

# Design and Construction of the Otira Viaduct

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## SYNOPSIS

The Otira Viaduct is a major prestressed concrete box girder bridge sited amongst majestic mountains in the Arthur's Pass National Park in the Southern Alps of New Zealand. The area's tectonic, geological, geotechnical and environmental conditions together with the terrain and active river conditions made this an unusually challenging and demanding project. The 445m long balanced cantilever bridge has spans up to 134m and piers constructed deep into avalanche material with rock compressive strengths of up to 250 MPa.

## 1 INTRODUCTION

The Otira Viaduct is a 445m long, two-lane, concrete box girder bridge constructed in the Otira River Valley. The site is in a national park, at an altitude of 800m in the Southern Alps of New Zealand and near Arthur's Pass, an important crossing linking the east and west coasts of the South Island.

The viaduct replaced a section of road traversing an actively eroding mountainside, before dropping in a series of steep hairpin bends known as "The Zig Zag". Gradients in this section were as high as 18%, and the corners extremely tight – restricting the length of vehicle that could use the route. Erosion of the slope beneath the highway had reached the point where a further slip was likely to cause permanent closure of the road.

To bypass this hazard, a major road realignment down the Otira River Valley was necessary. A range of innovative design and construction techniques were introduced to meet the challenges of what is one of the most demanding bridge sites in the world. The result – a four-span viaduct with central spans of 134m – is the longest span concrete bridge in New Zealand. The site and surroundings are shown in Figure 1.

The project is of national environmental significance as a major roading project constructed within a National Park. Environmental management and monitoring of impacts on the delicate and diverse range of flora and fauna were thus a critical part of the project.

The project was completed in 2000 and has since won a number of international awards.



*Figure 1 - Site view showing old road (the Zig-Zag) on left*

## **2 HISTORICAL BACKGROUND**

The Arthur's Pass route is believed to have originally been used by Maori who travelled over the alps to reach the rugged west coast and obtain highly valued greenstone (jade).

It was named after Arthur Dobson, son of the provincial engineer Edward Dobson, following its 'discovery' by Arthur during an exploratory trip in 1864.

Edward Dobson drafted a plan for a tunnel through the site. However funding would not permit this and instead a road was built and opened in 1866. In 1929 an earthquake caused a major section of the road to slip into the Otira River. In 1943 further erosion resulted in another realignment. Over time sections of the road were progressively shifted uphill to keep the route open and another zig-zag was added. However, the point was reached where this process could not continue and another major slip would have permanently cut the highway.

## **3 THE CHALLENGES**

### **3.1 Environmental and Aesthetic Issues**

The surrounding landscape is both visually spectacular and environmentally sensitive – hence its National Park status. Consequently, the bridge had to be sympathetic with its surroundings, providing an aesthetic, harmonious but also economic solution for its high profile site.

The effect of the project on the fragile sub-alpine environment was critical. To minimise the impact on the landscape, vegetation, wildlife (including protected bird species such as kea, blue duck and kiwi) and on the river itself, project-specific environmental management controls were developed and monitored to ensure compliance.

On the other hand, the effect of the inclement environment on the project was also significant. The area is one of the wettest and coldest in New Zealand. In addition to a rainfall of five metres per annum, the bridge site is subject to icing conditions during winter. These factors had implications not only for design but also for the construction approach.

### 3.2 Geological Issues

A spur at the southern end of the site (known as Deaths Corner) is the remnant of a huge rock avalanche which 2000 years ago dammed the valley, and ran well up the western bank. The avalanche, which was probably seismically induced, originated from the mountain ridge 700m above the riverbed. The river has subsequently cut a V notch in this dam. Figure 2 shows these features and indicates the avalanche source area. It also shows the road alignment prior to the 1929 Arthurs Pass Earthquake. In normal flow conditions much of the Otira River flows under the old landslide dam and underground through the rock avalanche materials in the valley. Foundations could not reach bedrock and had to be constructed in this avalanche debris.



*Figure 2 - Geological Features*

### **3.3 Geotechnical Conditions**

The rock avalanche material on which the bridge is founded is relatively compact and contains greywacke (sandstone) boulders with compressive strengths up to 250 MPa. The material is very abrasive and extremely variable in size varying from a silty sand matrix, up to blocks many meters in size. In addition, the ground water conditions are extremely variable and complex. The ground is highly permeable, with high water table levels, major underground water flows and artesian conditions prevalent at the toe of the Deaths Corner landslide dam.

### **3.4 Rockfalls**

Rock avalanches, or individual large rocks up to several metres in diameter, bounding or sliding down the steep adjacent mountain slopes posed a major threat to the structure. Special studies were carried out to enable the design of structures to protect the vulnerable bridge piers from rockfalls.

