1 SYNOPSIS
The Second Gateway Bridge duplicates one of Australia’s largest bridges. Designed to mirror the existing bridge appearance, it is nevertheless quite different in underlying detail. With respect to the foundations, this is particularly evident in the adopted method of bridge articulation. The approach spans require the foundations to be relatively flexible in the longitudinal bridge direction to prevent large stresses developing in the spans due to concrete creep and shrinkage, but still resist large axial and transverse loads. This paper commences with a brief description of the geology of the bridge site including the regional faulting in the underlying rocks, which influenced the design of several of the pier foundations. The paper then provides an overview of the foundation systems adopted for the bridge, and describes some of the more interesting design features of the adopted foundation system including:

- The rock socketted bored pile groups supporting the main river piers, including the use of Osterberg cell testing to verify axial capacity, and modifications to the design to accommodate faulting observed in rock cores.
- Spread footings on rock for the southern piers, including adoption of an eccentric footing at one pier spanning a sub-vertical altered fault zone.
- Use of a single row of rock socketted piles to provide increased foundation flexibility of support to piers with a critical influence on bridge span stresses.

2 BRIDGE LAYOUT
The Second Gateway Bridge duplicates one of Australia’s largest bridges at an offset of approximately 50m. Visually, the second bridge mirrors the existing bridge albeit with a wider carriageway to accommodate pedestrian and bicycle access. The main difference between the existing and Second bridges is in the articulation of the approach spans. The existing bridge used columns fitted with pinned bearings at both deck level and foundation level, such that the deck is free to shrink and expand, with longitudinal forces transferred to the abutments. To reduce maintenance of bearings and provide a structure with a higher level of redundancy with respect to earthquake and impact loads, the approach spans of the second bridge use piers with full moment connections at both deck and foundation level. As a result, longitudinal loads are shared between piers. The bridge is divided into four sections connected by halving joints. Pot bearings that allow longitudinal movement are provided at the halving joints and abutments. This layout is shown schematically in Figure 1.
This arrangement requires the pier foundations to be relatively flexible in the longitudinal bridge direction to prevent large stresses developing in the spans due to concrete creep and shrinkage, but still strong enough to resist the large axial and transverse loads.

3 GEOLOGICAL SETTING AND OVERVIEW OF FOUNDATIONS

The bridge is aligned approximately north-south, has a total length of approximately 1.6km and spans the Brisbane River. It is therefore not surprising that ground conditions would vary along the length of the bridge requiring a range of foundation solutions. A simplified illustration of the subsurface geology and bridge foundations is shown on Figure 1.
On the south side of the river, Triassic age sandstone, siltstone and low-grade coal of the Aspley-Tingalpa Formation occur at the surface. These rocks generally dip to the northeast at 5 to 15 degrees. The proximity of low to moderate strength rock to the ground surface towards the southern end of the bridge lends itself to spread footings in this area.

Approaching the river, the surface of the rock drops off, and continues to fall beneath the existing river, levelling out on the north bank of the river at around 25 to 30m below sea level. This fall in rock level represents the side of a wide eroded valley through which the ancestral Brisbane River would have flowed when sea levels were much lower.

At least two distinct coal seams were encountered on the south side of the river and extending beneath the riverbed. The coal tends to be more highly fractured and of lower strength than the sandstone/siltstone. Additionally the thickness of highly to extremely weathered rock and residual soil thickens towards the south bank of the river. The greater depth to good rock materials, together with the higher pier loadings from the main bridge span favoured adoption of groups of bored rock-socketted piles for the main span piers and transition piers (Piers 5 to 8).

Although the Triassic rocks remain undisturbed by significant folding, some faulting occurred during the Tertiary period as a consequence of crustal tension associated with the development of small sedimentary basins. These faults generally express themselves as high angle normal faults. Several of these faults were encountered in the foundations for the Second bridge. It would appear from the observed faults that they typically strike roughly perpendicular to the bridge alignment and dip at 60-80° to both the north and south. Displacement across the faults varies from a few centimetres to a few metres. Locations where faults were observed in the foundations are marked on Figure 1, and the effect these faults had on specific foundations is discussed later in this paper.

