

TAURANGA HARBOUR CROSSING STELLA PASSAGE BRIDGE — SEISMIC DESIGN

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ABSTRACT

This paper describes the seismic design of the Stella Passage Bridge which was constructed in 1986 to 1988 as part of the Tauranga Harbour Crossing, a road link between Tauranga and Mount Maunganui.

The bridge is a 12 span, 478m long incrementally launched continuous prestressed concrete box girder with expansion joints at both abutments. Seismic restraint is provided by means of shear keys to ductile cantilever single stem pier columns supported on large diameter bored pile cylinder foundations.

INTRODUCTION

The Tauranga Harbour Crossing was constructed between March 1986 and March 1988 to provide a direct road link between Tauranga and Mount Maunganui. The harbour crossing consists of a 478m bridge over the Stella Passage, a causeway constructed on a tidal sand bank and an 80m bridge over a secondary channel at the Mount Maunganui end. Hydraulic studies showed that the impact of the causeway on the bed levels in the harbour would be minor for the length of bridge provided. The general arrangement of the crossing is shown in figure 1.

The crossing was designed by Murray-North Ltd for Fletcher Construction Co Ltd who were awarded the design and build tender by the Tauranga Harbour Bridge Committee.

This paper describes the philosophy behind the selection of the structural form of the Stella Passage Bridge and the way in which it will resist seismic loads.

BRIDGE FORM AND CONSTRUCTION TECHNIQUES

The bridge form was decided during the tender stage after preliminary design and costing of various alternative types of structure by the contractor and the designers. An incrementally launched prestressed concrete box girder was selected as the most cost competitive form of construction for the bridge which at that stage was to be straight in plan but with a vertical curve. Subsequently the crossing was realigned by the client and a horizontal curve was

introduced over the bridge length. By compounding the horizontal and vertical curves to give a circular curve on an inclined plane it was still possible to construct the bridge using the incremental launching method.

The superstructure consists of a single cell concrete box girder continuous over all supports with expansion joints at both abutments. The span arrangement adopted was 34m, 10 spans of 41m, and 34m, giving a total bridge length of 478m between abutment bearing lines.

The incremental launching method has been described elsewhere [ref. 6]. In this case the segment lengths chosen were 13.67m (1/3 span) and the pushing abutment was at the Tauranga end. Figure 2 shows the superstructure partially launched.

The pier and Mount Maunganui abutment foundations consist of large diameter bored cylinders founded up to 32 metres below the sea bed. The Tauranga abutment was founded on raked driven tube piles designed to provide the large horizontal reaction for launching the superstructure. The pier cylinders were capped by a beam at sea level with single stem circular pier columns rising to support the pier capping beam on which the box girder bearings sat.

The bridge form is shown in figure 3.

SUBSOIL CONDITIONS

Several stages of subsoil investigations were undertaken over many years as the project moved through its various stages from conception to construction. To facilitate final design, a series of investigation bore holes were sunk over the length of the

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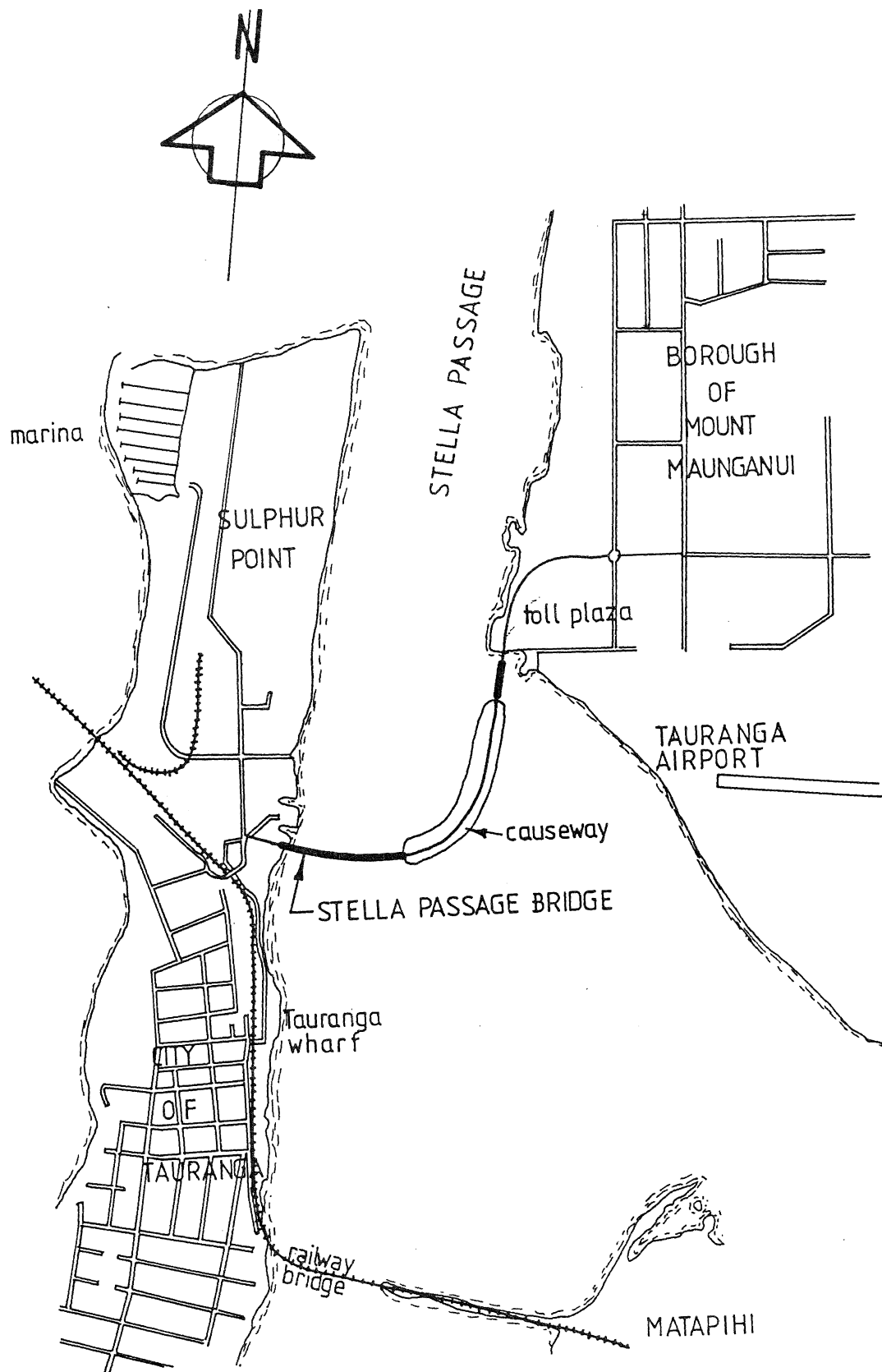


