Addendum - 2 February 2006

Previously this project was named 'Harbour Bridge to City' project and there may be references in this document referring to this name or its abbreviation: HBTC.

The name of this project has since changed and is now referred to as <u>'Vic Park Tunnel'</u>. Therefore, any reference to 'Harbour Bridge to City' or HBTC should now be taken to refer to Vic Park Tunnel or VPT.

- report

Harbour Bridge to City Project - Preliminary Geotechnical Assessment - Northbound Tunnel

■ report

Harbour Bridge to City Project - Preliminary Geotechnical Assessment - Northbound Tunnel

Prepared for

Transit New Zealand

By

Beca Infrastructure Ltd

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Revision History

Revision Nº	Prepared By	Description	Date
A	Keith Simpson	PGA – Northbound Tunnel	Oct 2005

Document Acceptance

Action	Name	Signed	Date
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Table of Contents

Appendices

Appendix A - Figures

1 Introduction

The Harbour Bridge to City (HBTC) corridor covers a length of approximately 2.4km of State Highway 1 (SH1) from the Auckland Harbour Bridge southern approach embankment to the Wellington Street Overbridge.

The HBTC project is a major corridor improvement project to accommodate existing and future traffic flows and enhance safety on the central motorway system by balancing the capacities of the Auckland Harbour Bridge and the Central Motorway Junction (CMJ) currently under construction.

Previous work presented several options to Transit New Zealand (Transit) for the HBTC project (Beca, 2001a, b). The geotechnical aspects associated with these options were reviewed and preliminary investigations conducted and reported (Beca, 2001 c, d, e).

Transit has since identified the Northbound Tunnel (NBT) as the preferred option, the scheme comprising a tunnel beneath Victoria Park for northbound only traffic combined with retention of the existing Victoria Park Viaduct (VPV) for southbound only traffic.

This preliminary geotechnical appraisal report presents a review of the previous work undertaken for the HBTC project, together with recent assessments in respect of the NBT option. Additional information in respect of geologic profile and groundwater conditions in the vicinity of the scheme has been sourced from Auckland Regional Council (ARC) records as well as previous Beca investigations not associated with the HBTC project. Further data has also been utilised from an investigation program currently in progress to assess groundwater conditions in the proposed project area.

We note that the scope of investigation carried out to date has been limited to that considered appropriate for Notice of Requirement and Resource Consent purposes. Further investigation will be necessary for detailed design.

Notice to Reader/User of this Document:

Should you be in any doubt as to the applicability of this report and/or its recommendations for the proposed development as described herein, and/or encounter materials on site that differ from those described herein, it is essential that you discuss these issues with the authors before proceeding with any work based on this document.

2 Scheme Description

The proposed HBTC northbound tunnel project comprises of works in two broad areas. The general arrangement is shown on Figures 1 to 4, Appendix A.

2.1 St Marys Bay (SMB) Section

Through the SMB section of the corridor, the new alignment stays generally within the existing motorway corridor. The motorway will remain at grade with split carriageways similar to the existing motorway. This section of motorway once complete will consist of:

- 5 lanes southbound and a bus shoulder lane.
- 5 lanes northbound.

The motorway carriageway will be widened on both sides and a new median will be constructed to accommodate the additional lanes. These works can generally be fitted within the existing corridor by utilising the existing shoulder and median areas. The landscaping strip between Westhaven Drive and the motorway will be affected by the work, as the lanes will be widened in this direction. A new narrower landscaping strip will be provided to replace the removed strip.

Other works required through this section of the corridor include new sign gantries, minor stormwater works, new concrete safety barriers, new concrete edge retaining walls, new motorway lighting and noise walls.

There will generally be no encroachment beyond the existing fence line on the SMB side of the motorway. Some improvements such as landscaping, footpaths and noise walls will be located outside both the existing motorway designation and fence lines in places.

The primary exception to this is the Shelly Beach Road overbridge. To allow space for an additional northbound lane, one existing bridge pier is to be relocated about 5m south (closer to the abutment and cliff). This will involve rebuilding the southern abutment and constructing a new pier, cutting the bridge deck at mid span and sliding the southern portion of the deck onto the new abutment. The deck void is then infilled to create the lengthened bridge.

In addition, a pedestrian bridge is proposed to be constructed across the motorway near Jacob's Ladder. Modifications to the Fanshawe / Beaumont St intersection are also proposed.

