

WESTGATE FOOTBRIDGE, AUCKLAND

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ABSTRACT

The Westgate Footbridge provides a new pedestrian and cyclist connection across four carriageways of SH16 near the Westgate shopping centre in Auckland. The bridge is 3m wide and 220m long and consists of two separate structures. A curved steel truss under the deck provides a 120m long ramp which allows disabled access gradients to rise 8m over the motorway. A straight 100m long through truss crosses the motorway with 6 spans and minimal structural depth.

The paper presents the background to the project, the use of steel and the key issues associated with the design and construction of the project.

Background

The Westgate Footbridge is a joint NZTA and Auckland Transport initiative to provide a new crossing of the motorway to connect the communities Massey, West Harbour and the shopping center. It provides pedestrian and cyclist access between Oreil Avenue and Westgate Drive. The footbridge addresses a key safety issue of pedestrians crossing the SH16 carriageway in the vicinity of Westgate Wetlands and Lendich Reserve. Illegal crossings of the motorway either at grade or using a storm water culvert nearby have been noted. CCTV footage was used to monitor pedestrian movement across SH16 and found that in one particular Saturday 40+ people crossed the motorway. In 2004 a pedestrian was killed while crossing SH16.

In 2010, New Zealand Transport Agency (NZTA) and Auckland Transport agreed funding to deliver the project as part of a design and construct contract. Construction commenced in March 2012 with the completed bridge opened on Thursday 31st January 2013.

This paper discusses the project requirements, development of bridge concepts and design and construction of the bridge.

Project requirements

Site description

An aerial photograph of the site location is given in Fig. 1. The proposed bridge was required to be within the current SH16/18 designation and council park land, that is Westgate Wetlands (to the right of the figure) and Lendich Reserve (to the left of the figure).

Westgate Wetlands is a stormwater storage facility which also has a walking track and rest stops around the perimeter. The Wetlands are landscaped with a bark/mulch base and small scrubs and brushes throughout. The walking track is accessed from Westgate Drive which leads directly into the Westgate Shopping Centre. On the southern boundary there is an existing bank. The proposed footbridge would need to connect to Westgate Drive.

At the bridge location the motorway consists of 4 separate carriageways, SH18 Eastbound, SH16 Northbound, SH16 Southbound and SH16 Southbound on-ramp. The provision for a future cycleway connection alongside the SH16 Westbound carriageway is also allowed for. Vehicle clearance meets the requirements of the TNZBM with minimum vertical clearance of 6.2m and a minimum rollover horizontal

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clearance requirement of 650mm to structures located behind barriers.

Lendich Reserve is located approximately 6m below the SH16 carriageway. The reserve consisted of large pine trees with dense over growth surrounding a steep bank down to a creek. There are a number of houses located around the perimeter of the reserve. Fig. 1 shows the park after the site was cleared of large tree for construction.