FIGURE 1. TAURANGA HARBOUR CROSSING: GENERAL ARRANGEMENT.

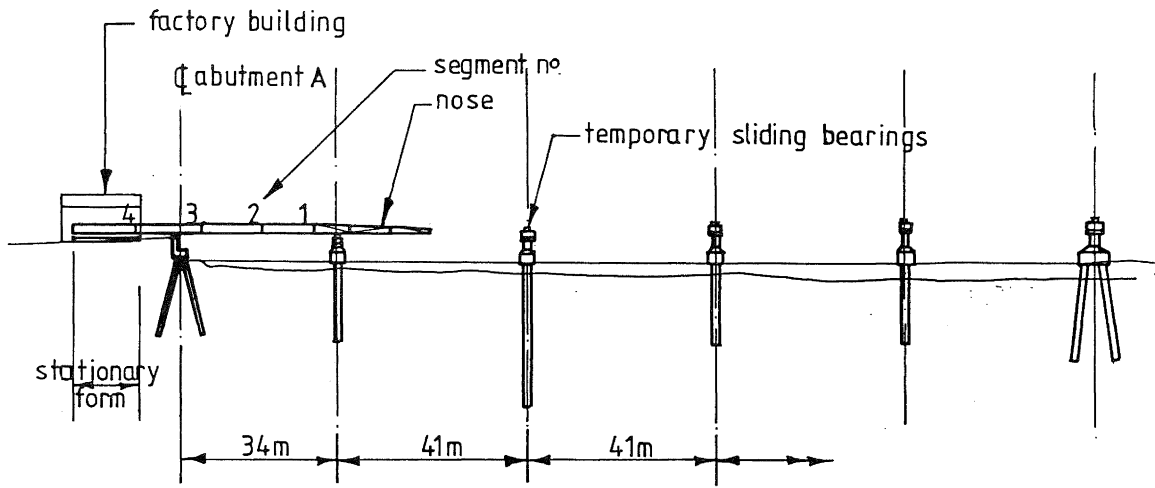
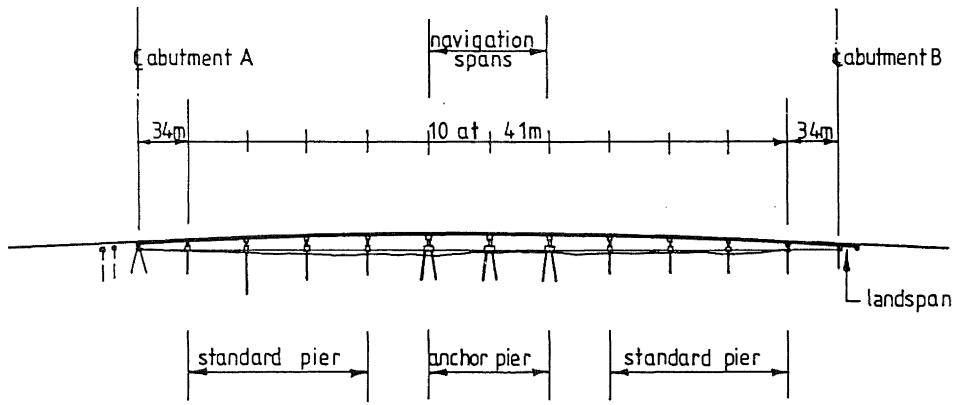
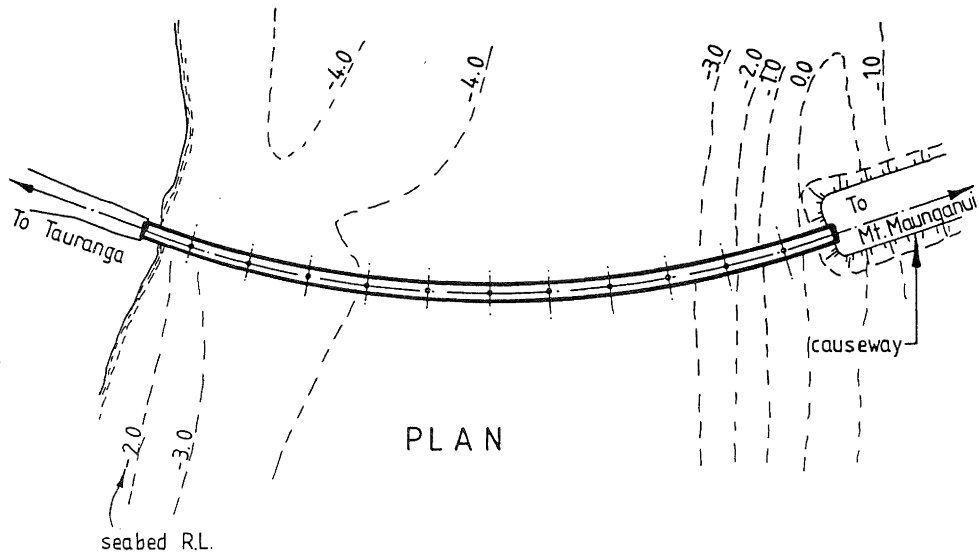