2.2 Victoria Park to Wellington Street Overbridge Section

The existing Victoria Park Viaduct (VPV) will be retained to provide 4 lanes in a southbound direction only. The retention of the viaduct will involve minor upgrading to include seismic strengthening of the piles and pile caps and ASR mitigation. The existing Cook Street off ramp will remain unchanged.

A three-lane tunnel will be constructed for northbound traffic. The tunnel will be located to the west of the existing viaduct through Victoria Park having a maximum depth (to underside of floor slab) of 12m. The tunnel includes a fully covered section (about 450m in length) under Beaumont Street, Victoria Park and Victoria Street. This section will be constructed by "cut and cover" methods.

The southern approach (uncovered section) to the tunnel starts about 200m north of the Wellington Street overbridge and extends down at a gradient of about 6% to the tunnel portal located near the existing Birdcage Hotel.

The northern tunnel portal is located opposite the Victory Christian Church and ramps up at a gradient of about 4% to join the at grade section near the northern end of the Ngapona car park.

Both the southern approach and northern exit are of open box section, i.e. they have no roof.

As part of the works, relocation of the Birdcage Hotel approximately 30m-50m up Franklin Road will be carried out.

Other works required through this section of the corridor include new sign gantries, minor stormwater works, new concrete safety barriers, new motorway lighting and noise walls.

Major service relocations will also be required in this area. A main stormwater culvert passes through Victoria Park and conflicts with the tunnel. Relocation will involve constructing a new stormwater pipe on the west side of the new tunnel from Weld Street, across Victoria Street. The pipe will cross above the new tunnel immediately north of the Campbell Free Kindergarten building and then run to the east of the existing viaduct to connect to the existing culvert. Other stormwater relocations will be necessary in the vicinity of Lower Union Street and Victoria Street in conjunction with the relocation of the main stormwater culvert. In addition, Watercare's Orakei main sewer passes just south of the south abutment of the VPV between Weld Street and Drake Street. This 1.5m x 2.2m egg-shaped sewer will be diverted to the south to enable it to pass under the tunnel.

3 Site Investigation

3.1 Previous Investigation

The preliminary geotechnical appraisal of March 2001 (Beca, 2001c) prepared for the HBTC options assessment reported on the results of a site inspection and review of existing geological and geotechnical data.

A geotechnical investigation for the options assessment was undertaken between May and July 2001 (Beca, 2001d, e). The investigation consisted of 21 Nos. machine boreholes (BH) to depths up to 18m, 37 Nos. cone penetration tests (CPT) and 12 Nos. Hand Augers. Standpipe piezometers were installed in six selected boreholes for groundwater level monitoring and limited permeability testing. The locations of the investigation points are presented on Figures 1 to 4, Appendix A.

Additional information regarding geologic profile and groundwater levels along the length of the scheme has been sourced from Auckland Regional Council bore records and previous Beca investigations (Beca, 1985, 1987, 1993) at the following sites:

- Shelly Beach Road;
- Western Reclamation – various sites;
- Beaumont Street (Beaumont Quarter, Victory Christian Church)
- Victoria Park;
- Victoria Street;
- Drake Street;
- Gaunt Street; and
- Central Motorway Junction (CMJ)

3.2 Current Investigation

The previous geotechnical investigations for the HBTC project noted above were primarily for the (above ground) West Side Widening option and therefore did not adequately address groundwater issues associated with the NBT option. Additional data on existing conditions and potential effects is required for both design and consenting. Additional data is also required to support a significant decision on whether the tunnel is to be drained or tanked. This has major effects on groundwater levels and flows, contaminant migration and tunnel floor design and cost.

A further six machine boreholes to depths up to 16.5m were undertaken to provide this additional data. Field work was performed between 23 September and 4 October 2005 and comprised open barrel coring through surface sediments with shear vane testing and SPT testing at approximately 1.5m intervals. Thin walled tube samples were taken in soft cohesive deposits. Boreholes were advanced to effective refusal by open barrel, thereafter boreholes were advanced with triple-tube rock coring techniques. In-situ permeability testing was conducted in the boreholes as they were advanced to facilitate preliminary groundwater modelling. All bores were completed as standpipe piezometers to allow for long-term groundwater monitoring, some of which may be utilised for construction monitoring.