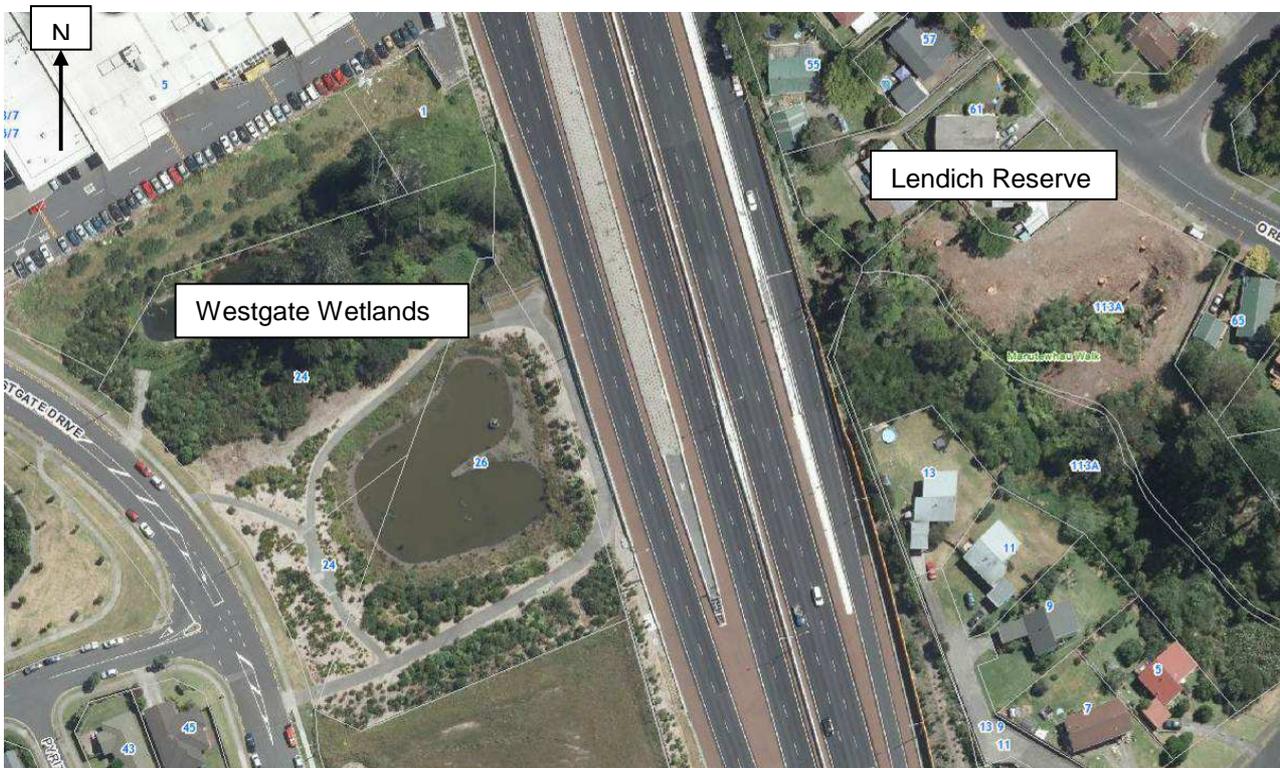


Figure 1. Site location – Source Auckland Council Maps.

Bridge requirements and constraints

The bridge was to meet the following requirements and constraints;

- Provide access across SH16 from Oriel Avenue to Westgate Drive
- 3m wide footpath for combined pedestrian and cyclist traffic
- Vertical gradients to comply with New Zealand Building code
- Vertical and horizontal clearance requirements for the bridge structure within the carriageway
- Steep banks and creek in Lendich Reserve which is 6m below the carriageway
- Bridge alignment should minimise views to housing around the perimeter of the reserve
- Constructability, difficult access with the carriageway and step terrain in the reserve

Design solutions

Concept design

A number of concepts were considered to address the constraints noted above. To achieve a cost effective solution the bridge needed to be as short as possible. On the western approach (Westgate wetlands) an existing high point was identified as a natural feature for the bridge to land. On the eastern approach (Lendich reserve) a curved alignment was needed to provide sufficient length to reach existing ground using access ramp gradient requirements, avoid the steep banks and creek, and maximise the sightlines to the existing houses around the reserve perimeter.

A structural form with minimal depth over the carriageway was desired so that bridge length was kept to a minimum. A through truss or arch structure was considered to fit this criteria and this form would be best suited to a straight alignment across the carriageway. The curved alignment on the eastern approach suited a structural form that allowed unobstructed views of the reserve, resist torsional loads and able to be constructed with limited access.

Large span options across the carriageway were considered during the concept design phase. The concept of a transitioned top and bottom chord was developed with the purpose of minimising structural depth over

the carriageway (through structure) and having unobstructed views over the reserve (structure underneath), see Fig 2a. However, these large span concepts were considered to be too difficult from a constructability perspective and where also a bold statement visually that was not suited to the site. It was decided that piers should be placed between each carriageway to limit the span lengths and weights for construction, see Fig 2b. The transitioned top and bottom chords were simplified into two separate bridges.