### **3.5 Seismicity**

Otira is located in the highest earthquake hazard area of New Zealand. The site is located some 20km southwest of the active Alpine Fault which forms a section of the Australian-Pacific plate boundary. There are 14 known active faults within 50km of the site, capable of generating Magnitude 7 to 8 events. Of these faults 5 are within 20km, with the nearest 2 km from the site. The high seismicity has historically caused major slips and rockfalls, including the dislodgement of boulders from the mountain slopes which rise 700m above the site.

Accordingly a site-specific seismic hazard study was carried out.

### **3.6 Hydraulics**

The Otira River is slowly degrading at the site and several years ago a ‘nick point’ was constructed near Deaths Corner by placing imported rip rap across the river. The river can rise very rapidly in flood conditions and has caused significant erosion in the project area in the past. The progressive failures threatening the old highway were initiated by lateral erosion and down cutting of the toe of the slope by the river. Loss of control of the river could seriously affect the project and deep foundations, together with scour protection, have been provided to mitigate this risk.

### **3.7 Geometrics**

Geometrics are very constrained by the topography, which is extremely steep and rugged. The gradient of the viaduct varies from 12 to 16%. Although substantially less than the existing road gradients, this is well outside accepted North American and European standards of 6 to 8% for highways subject to icing conditions. To mitigate against this, the winter gritting regime previously used on the road has been maintained, together with the trial use of Calcium Magnesium Acetate (CMA) as the use of salt (NaCl) is not permitted on New Zealand roads. This strategy has proved successful.

### 3.8 Cultural

Several very large rocks located in the river valley have cultural significance to the local Maori as markers for the greenstone trail. These sacred rocks (known locally as Kaitiaki) limited the available pier positions and required careful protection during construction.

## 4 DEVELOPMENT OF BRIDGE CONCEPTS

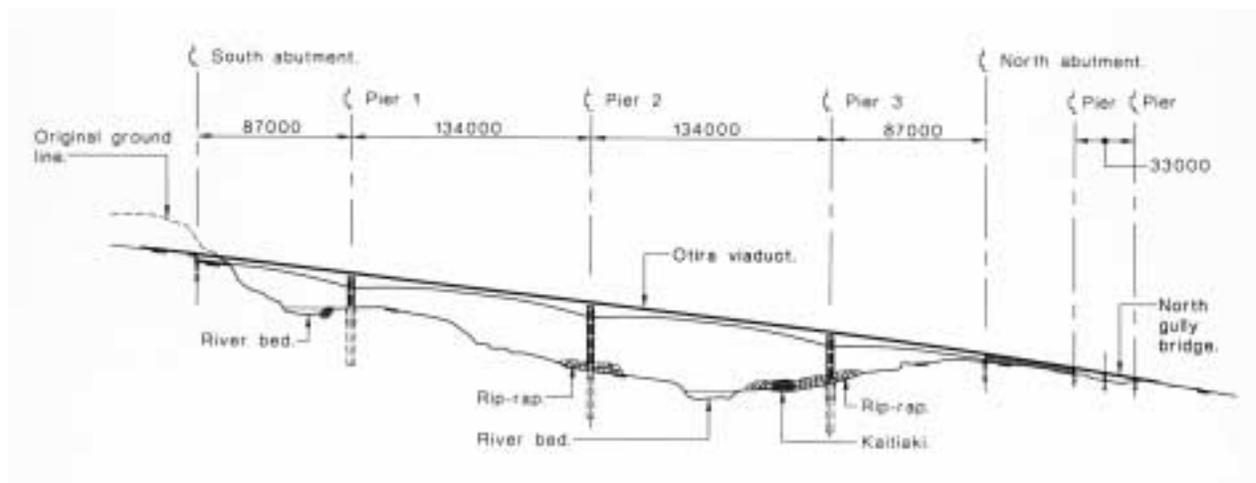
Several alignments were investigated to overcome the various constraints posed by the challenges described above. These varied from a realignment across the river into a cutting in the far bank, to an on-grade highway beside the river. During this study Professor Christian Menn from Switzerland visited the site providing valuable review inputs and design recommendations.

After comparing the options, a gently curving design following the river valley was chosen. A range of bridge spans and types was investigated varying from a three span option with a 225m main span down to a seven span option with 84 m spans.

The investigations showed the feasible options for this project to be:

- Four span bridge - balanced cantilever in-situ concrete box girder
- Seven span bridge - balanced cantilever in-situ concrete box girder or steel truss bridge or steel box girder

These options were taken through for further study and the four span concrete structure best fulfilled the project requirements and was also the lowest cost option. This structure has spans up to 134 m long and piers up to 45m high as is shown in Figure 3.



*Figure 3 - Bridge Elevation*

## **5 BRIDGE DESIGN**

### **5.1 Aesthetics**

To compliment the gently curved alignment, the shape and geometry of the bridge were carefully crafted. Features designed to enhance its appearance included sloping the box girder webs, gently curving the web to deck soffit junction, shaping the superstructure/pier top connection and adding special finishes to the pier sides. These features are illustrated in Figures 4, 5 and 8.