FIGURE 2. BRIDGE LAUNCHING.



BRIDGE ELEVATION



PLAN

FIGURE 3. BRIDGE ELEVATION AND PLAN.

bridge. During construction the data base was widened by CPT testing at each pier position to assist in final selection of the founding level.

All materials encountered in the offshore bore holes were sands, silty sands, sandy silts, shell or gravels. Raymond numbers and CPT cone resistance provided in situ soil strength data needed for foundation design. Soil stiffness under both horizontal and vertical loadings were interpreted on the basis of published correlations for the types of soils encountered.

DESIGN STANDARDS

The design standard for this bridge was the M.W.D. "Highway Bridge Design Brief" CDP 701/D September 1978 and the N.Z. National Society for Earthquake Engineering design philosophy for seismic design of bridges [2]. These two documents have been combined in effect in a draft update of CDP 701/D dated January 1985. The revision affected Section 2.4 of CDP 701/D, Earthquake Resistant Design, which is consistent with the NZNSEE approach but expands on its application. The January 1985 revision was used for the design but modified by the use of the inelastic design spectra published by Priestley and Park [3] (see figure 4).

The design conformed with the usual NZ standards and where appropriate the conditions of BS5400 [4] were applied.

DESIGN PHILOSOPHY

To a large extent the design philosophy for the bridge was tied to the choice of the

construction method for the superstructure. The incremental launching resulted in a continuous box girder superstructure of constant cross section without expansion joints from one abutment to the other. Although the concrete was up to eighteen months old when the superstructure was in its final position and connected to the substructure, the remaining shortening due to creep and shrinkage was calculated to be 256mm and design thermal shortening of the deck was ± 112 mm. The design philosophy adopted had to accommodate these superstructure shortenings whilst still providing horizontal restraint to traffic, wind and seismic loadings.

The bridge vertical geometry was a summit curve with maximum pier height at midspan and short piers near the abutments. The centre piers would therefore be the longest offering the greatest potential for ductility. Shipping lanes were also central requiring the central piers to be designed for ship impact in addition to normal loadings.

Taking these factors into consideration it was decided to provide longitudinal restraint to the bridge superstructure at the three central 'anchor' piers. Lateral restraint is provided at each pier and the abutments.

LONGITUDINAL SEISMIC BEHAVIOUR

Once the superstructure was in its final position it was connected to the three central anchor piers by shear keys. These shear keys consist of a heavy fabricated steel box section concreted into the pier column capping beam and protruding through the box girder soffit slab (see figure 5). The shear keys were installed in an oversized boxout in the pier capping beam before the

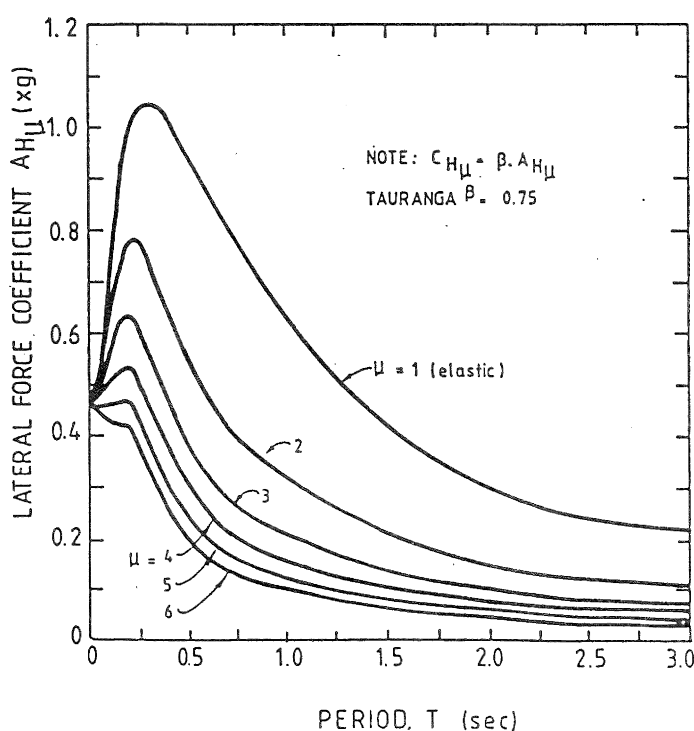


FIGURE 4. ZONE B INELASTIC DESIGN SPECTRA.