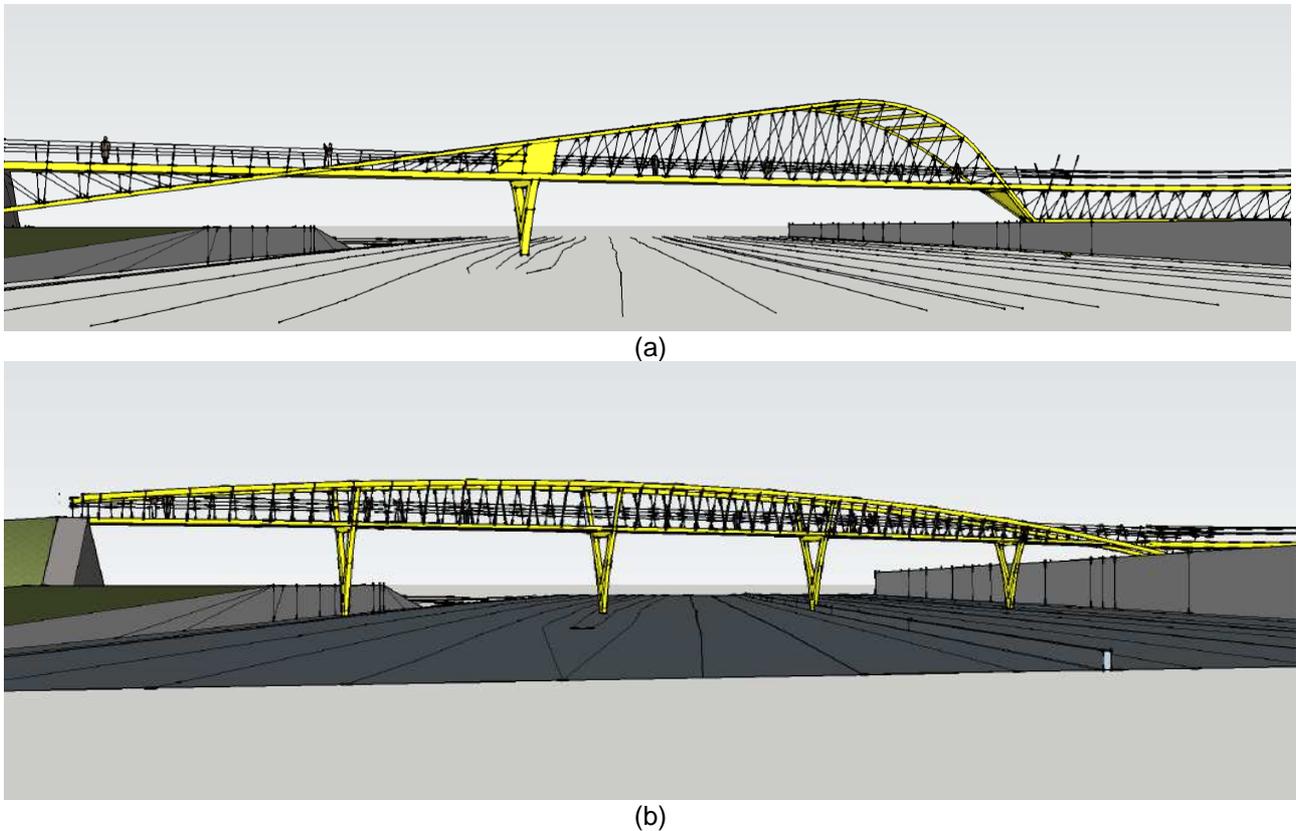


Figure 2. Early concepts across the carriageway (a) large span arch (b) truss with spans between carriageways.

Preferred concept

The preferred concept was chosen for detailed design. The bridge consists of three bridges which are; span 1 approach, bridge 1 over the carriageway and bridge 2 over the Lendich Reserve. The key features of the design are listed below and discussed in further detail in the following sections;

- Through truss with sloping webs and variable depth across SH16, approximately 100m long
- Underslung curved truss over the reserve, approximately 120m long
- Twisted twin curved column piers in the carriageway
- Steel CHS sections used for trusses and piers
- Throw screens
- Handrail with LED Puck lighting

A prestressed concrete double tee 10m span was used on the western approach (Span 1) as landing the truss which was approximately 3.5m high over shallow ground would be visually imposing.