### **5.2 Design Standards**

The bridge was designed to comply with the standards contained in the Transit New Zealand Bridge Manual (TNZBM). Dead Loads are still critical for long span balanced cantilever bridges and, at the commencement of design, typical trial-mix densities of the specified concrete were obtained from local suppliers. Based on experience a 1 to 2% margin was allowed within the design for form swelling and construction tolerances.

Both ultimate and serviceability limit state checks were carried out in accordance with the TNZBM. The serviceability limit state adopted was zero tension in the superstructure under normal traffic loads but a partial prestress design under traffic load plus differential temperature, with a maximum crack width of 0.3mm.

The TNZBM requires an ultimate limit state design earthquake event return period of 1000 years. At this event level, repairable damage is acceptable provided emergency traffic can continue to use the facility.

In an “event with a return period significantly greater than the design value” collapse must be prevented and repairable damage and traffic load limitations are accepted.

Due to the seismicity of the site and the close proximity of a number of active faults, a site specific seismic hazard study was carried out. The study confirmed the high seismicity of the site and recommended peak ground acceleration and response spectra levels in the order of twice those specified in the TNZBM. The study indicated the 1000 year return period hazard to have an estimated peak ground acceleration of 1.0g and for 500 years a pga of 0.75g.

The bridge was designed for the hazard levels recommended in the above study using a 1000 year return period as recommended by the TNZBM. A maximum structure displacement ductility factor of 6 was adopted as specified for the TNZBM for bridges where the areas of potential inelastic demand and damage are visible and accessible. Capacity design procedures were applied with inelastic seismic demands designed to occur at the top and bottom of the hollow box bridge piers.

The bridge was designed for a 100-year life in accordance with the TNZBM. 28 day concrete strengths were 40 MPa for the superstructure and 30 MPa for other components. All pier and

superstructure concrete had 6% air entrainment to provide freeze thaw mitigation. Reinforcing steel was generally grade 430 MPa for principal reinforcement and 300 MPa for stirrups.

### **5.3 Structural Form**

For the reliable seismic performance of this bridge it was considered essential that the piers and superstructure be monolithic, without articulations. However the piers are relatively short and stiff thus attracting high loads from creep, shrinkage and temperature deformations of the structure. It was not possible to reduce the pier size further without comprising the safety during construction without the introduction of a temporary support pier. The use of twin leaf piers is sometimes used to overcome this, however due to the very severe seismic strength and ductility demands this was not considered feasible. Instead, before the final two cantilevers were made continuous by completing the mid-span closure pours, the superstructure was jacked apart locking large loads into the end piers. The bridge is supported on twin elastomeric bearings at each abutment.

The structural form is illustrated in Figures 3 and 5.

### **5.4 Seismic Design Philosophy**

As noted above the bridge was designed with ductile piers using capacity design principles so that plastic hinges are fixed to the columns. These hinge locations are preferred to protect the superstructure and piles from significant inelastic seismic demands.

At the abutments the bridge is supported on large elastomeric bearings and is effectively base isolated. The bearings and abutment lateral support systems have been design to ensure reliable performance in major seismic events. The bearings are each 1000 mm square and 470mm high.

Under longitudinal seismic actions the bearings reach about half of their available displacement capacity, in the design 1000 year return period earthquake, and thus have sufficient reserve capability to cope with much larger events.

Under transverse seismic actions the displacement demands are significantly larger. In the 1000 year event the displacements are in the order of 400mm, which is within the bearing capacity. To ensure reliable performance in a larger event large shear keys are provided each side of the superstructure (with a 400mm gap provided). The shear keys are designed to hinge the single pile supporting the abutment so using capacity design principles. Thus under transverse seismic actions reliable performance is assured in large seismic events.

### **5.5 Foundations**

The foundation conditions presented a major construction challenge and had a major influence on the bridge type and spans. Particular issues included:

- steep hillsides and severe flood/scour conditions around piers
- large seismic forces

- construction in permeable, abrasive, avalanche materials containing extremely hard and massive rock

