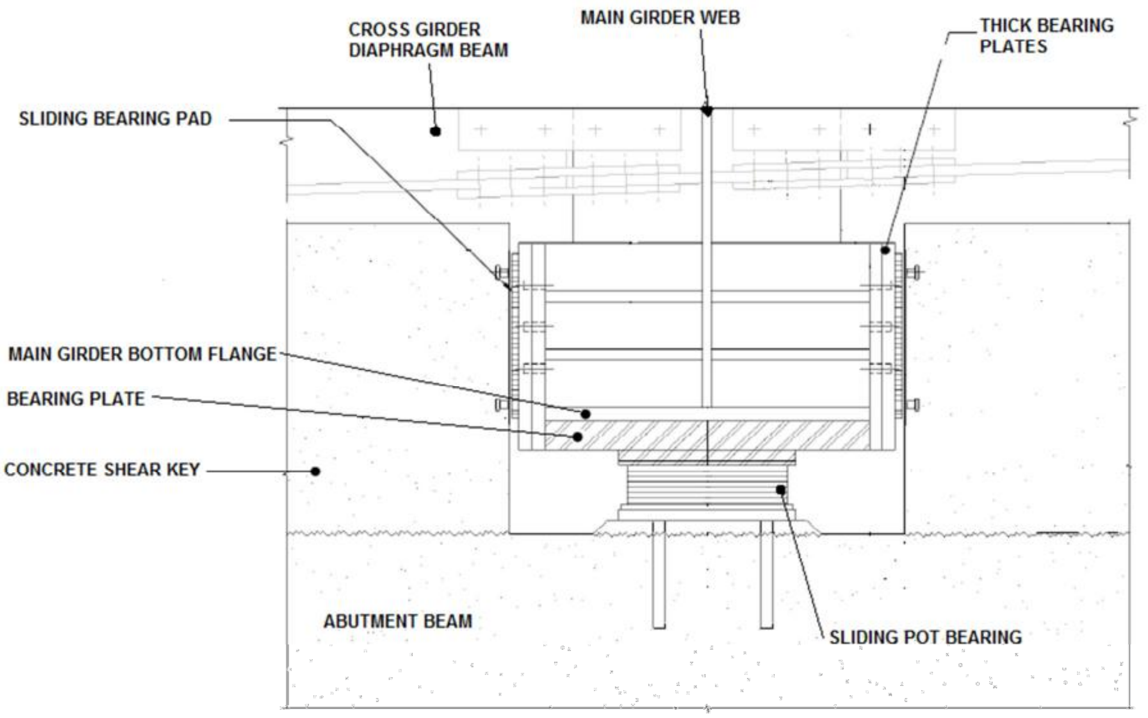

Figure 4 Abutment sliding shear key detail.

Ground Improvements were undertaken at the abutment and pier locations to restrict lateral spreading of the ground under a seismic event down to approximately 23m. Under a ULS seismic event the ground is expected to permanently displace in a direction towards the river. To design for this displacement slotted shear keys have been adopted at the piers to allow the columns, pile cap and piles to move up to a limited displacement freely relative to the superstructure.

A steel superstructure is typically advantageous over a reinforced concrete superstructure for seismic design as the total superstructure weight is typically less than an equivalent concrete superstructure. A benefit of having a lighter superstructure means the foundations are also usually smaller, and the seismic design of the substructure is more efficient with a smaller seismic weight, providing material savings in the substructure.

Steel Construction Methodology

Bridge Construction commenced with steel work fabrication in mid 2012. Approximately 970T of structural steelwork has been fabricated and painted by Eastbridge in Napier, New Zealand.
The construction methodology of the bridge is simple and requires relatively small cranes which are economic to run.

The design was done considering the construction methodology through consultation with the contractor.
Six main girder splices were designed at the points of contraflexure in the main girders for the final design condition, this is the point on the beams with the least amount of bending moment, and hence the most favorable location for a splice. This allowed the splice connections to be small.

The main girders were fabricated into 6 lengths of approximately 30m each, with each section being lifted separately. This meant 24 beam lifts that weigh 30T or less. Due to the river constraint, the river span section and pier section are spliced together on the ground and lifted in as a single beam, approximately 60m long and weighing 60T. Figure 5 below shows one of the 30m long haunched main girder sections being lifted as a single beam. A 150 tonne crane is used for the lifts of the single piece beams. The river span section requires a 250 tonne crane to lift the 60m spliced section, as shown in Figure 6 below.

Figure 5. Lifting of main girder sections – 30m section.

Figure 6. Lifting of main girder sections – 60m long spliced section.

Conclusions

The use of steel-concrete composite construction provided an optimal solution for the Kaituna River Bridge for the Tauranga Eastern Link Project and the following conclusions can be made about the design, innovation and efficiency of the final design:

1. Steel-concrete composite construction allowed for a simpler open approach to the local road and river crossing removing the need for a heavy sand embankment between the river bridge and local road overpass.
2. The unique four girder ladder deck was the most efficient in terms of steel weight and estimated constructability.
3. The fatigue life of the structure including the effect of Articulated Dump Trucks used during construction was assessed with a new method using linear interaction and found to be satisfactory when designing for the requirements of AS5100.6.
4. Slender beams and haunched main girders optimized the efficiency of the steel structure with a superstructure steel weight of 200kg/m^2 of deck area.
5. A slip factor of 0.5 was used in the design of the main girder splices, which provided material, time and cost savings for the splices.
6. Efficient layout of the cross girders and by designing the lateral load resisting system to be offset from the supports allowed for a reduced amount of steel required for the pier diaphragms.
7. The beam sections were lifted in individually and spliced in the air or on support towers, allowing the majority of the lifts to be done using a relatively small 150T crane.

Acknowledgments

The authors would like to acknowledge the following companies: Fulton Hogan Heb Construction Alliance; NZTA; Eastbridge Ltd; Napier Sandblasting Co Ltd; URS New Zealand Ltd; Opus International Consultants Ltd, Peters and Cheung Ltd, Bartley Consultants Ltd.

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