The chosen concept across the carriageway is shown in Fig 3a and b. The concept has the minimum structural depth at the central pier across the carriageway and the bottom chord follows the vertical alignment requirements to achieve both vertical clearance and gradients. The spans vary in length between approximately 16 – 20m. The top chord forms a straight line between the central pier and is at a gradient to achieve equal heights at the ends of the truss. The shape creates an optical illusion of a sagging top chord. For ease of construction the spans were simply supported, however, to achieve the visual effect in the shape a continuous chord was required. To respect the form of structural action, an in-fill piece distinct from the top chord was used between the spans, see Fig 3b. To keep the sense of space across the footbridge the truss diagonals were sloped at 1:5 outwards, see Fig 3c, initially a slope of 1:3 was considered. This also creates a visual feature with the top chord moving inwards/outwards with the dimension between the top chord

decreasing and increasing. Late in the detailed design phase the addition of throw screens over the carriageway was requested. After investigating several options a concept was developed that accentuated the shape of the truss and the sloping sides, see Fig 3c. The throw screen mesh was made from stainless steel wires. The alignment of the bridge to the carriageway was skewed and to keep the ends of the truss square a skewed support was required. A twisted V-pier was developed and was considered to be an interesting feature of the bridge that related to its alignment, see Fig 3b.

For the spans across the reserve a curved underslung truss was adopted with spans of between 20 to 23m. The underslung truss allowed unobstructed views of the reserve and better sightlines for the safety of users (speeding cyclists and line of sight). A truss was adopted to keep the structural form the same. Planting provided around the reserve will provide a 'tree top' walk in future years once the trees have established.

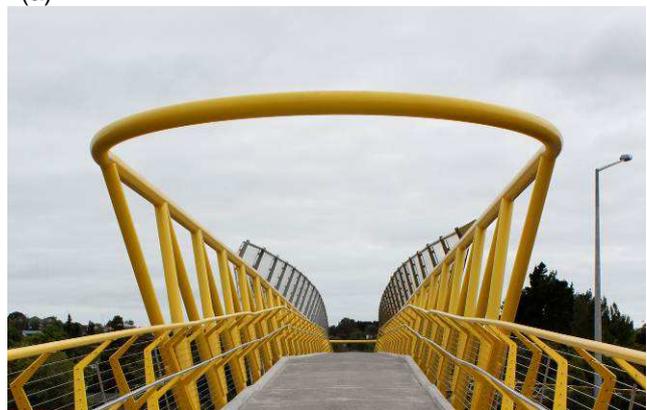
To keep with the triangular form of the truss, the concept for the handrail post was developed in which equal spacing between the handrail supports was achieved with angled twin posts, see Fig 3d. Stainless steel wires at 80mm crs vertically were used for in-fill and stiffened double plate handrail posts were provided at the tensioning locations for the wires. A top rail at 1400mm and lower rail at 900mm was provided for cyclists and pedestrian respectively. LED Puck lighting was provided in the lower handrail to illuminate the deck.



(a)



(b)



(c)



(d)

Figure 3. (a) Bridge 1 over the carriageway (b) twisted pier and in-fill piece (c) sloped truss webs and throw screens (d) handrails.

Detail design and analysis

The preferred concept was developed and detailed to produce drawings for construction. Design of the bridge was in accordance with Transit New Zealand Bridge Manual (TNZBM, 2003). General arrangement drawings for reference are included in the appendix. The key design challenges were;

- Complex geometry of the trusses
- Design of bridge 2 continuous space frame truss
- Stability of the bridge 1 top chord
- Seismic design of the structure

These challenges and the design of the bridge are discussed below.

Superstructure

Two different trusses were used in the design with a through truss for Spans 2-5 (Bridge 1) across the carriageway and an underslung truss for Spans 6-11 (Bridge 2) over the reserve, see Fig. 4. Both trusses were constructed from Grade 350 steel CHS sections that were shop welded. The truss nodding was detailed for zero eccentricity at the joints and the joint capacity was evaluated using the guidance in CIDECT design guidance (CIDECT, 1995). Tension/compression bracing was provided at the bottom and top chord levels of bridge 1 and 2 respectively.

