New MetroRail City Project- Overview

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ABSTRACT: The New MetroRail City Project was designed and constructed during the period February 2004 to October 2007 in Perth, Western Australia. It formed the northern part of the Perth to Mandurah Railway project. The project included construction of two underground train stations, twin bored rail tunnels of 1.5 km combined length, and cut and cover tunnels and dive structures of approximately 1 km total length. It is the first major underground construction project within the Perth Central Business District. This paper provides an overview of the project and describes the key features of the undertaking from a risk management perspective.

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1 INTRODUCTION

The New MetroRail City (NMC) Project was completed in 2007 in Perth, Western Australia and involved a major upgrade and expansion of the city's electrified rail network. The project formed the northern part of the Perth to Mandurah Southern Suburbs Rail (SSR), which in total doubled the length of Perth's railway system, included 11 new stations, and added 93 new railcars onto the network, involving a total capital works budget of more than A$1.6 billion. The SSR linked the rapidly growing South-West Corridor with the Perth CBD and its existing rail network.

The work was administered by the Public Transport Authority (PTA) on behalf of the Government of Western Australia.

The NMC Project was constructed by Leighton Contractors under a A$334 million design, construct, and maintain contract with PTA. Leighton sub-contracted the whole of these works to the Leighton Kumagai Joint Venture (LKJV).

The NMC Project included construction of two underground stations, twin bored tunnels, cut and cover tunnels and dive structures. The City Project built approximately 2.7 km of new railway from Roe Street to the Narrows Bridge, as shown in Figure 1. It was the first major underground construction project within the Perth CBD, apart from multi-level basements for high-rise structures.

The NMC Project permanent works were designed by Maunsell-GHD with Independent Verification of these designs by SKM-SMEC. Specialist advice, in the development of geotechnical design parameters and seismic modelling was provided by Coffey. Temporary works were designed by Geoconsult (Singapore) and BG & E, with Independent Verification of these designs by Golder and Van der Meer Group. Specialist design advice was obtained by PTA from WorleyParsons and Connell Wagner.

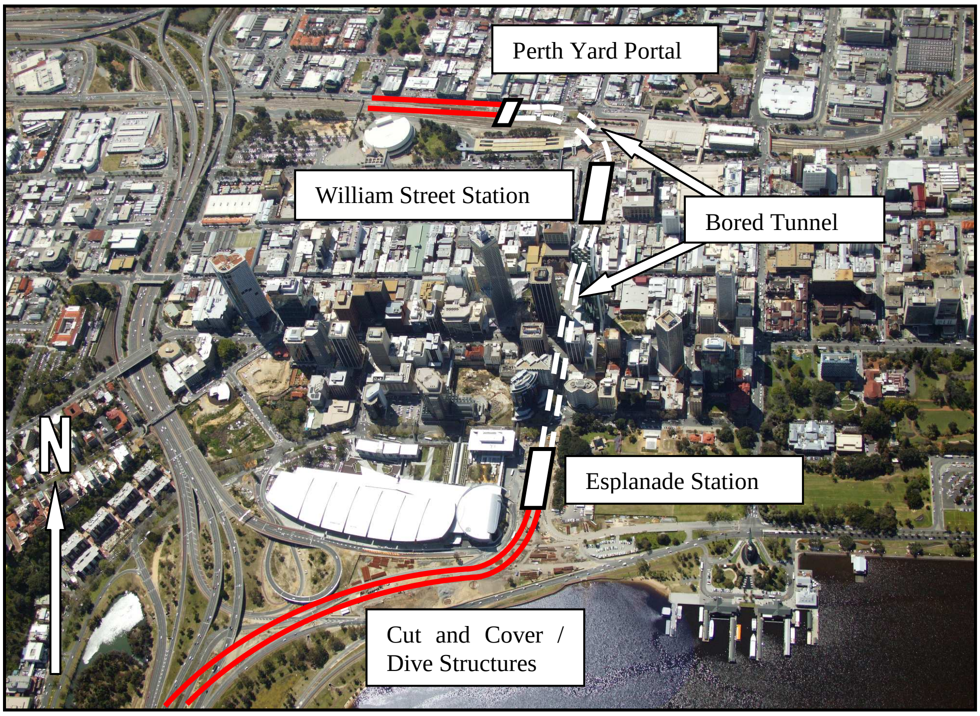

Figure 1: NMC Project Site Layout

2 PROJECT BACKGROUND

As shown on Figure 1, the northern end of the route commences within the Perth Railway Yard adjacent to Roe Street. It then extends eastward within the rail reserve and the adjacent Roe Street road reserve before turning to the south in a 135m radius curve and passing beneath the existing rail tracks and the heritage-listed Horseshoe Bridge that carries William Street over the rail tracks. South of Wellington Street, the alignment is located initially to the east of the William Street road reserve in land that was purchased by the WA State government to facilitate project development, and has since been on-sold to a private developer to develop as high rise towers above and adjacent to the William Street underground station. The project alignment then passes beneath the Murray Street mall and four city buildings ranging from 2 to 5 storeys in height, that are supported on shallow footings, generally with one basement. The tunnels are then located beneath the road reserve until reaching Mounts Bay Road. The alignment then turns to the west beneath the William Street road overbridge (since demolished) and passes through the Narrows Interchange towards the Narrows Bridges. The route passes beneath the southbound Kwinana Freeway carriageway and approaches the Swan River between the two Narrows Bridges. Chainages along the route are referenced to Chainage zero (Ch 0) in the centre of the William Street Station, with increasing chainages to the south expressed as Perth to Mandurah (eg Ch 850PM) and increasing chainages to the north as Perth to Butler (eg Ch 220PB).

The significant underground features of the project include:

- William Street Station, extending over a length of 139m and with rail level located approximately 17m below ground. The station is linked to the existing central Perth Train Station by an underground concourse beneath Wellington Street. The station was formed using diaphragm walls and top-down in-situ concrete construction methods.
- The Esplanade Station, extending over a length of 138m and with rail level located approximately 9m below ground. This station was constructed bottom-up using in-situ concrete constructed within temporary sheet pile walls.
- Roe Street cut and cover tunnel and open dive structure, extending over a length of about 300m. At its deepest point, the rail level is approximately 9 m below the ground surface. The Roe Street structures were built of in-situ concrete inside temporary and permanent sheet pile walls, generally using bottom up methods. In the deeper, strutted section of the Perth Yard Dive structure a semi-top down construction method was used.
- Twin bored tunnels extending from the eastern end of the Roe Street cut and cover section to the William Street Station and from there through to the northern end of the Esplanade Station. Within the bored tunnels, the rails are between about 9 and 21 m below ground surface. The total length of bored tunnels is approximately 1.5 km. The 6.16m internal diameter bored tunnels have a 275mm thick segmental lining.
- Foreshore cut and cover tunnel, open box structure and dive structure, extending from the southern end of The Esplanade Station until the rail track rises to ground surface between the two carriageways of the existing Kwinana Freeway. These structures were nearly all formed of in-situ concrete within temporary retaining walls.