In addition, shallow investigation drilling up to 10.5m depth for the assessment of contaminated ground was undertaken immediately prior to the above investigation phase between 19 and 23 September 2005. In-situ permeability testing could not be carried out during drilling of theses environmental boreholes, however they have been completed as standpipe piezometers for shallow groundwater monitoring.

A total of 13 piezometers have been installed at locations as shown on Figures 1 to 4 Appendix A, with baseline and initial readings already undertaken.

Further work to comprise a pumping test (dependent on results of environmental investigations) to obtain additional permeability data is proposed for late 2005. The results of the pumping test together with monitoring data obtained from the piezometers will facilitate calibration of a 3D groundwater model to evaluate long and short-term (construction) effects.

Factual and interpretive reports accompanying the above investigations are in progress, and where available the new data has been incorporated into this report.

4 Site Description

The HBTC corridor is an approximately 2.4km length of State Highway 1 (SH1) extending from the Auckland Harbour Bridge southern approach embankment in the north to the Wellington Street Overbridge in the south.

The corridor generally occupies low-lying (+3mMSL to +5mMSL) reclaimed land of the original Freemans Bay embayment and shore platform and St Marys Bay foreshore, as well as naturally infilled drainage gullies and channels originating from the surrounding elevated land and former shoreline cliffs to the south and southwest. The southernmost portion of the corridor occupies the elevated land above the former shoreline.

The HBTC corridor may be subdivided into three sections of similar topography, sub-surface condition and existing road construction, each section having broadly similar geotechnical issues.

Refer to Geotechnical Site Plan, Figures 1 to 4, Appendix A, for the motorway alignment and surrounding areas.

4.1 St Marys Bay to Victory Christian Church CH150 to CH1450

The alignment through St Marys Bay is located on reclaimed foreshore and is bounded to the south by the natural (former shoreline) cliff line which rises to the south from crest elevations of about +20mMSL. To the north, the corridor is bounded by Westhaven Drive, Westhaven Marina and associated reclamation developments.

The corridor and existing motorway extending south from the Harbour Bridge approach embankment is at grade (MSL+3m to +5m) with four lanes each way along the St Marys Bay foreshore to the northern abutment of the VPV.

At the west end, the corridor alignment merges with the Harbour Bridge on-ramp from Curran Street whilst the Shelly Beach Road overbridge off-ramp from the Harbour Bridge crosses the alignment at about CH350.

At the east end, the VPV northern abutment begins to rise above the reclaimed foreshore including the Fanshawe Street southbound off-ramp and the Beaumont Street/Fanshawe St northbound on-ramp.

4.2 Victory Christian Church/Victoria Park CH1450 to CH1950

The corridor occupies the reclaimed foreshore and tidal platform of St Marys Bay and Freemans Bay forming Fanshawe St, Beaumont St and Victoria Park, and extends to the base of the former natural shoreline south of Victoria St. The proposed tunnel alignment also encompasses a former natural low-lying (~+8mMSL) shoreline promontory (the site of the Victory Christian Church carpark) immediately south/southwest of the reclaimed foreshore.

The VPV forms the dominant surface structure through this section crossing Victoria Park. The VPV is a curved concrete bridge 590m in length providing dual lanes each way. The north abutment of the Viaduct begins west of Beaumont Street and curves around the former shoreline promontory before continuing south across Victoria Park. The south abutment is located adjacent to Drake Street. SH1 northbound on-ramps and southbound off-ramps are located at the intersection of Fanshawe Street with Beaumont Street.

Other structures within or close to the corridor include the Newton Sports Club, Campbell Free Kindergarten, Victoria Park Bowling Club, skate park and petanque square.

4.3 Victoria Park to Wellington Street CH1950 to CH2400

South of Victoria Park, the corridor occupies a ground surface which rises up from the reclaimed shoreline at about +5mMSL to the Wellington St Overbridge at about +30mMSL. The existing SH1 continues as a dual lane route south from the VPV with ramps located at Cook Street (southbound off-ramp) and Wellington Street (northbound on-ramp).

West of the existing SH1 route, the ground surface slopes down towards an approximately north oriented drainage channel extending approximately from the Wellington St/Hepburn St intersection towards the Franklin Rd/Victoria St intersection. The proposed tunnel approach ramp is located within the upper portions of these western slopes, the upper slopes having also been modified during works for the existing SH1 construction.

The proposed tunnel approach through this section is generally within filled but otherwise undeveloped ground, however the northern portion intersects the existing Birdcage Hotel which is to be moved for construction.