Beneath the northern river bank, the ancestral Brisbane River in the Tertiary and early Quaternary period appears to have been a fast flowing high energy river. This is evidenced by the top of the rock having been eroded to expose relatively strong rock at its surface, overlain by a layer of coarse river gravels. In one of the bored piles at Pier 14, the gravels formed a thick dense layer that was quite difficult to drill and contained some rounded river boulders. Similarly at Pier 10, approximately one third of the driven piles refused near the top of these gravels.

After deposition of the gravels, sea levels subsequently rose and interbedded clays and sands were deposited through the Quaternary period in both an alluvial and estuarine environment to levels probably extending above the current ground levels. Prior to the Holocene period (10,000 years before present day), sea levels again fell and palaeo-channels were cut by the river into these sediments. As a result these sediments are over-consolidated to a reasonably stiff/dense state. The current Brisbane River sits in the southernmost of these palaeo-channels, but several other channels are evident beneath the northern bank of the river and extending further north beyond the bridge.

Through the Holocene period, sea levels again rose, and soft estuarine clays were deposited into the paleo-channels. This deposition continues to the present day in the bed of the Brisbane River, where clay continues to be deposited in the mangroves, and clay shear strength as low as 3kPa was measured. On the northern bank of the river, there appears to have been some consolidation of the upper
estuarine clays by desiccation and minor surcharging from 1 to 2m of sand reclamation. The shear strength of the estuarine clay beneath the northern bank is therefore about 15kPa at about 3m depth, increasing with depth at a rate of about 1.5kPa/m. In addition to being of a very low strength, these soils are also highly compressible and prone to long term consolidation and creep settlement.

These soft estuarine clays are therefore unsuitable as a founding material for bridge piers, and piled foundations extending to the gravels and rock were therefore required. The soil properties also pose significant challenges for the stability and settlement of the approach embankment to the north abutment as well as the islands constructed for the river piers.

Further discussion of some of the more interesting features of the foundations is included in the following sections:

4 MAIN SPAN BORED PILE FOUNDATIONS

The main span pier foundations for the existing bridge comprised 48, 1.5m diameter bored rock socketted piles raked in both the longitudinal and transverse directions at 1:6 to 1:12 to assist with resisting lateral loads on the pilecaps. The piles were all physically inspected, and the piles for Pier 6 were under-reamed to increase end bearing resistance and therefore reduce socket length. During construction of Pier 7, highly variable rock conditions were encountered including extensive zones of material described as “breccia” up to 6m thick, and several instances of steeply dipping faults. As a result of these conditions, the lengths of the piles were altered during construction. The piles were cased down to sandstone, with casing toe levels varying from RL-33 to RL-43. Pile toe levels varied from RL-38 to RL-52 with socket lengths ranging from 3 to 15m.

By contrast the main piers for the Second bridge were designed with 24, 1.8m diameter vertical piles, designed to resist lateral loads using portal action, with correspondingly much larger bending reinforcement requirements than the original piles. The reduction in the number piles for axial load was achieved through adoption of longer rock sockets, designed to resist the loads predominantly in shaft resistance. A comparison of the existing and proposed piles at Pier 6 is shown on Figure 2.

Current occupational health and safety practices meant that it was impractical and undesirable to carry out physical inspection and cleaning of rock sockets up to 60m below sea level in the middle of a river, and preferable to construct piles without any dewatering of the excavation. Airlift cleaning of sockets and tremmie concreting was proposed. However, concerns remained about the cleanliness of the rock sockets and the possibility of variable rock conditions as encountered at the existing Pier 7.

To address these concerns, several construction procedures were developed for the main piers which included: inspecting the base of all piles using a special down-hole camera contained in an open ended diving bell (SID); Osterberg Cell axial pile load tests of two test piles plus two production piles; and continuous cored boreholes at every pile location in the main piers.
Figure 2 - Section Through Pier 6 of Existing and Second Bridge Foundation

Figure 3 - Osterberg Cell Test Pile 1 (Pier 6)
Osterberg cell testing is a type of static load test which uses high pressure hydraulic jacks cast into the pile part way down the socket at a position calculated such that the shaft resistance above the jack is similar to the combined shaft and base resistance below the jack. Strain gauges are cast into the pile at various levels to allow the distribution of shaft resistance down the pile and at the base to be determined. Telltale displacement rods are also cast into the pile to allow the toe level and jack displacement to be directly measured.