Major components of the project were required to be designed for a design life of 120 years. Particular durability requirements were specified in the NMC Project Scope of Work and Technical Criteria. Limits on long term seepage into the underground structures were specified, requiring care with design and construction of waterproofing details. Limits were specified on long term rail movements, requiring substantial quantities of piles to resist vertical loading (including compression due to ongoing ground subsidence in the reclamation), tension forces (buoyancy forces on underground structures) and lateral loads (lateral movements of reclamation fills). Minimum monitoring requirements were specified to measure movements of buildings, infrastructure and services, and ground and groundwater levels. Limits were placed on changes to groundwater levels.

Topography along the route ranges from ground levels at about 2 m above sea level in the foreshore area to about 15 m above sea level in the CBD where the bored tunnels are at their deepest, to about 11 m above sea level in Perth Rail Yard where the project rejoins the existing rail network.

3 HISTORICAL DEVELOPMENT ALONG THE ROUTE

European settlement of the Swan River Colony, which later became the City of Perth, occurred progressively during the 1800's. Several major developments associated with the growth of the city had the potential to impact on the project and are described in this section.

3.1 *Old Northbridge lake system*

The extent of lakes in the vicinity of the northern section of the project alignment taken from a plan dated 1838 is shown in Figure 3. Lake Kingsford is seen to cover much of what is now the Perth Central Train Station. In particular, a section of the new rail alignment adjacent to Roe Street is within the original lake area, covered by Lakes Kingsford, Sutherland and Irwin.

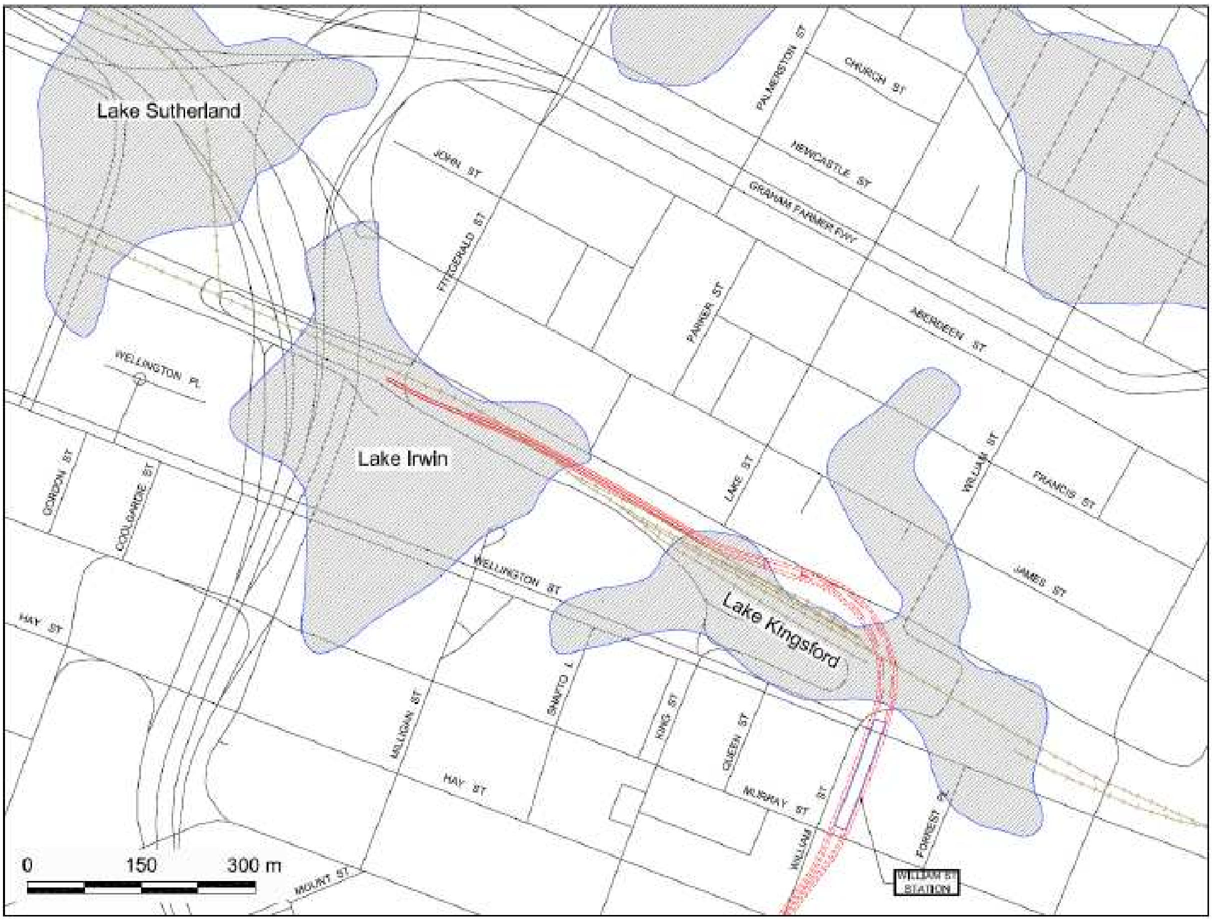

Figure 2: Old lakes near project alignment

In some old lake areas thicknesses of 1 to 2 m of organic peaty material formed. The organic materials were generally at shallow depth (1 to 2 m maximum) since the lakes were formed by groundwater exposed in shallow interdunal swales. Filling of these lakes to eliminate mosquito problems and create more land for development in the mid 1800's resulted in variable ground conditions. Subsequent land use as railway yards for the past 150 years has caused the old lake infilling to generally comprise sand with traces of organic matter, occasional zones of peat and railway ballast, cinders and timber sleepers.

3.2 Foreshore reclamation area

Over a period of about 150 years, a substantial part of the Perth foreshore was reclaimed from a large tidal estuary known as Perth Water. The Esplanade Station and foreshore cut and cover sections of the project are located within this reclaimed area. Early development of this area during the mid to late 1800's included the construction of a filled causeway leading to a timber jetty. A channel was later dredged and formed to improve navigation to the jetty. By about 1900, filling of the Swan River had extended out about 100 m south of the original shoreline and on both sides of the original causeway. These features are shown in Figure 3. This area was used for a number of purposes, including for the support of military aircraft during World War II.