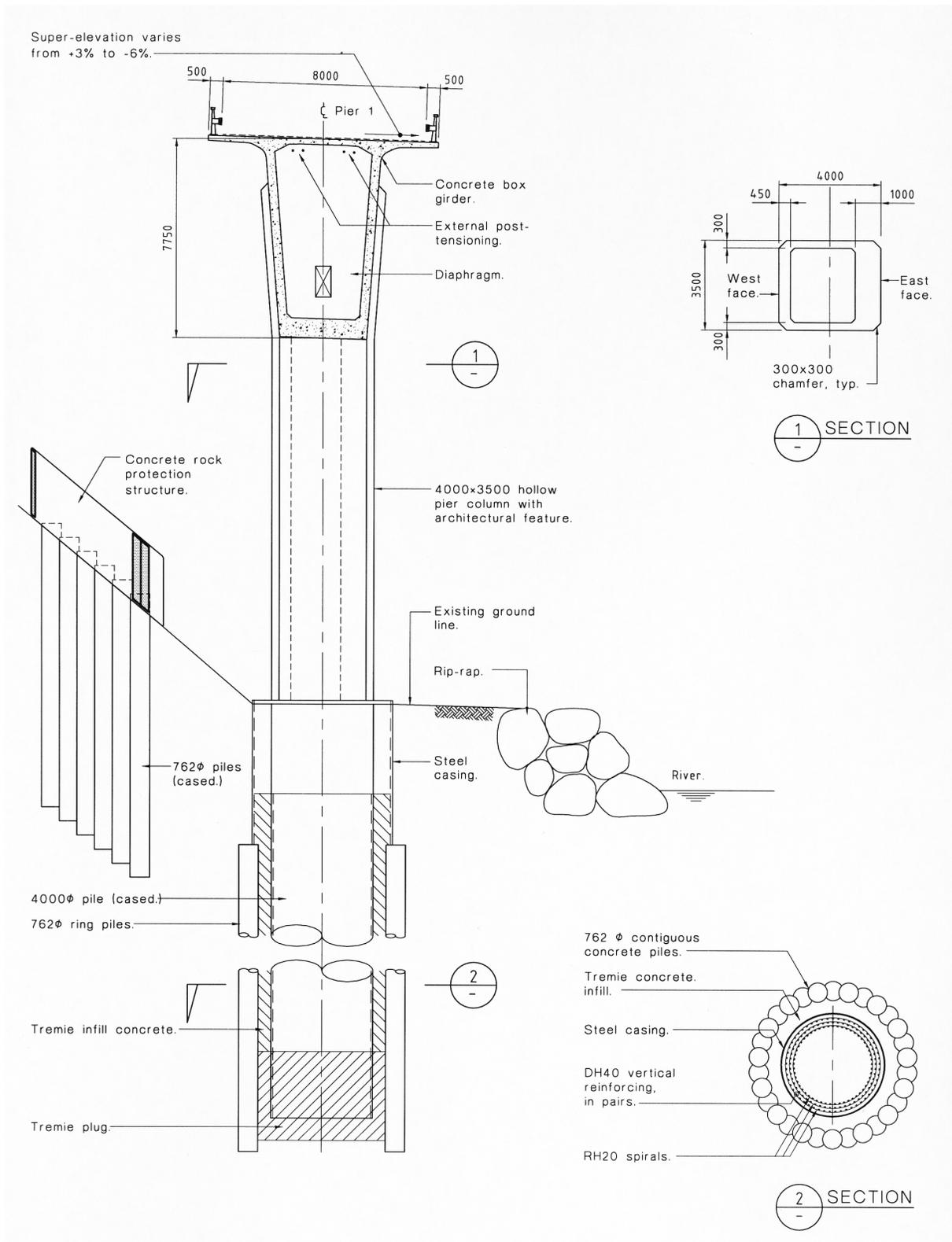


*Figure 4 - General view of site*

These conditions dictated a relatively long span bridge, which in turn had implications for foundation design. The most appropriate solution was deep, 4m diameter, single-cylinder foundations, up to 25m long, under each pier position (refer Figure 5).

## **5.6 Piers and Abutments**

The bridge is supported on single concrete box piers up to 45m high, constructed over each foundation cylinder. The boxes at piers 1 and 3 have walls of different thickness to reduce the effect of dead load eccentricity moments which are induced because of the horizontally curved superstructure. The varying wall thicknesses shift the section centroid laterally. This was considered necessary to minimize lateral deflections during construction and to mitigate against “ratcheting” of the structure in one direction during a major seismic shaking transverse to the bridge. To reduce such effects the out-of-balance dead load effects need to be minimised.



**Figure 5 - Section at Pier 1**

The hollow box piers are detailed for plastic hinge formation at the top and bottom, and the cylinder foundations (and superstructure) are designed for the resulting overstrength actions under seismic loadings. Thus in the event of a major seismic event damage can be inspected and repaired if required. Column overstrength moments were calculated using a reinforcement yield strength of 1.25  $f_y$  and a concrete strength of 1.5  $f'_c$ . To provide the confinement to the concrete and restraint to longitudinal reinforcement necessary to achieve the design ductility, closed Ø10mm stirrups were provided in the plastic hinge zones. Typical pier details are shown in Figures 5 and 6.

The bridge is supported on elastomeric bearings at the abutments. Seismic loads transverse to the bridge centreline are resisted at the abutments and by cantilever action of the piers. Seismic loads along the bridge are carried by the framing action of the piers and a contribution from the abutments through the elastomeric bearings. Under a very large seismic event when large displacements may occur, the top of the abutment wall is designed to be knocked off by the projecting deck slab. In addition a concrete 'fail safe' plinth has been provided just beneath the bridge to support the end of the span in the event of failure of the elastomeric bearings.