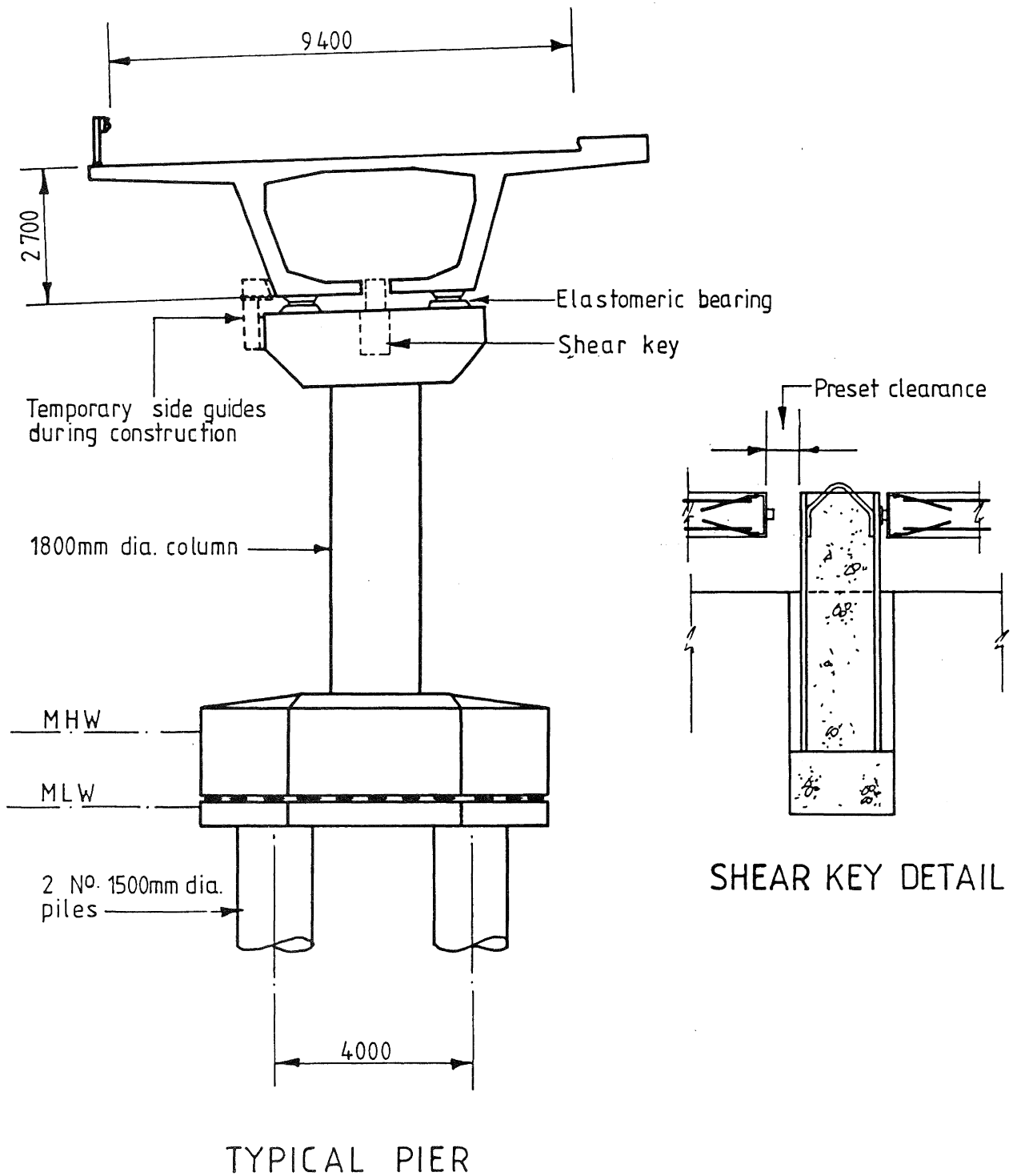


FIGURE 5. TYPICAL PIER AND SHEAR KEY DETAILS.

deck was launched over the piers. The deck soffit slab had a boxout at the final pier position and the shear key was lifted into place and concreted into the pier. The box girder soffit slab was then formed up around the shear key so that at each anchor pier both longitudinal and transverse forces were transmitted by bearing on the shear key at the level of the centre of the box girder soffit slab. The shear keys at the three anchor piers were set at a tight fit so that they shared seismic longitudinal loadings. The central pier was then the centre of fixity and was subjected to no environmental loadings due to shortening. The adjacent anchor piers were deformed elastically by shortening once the nominal construction separation at the shear key bearing surface was taken up.

With the box girder restrained longitudinally at the central anchor piers it could be released at all other standard piers and both abutments.

The bearings selected for the piers were laminated elastomeric and these could accommodate limited shear deformation of magnitude dependent on the number of laminations. The bearing heights increase with distance from the centre of longitudinal fixity, the anchor piers.

Although free longitudinally for environmental shortenings, the bearings required protection from excessive shear deformation during earthquakes. This was achieved by providing shear keys at all standard piers similar to those at the three central anchor piers but with slotted holes in the soffit slab. These provided lateral restraint and limited longitudinal movement. The settings were calculated to take account of remaining shortening, thermal movements, and pier deflection during launching. To reduce bearing shear requirements, the pier heads were predeflected by jacking against the superstructure before the shear key stops were set.