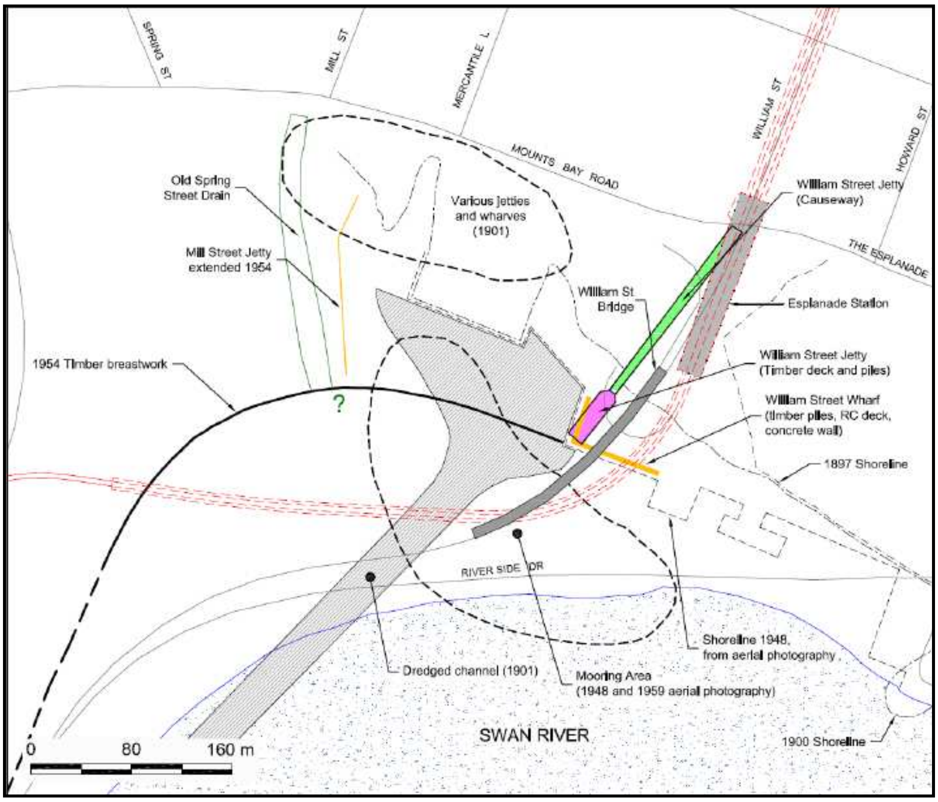

Figure 3: Historical development in foreshore area

In 1954, reclamation of an area of about 29 hectares was commenced to enable construction of the Narrows Bridge. Reclamation was carried out by placing sand hydraulically behind a perimeter bund. It is understood that the bund was formed by first dredging oyster shells as light weight fill and then covering this with about 2 m of sand. This bund was subsequently penetrated by timber sheet piling breastwork. This so-called "old" reclamation, was essentially completed by January 1958. A large proportion of the hydraulically placed fill comprised estuarine mud capped by a layer of sand at about RL 2.5 m.

A "new" reclamation of an additional 8 hectares area followed the "old" stage in order to accommodate the Narrows Interchange road works. The "new" reclamation continued until about 1970. To accelerate the large settlements expected across this area from embankment loading, the compressible foundation soils were surcharged. Some 43,000 vertical sand drains each of about 450mm diameter were installed in 1964/1965 to accomplish this.

Parts of the original river bed were observed to have settled up to 7 m under surcharge loads, contributed by filling about 5 m above final design levels. The total depth of fill including surcharge since reclamation commenced, in some places, exceeded 20 m. Residual (secondary) settlements of up to 0.7 m or more were expected over a long period of time following construction. This magnitude of estimated settlement is consistent with the results of subsequent settlement monitoring.

3.3 Old ground anchors

More than 150 ground anchors were known to exist beneath William Street along the alignment of the proposed bored tunnel. The anchors were installed for temporary support of basement retaining walls for possibly 6 buildings. Anchors typically comprised twisted strands of high tensile steel wire grouted with cement (over fixed lengths of 6 to 10 m) within holes of about 80 mm diameter and with free lengths of about 8 to 10 m. More than one row of anchors were employed on deeper excavations. It was reported that the anchors were de-stressed as basement construction progressed, however, this practice may not have been followed in all cases.

4 SITE INVESTIGATIONS

The geological and geotechnical setting for the NMC Project works is complex, as may be seen from only a brief consideration of the geological description and history of the site. Six phases of site investigation were carried out for the project as summarised in Table 1. Phases 1 to 4 were managed by the PTA. Phases 2 and 3 involved input from tendering consortia. Phases 5 and 6 were managed by LKJV. A substantial Geotechnical Interpretive Report was prepared by LKJV at the completion of the Phase 5 investigations, summarizing all findings and recommendations for design at that stage.

Phase	Timing	Details
1	Prior to calling for expressions of interest	Scoped and commissioned by PTA
2	During Expression of Interest (EOI) stage	Scope proposed by five EOI participants and commissioned by PTA
3	During the Tender design stage	Scope proposed by two Proponents. Partial scope commissioned by PTA
4	During Tender evaluation stage	Scoped in consultation with Preferred Proponent. Partial scope commissioned by PTA
5	Immediately following award of contract	Scoped and commissioned by LKJV
6	During construction	Scoped and commissioned by LKJV

Table 1: Phases of site investigation.

Separate investigations were conducted by environmental consultants into the occurrence and characteristics of acid sulphate soils and contaminated materials and groundwater along the route.

5 GEOLOGICAL MODEL

The historical borehole records, six phases of site investigation and construction boreholes resulted in a total of approximately 900 locations that were test pitted, drilled or probed to provide geological, geotechnical and/or hydrological information for the project. This total figure includes approximately 200 historical drill holes and CPT probes.

During the Phase 1 to 5 investigations the geological model of the site was subdivided into 6 to 8 units (depending on investigation phase). These engineering geology units with an assigned symbol (eg SS, Spearwood Sand) were used throughout the project for design and construction purposes. An example of a geological section produced during the project is shown in Figure 4.

During the construction period several variations to the geological model were created in response to observations made during excavation and for construction requirements. These later variants identified up to 18 separate geological units and interpreted several new geological systems. These later interpretations were developed from bored tunnel spoil logs and observations made during excavation of the TBM launch box.

The refined geological models were used for back analysis and predictive work. Refined geological models were not continuous across the project with separate models created for the Esplanade, bored tunnels, William St Station and Perth Rail Yard.