Sacrificial test piles constructed identically to the production piles (but with diameter reduced to 1.5m to reduce jack loads), were constructed adjacent the main piers and tested to a peak load of approximately 3.1 times the design Serviceability Limit State (SLS) load. The results from the sacrificial test pile at Pier 6 are shown on Figure 3. In addition, a production pile was tested at each main pier to 1.2 times the design Ultimate Limit State (ULS) load (about 1.5xSLS) and the jacks grouted on test completion to transfer load to the base in service.

The sacrificial test piles demonstrated an axial capacity and axial stiffness in excess of requirements. The shaft resistance in the bottom part of the test pile at Pier 6 was fully mobilised and was greater than assumed in the design. The test load was insufficient to fully mobilise the base resistance or the shaft resistance in the upper part of the pile. The production pile tests also confirmed axial stiffness in excess of requirements.

The borehole cores at each pile indicated ground conditions inconsistent with the design assumptions at both of the main piers. In both instances, the varying ground conditions could not have been reasonably identified from the existing boreholes used at design phase.

At Pier 6, a sheared bedding parting was identified cutting across the pile group below the proposed pile toe level. Reanalysis of the pile group to take into account the presence of a weak layer beneath the piles indicated there was some reduction in foundation stiffness (conservatively up to 20%), but it was not sufficient to adversely affect the pier performance. Secondly it was determined that the presence of a weak layer beneath the pile toe had negligible effect on pile capacity if it occurred greater than 3m below the pile toe. Where core logs indicated the pile toe would found less than 3m above the weak layer, the pile toe was extended to just below the weak layer.

At Pier 7, several of the cores encountered highly fractured zones including some materials that had been sheared to clay. This appeared to be a continuation of the same features that had been observed in the pile excavations for Pier 7 of the existing bridge. At the same time, excavations for the footing at Pier 2 exposed an intersection of two faults (see photo insert in Figure 4), and a faulted zone of fractured rock about 200mm wide was observed in bulk excavations near Pier 4. Taking all this into account, a geotechnical model to explain the observed conditions in the borehole cores was developed as shown on Figure 4.

The conclusion drawn from this model was that the rock condition in a 50mm diameter rock core could not necessarily be used to infer conditions about the rock conditions in a particular 1.8m diameter pile socket. However, it suggested that there was a high likelihood that several of the piles in the 24 pile group would encounter the poorer quality rock over part of the shaft or base.
To assess the potential impact this could have on the performance of the pile group, a series of sensitivity analyses of individual piles and the pile group were carried out. From this analysis it was determined that the assumed rock mass modulus, shaft resistance and base resistance should be downgraded for all piles. To compensate for this, the more highly loaded piles around the corners of the pile group (12 of the 24 piles) were increased in length to provide reserve geotechnical capacity; and the stiffness of the pile group was conservatively reduced by 20 to 25%. As with Pier 6, this reduction in stiffness was found not to adversely affect pier performance. As an added precaution, a procedure was introduced to lower the pile toe level of any individual pile where SID camera inspection of the base indicated faulted material over a significant proportion of the base area.

Figure 4 - Pier 7 Adopted Geological Model and Pile Design Assumptions

Figure 5 - Pier 1 Eccentric Footing Spanning Fault
5 SPREAD FOOTINGS – SOUTHERN APPROACH PIERS

Spread footings on rock for the southern piers needed to balance bearing capacity requirements with footing flexibility requirements. To reduce bending moments in the piers and stresses in the deck, it was desirable that the spread footings were reasonably flexible in the longitudinal axis of the bridge. The design of footings therefore needed to consider a range of possible rock modulus, and the footings were made relatively narrow in the longitudinal axis of the bridge to make them rotationally less stiff.

During construction, all footing excavations were reviewed by a geotechnical engineer for consistency with the design assumptions. At Pier 1, a wide fault zone that had weathered to the consistency of hard clay was exposed in part of the footing excavation. The bearing capacity and modulus of this fault material was established from plate load tests, and was found to be much lower than adopted in the design of the footing. The design of the pier and deck had already been finalised at this stage. To avoid having to redesign these elements it was necessary to resize the footing so as to provide adequate bearing capacity, while not significantly changing the rotational stiffness of the footing. This was achieved, (after several iterations of different footing shape and sensitivity checks for different modulus materials), by using an eccentric footing as shown on Figure 5.