5 Site Geology

5.1 Assessment of Site Geology

The geology of the study area is shown on the Geotechnical Site Plan (Figures 1 to 4, Appendix A) and long-section (Figure 5, Appendix A) and described below. The extent of geologic units have been derived primarily from the geologic map Sheet R11 Geology of the Auckland Urban Area 1:50,000 (Kermode, 1992) as confirmed by the previously reported investigation drilling, construction records, field mapping of outcrops and surface morphology, together with results where available from the current investigation. Conditions between investigation points will likely vary and continuity of layers should not be automatically assumed.

5.2 Waitemata Group (East Coast Bays Formation)

The East Coast Bays Formation of the Waitemata Group forms the bedrock unit throughout the study area and outcrops along the old cliff line and foreshore throughout the St Marys Bay area and in cuts formed below Union Street. The pre-reclamation shoreline follows the approximate boundary of Victoria Park.

This formation comprises interbedded sandstone and siltstone, which are very weak to weak in terms of rock strength. These rocks weather to soft to very stiff silts and clays and medium dense to dense sands that can be up to 10m thick. Where exposed on the cliff line, the upper material comprises such residual Waitemata Group soils.

Although not positively identified within previous investigations the formation is known to contain occasional interbedded lenses of stronger, more resistant Parnell Grit (as mapped at Point Erin). Bedding in the study area generally dips towards the northwest at about 15° however localised areas have been mapped to range between 5° and 23°.

5.3 'Lower' Tauranga Group - Undifferentiated Alluvium

These sediments unconformably overlie the East Coast Bays Formation and are predominantly estuarine and terrestrial sediments consisting of soft to very stiff silts, sand, clay-silt, peat and colluvium comprising weakly cemented sub-angular siltstone/sandstone derived gravels. These deposits are interpreted as deposited during the Pleistocene (1.7million to 10 thousand years ago) in low areas of the underlying topography. During the course of their deposition, fluctuating sea levels have resulted in both variations in the types of sediments deposited; sand, clay/silt and peat and repeated cycles of deposition and erosion. This discontinuous and variable depositional environment is reflected in the variability of this unit where encountered in boreholes.

5.4 'Upper' Tauranga Group – Littoral (Recent Marine) Sediments

Recent (Holocene) marine sediments occur in the low lying harbour area and its tributaries overlying the older Tauranga Group Sediments and Waitemata Group rocks. These sediments have been deposited below present sea level in the last 10 thousand years and have therefore not experienced significant sea level changes and associated over-consolidation. In the HBTC corridor these sediments are now covered with fill from the reclamation activities along the original foreshore and shore platform (St Marys Bay and Victoria Park area), and out into the harbour.

This formation consists of varying depths of very soft or loose, unconsolidated recently deposited silty and sandy marine sediments. These marine deposits are subject to considerable consolidation settlement when surcharged and are considered prone to liquefaction where sandy (Beca, 1995).

5.5 Fill

The areas of fill material noted on the geologic map are mapped as "Construction Fill" as a part of the reclamation along the low-lying harbour areas. These have been further differentiated based on information from various reports and previous investigations:

a. Construction Fill. This all-inclusive description includes a mixture of clay to gravel sized materials in a variable state of compaction and may include demolition debris (Kermode, 1992).

b. Hydraulic Fill. This material forms the St Marys Bay and Westhaven reclamation up to around MSL. It consists of very soft silts and sand and is considered prone to liquefaction (Beca, 1995).

c. Rolled Clay Fill was placed beneath motorway pavement areas over hydraulic fills. The clay was compacted in layers and is described in part as firm to stiff mottled grey, yellow and brown clayey silt, highly plastic (Beca, 1995 & 1997b). "Clay" fill was also used in the reclamation of selected areas of Westhaven Marina.

d. Rock Sea Walls were constructed along the seaward edge of the original reclamation along the St Marys Bay foreshore (Beca, 1995). Additional walls were subsequently constructed as the reclamation was extended to accommodate the motorway widening and progressive development of the marina.

Where identified, the stages of reclamation and construction of seawalls and fills have been shown on the Geotechnical Site Plan Figures 1 to 4, Appendix A.

5.6 Faulting

Faults have been mapped or inferred (Kermode, 1992 and High, 1972) in exposures of Waitemata Group in the study area. These faults are widely regarded as being inactive as no evidence of displacement of quaternary deposits has been identified in this area (Hull *et al*, 1995). Refer Geotechnical Site Plan, Figures 1 to 4, Appendix A.