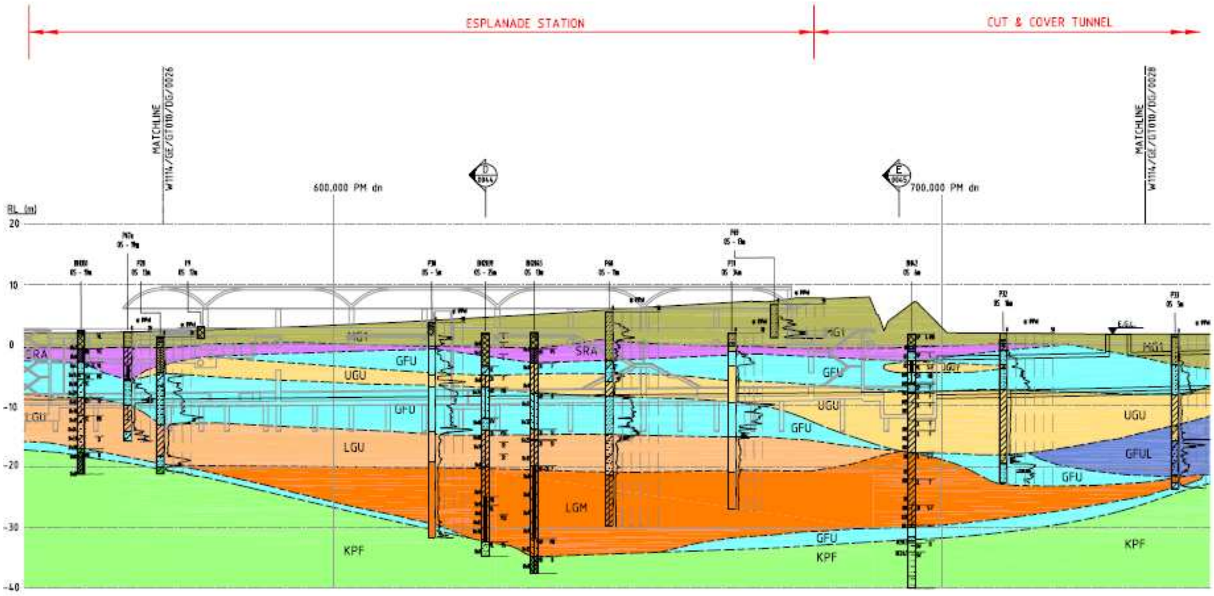

Figure 4: Typical Geological Long-Section prepared prior to construction, showing anticipated ground conditions around Esplanade Station.

Legend: MG - fill material (Made Ground) - comprises sand with varying amounts of refuse and other materials.
SS - Spearwood Sand - comprises a quartz aeolian sand that forms a ridge in the CBD with a maximum thickness of approximately 8m with the highest point between Hay and Murray Streets.
LSA - Lake System Alluvium - comprises thin layers of dark brown to black, organic silty sand, clay and silt.
SRA - Swan River Alluvium – comprises quartz sand that underlay all the areas that was previously part of the Swan River.
GFU - Guildford Formation Undifferentiated - comprises a clayey upper section underlain by sandy material.
Kngs Park Formation - comprises argillaceous sandstone with rounded clear quartz sand in a dark grey-brown shaly matrix, or dark grey to black, clayey to sandy siltstone.

6 GEOTECHNICAL HAZARDS

A number of geotechnical hazards relating to uncertainty and unknowns were identified in the earlier phases of investigations, which were not able to be directly quantified within the geotechnical recommendations. These were all managed by further investigations and analysis during construction, and were subsequently encountered and managed during construction. Broadly they fell into the categories of unforeseen ground conditions or materials and unexpected variability in geotechnical or hydrogeological parameters. These included:

- Existing disused ground anchors in the William Street road reserve that may be encountered during tunnel boring. In the event, the vertical and horizontal alignment of the bored tunnels was successfully adjusted during the design process to pass beneath these obstructions, and the TBM tunnelled up William Street twice without incident, but the William Street compensation grout construction shaft encountered a number of anchors in front of a building that had not been previously identified as having basement anchors. Temporary sheet pile wall construction was successfully completed for this 6m deep shaft.
- Ground movements due to excavation and dewatering could have potentially caused damage to buildings and other infrastructure. Considerable effort was put into managing these risks.
- Ground conditions were expected to be variable in terms of both material types and material strengths, particularly in certain areas along the alignment. In the foreshore, paleochannels incised into older formations were known to have subsequently filled with younger sediments, the more problematic infilling being the Swan River Alluvium (SRA) on the foreshore. Design of foundation systems considered future long term 100 year settlements (up to 250mm) and lateral movements (up to 70mm) in foreshore paleochannels and overlying ground. Over 350 bored piles of 750mm diameter socketed into the KPF were used to support Esplanade Station, cut and cover and open dive tunnel structures through the foreshore reclamation.
- The highly variable Guildford Formation and Spearwood Sands were known to contain cemented zones with strengths equivalent to very low to low strength rock although higher strengths can also occur. Cemented zones are generally less than 1m thick and could have affected diaphragm wall and sheet pile installation. Occasional driving difficulties were experienced with sheet piles. Preboring or predriving was used to assist penetration in such conditions.
- Damage to older buildings founded on relatively poorly compacted ground or poorly constructed footings along or near the alignment could have occurred due to vibrations induced during construction or subsequent operation of the railway. Significant investigation, monitoring and management of such risks was incorporated in the project.
- The presence of old structures buried within the foreshore reclamation and possibly elsewhere was identified. Buried structures included:
    - Sheet piles or other steel or concrete parts of former building foundations and retaining structures
    - Retaining structures associated with previous Swan River foreshore reclamations and rubble used as reclamation material (also within Lake Kingsford)
    - Abandoned old wells
    - Drill rods from site investigations

  Old timber jetty piles in the foreshore required removal during excavation of cut and cover tunnel works, and a 24mm diameter steel bar was encountered by the TBM in Perth Rail Yard and had to be removed from the screw conveyor within the TBM.
- Difficulties were anticipated with grout takes and extent of permeation within variable or interbedded soils. Grout set may be inhibited by relatively small organic contents in the host soil. Later laboratory trials indicated this was unlikely to be a concern and it did not eventuate on site, during the considerable grouting and deep soil mixing activities undertaken.
- Variability in groundwater conditions was anticipated. It was considered that a number of confined aquifers existed within the Guildford Formation. The continuity of these aquifers could not be quantified with any certainty until construction dewatering commenced. This

was further complicated by evidence that existing drained or pumped basements and irrigation bores had an effect on the natural groundwater environment. This variability meant that predictions of construction inflows and drawdowns, as well as long term groundwater levels contained uncertainties.

- Discharge of pumped dewatering groundwater to sewer or stormwater system was required. Discharge water quality may be a significant practical issue. Water treatment plants were established around the site. Wherever possible, groundwater was returned to aquifers as recharge, but on occasions excess groundwater was discharged to sewer or stormwater systems as appropriate.
- Previous slope failures and resulting disturbed, lower strength ground zones around the edges of the reclamation were identified, particularly where deeper SRA deposits existed. Design of temporary and permanent works considered these possibilities. No additional issues arose during construction.
- Running sands below the water table in MG, GF and SS units were identified as a construction risk. Several sinkholes formed up to 2.5m in diameter in sands at locations of ground loss (eg minor breaches in sheet pile walls, joins between dissimilar walling such as sheet piles and grouted soil masses). No such events were experienced during TBM tunnelling. Sinkholes were repaired by backfilling and cement grouting.
- Contaminated materials and acid sulphate soils (ASS) were identified on site. Significant construction efforts were required to manage these issues.