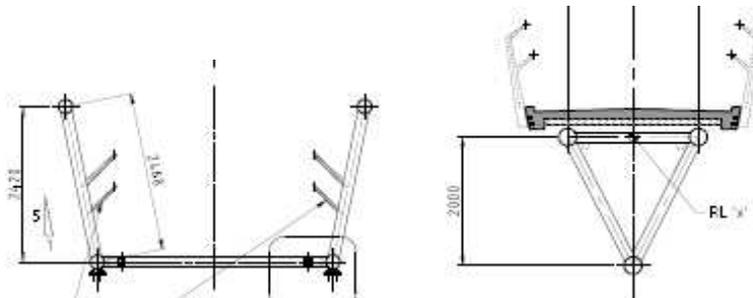


Figure 4. Cross section of trusses, bridge 1 (left) and bridge 2 (right)

The geometry of bridge 2 truss was complex due to the curved alignment and gradient. To achieve a level surface across the deck, the top chords were required to be offset vertically from each other at either ends. This offset swapped over at mid span to give a twist in the top chord. This was required to counteract the effect of vertical height differences caused by the inner and outer chord lengths at a constant gradient. The complexity of the geometry was further compounded, as the required shape of the chords was in the form of an ellipse which is difficult to bend. Instead a radius was used to average the ellipse. A 3d autocad model of the structure was used during design to ensure the complex geometry was captured in the construction drawings.

Due to the curved alignment of bridge 2, a triangulated closed form truss was adopted to take torsional loads through the structure. The truss was also designed to be continuous so that torsional loads could be efficiently transferred around the structure, thus limiting the need for tension bearings to hold down the truss. From analysis one bearing had a small reaction in tension under permanent loads, located at Pier 6 on the inner radius of Span 7. Live load analysis considered patch loading over the spans with the reductions allowed in (TNZBM, 2003) for loaded lengths in excess of 30m. Eccentric live loading across the deck was considered to maximise the torsion effects in the truss. Under live loading no further tension bearings were required.

For erection of the continuous truss in the field, bolted splices were provided in the top chords at the span ends and a small bottom chord segment with a bolted flange connection at either ends was provided. An erection sequence and temporary propping arrangement was developed to avoid tension in the bearings.

For stability the bridge 1 truss relied upon u-frame action from the truss diagonals and bottom chord to restrain the top chord from buckling. An elastic buckling analysis was conducted to determine the effective length of the top chord using a 3D model of the spans. The results obtained from the analysis were verified with a simple beam model with springs representing the stiffness of the diagonal supports. Effective lengths of approximately 1.3 to 2.3 were obtained depending on the height of the chords. Effective lengths for the diagonals for in-plane and out-of-plane buckling modes were obtained from BS5400-3:2000.

A precast concrete deck was used for both bridges. For bridge 1 the deck was designed to span longitudinally between the truss nodes. For bridge 2 the deck was designed to span transversely between the chords and loads were applied to the truss not at the nodes. The precast unit for bridge 2 had curved kerbs which varied in height to account for the twist so that the kerb top of adjacent units matched. An in-situ topping was used to form the levels of the footpath and take up the tolerances from the precast units. The precast units were supported on rubber strips with packing provided on the bridge 2 to accommodate the lack of fit in the geometry.

Substructure

The piers were constructed from Grade 350 steel sections. The piers were designed as twin curved columns with a transverse member connecting the tops, see Fig. 5. Bracing was provided to resolved forces due to the changes in inclination of the columns. Piers 2 -5 which were located in the carriageway were twisted. As the CHS sections can only be bent in one direction the shape was achieved by inclining the pier to the vertical. The piers were detailed for future bearing replacement with jacking locations provided.

The piers were supported by a Reinforced Concrete (RC) pile cap and a single 900 dia or 1200 dia bored RC pile. A bolted flange connection to the pile cap was designed without stiffeners to connect the pier to the pile cap. A single pile was adopted as adequate geotechnical capacity was available to support the bridge on a single pile, the frame action obtained with a pile group was undesired as pile tension would develop, and from a seismic perspective the capacity of the pier is the same in all directions. The piles were found into unweathered East Coast Bay's Rock which was located between 7 – 12m below the ground surface. Due to the vicinity of the piles to the steep banks, the stability of the bank was considered when assessing pile lateral capacity.