6 DRIVEN PILES – NORTH APPROACH

For most of the northern piers, the pier height provided sufficient flexibility that the stiffness of the foundation was not a major issue. Due to the large depth of soft alluvium, and the presence of gravels overlying the rock, driven pile groups (as also adopted for the existing bridge) were the preferred foundation solution. Groups of 40 to 45 driven prestressed concrete octagonal piles beneath a 2m deep pilecap were adopted. The number of piles was varied from pier to pier depending on the applied loads, but the pilecap geometry was kept constant for ease of construction.

Hard driving had been experienced in the original bridge, and it was uncertain whether piles would found on the rock or in the dense sands and gravels overlying the rock. Piles needed to be driven in two lengths, and the splice was required to be at least 10m below the pilecap to satisfy the requirements of the project deed. Pile lengths and details were selected so as to provide interchangability in piles, so that the pile length used in a pile group could be selected during construction based on the performance of previous piles in the group to ensure the splice was at the right depth. Piles were dynamically tested to ensure pile capacity had been achieved. As it turned out, the majority of piles penetrated through the gravels to the rock. A notable exception was Pier 10 where about one third of the piles founded in the gravels.

Lateral loads on the pile group were resisted by a combination of portal action in the piles and passive resistance on the pilecap. As a result of the former effect, piles had to be detailed for quite high bending moments with extensive use of reinforcement in addition to the prestressing strand in the upper pile sections.

Piles also had to consider downdrag effects from consolidation of the soft estuarine clays. In assessing the magnitude of downdrag, pile group effects were taken into account. This showed that the maximum feasible downdrag on each pile in the group was between one third and one half of the downdrag on a single isolated pile.
At Pier 16, several piles in the north eastern corner of the pile group went from very easy driving in soft estuarine clay to sudden refusal on rock. Dynamic pile testing indicated that five piles in this area had been broken by the sudden increase in driving stress. By contrast, capacity was achieved more gradually over a penetration of 2 to 3m into the rock over most of the rest of the pile group. It has been hypothesised that a fault in the rock could have occurred at this pilecap. This would explain the differing pile driving characteristics if the materials on one side of the fault were moderate strength sandstone, and the materials on the other side of the fault were weaker mudstones. To reinstate the capacity of the pile group, the pile group was reanalysed taking into account some redistribution of load between piles, and three extra replacement piles were driven in the northeast corner of the group between the broken piles.

7 FLEXIBLE PIERS – NORTH APPROACH

For the northernmost section of the bridge (Pier 14 to Abutment B), the relatively short piers would have generated excessive stresses in the piers and bridge spans due to the restraint provided by a driven pile group. To prevent these high stresses developing, Piers 14 and 17 were designed as a single row of 1.8m diameter bored rock socketted piles. This provided adequate strength and stiffness in for both vertical and transverse loads, but allowed the piers to be relatively flexible in the longitudinal direction.

8 SOUTH ABUTMENT SPILL-THROUGH

The southern abutment of the existing bridge straddles a ridge of rock. The abutment is a concrete box structure to resist the longitudinal loads from the approach span, and the height of approach embankment is about 10m. By contrast, the Second bridge sits on the side of the ridge on a spill-through embankment 24m high from crest to toe. A single row of rock socketted piles was originally proposed to support the abutment headstock, but the rock sockets were changed to spread footings and concrete columns during construction to avoid re-mobilisation of the pile rig. The general arrangement of the abutment is shown on Figure 6.

The abutment headstock supports a 4.5m height of soil, and the load from this soil generates significant shear forces and bending moments in the columns. Passive earth pressure on the front of the columns is required to resist these forces, but this in turn reduces the stability of the spill-through embankment. The columns, footings and embankment were therefore designed using finite difference analysis to ensure that the column bending moments, abutment movements and overall embankment stability were satisfied. Analysis also had to consider the torsion generated in the abutment due to varying column length.

9 NORTH ABUTMENT REINFORCED SOIL WITH ISOLATED FOUNDATION

The approach embankment to the northern abutment is up to 14m high and is founded over soft compressible estuarine clay. To provide adequate stability and settlement characteristics of the embankment, ground improvement piles were extensively used beneath the embankment. To minimise the embankment footprint (and therefore the amount of ground improvement), a vertical sided reinforced soil structure was adopted in the vicinity of the bridge abutment.
Figure 6 - Abutment A General Arrangement

Figure 7 - Abutment B General Arrangement

Figure 8 - Working Platforms and Causeway
A key requirement of the project deed was a bridge design life of 300 years. To date it has not been possible to demonstrate a reinforced soil structure with a 300 year life. It was therefore decided to support the bridge abutment structure independent of the reinforced soil embankment so that in the event the reinforced soil wall became unserviceable at some time in the future, it could be reconstructed without adversely impacting on the bridge foundations. This solution works well, but requires some careful detailing to accommodate differential movements both during construction and in-service.