The abutments were set on sliding pot bearings which could accommodate full plastic longitudinal seismic movements as well as environmental shortening.

Under longitudinal service loadings the anchor piers provide restraint, the standard piers resist bearing shear forces and the abutments experience bearing friction forces. Under longitudinal seismic loading the anchor piers will control the superstructure deformation response and that deformation will be imposed at the standard pier positions. Initially the standard piers will respond independently of the superstructure but damped by the interconnecting bearing shear force. However once the available travel at the shear key slots is used up the standard piers will resist load from the shear keys.

The anchor piers are relatively stiff longitudinally, being supported on four opposing longitudinally raked large diameter bored cylinders. The standard piers are flexible longitudinally, being supported on pairs of bored cylinders forming a transverse portal frame but cantilevering longitudinally. In this manner the standard

piers remained elastic during the full plastic deformation of the anchor piers.

Any contribution the standard piers make to resisting the seismic forces or deformation of the superstructure longitudinally were ignored in the design of the anchor piers. Preliminary calculations indicated that the standard piers would contribute less than ten percent of the required seismic resistance longitudinally.

TRANSVERSE SEISMIC BEHAVIOUR

Under transverse seismic loading the box superstructure will act as a stiff horizontal beam on spring supports at all piers. The total load will be shared between piers according to their stiffness and position and the stiffness of the superstructure. Initially the design intent was to have no lateral restraint at the abutment as their significantly greater stiffness would otherwise attract excessive loading. This led to both excessive loading and excessive ductility demand on the first and shortest pier at each end. It also introduced high lateral bending moments in the box girder at those piers which would have necessitated additional deck prestress. These problems were solved by the addition of two seismic linkage devices at each abutment. These consisted of machined flat plates which will be deformed plastically under transverse seismic loading thus providing a predictable support load at the deck ends. The connection to the seismic linkage slides longitudinally so that the deck remains unstrained in that direction at the abutments (see figure 6).

EFFECTS OF CURVATURE

The bridge is curved in plan although the deflection from pier to pier is only 3.4 degrees.

Longitudinal forces generated near the abutment and resisted at the central anchor piers impose a small lateral load at each standard pier in between. Because the stiffness of the standard piers is relatively high transversely this is accomplished with very little bending moment being induced in the superstructure.

Because the deflection angle at each pier is small, the transverse load applied to each pier is also small.

Thus, for simplicity, the bridge could have been analysed as a straight structure with separate analysis longitudinally and laterally. This was confirmed by the three dimensional modelling actually carried out.

POST ELASTIC BEHAVIOUR

Longitudinally

Longitudinally the superstructure seismic forces will be resisted by the three anchor piers which will be loaded by a horizontal shear applied at box soffit slab level. This shear, together with the pier inertial forces, will cause a cantilever bending moment in the pier columns. The column was