7 BUILDING PROTECTION

The NMC Project tunnels and underground structures were built in close proximity to and underneath a wide range of city buildings, infrastructure and underground services. It was recognized that elimination of ground movements and damage to adjacent facilities was not possible. However, state-of-the-art practices existed for design and construction whereby such ground movements and damage could be minimised within tolerable limits that would be acceptable to project stakeholders including building, infrastructure and service owners, the PTA and the project insurers. A risk sharing approach was adopted in the Project Deed, whereby a defined low level of incremental building damage caused by the project was allowed. Rigorous pre-and post-construction condition surveys of nearby facilities were used to assess any incremental damage and repairs were carried out to reinstate facilities to their pre-existing condition, with cost sharing for repairs between LKJV and PTA under certain circumstances. Monitoring and protection of vulnerable facilities as determined from pre-construction assessments was required to manage the associated risks.

Building protection was undertaken as follows:

- Prepare schedules of all existing buildings, infrastructure and services within or partly within predefined monitoring/assessment zones

- Estimate likely ground movements due to cut and cover excavations and bored tunnelling

- Estimate construction impacts on buildings, infrastructure and services

- Identify critical areas or structures where damage may exceed defined allowable levels

- Design and install required protective works to limit degree of damage in critical locations

- Design and install arrays of building, surface and subsurface instrumentation and management systems

- Manage a system of pre- and post-construction condition surveys on nearby facilities

- Undertake repairs to damaged buildings, infrastructure or services.

The management system required for building protection included the following documents:

- Geotechnical Interpretive Report
- Ground Settlement, Building Protection and Repair Plan
- Instrumentation and Monitoring Plan
- Building Protection Management Plan
- Property Condition Surveys
- Building Protection Assessments
- Various Method Statements and Safe Work Methods

7.1 Estimation of likely ground movements

Greenfield ground surface movements were estimated using current theory for excavations and bored tunnels. Analysis included the effects of subsurface conditions, geometry of structure to be built, type of excavation support, method and sequence of construction and any ongoing background ground settlements unrelated to the project works. Where appropriate, superposition of construction effects was used to determine overall likely ground movements. This included, for example, the construction of the second bored tunnel adjacent to the first bored tunnel, or at interfaces between bored tunnels and open excavations.

The transverse surface settlement profile due to bored tunnelling was assumed to have the form of an inverse Gaussian normal distribution curve (RJ Mair, et al 1996, O'Reilly et al, 1982 and 1991, Peck

1969). The maximum surface settlement that would occur along the tunnel centerline was assumed to be directly related to volume loss and inversely related to horizontal distance of the inflection point of the settlement trough from the tunnel centreline. Based on the maximum surface settlement, the vertical and horizontal ground movements at arbitrary points in space were calculated using the methods described by O'Reilly et al (1991). The longitudinal settlement trough was assumed to correspond to a cumulative probability curve as described by Attewell & Woodman (1982).

Volume loss parameters for design, volume loss requirements for building protection, and additional volume loss due to tunnel curved alignment were assessed. These volume loss parameters were supported by the results of numerical analyses. The results considered ground movement analysis for a regular design case, a ground treatment case for areas where building protection was required, and an exceptional case which involved a large volume loss value to allow for an accidental drop of TBM face pressure to 50% of target pressure.

Excavations were supported by the following retaining wall systems:

- Temporary steel sheet pile walls with struts
- Combination of temporary steel sheet pile walls with soldier piles and struts
- SMW (soil mixed wall)
- Diaphragm walls

Numerical software packages (such as FLAC, PLAXIS, WALLAP, SEEP/w and MODFlow) were used for the assessment of wall deflections and seepage analyses associated with excavations and support systems. Where appropriate, undrained behaviour of soils was considered in the analysis. Additional analyses took into consideration the effects of groundwater dewatering and recharging measures.

The type and scale of excavations as undertaken in the project, had not been carried out in Perth before and correlation of analytical studies to actual case studies was not readily available. A semi-empirical approach was used to derive surface settlements from numerically calculated deflections of retaining walls. There are a variety of methods for predicting the magnitude and distribution of ground movements caused by braced open cut excavations. The semi-empirical method adopted for settlement analysis was mainly based on that proposed by Caspe (1966) and modified by Bowles (1997). Using this approach, additional deformations due to installation/removal of retaining walls can also be considered. The analysis assumed good workmanship and early placement of strutting. The approach included an allowance to increase the settlement to model the extraction of sheet piles.

The likelihood of settlements exceeding predicted values was considered. Construction methods and sequences have a significant influence on ground surface settlements for both bored tunnels and cut and cover excavations such as tunnels and stations. For example, if construction methods or sequences change, wall deflections would probably change from the design estimates. The effects of changed construction procedures were reviewed in relation to previously estimated ground movements. The results of ground movement monitoring were also continuously reviewed in relation to earlier estimates of ground movements due to construction. A hierarchy of management procedures and emergency actions were put in place to manage and control unexpected exceedances of predicted ground movements by construction related changes.

7.2 Assessment of construction impacts

Construction impacts on road surfaces, footpaths and kerb lines were assessed based on anticipated surface settlements and related horizontal movements. The project aimed to limit construction impacts to within preset damage categories.

The assessment of potential damage to services and utilities was based on calculated ground displacements and, for rigid utilities such as pipes, on the sectional forces derived from the respective greenfield settlement profile.

The methodology for building damage assessment was based on limiting tensile strains using the approaches of Mair et al, (1996). For each property, a Building Protection Assessment was undertaken that considered the summation of damage impacts from the:

- Initial damage category (taken from the Property Condition Report (PCR)); plus
- Predicted maximum damage due to construction.

The result was the maximum damage category that could be expected. Building protection was required if the "incremental" damage due to construction exceeded the following limits:

- For heritage structures – very slight (up to 1mm crack width);
- For other structures – slight (up to 5mm crack width).

The main structures that required protective measures were:

- Wellington Building (see paper "Structural Design Of Stations, Cut And Cover And Dive Structures" in this seminar)
- Mitchell's Façade (sawn into 9 pieces and temporarily removed and stored off site)
- Horseshoe Bridge (timber propping under brick arches)
- The four buildings under which the TBM passed - Walsh's, Hungry Jacks/KFC, HBF and Friendlies Chemist
- Perth Station western concourse footbridge (portal frame knee joints temporarily released to give rotational freedom)
- Claisebrook Main Sewer (new plastic lining inserted along length considered potentially "at risk").