The piers are relatively short and stiff, attracting high loads from creep, shrinkage and temperature deformations of the superstructure. However, it was considered essential for the reliable seismic performance of the bridge that the piers and superstructure were monolithic, without articulations. Accordingly, before the cantilevers were made continuous by completing the mid-span closure pours, the superstructure was jacked apart by 40mm in each of the main spans.

## **5.7 Superstructure**

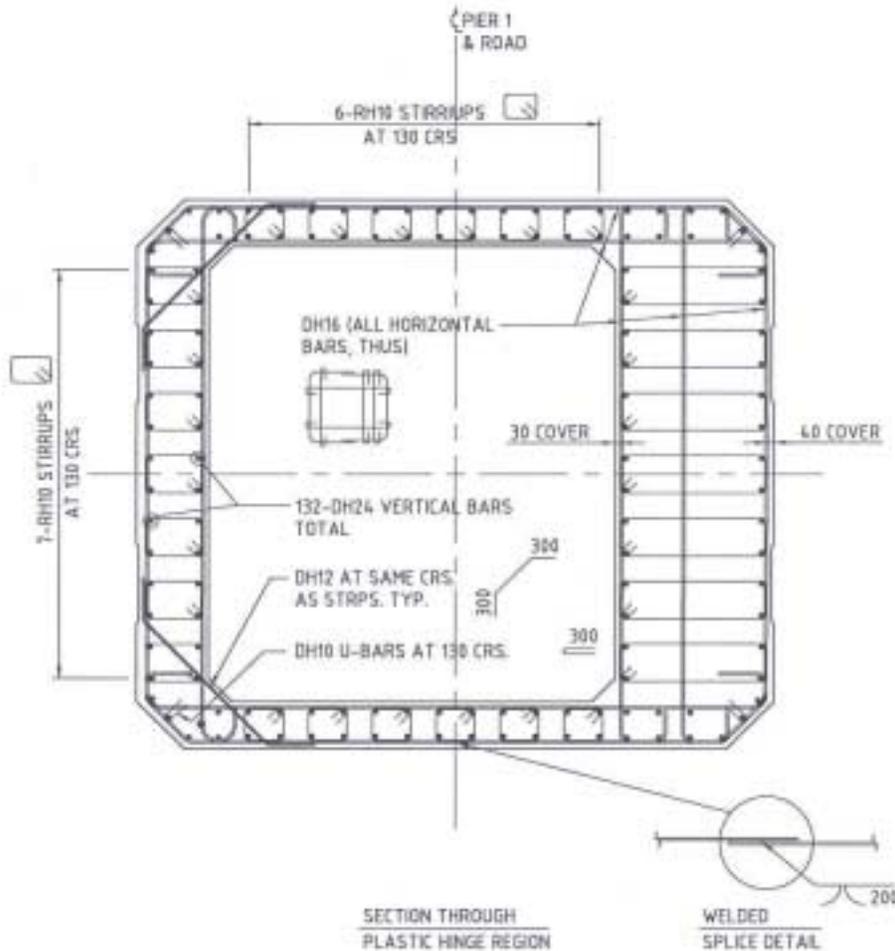
The bridge superstructure comprises a four-span, 445m long, balanced cantilever concrete box girder with a maximum span of 134m. The box cross section is shown in Figure 5 and a typical cantilever on Figure 7. The cantilever cables for the box girder comprise twenty 19/12.7mm strand cables and the continuity cables involve up to twenty two 12/12.7mm strand cables. Two external 12/12.7mm strand cables were also provided for the full length of the bridge as this was found to be a very effective method of gaining additional control of mid-span stresses in the continuous structure. To allow for dead load variations, provision was made for the installation of two additional 12/12.7mm external cables, which could be installed should traffic or dead loading increases occur in the future.

Web thicknesses vary from 350 to 300mm and quarter-span diaphragms have been provided.

## **5.8 Rockfall Protection Structures**

Large rocks regularly bound or slide down the steep mountainsides. To mitigate against rockfall damage to bridge piers, rockfall protection structures were provided above Piers 1 and 3 and a rock bund deflection mound provided at Pier 2 (refer Figures 5 and 8). A special study was carried out using the Colorado Rockfall Simulation Program, a computer model which estimates rock velocities and trajectories. The rockfall protection structures were accordingly designed to

withstand a 4.5m diameter rock travelling at 10m/sec. Maximum bounce height was estimated at 0.4m. The large V-shaped structures are supported on steel encased concrete piles, which have been designed for the ductile action required and can withstand several design impacts before their integrity is compromised.