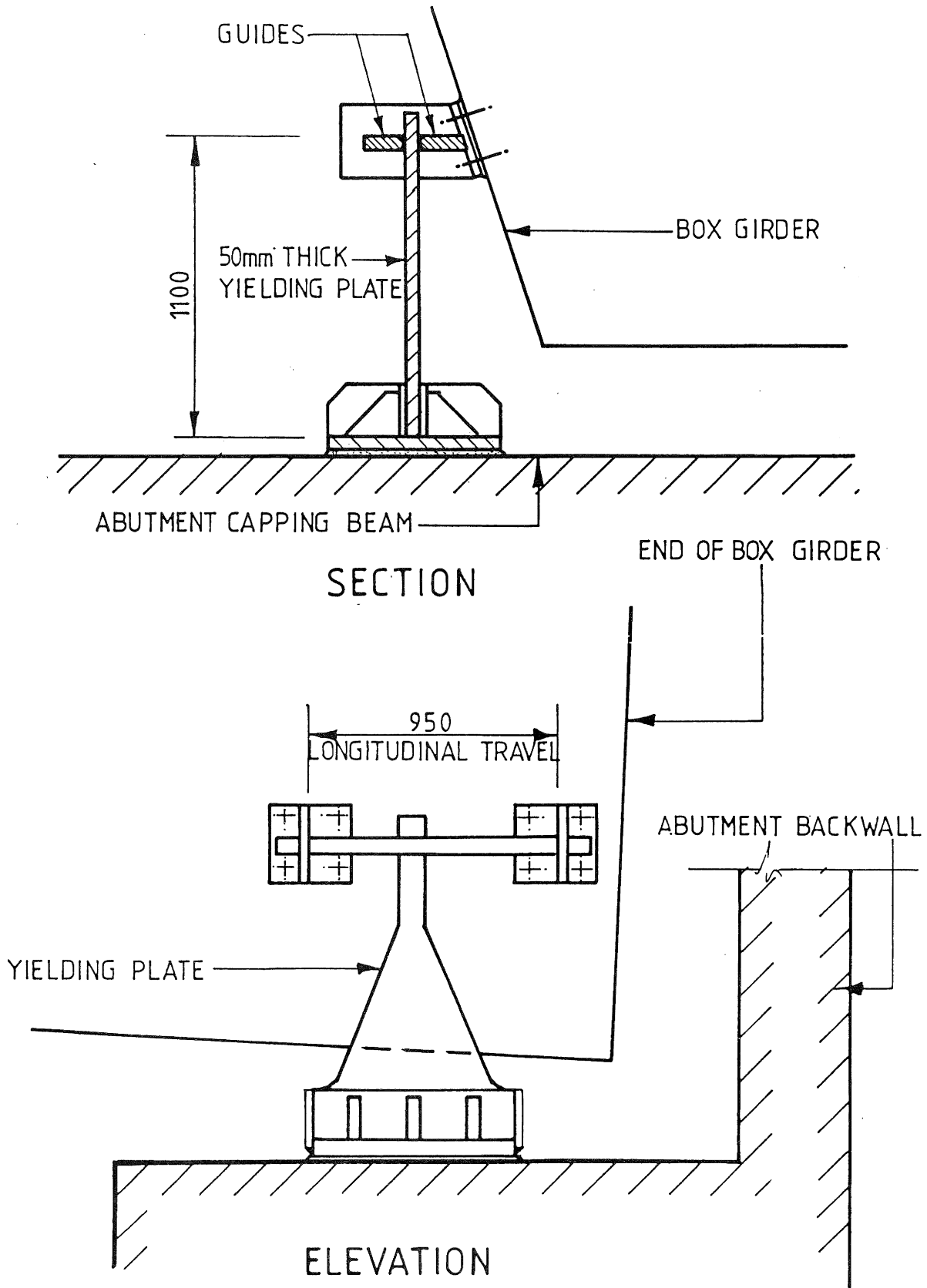


FIGURE 6. ABUTMENT LINKAGE DEVICE.

designed to form a plastic hinge at its base under the design seismic shear force. The capacity seismic shear was then established and applied to the pier structure. The pile capping beam, piles and shear key were then designed to remain elastic by the capacity procedures laid down in the relevant material codes. Under this loading the top sand strata were identified as having a potential to liquefy and the over-capacity design allowed for liquefaction to five metres below seabed at all piers and to ten metres below sea bed at any one pier.

The plastic deformation of the superstructure was established and used to set clearances at the abutment. That deformation was also applied to the standard pier heads, corrected for movement tolerance permitted by the shear key slotted guides, and the standard piers all remained elastic under the imposed deformation and self weight inertial effects.

Transversely

A similar capacity approach was applied to the transverse earthquake loading. All piers were designed to hinge plastically at the base of the pier columns with the pile capping beams, piles and shear keys remaining reliably elastic even with liquefaction of the upper strata. The abutment seismic linkage devices were sized so that they could accommodate the full plastic transverse deformation whilst providing the desired restraining force.

ABUTMENTS

The abutments were isolated from the remaining structure longitudinally by providing sliding bearings and seismic clearance to the back wall and transversely by the linkage device which imposed a controlled force. The Tauranga abutment provided the jacking reaction for launching the superstructure and it was that condition which totally controlled the abutment design. The abutment will remain elastic during seismic loading.

At the other abutment the bridge runs onto the causeway constructed over a sand bank. A study of the causeway design indicated the potential for liquefaction of the sands underlying the causeway fill [5]. The earth pressure forces on a conventional abutment could not be sustained under that condition and the design was modified to introduce a land span. That consists of a 12m span of double hollow core units running between the main bridge abutment and a sill footing. That permitted elimination of the filled earth behind the abutment back wall and capping beam which reduced the loadings on the abutment during liquefaction under the causeway. To avoid soil arching and to achieve a reliable upper bound force for the soil moving round the abutment piles, a single large diameter cylinder was selected. The abutment design was controlled by the liquefaction case and the pile was designed to remain elastic under that condition.

The landspan also acts as a long settlement slab to reduce the effects of differential settlement between the approach fill and the bridge structure.

CONSTRUCTION EARTHQUAKES

The launching of the superstructure took in excess of one year during which time the superstructure was supported on sliding pads at all piers. Transversely side guides were required on the down hill side at each pier to assist in the control of the horizontal alignment of the launching as the leading segment arrived at the pier. These side guides consisted of a steel beam stressed onto the pier capping beam and cantilevering up the side face of the box girder and plated to the profile of the box girder edge. As the box girder arrived at a pier PTFE pads were fed between the box girder concrete and the temporary bearing and side guide so that the box girder could slide freely over the bearings and past the side guide. The side guides were available for transverse seismic shear transfer between the box girder and the piers during construction. Additional guides were installed on the uphill side on alternate piers only.

Longitudinally the box girder was jacked uphill from the Tauranga abutment and after launching one segment the abutment sliding bearing was lowered leaving the box girder sitting on brake pads at the abutment whilst the next segment was constructed. The jacking system, consisting of pulling bars linking an end frame through the box girder to the abutment, remained in place. For one direction of earthquake loading the downhill component of weight and the abutment brake and pier bearing frictions before relative movement could take place. Downhill slipping of the superstructure was restricted by the jacking bars, abutment brake and pier bearing friction but assisted by the downhill component of weight. The consequences of the longitudinal earthquake forces exceeding the restraint capacities were not likely to be significant as the superstructure would just 'walk' across the piers during successive cycles. Following such an event the box girder could be jacked back to the required position.

The construction earthquake load combination of CDP701/D was applied which resulted in designing for half of the loads that would be required to be sustained by a permanent structure. The controlling element was the brake pad support block on the abutment and it was designed to yield before the permanent works were overstressed. Because the single point fixing arrangement directed the total longitudinal superstructure earthquake forces to the abutment during construction, there was potential for a significant overload of the stiff raked pile group at that point if the brake pad block did not yield first.