The bridge abutment piles were installed before the reinforced soil wall was constructed. Although, the reinforced soil structure is supported on ground improvement piles, there is nevertheless a proportion of the embankment load that is transferred to the soft estuarine clay, resulting in both settlement and lateral soil movement. As the four bridge abutment piles are in almost a straight line they are quite flexible in the longitudinal direction (similar to Piers 14 and 17), and the soil movement due to placement of the reinforced soil was predicted to cause sway of these piles. To assess this, the construction sequence was modelled using finite element methods. This analysis indicated placement of the reinforced earth could result in up to 70mm of longitudinal displacement of the abutment at bearing shelf level. The design of the bearing shelf was therefore modified to accommodate this movement.

Although the bridge deck is supported at the abutment on pot bearings, due to the flexible nature of the piles in the longitudinal direction, bearing friction was nevertheless predicted to cause longitudinal movements of the abutment at deck level of around 30mm. Although the structure can readily accommodate the stresses generated by this movement, the run-on slab between the abutment and the reinforced soil would have needed to be fitted with a substantial movement joint to provide a serviceable bridge. Instead it was decided to anchor the run-on slab to the reinforced soil using piles with sufficient lateral capacity to resist the bearing forces, and pin the run-on slab to the abutment such that rotation of the run-on slab to accommodate post-construction embankment settlement could occur, but longitudinal movement was restrained.

**10 ARRESTOR/CONSTRUCTION ISLANDS**

In order to construct the river piers for the Second bridge, temporary construction platforms were constructed out into the river. On the north side of the river a Causeway was also constructed to gain access to Pier 7 (refer Figure 8). In the completed works, the Causeway is to be removed and the construction platforms trimmed back to below low tide level to form arrestor islands around the river piers to provide additional protection from ship impact. This is a significant departure from the existing bridge which used floating plant and a piled causeway to construct the river piers.

These platform works were constructed on very soft estuarine clay which is prone to large consolidation settlement and causes significant problems with embankment stability. This was particularly evident in the construction of the Causeway where, despite use of a high strength geofabric, placement of rock fill caused a large volume of clay to be displaced in a mud wave on either side of, and in front of the advancing causeway (refer Figure 9). For construction of the Causeway and the Pier 6 platform, the adopted methodology was to intentionally cause a mud wave to form to
displace the softest mud, and then allow strength gain and consolidation to occur in order to place fill to the final levels.

To ensure that the completed platform had an adequate stability under construction loads, the platform was subjected to a surcharge load back-calculated to provide a factor of safety of at least 1.3 under the operating condition. On several occasions surcharging induced instability in the leading edge of the works, but it was noted that following failure, the platform could be surcharged without causing further failures. This suggests that the sheared mud rapidly consolidates and gains strength.

![Figure 9 - Causeway Construction](image)

For construction of the Pier 7 platform in close proximity to the shipping channel it was decided that the risk of instability due to intentionally causing a mud wave was unacceptable. Dredging of the softer soils was therefore carried out prior to rock placement.

Consolidation settlements that occur during construction are managed by periodic topping up of the platform. The platform acts as a surcharge on the long-term condition and therefore post construction settlements of the arrestors are expected to be relatively small requiring only minor top-ups as part of rip-rap maintenance.

11 CONCLUSION

A key element in the delivery of the foundation systems for the Second Gateway Bridge was a high degree of co-operation between the design and construction teams, including a continued involvement of the designers through construction. This allowed adoption of observational techniques to modify designs in response to observed ground conditions and test results. As it turned out, there were a few unexpected ground conditions encountered which required more substantial design changes during construction than were anticipated. However, because the team had already been set up to manage construction modifications these changes occurred efficiently without significant delays to the construction program. The end result is a series of elegant design solutions to varying ground conditions which achieve a robust foundation system for the bridge while facilitating constructability.

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