Despite the project damage performance criteria being based on "incremental" damage, some buildings already had damage in the Severe category of Table 4, prior to the project works commencing. In these cases the relevant property manager was notified and remedial works recommended.

7.3 Post-Construction Condition Reports and Repairs

No major (catastrophic) problems were encountered with damage to buildings, infrastructure or services due to the construction works. Ground and structure movements adjacent or above the works were typically in the range 0 to 5mm, with some buildings observed to move 5 to 20mm and several structures were observed to move 20 to 30mm. Observed building damage due to the works ranged generally negligible to slight in a few isolated cases.

Post-construction condition reports were conducted on over 30 buildings where either complaints of alleged damage due to construction were made, or minor damage was known/suspected to have occurred. At the time of writing, repairs were being undertaken to several facilities and owner sign-off on post-construction reports was progressing.

It is considered that the building protection programme on the project achieved its primary goals of minimising the effects of construction on the CBD environment, within reasonable bounds and applying best-practice.

8 MAJOR TECHNICAL RISKS

Major technical risks were embedded in the NMC Project. These arose from:-

- the complex, variable, saturated, unconsolidated and unstable ground conditions;
- the possibility of excessive surface settlements resulting from underground excavation, causing damage to structures constructed above ground, including the retention, without damage, to historic buildings and façades;
- the uncertainties associated with specification of the various operational settings for the TBM in order to achieve minimal ground loss and consequent surface settlement;
- the uncertainties associated with the requirement to cope with obstructions in the ground, such as buried ground anchors and undocumented underground constructions, while still maintaining an acceptable rate of advance of the TBM. It is significant that the locations of these obstructions were generally unknown prior to the commencement of excavation, adding to the uncertainty, risk and complexity of dealing with them;
- very close spacing of the rail tunnels and in particular the influence of excavation of the second tunnel on the construction of already completed works;
- excavation through acid sulphate soils;
- the control of water and the ingress of soil into excavated voids during launching and arrival of the TBM. The ingress of these soils carries the further risk of causing ground subsidence and in extreme cases the formation of sink holes with the attendant risk of damage to surface structures;
- the very tight contract time frame. It was notable that this required a construction period which was significantly shorter than standards accepted elsewhere in the world;
- The Guildford Formation through which much of the bored tunnel was constructed is a highly variable unit with clay and sand regions. This high variability could easily have led to steering difficulties for the TBM, increased ground loss and the imposition of uneven loading on the tunnel lining.
- Coupled to the variability of the Guildford Formation, was the potential for and the reality of variability in the groundwater conditions. During construction it was confirmed that a number of confined aquifers exist within the Guildford Formation. It was not possible to confirm the continuity of these aquifers with any certainty until the construction dewatering commenced. This meant that the pumping and dewatering requirement for the project could not be known with certainty until the project was underway.

9 PROJECT NARRATIVE

In designing and constructing the NMC Project works, Leighton and its design team had to deal with a range of ground conditions, from soft clayey materials, largely in the paleo-channel and the interdunal lake deposits, sandy soils of the remnant dune systems, as well as encountering the harder Kings Park Formation in some locations.

In addition to the different materials encountered along the alignment, a complex system or groundwater regimes also had to be dealt with. At many locations excavation and tunnel construction had to be carried out in saturated ground conditions.

This variability in natural materials and groundwater regimes encountered along the alignment contributed to both the complexity of the overall construction task and the significant level of uncertainty that had to be managed by Leighton during construction of the project.

9.1 Foreshore and Esplanade Station

The underground station at the Esplanade, and the foreshore and Perth yard dive structures, were constructed using the ‘cut and cover’ method.

The major technical challenges in this section of the project included:

- The need to stabilise the very soft alluvial paleo-channel soils and fill in which open and cut and cover sections of the rail tunnel were excavated along the foreshore;
- The use of deep soil mixing to stabilise soils at depth along the tunnel alignment which were originally of variable strength, stiffness and consistency;
- The innovative use of low vibration techniques to install sheet piling in the Perth Yard dive structure;
- The resolution of difficulties arising from the need to construct a freeway underpass with a large skew angle;
- The unexpected need to deal with highly variable depths to rock while still ensuring adequate toe resistance of the driven sheet piling used to retain soil beyond the excavated cut and cover tunnels;
- The need to balance earth pressure forces acting on the TBM launch box constructed adjacent to the Esplanade station, and the general need to ensure stability of the ground in this general area;
- The need to provide dewatering of the excavations while at the same time minimising surface settlements of the surrounding ground; and
- Recognition of the fact that for many of the permanent structures the critical design loading would probably occur at some interim stage during construction, not necessarily at the end of construction.

Significant technical risk was associated with the foreshore works because of the need to deal with difficult ground conditions during the cut and cover operations, including unstable sandy alluvium and reclaimed ground, as well as the need to dewater the works during construction. It was also necessary to provide adequate resistance to uplift forces acting on the floor slab of the tunnel structure due to groundwater pressures. These challenges were met by the use of a particular technique of cut and cover construction that has had only very limited use, if at all, previously in Australia.

Deep mixing of cementitious materials with the alluvium and fill was conducted prior to construction of the tunnel section in order to provide a stronger, more stable layer beneath the excavated cut and cover tunnel. While this technique was used quite extensively in the foreshore region, its use was first trialled in the Perth Yard area as part of the box structure constructed to receive the TBM at the completion of its drilling of the tunnels.

A further source of technical risk in the foreshore area was associated with the Kings Park Formation (KPF), the hard rock layer that underlies the fill and the paleo-channel in this area. The variability in the depth to the KPF meant that at some locations the sheet piling driven to retain the cut and cover excavation was prevented from being driven to the design depth because of the high resistance provided by the KPF.

The underpass structure constructed beneath the freeway in the foreshore area is a boxed tube structure consisting of cast in place secant piles forming the side abutments and cast in situ base and deck slabs. A defining feature of this structure is the fact that it is skewed in plan with respect to the freeway. Indeed, the skew is so extreme it has resulted in a significant misalignment of the forces acting on the two sides of the box structure arising from the lateral earth pressures acting behind the abutments. This misalignment would tend to rotate the structure as a rigid body about a vertical axis and would possibly tend to distort it as well.

This defining feature of the structure presented a unique challenge for the designers of the underpass and required them to conduct sophisticated two- and three-dimensional finite element analyses to demonstrate that the overall rotation of the structure about the vertical axis would be within acceptable levels. A consequence of this particular investigation was a significant change to the original concept design, most importantly identifying that the use of tie anchors to resist the rotational moment caused by the earth pressure loads was unnecessary. Sophisticated numerical analysis indicated that friction between the abutment walls and the soil behind the walls was capable of providing adequate rotational resistance.