In fact, during the Bay of Plenty earthquake of March 1987 quite strong motions were felt at the site. The launching nose, which was cantilevering some distance at the time moved laterally quite violently. The longitudinal component must have been quite high also because the brake pad block just started to yield.

No other damage was sustained.

BRIDGE HARDWARE

Pier Shear Keys

The seismic shear transfer between the box girder and the piers is by means of cantilever shear keys cast into the pier capping beam. These consist of steel box sections fabricated from 30mm mild steel plate, hot dip galvanised and concrete filled after installation. Where guiding sliding side surfaces were required at the standard piers, this was achieved by welding on a side bearing plate and facing it with stainless steel which was to bear on PTFE fixed to the box girder soffit slab boxout (see figure 5).

The shear keys were designed to transfer capacity shears whilst remaining elastic and are not expected to sustain any damage in an earthquake.

Pier Bearings

The pier bearings are laminated elastomeric bearings which deform in shear under both environmental shortenings and longitudinal earthquake. The shear keys have been set to restrict the shear deformation to within a permissible limit.

Abutment Bearings

These are sliding pot bearings with sliding top plates sized to accommodate the full plastic displacement of the superstructure. No damage is expected during an earthquake.

Abutment Seismic Linkage Devices

These provide a means of transferring a reliable force during transverse earthquake

loading. They are fabricated from machined ductile mild steel plate and are hot dip galvanised (see figure 6). During the design earthquake they are expected to be subjected to plastic deformation. After a major earthquake the linkage devices will require replacement as they can sustain only a limited number of cycles of fully plastic deformation.

Expansion Joints

The service movements at each abutment from shortenings, temperature and traffic braking forces were calculated to require expansion joints to accommodate total movements of 291mm at abutment A and 267mm at abutment B. Fully plastic seismic longitudinal movements were calculated to be ± 336 mm and it was recognised that expansion joints could not accommodate these in addition to the environmental movement range. The code however requires that one quarter of full plastic earthquake deformation be provided for within the expansion joints.

No expansion joint could sustain the expected transverse earthquake movements at the abutments (± 360 mm plastic).

The solution adopted at abutment A was to provide a short link span at the abutment with an expansion joint at each end. One expansion joint was designed to provide for most of the longitudinal movement and is protected from transverse movements. The other joint, which provides only a small part of the longitudinal movement, is located where transverse movement can take place in an earthquake (see figure 7). The rubber part of the strip seal expansion joint might be destroyed and the linkage bolts deformed but

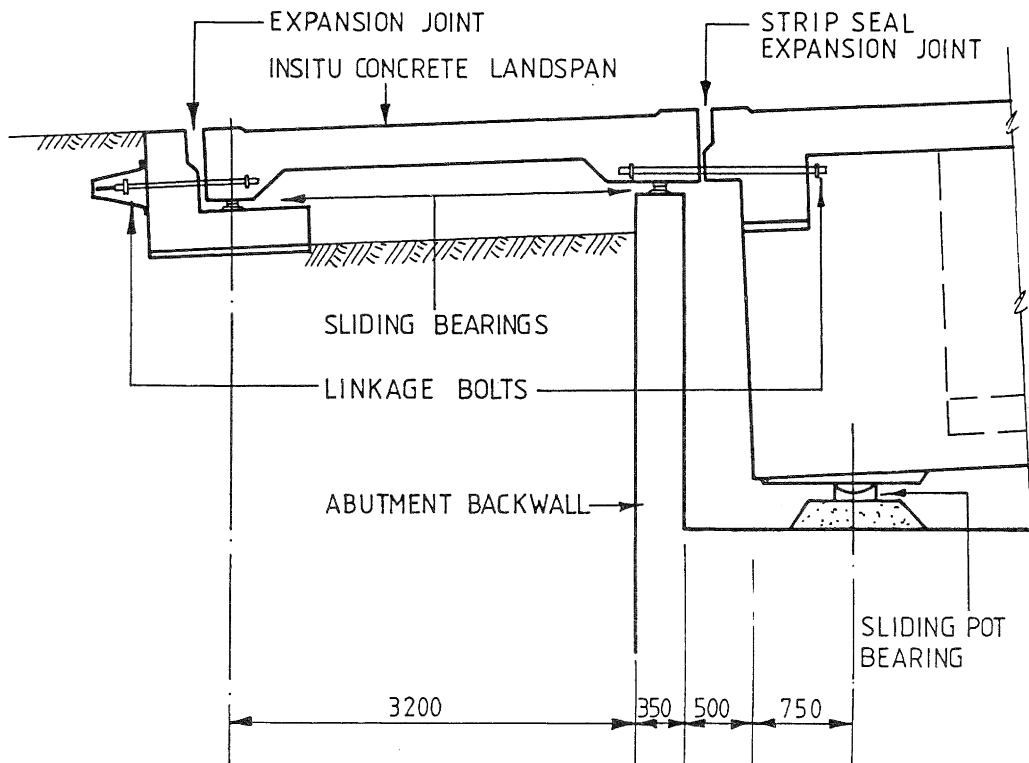


FIGURE 7. TAURANGA ABUTMENT LANDSPAN.

both are easily replaced and neither would affect the serviceability of the bridge. The same system was adopted at abutment B making use of the land span which had been provided to relieve liquefaction forces.

During post elastic longitudinal movements of the box girder the system of linkage bolts and rubber buffers are designed to move the landspan sill footing relative to the approach earthworks.