**Figure 6 - Reinforcing details for Pier 1**

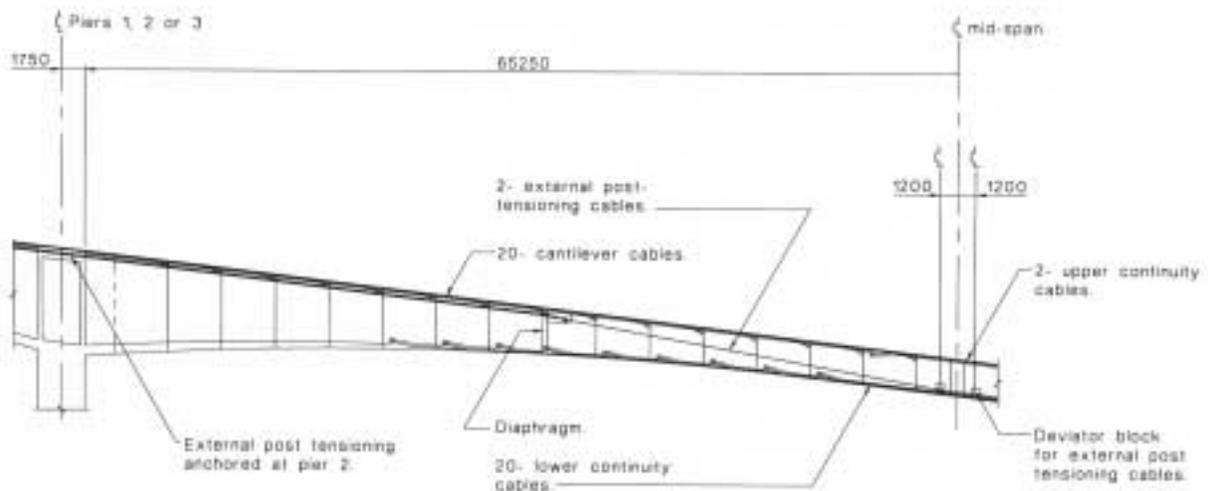
## 6 CONSTRUCTION

### 6.1 Construction Loading

As discussed above under ‘Structural Form’ the piers needed to be kept as slender as possible to limit loadings induced by longitudinal shortening of the superstructure. As a result the piers had limited capacity to carry out of balance moments during construction, in conjunction with seismic effects. However, as allowed by the TNZBM, seismic forces during construction were reduced to one third of normal design values to reflect the lower risks posed by the relatively short durations of the critical loadings. Even so it was found necessary to either limit the pours for the final segments to half size pours, or else to counterweight the cantilever by half a segment. The contractor opted for the latter option.

## 6.2 Foundations

Construction of the 25m long, 4m diameter cylinders into the rock avalanche debris in the conditions previously described was a major challenge for this project. This was recognised at an early stage in the project and the tender documents encouraged the use of innovative methods of construction. To provide for a reasonable degree of risk sharing for the very difficult ground conditions the contractor was required to install three additional boreholes at each pier location. This information was evaluated and, where the contractor could demonstrate conditions more adverse than those detailed in the tender documents, a claim for additional costs was permitted. This risk sharing resulted in a modest cost increase at one pier where an extraordinarily large (30m) boulder was uncovered.



*Figure 7 Elevation of typical span*

The contractor devised a method of construction which met the demanding conditions and minimised ground relaxation and environmental impacts. This involved the use of state of the art, down-the-hole drilling technology.

This method involved the construction of a concrete pile secant wall around the outside of the permanent pile footprint allowing later excavation inside the concrete pile coffer dam for lowering of the permanent steel liner and reinforcement. As dewatering was not possible, the work was carried out below water until the pile casing had been secured with a tremie plug and tremie in-fill concrete (refer Figure 5). The pile could then be constructed in the dry.

The secant pile holes were 762 mm in diameter and drilled with a “Numa” Down Hole Hammer (DHH) percussion drill purchased especially for this project. The bit used was an under-reaming bit with a driving collar, effectively bottom driving a casing during drilling. The drill rig has a 35m high leader mounted on a 40 tonne crane base. The drill bit was rotated with a 6 t-m Casagrande rotator and three No. 1000 ft<sup>3</sup>/minute compressors supplied air for the percussion

hammer and cutting return. The average drilling rate at Otira was about 6m/hr. However, drilling rates as fast as 15m/hr were recorded on the site.

These holes were drilled on a 'hit and miss' basis and tremie concreted with unreinforced 17MPa concrete prior to drilling the intermediate overlapping holes. The casings were withdrawn with the tremie pipe during concreting using a crane suspended vibro-hammer.

The abrasive and extremely hard greywacke rock resulted in significant wear and tear on the drilling equipment.

The ground inside the secant pile wall was excavated using conventional clamshell techniques, supplemented by drilling of relief holes to help break up any large rock masses. Much of the excavation was undertaken with no visibility as the ground water level at each pier was within six or seven metres of the ground surface. The effects of flowing groundwater in the porous avalanche debris caused concern about the integrity of the secant pile ring due to the washing out of cement. Core drilling of piles at Pier 2 and 1 was consequently carried out and indicated intact concrete full depth.