Another consequence of the skew of the underpass was the large torsional moment that had to be resisted by the deck slab. Sufficient resistance was provided in the design by the innovative use of the concrete barrier located at the freeway edge to help resist this torsional moment. The barrier was stiffened by using a much higher density of steel reinforcement than would normally be the case.

Construction of the box excavation used to launch the TBM on its northbound journey towards the William Street station also required the adoption of an innovative design solution. At the concept design stage it was envisaged that the TBM launch box and the Esplanade station would occupy the same excavated cavity. However, for various reasons it was decided at the design development stage that the main cavern of the station and the launch box for the TBM would need to be separated. The revised design of the cut and cover excavation of the launch box generated additional complications, which included the prospect of a major imbalance in earth pressure forces acting on the driven sheet pile retention system. Potentially this imbalance would arise when excavation commenced in the cut and cover tunnel section between the launch box and the Narrows Bridge. Much of the balancing earth pressure force acting on the southern side of the launch box would be removed as the cut and cover tunnel was excavated.

Leighton adopted a unique and ingenious solution to this problem, which carried some attendant technical risk. Research in the form of sophisticated structural and geotechnical analysis was required to ensure that the earth pressures acting on the northern side of the launch box could be resisted by friction forces developed along a length of the cut and cover tunnel. The launch box and the sections of the sheet pile walls adjacent to the launch box had to be strengthened to allow adequate transmission of the longitudinal force generated by this arrangement.

It is also noted that problems were encountered during the driving of sheet piling used to support the ground surrounding the launch box. At some locations the surface of the hard Kings Park formation was encountered up to 5 m above the level expected from interpretation of geotechnical bore logs and soundings. This presented a problem with ensuring adequate toe support for the sheet piles, so that they did not “kick in” after the soil inside the launch box was excavated. This occurred despite an intensive geotechnical investigation prior to construction. This occurrence highlights the high technical risks that are often associated with major excavation works of this type and the need to adopt a flexible, adaptable approach to resolving the geotechnical issues.

9.2 Perth Yard

Design and construction of the dive structure in Perth Yard also had associated with it significant technical risks and posed challenges that required the development of innovative solutions. This structure was constructed using a modified top down approach combined with the novel installation of the “temporary” earth support system. Sheet piling was used instead of the more usual diaphragm walls to support the sides of the excavation, and the sheet pile sections were installed using a vibration method (the ‘Giken’ silent piling system) rather than the more usual method of pile driving,. This method of installation was adopted because of the need to minimise noise and the dynamic effects of pile driving on neighbouring buildings and the working railway system. Trials were conducted by LKJV of both the Giken piling system and the use of deep mixing techniques to strengthen and stabilise the weak soils beneath the dive structure. The need for this testing provides further evidence of the significant technical risks associated with these works.

The sides of the excavation required to construct the dive structure were eventually supported by permanent concrete walers and struts. A novel feature of this structure is the use of the sheet piles as permanent tension anchors to resist the uplift forces arising from the action of the groundwater pressures on the floor of the dive structure. This is an unusual use for sheet piles; they are normally used only to provide temporary support against lateral loads. However, by adopting this approach Leighton avoided the need to remove the driven pile sections and the need to install a separate system of anchors to provide the required uplift resistance.

Construction of the dive structure within the very tight constraints imposed by a working railway system also called for unusual construction methods and procedures. The use of the Giken system to install the sheet piling overcame the need to use large pile driving equipment, which would have been very difficult within the constraints of the Perth Yard site.

9.3 Tunnelling

Twin bored tunnels were constructed using a Tunnel Boring Machine (TBM) under the Perth CBD from Esplanade station to Perth Yard. The TBM was required to pass through saturated soils and highly unstable ground beneath and close to numerous existing structures. It was a specific requirement of the project to preserve these existing structures and to minimise damage to them as a result of the tunnelling works.

The twin tunnels are unique in several ways. They are the first twin tunnels to have been drilled through soft, saturated and sandy ground conditions in Australia and, significantly, they are relatively shallow and pass beneath numerous existing structures, as previously mentioned. These unique features have required Leighton to research and develop technical innovations to ensure the project could meet all contractual requirements, particularly those related to ensuring that surface structures are minimally affected by the tunnelling.

The earth pressure balance (EPB) system was used to support the ground at the cutting face of the TBM. In this method the pressure of the drilling mud is used to control the stability of the ground beyond the tunnel face. While the EPB method has been used previously, the pressure setting adopted is critical in controlling the effects of the advancing tunnel excavation on the surrounding ground and any surface or buried structures close to the advancing tunnel face. For example, if the pressure were too high, heave of the ground surface could result. Conversely, if the pressure is allowed to reduce significantly below its target setting, excessive ground settlement could occur. In both cases there would be significant consequential risk of causing damage to surface structures.

The selection of an appropriate pressure in the EPB method is usually site specific. It is also important to note that the target pressure may not remain constant throughout the tunnelling because ground conditions can and usually do change along the tunnel alignment.

To ensure that an appropriate pressure is always maintained requires the use of careful monitoring of the ground movements, including surface movements, and the adoption of a feedback loop to make the necessary adjustments to the TBM operational settings when required. Leighton adopted a very comprehensive monitoring system that was innovative in a number of ways. It included monitoring of both ground and building movements using a range of instruments. Sophisticated and very sensitive instrumentation was deployed, so sensitive in fact that it was able to detect an Indonesian earthquake of magnitude 7.6 that occurred in the Banda Sea on the 28th January 2006.

In addition to careful management of the face pressure during tunnelling with a TBM using the EPB method, the material being excavated must remain in a workable condition, so that it can easily be removed using the TBM’s muck extraction system. At various locations along the twin tunnels the excavated soil had to be pre-conditioned to allow for its easy removal. This was achieved with the use of foams and other substances including polymers being injected into the ground at the face.

Behind the advancing TBM the ground is supported and the tunnel cross-section is maintained by placement of segmental concrete ring structures. These rings also provide sufficient longitudinal resistance to the TMB as it “pushes” its way forward during the drilling operation. Placement of the rings is also a critical operation in terms of controlling potential ground movements and thus it has attendant risk. The concrete ring segments used in the twin tunnels were required to comply with a strict specification with regard to size, strength and other mechanical and hydraulic properties. So tight was this specification with regard to water permeability (less than or equal to 10^{-12} m/sec) and strength, that the manufacturer (Humes) was required to develop a special concrete mix for this purpose.