COMPUTER MODELLING

Stages of Modelling

Extensive computer modelling was undertaken in the design of the bridge for launching and final service loading conditions. This section describes only that related to the final seismic design.

At the tender stage sufficient preliminary design had been undertaken to estimate main member sizes. At the start of final design further preliminary calculations were undertaken to verify these so that computer modelling would start with best estimates of member stiffnesses.

In order to establish the total structure behaviour under seismic loading it was decided to model the total bridge as a space frame. That model would provide data on load sharing between piers, design effects on the superstructure box and displacements. If the model were detailed enough it could also have been used to establish design effects on the pier structural elements. However that would have necessitated modelling the soil support on all piles in all directions, a daunting task. To overcome that problem the computer modelling was carried out in stages with the first step being modelling of an individual pier.

Piers

The stiffness of each pier varied significantly due to the variations in pier column length, seabed level and subsoil properties. What was required for total structure modelling was the stiffness in each direction of each pier at the pier shear key and bearing positions. The first step was to set up a model of a complete pier structure including the modelling of the soil support at one metre intervals down all piles as a series of springs. A best estimate of soil stiffness was established by correlating available test data with published parameters for each strata at the pier being modelled. These stiffnesses were applied to the structural elements which were modelling the soil support (springs) and upper bound capacities for the reaction were set based on passive soil capacity. This detailed pier model was then loaded by horizontal "unit" loads in each direction at the shear key level, vertical unit loads at one and both bearings, self weight and self weight inertia. The modelling was repeated for a range of soil stiffness parameters to test for sensitivity and for no soil support for the top 5m and top 10m to model for liquefaction of the upper layers. Furthermore the yielding of top layers of soil was

modelled using an iterative process by replacing springs with constant forces once yield had been reached.

Thus the magnitude of the unit load applied to determine a pier stiffness had to be of the order of the real load so that the non elastic behaviour of the soil was modelled realistically.

That detailed model provided a relationship between load and displacement at shear key level and at bearing positions. A simplified structurally equivalent pier without soil springs was then established to give matching displacement stiffnesses for a greatly reduced number of structural elements. This equivalent pier was then used in the model of the total structure and the procedure was repeated for all piers.

Subsequent to running the total model the detailed model for each pier was used for the final design of the pier members and for pier ductility analysis.

Total Bridge Structure

The total bridge was then modelled as a space frame supported on the equivalent piers and sliding both ways at the abutments.

Transverse seismic forces were applied based on a first estimate of total structure ductility with the centre of mass in its calculated location. Further load cases were run with the code specified torsional moment to account for either the centre of mass or centre of resistance not being as predicted. The results of these runs indicated that the first piers in from each end were highly loaded and would be first to form plastic hinges. At that stage the decision was made to introduce the seismic linkage devices at the abutments and the desired yielding force of the devices was established and the total model re-run. The model then provided design data for the earthquake effects on the superstructure and the load sharing between piers.

The model was also run with reduced stiffness piers to simulate five metres of soil liquefaction over the whole site and with individual piers in turn subjected to local ten metre deep liquefaction.

A similar process was applied for longitudinal earthquake loading but that was found to govern only the three central anchor piers.

Dynamic Analysis

The total structure was analysed using EASE-2 and the frequency and deflected shapes of various modes were determined horizontally and vertically. This analysis was used to estimate the period T for determination of seismic loads for the static analysis.

The potential cost savings available by using dynamic analysis for detailed design were likely to be minor. It was considered that pursuing these minor savings could not be justified for the degree of reliability that could be placed on the soil/structure interaction modelling and therefore the

equivalent static analysis was used for final design.

Interactions Between Models

The procedure described above is somewhat simplified as in practice an iterative process of adjusting the models was required to refine the design of members and to establish a total structure ductility and fundamental period. Fortunately the natural period was long enough to be on a fairly flat part of the response curve. Before design there was concern that some piers would become plastic much earlier than others which would place large ductility demand on those piers. In the event the computer modelling demonstrated that although successive piers yielded the demand on any one pier for ductility was not excessive. The total structure ductility needed to establish the design lateral force coefficient was therefore able to be selected from within a small range of pier ductilities.

SUMMARY

The computer analysis led to the adoption of total structure ductilities of 2.0 transversely and 3.0 longitudinally. The natural period was calculated to be 1.4 sec transversely and 1.85 sec longitudinally. The lateral force coefficient obtained from the inelastic design spectra for these values was 0.12 transversely and 0.075 longitudinally.

The design was based on these seismic coefficients which controlled the sizing of the pier columns and the column reinforcement. The over-capacity factor (ratio of maximum shear to cause plastic hinging at maximum member strength to minimum shear to cause dependable first yield moment) was 1.6 and the seismic shears were increased by this factor to establish the capacity design effects for the remaining parts of the structure.

CONCLUSIONS

The bridge was designed rigorously in accordance with the relevant code requirements. Furthermore, the structural form selected should ensure that it performs well in the design earthquake.

By concentrating longitudinal resistance to the centre of the bridge the out of phase effects which are difficult to design for in a long structure should be minimised.

The continuous superstructure and the careful attention to overstrength connections should ensure that the bridge is robust in the extreme earthquake, and that the ultimate failure mode is a favourable mechanism.

The greatest unknown is the extent and effect of liquefaction at the site. However the deep foundations and robust pile sections should minimise the consequences of liquefaction on the structure itself.

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