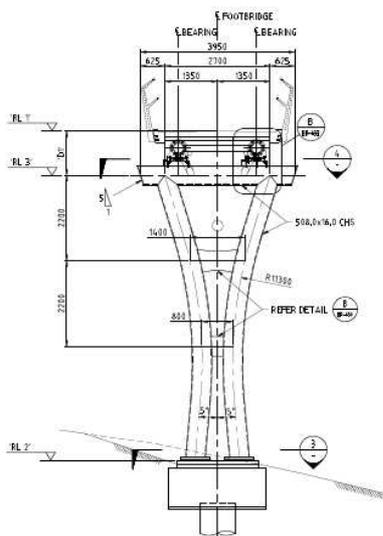


Figure 5. Typical pier arrangement.

Articulation

An expansion joint separates the straight and curved bridges at pier 6 with a simple stainless steel cover plate and drainage details to catch deck water. Additional expansion joints were provided at Pier 1 and Abutment B. Elastomeric bearings were adopted for the concrete approach span and pot bearings were used for the steel truss with fixed, sliding or free bearings.

Seismic design

Design philosophy

The lateral load resistance system of the bridge is provided as truss action from in plane bracing at deck level to transfer lateral loading to the piers via the bearings. Cantilever action of the piers transferred moments and shears into the piles through the holding down bolts and pile cap. These actions caused deformations in the piles and were resisted by the soil.

For this type of structural arrangement the TNZBM allows plastic hinges to form in the piles with a maximum ductility factor of $\mu = 3$. In this case a more robust solution was adopted and the structure designed to be

nominally ductile with $\mu = 1.25$. Although the structure will essentially remain elastic under a seismic event it was considered prudent to meet the provisions and detailing requirements for a ductile structure. Horizontal and vertical seismic responses were considered to act non-concurrently except where allowed for in the load combinations given in the TNZBM.

Capacity design principles were adopted and the piers and holding down bolt arrangement were design for the over-strength capacity of the piles. To prevent the simply supported spans from unseating, longitudinal linkage was provided between the adjacent spans using mild steel bars. No linkage system was provided at the piers where the truss was continuous. Shear keys were provided to restrain the bridge transversely. Holding down devices were provided at bearing locations for Spans 7 – 11 where in an seismic event the dead load reaction was relieved by 50% or more and in many cases a tension capacity was required.

Expansion joints were provided at Pier 1, Pier 6 and Abutment B. Impact between the bridges or bridge and abutment was not desired and a gap was provided to prevent this occurrence in the ultimate limit state seismic event.

Modal analysis

Due to the curved geometry and articulation of the bridge, modal analysis was considered to be necessary to obtain the horizontal seismic demands. The three bridges were articulated so that relative movement at the adjoining pier was allowed in the longitudinal direction, however, transversely the bridges were linked. A 3D dynamic model of the entire bridge was created in which the stiffness and mass distribution was modelled to accurately represent the structural system. To account for cracking in the RC piles an effective member property was determined from moment-curvature analysis using a confined concrete model.

Modal analysis was conducted using an array of angles of attack with a 30 degree increment to find seismic demands. In accordance with TNZBM, the number of modes included in the analysis was selected to ensure a minimum of 90% mass participation in both orthogonal directions. The Complete Quadratic Combination (CQC) method was adopted to combine the results from each mode. This was considered to be appropriate given the close spacing of the modes and potential coupling of longitudinal and transverse models due to curvature. The individual modal analyses were combined using the 100% and 30% orthogonal contribution to obtain demands at 30 degree increments. Horizontal torsion and rotational inertia effects were considered to be adequately accounted for in the 3D modal analysis.

Displacements were estimated from the modal analysis with multiplication by the ductility factor. Of particular interest were the relative movements between the two bridges at the expansion joint on the intermediate pier. Longitudinally it is possible that the bridges may be moving out-of-phase and could impact one another. The required gap to prevent pounding was obtained from the modal analysis.

Nonlinear static push-over analysis

Seismic demands were obtained for each pier from modal analysis. These demands were compared against the capacity of the pier as determined using a nonlinear static push-over analysis of an individual pier. Using a model of the individual pier with dead loading applied, the pier was pushed with an incremental lateral load until a collapse mechanism was reached. Several angles of attack were assessed to define an interaction curve between longitudinal and transverse demands. P-delta effects were included in the analysis which was considered necessary given the structural configuration. However, some conservatism exists in that the piers are to some extent restrained by adjacent piers via the truss although ignored in the analysis. A secant stiffness value for the pile at yield was adopted from above. Soil springs representing the reaction of the soil were removed and replaced with a force representing the yield capacity of the soil once yield criteria was met.