The use of the TBM technology requires the construction of relatively large chambers in the ground from which the TBM is launched and at which it is received at the completion of drilling the required length of tunnel. The launch and recovery (arrival) of the TBM were specific operations associated with high technical risk, with consequent commercial risks. The major technical risks arose from the need to prevent excessive ingress of groundwater and soil into the launch and recovery chambers. Excessive ingress of these materials would result in ground subsidence and the potentially damaging consequences for surface structures.

Leighton managed these risks using a range of innovative and technically advanced methods, some of which, were used for the first time in Australia, and probably elsewhere in the world. In the main, Leighton adopted ground improvement techniques for this purpose, including the use of glass fibre re-inforcement of the ground, the use of deep mixing with cementitious materials effectively to form gravity retaining structures in the ground, and the use of rubber rings attached to the station blocks to help form a seal at the launch and recovery chambers.

Another technical innovation in the tunnelling phase was the use of novel technology to “sense” the presence of buried objects such as old ground anchors, wells and undocumented services ahead of the tunnel face. This technology has not been deployed previously on a tunnelling project in Australia, and quite probably elsewhere.

Compaction grouting was used at various locations along the route of the bored tunnels as a means to control the settlement of surface structures. This technique has been used previously in tunnelling operations for a similar purpose in London during the construction of underground railway tunnels in the vicinity of the Houses of Parliament. However, it is believed to be the first time this technology has been applied to a tunnelling operation in Australia.

The boring of closely-spaced twin tunnels through soft ground carries with it some significant technical risks. These are mostly associated with the impact the boring of the second tunnel may have upon the tunnel already constructed. In principle, the earth pressures acting on the lining of the first tunnel should be reduced to a degree by the boring of the second tunnel. Depending on the magnitude of this pressure reduction, the lining of the first tunnel could experience bending. In extreme cases such bending could be detrimental to the integrity of the segmented lining, since it is designed primarily to resist the earth pressure loads in ring compression. The uncertainty regarding the magnitude of this interaction prior to construction of the tunnels is indicative of the technical risk encountered by the designers and contractors of these twin tunnels.

In the section of Perth Yard north of Williams Street station, the alignment of the twin tunnels involved very tight curvature and shallow ground cover, i.e., approximately one-half of the tunnel diameter at its shallowest. These design constraints are necessarily accompanied by high technical risk. Very careful control of the “steering” of the TBM through the weak, saturated ground in this region would have been required to manage these high risks.

Tunnel excavation always carries with it the potential to induce ground movements around the tunnel and therefore the potential to induce movements of buildings and structures located in the vicinity of the tunnelling works. For this reason all buildings and structures within a pre-determined zone were monitored during the advancement of the TBM and excavation of the station caverns. Similarly, the major services located within the monitoring zone were also targeted for monitoring.

A comprehensive and sophisticated suite of instruments was installed to allow this monitoring to proceed. In the writer's experience this is one of the most, if not the most extensive and sophisticated instrumentation programs set up to monitor the effects of soft ground tunnelling in Australia. It would rank with best practice anywhere in the world.

In the context of developing and using relatively novel technology during the tunnelling phase, it is also worth noting that Leighton also made extensive use of monitoring technologies such as CCTV to inspect services, including lined sewers, in advance of the TBM. Monitoring of existing services both before and after tunnelling allows accurate assessment of the effects of the TBM operations.

Overall, the successful execution of this tunnelling phase of the project has required the development of clear and effective communication strategies between the various contractor units working on the project. For example, it is clear that good communication was required between the crews monitoring the performance of surface structures and those crews manning and operating the TBM. It is doubtful if LKJV would have been as successful as it was in causing minimal disruption and damage to the surface structures had this effective communication not existed. This in itself is considered to be a major achievement that required the development of appropriate communication strategies.

9.4 William Street Station

The construction of William Street station and the tunnel breakout zones required significant innovation and the development of novel construction techniques. This arose from the requirement to excavate the deep station cavity and erect the station structure while at the same time retaining and protecting nearby heritage structures and building façades.

The station was constructed using a specially adapted form of the 'top down' construction method, in which the upper-most elements of the final structure are constructed before excavation below them and subsequent construction of the lower structural elements. For the William Street station this process had to be achieved in soft, saturated and unstable ground conditions.

Construction of the station cavity required the installation of 30 m deep diaphragm walls in saturated sands, the provision of horizontal and vertical wall support, the provision of an anchoring system effectively to hold down the floor of the cavern allowing it to resist the upward ground water pressures, and the development of a system to provide permanent waterproofing of the completed cavern. All of this had to be achieved on a site with tight constraints on working space and with minimal disturbance to surrounding buildings, and in particular the preservation of the historically important Wellington building.

The solution of these design and construction challenges for the William Street station required a number of important technical innovations. Some of these are identified in the following paragraphs.

The use of diaphragm wall technology to depths not previously experienced in Perth (30 m) was a unique feature of the project.

The use of barrettes as composite supports acting: (a) in compression to support the roof of the station; and (b) in tension beneath the floor of the station, where they provide anchorage for the floor, was a unique aspect of the project. Both compression and tension (pull out) testing was conducted on these barrettes to prove their capacity. Normally barrettes are required to carry compressive loads only, not both tension and compression.

A dewatering system was designed and used to maintain dry working conditions inside the excavation, together with the provision for recharge of the groundwater beyond the excavation in order minimise surface settlements of the surrounding area. While the principle of dewatering of this nature is not particularly novel, each project requires the development of site specific technology and unique solutions.

The use of novel telescopic excavators for excavation to depths of 18 m is particularly notable. Excavators of this type are very useful in areas with tight access constraints.

A novel method was developed to underpin the Wellington building in order to support it while excavation and construction of the William Street station was carried out beneath the building. This novel method involved clamping the masonry walls of the building, the installation of micro piles eventually to support the building, and the construction of a slab on the ground to transfer the weight of the building to the piled raft composed of the slab and the micro piles.

Preservation of the façade of the Mitchell building was also an important requirement of the project. The method adopted effectively involved cutting up the façade into 9 sections, removing the sections from the construction site only to be returned and re-erected after the construction of the William Street station was completed.

10 MAJOR PROJECT ACHIEVEMENTS

Numerous technical innovations were implemented in the execution of the NMC Project works. In summary they included:

- the first road or rail tunnel of this scale in Perth constructed using a TBM. It is the deepest, and also the shallowest, soft ground tunnel constructed to date in Western Australia;
- The achievement of very rapid construction for a project of this nature, significantly faster than normally accepted practice elsewhere in the world;
- Execution of a project with an effective design life of 120 years, which is generally 20% greater than previous practice in Australia;
- The development and use of novel ground stabilisation measures such as foams, polymers and bentonite for enhancing stability at the face of the TBM;
- The development and installation of overhead tractive supports in the tunnels;
- The design and installation of novel tunnel ventilation and fire protection systems; and
- The design and installation of new noise and vibration isolation systems.