Following excavation the permanent 4m diameter casings were lowered into the excavation and located prior to tremie concreting the plug and later the annulus outside the steel casing. The steel casings were dewatered to allow placement of the reinforcement and pile concrete in the dry. Construction details of a typical pier are shown in Figure 5 and schematically in Figure 9



*Figure 8 - View at Pier 1*

### **6.3 Piers**

The pier box columns were constructed using 4.5m segment jump forms.

### **6.4 Superstructure**

The superstructure was constructed in-situ using balanced cantilever construction. Two sets of cantilever carriages (i.e. four carriages in total) were used for this project. At pier 2 the 8m long pier table was supported off the pier prior to erection of the carriages. At Piers 1 and 3 the pier tables were extended to 16m with the aid of ground supported falsework.

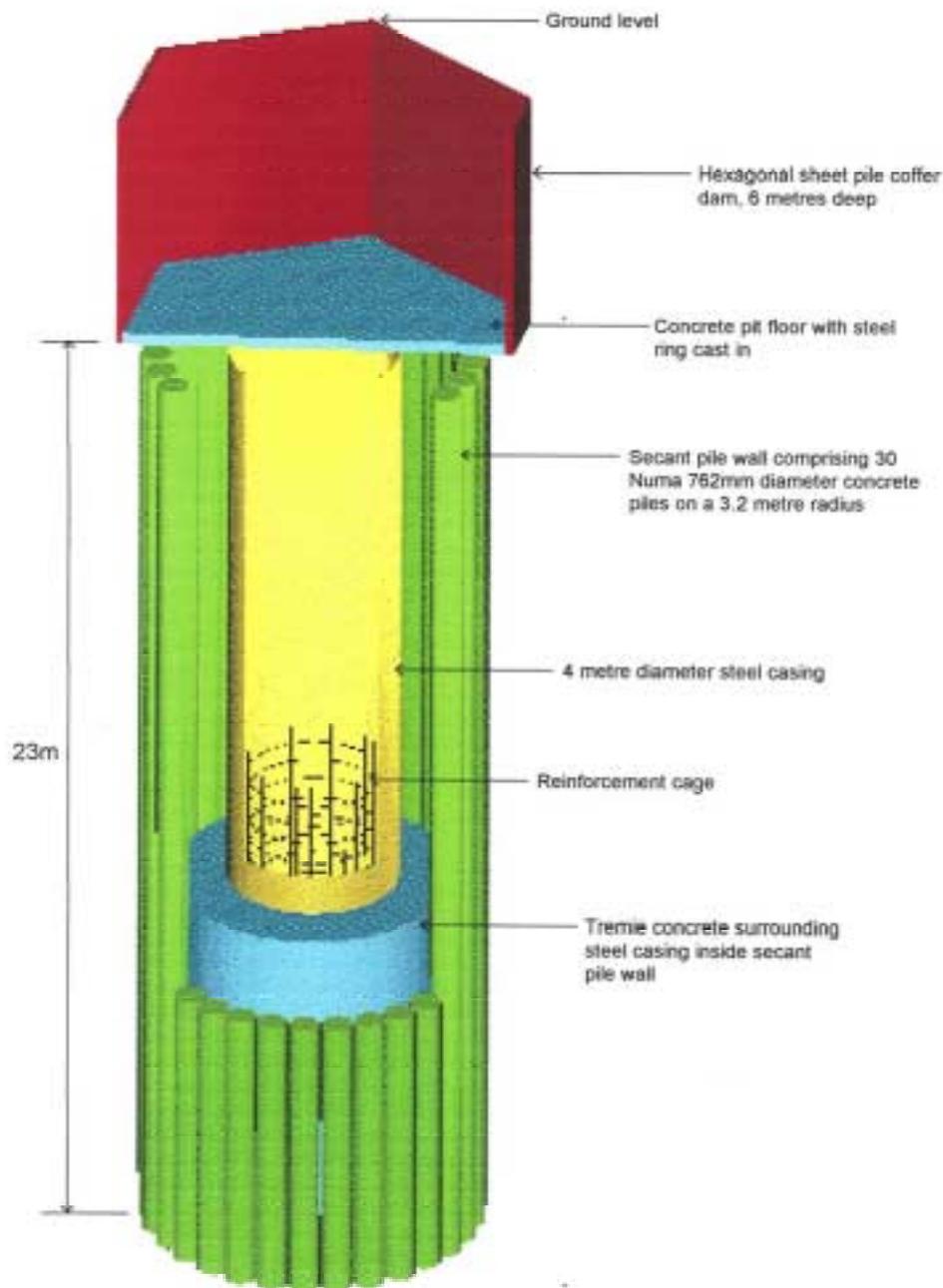
The superstructure was constructed in 4m segments, each cantilever being 16 segments either side of the pier. The contractor design and manufactured carriages to support the hanging framework for each cantilever. Rather than using kentledge, the carriage support beams were stressed down to the deck at the rear. The carriages were self-launching, using hydraulic rams to push forward the support beams for the next segment. The specified 40 MPa concrete was supplied from a batch plant set up for the project outside the National Park in Otira, some 6km from the site. This required negotiating the treacherous 18% gradients over the 'Zig Zag' but only one major concrete pour had to be stopped due to icy conditions halting the concrete trucks.