Pedestrian dynamics

To check pedestrian comfort levels on the bridge TNZBM references BD37/01 to check the serviceability accelerations of a single pedestrian. Guidance for complex structures, e.g. curved alignment is however limited. Further guidance was sort from the FIB Bulletin 32 (FIB, 2005). Both the vertical and lateral accelerations from a single pedestrian and a group of pedestrians were considered using an equivalent viscous damping ratio of 1%. Analysis showed that the accelerations were within the limiting criteria. In service behavior of the bridge on opening day was good and no noticeable vibrations were felt when walking with a large group people across the bridge.

Construction

Construction of the bridge was completed in early 2013 and the bridge was officially opened to the public on 31st January 2013. Some of the key construction challenges were;

- Construction access in the live traffic carriageway and steep banks and creek of Lenrich reserve
- Fabrication of complex trusses and piers all with unique member dimensions
- Erection of bridge 2

During construction access across the creek to Piers 6-8 was achieved by installing temporary staging between Spans 7 & 8. This staging allowed pile rigs and a mobile crane access to this area. Access to Piers 9 and 10 was achieved using an existing footway with a sheet pile wall used to shore the steep slopes.

Fabrication

Fabrication of the piers and trusses was undertaken nearby by D&H steel in West Auckland. The large CHS sections for the piers were bent in Australia. Fabrication was completed in the following order, piers, bridge 2 and bridge 1 trusses. The handrails for bridge 2 and the abutments were fabricated by Total Bridge Services. All elements of the piers and trusses were unique, however, the most challenging geometry was bridge 2 trusses. The trusses as discussed above have complex geometry and were assembled in a jig.

Pier erection

The bridge 2 piers, Piers 6-10 were erected first. The piers were transported to site by truck and lifted into place with using a 150T mobile crane, see Fig. 6. The piers were landed onto leveling nuts and the base plate was grouted afterwards. The piers were approximately 5.5 to 6.5T each and Pier 9 was lifted with at a 41.5m radius. Piers 1-5 in the carriageway were erected during night closures.



Figure 6. Pier 8 erection.

Truss erection

Bridge 2 was erected first and as the truss is continuous in the final condition an erection sequence and temporary propping arrangement was required to ensure stability during construction. The trusses were transported to site by truck and each span was landed onto the bearings already installed on the pier. The detail at the bearing had a shoe for each truss span so that the trusses could be landed independently on a common bearing. The erection sequence was chosen to avoid the development of tension in the bearings and was as follows;

- Lift Span 10 and release with prop at mid span
- Lift Span 11 and complete top and bottom chord splices, release span
- Lift Span 8 and release with prop at mid span
- Lift Span 9 and complete top and bottom chords splices, release span
- Lift Span 7 and complete top and bottom chord splices, release span
- Remove props and cast in situ deck from span 7 to 11

The truss spans were lifted in to place using a 200T mobile crane without the precast deck. Fig. 7 shows all spans in place with the temporary propping installed. Erection of the spans took approximately 5 days with site drilling of some of the bolt holes require to provide tolerance.

For bridge 1 the spans were fitted out off site with the precast concrete panels, throw screens, hand rails and lights installed before delivery. This allowed the deck to be used as a work area when finishing off the details and limited the number of night closures required. The trusses were transported to site and erected during separate night closures of each carriageway. Span 5 was approximately 47T in total. The bearing bolt holes were drilled on-site at one end of the truss to provide tolerance.



Figure 7. Bridge 2 span 8 lift (left), Span 6 being lifted into place.

Conclusions

Westgate footbridge is a new bridge that provides safe access across SH16 near Westgate shopping centre. The bridge is an innovative solution with extensive use of steel for both deck and piers. It adopts a distinctive structural form driven by alignment criteria across the carriageway and a curved alignment that allows views over Lendich reserve.

References

- CIDECT, 1995. Design guide for circular hollow section under predominantly static loading, CIDECT.
- FIB, 2005. Guidelines for the design of footbridges, FIB, Lausanne, Switzerland.
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