The endspans adjacent to the abutments were built on ground supported formwork, and connected to the superstructure with continuity tendons on completion of the adjacent cantilever sections.

The formwork for the superstructure was complex due to the box shape and changing geometry of the bridge at each segment. The superstructure formwork combined 4mm steel plate and plywood framework with adjustable wall heights and base widths to cope with the changing cross section and changing super-elevation of the bridge. As the superstructure proceeds towards mid-span the box section becomes shallower. The internal plywood web forms were constructed in removable segments to enable reduction in height during construction. The outer steel forms did not reduce in size towards mid-span but hung down beneath the permanent works. Hinges incorporated into the form designs allowed rotation of the deck (super-elevation) with respect to the webs of the box.

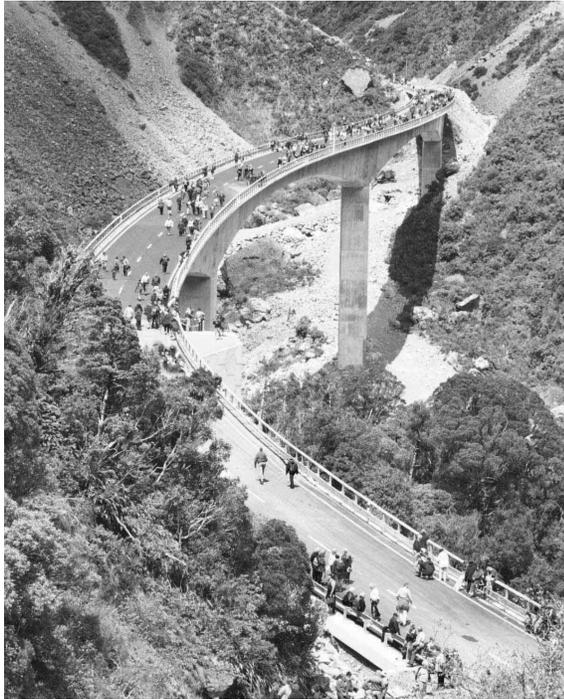
Geometry control was a critical item and the target levels for each segment were adjusted as construction proceeded to ensure the desired profile and facilitate closure. Geometry control measurements were particularly difficult due to the variable and often unpleasant weather conditions on the site including heavy rain, cloud and mist. For the first winter, the travellers were fully enclosed with a roof and side tarpaulins for the benefit of the construction workers. The forms were electrically heated during the winter with strapped-on cables and insulated with polystyrene-backed ply to ensure proper curing and early strength gain in the concrete. Radiant heaters were used above the bridge deck and fan heaters inside the box maintained acceptable temperatures after pouring.

The bridge piers could not carry the full out-of-balance loads of the last two segments of each cantilever. Rather than constructing these in half segments, the contractor chose to maintain balance using counterweights placed on the deck equivalent to the out-of-balance movement of half a unit.



*Figure 9 - Schematic of Pier Foundation Construction*

Construction of cantilever segments was targeted at a weekly cycle, with stressing occurring on day two or three at a concrete strength of greater than 25 MPa. Once the initial learning curve was over this cycle was generally achieved.



*Figure 10- Opening day!*

The completed bridge is shown in Figure 10.

## **6.5 Deflection Control and Monitoring**

During construction, dimensions were measured, on-site cable forces determined and concrete densities were continually monitored as part of the deflection control procedures. Several units prior to closure of each span, the stressing loads for the two additional 12/12.7mm cantilever cables were chosen and stressed appropriately to achieve the desired load balance.

Subsequent to completion of the bridge, a regular deflection-monitoring programme has been implemented. In addition, the horizontal position of the rock protection structures is measured to monitor their location and thus the

integrity of their supporting ductile piles.

## **7 CONCLUSION**

The Otira Viaduct project involved the design and construction of a major bridge at a site which was very demanding by world standards.

The challenges posed by very difficult foundation conditions, extreme alpine weather and construction in a sensitive environment were met by incorporating features and techniques which have not been used before for bridge construction in New Zealand. Special features of the project included use of a mixed internal/external post-tensioning system, major pier rockfall protection structures, use of large state of the art down-the-hole hammers for the foundations and use of enclosed, heated work spaces and forms.

Completion of this high risk project in 34 months – more than one month ahead of time and also below budget – is a tribute to all involved. This was made possible by pro-active management, a partnering approach to problem solving and the provision of equitable risk sharing provisions for the geotechnical conditions.

## **8 ACKNOWLEDGMENT**

The author wishes to thank Transit New Zealand, McConnell Smith Ltd, Department of Conservation and other parties who made this a successful project.