A site-specific hazard assessment has previously been undertaken for the northern end of the study area as part of the Auckland Harbour Bridge seismic assessment (Beca, 1995, 1997b). For the current design however, seismic loading in accordance with NZS1170.5 (2004) and the Transit Bridge Manual – Section 5 (2004) will be utilised.

6 Geologic Profile

The sub-surface profile along the NBT route is summarised on Figure 5, Appendix A.

6.1 St Marys Bay to Victory Christian Church CH150 to CH1450

6.1.1 Description of Profile

The bedrock surface slopes down into the harbour from the base of the cliffs. Where marine deposits are present they tend to lie in localised depressions or erosion channels in the wave cut platform. Piling records at the Shelly Beach flyover abutments show a 3m layer of loose deposits underlying the reclamation fills. At the Auckland Harbour Bridge south anchorage, marine deposit thickness increases to as much as 15m (Beca, 1995, 1997a, 1997b).

In the St Marys Bay and Westhaven area, the motorway is constructed on reclamation placed on the wave cut platform beneath the cliffs and on alluvial sediments infilling low areas in the rock profile e.g. St Marys Bay.

The original reclamation works were undertaken during the late 1950's to form the southern approach to the Harbour Bridge. Behind the rock bund seawall (to about +3.7mMSL), hydraulic fill was pumped in place to a level of +2.1mMSL using a suction dredge. Construction records reveal that limited discharge points were adopted resulting in segregation of the finer soils that dispersed. These fines tended to concentrate at the outfall weirs and are likely to be weaker and more compressible than for other areas. Settlements in excess of 300 mm were recorded during the construction.

Pockets of soft marine sediments and intertidal mud are likely to remain, sandwiched between the hydraulic fill and Waitemata Group Formation.

For the motorway pavement subgrade, compacted clay was placed on the top of the hydraulic fill. This material was specified to have a CBR > 10% (Beca, 1995, 1997a, 1997b). Remaining areas either side of the pavement formation were topped up with hydraulic fill to finished level at about +4.0mMSL.

Since these works, further reclamation has been undertaken. Ports of Auckland Ltd report that their reclamation, of approximately 4ha has been formed from hydraulic fills, Waitemata Group rock excavated from the marina basin and controlled clay fills. These staged reclamations were each faced with rock sea walls. We understand that earlier walls have remained in place and were buried by subsequent reclamation.

Table 6.1.1 presents a summary of the sub-surface profile through this section.

Table 6.1.1 – Sub-surface Profile – St Marys Bay

Name	Description	Depth to Top (m)	Thickness (m)
Rolled Clay Fill	Based on descriptions from historical data. Not encountered during recent investigation.	0	Approx. 2.0
Fill	Soft to hard gravelly silts and loosely packed organics, gravel, building waste.	0	1 – 4.75
Hydraulic Fill	Very loose to loose silty sand.	1 – 4.4	0.55 – 2.5
Recent Marine Sediments	Very soft silt. Loose silty fine to medium sand (shells in some boreholes)	2.15 – 5	0.15 – 1.3
Tauranga Group Sediments	Firm to very stiff silt-clay and sandy silt (shells and peat in some boreholes).	3.45 – 5.6	0.65 – 8.05
Waitemata Group (Ex.Wk)	Highly to moderately weathered sandstone and siltstone.	4.2 – 11.4	1.1 – 2.65
Waitemata Group (V.Wk)	Fresh to slightly weathered sandstone and siltstone.	4.2 – 12.5	N/A

6.1.2 Groundwater Levels and Permeability Testing

Groundwater is generally encountered at about 2 to 2.5 m depth (RL 0 to 0.5m).

Reverse slug (rising head or recovery) permeability testing was attempted during the 2001 investigation using a manual bailer in standpipe piezometers installed within machine bores BH02 and BH05 (Beca, 2001e). Results are summarised below in Table 6.1.2.

Table 6.1.2 – Permeability Test Results – St Marys Bay

Borehole No.	Analysis Method	Hydraulic Conductivity (m/s)		Screened Aquifer Description
		Early Time	Late Time	
BH02	Hvorslev	3E-6	8E-7	0.6m - Fill 2.5m - Hydraulic Fill
	Bouwer – Rice	4E-6	9E-7	
BH05	Permeability Testing performed however testing abandoned, as measurable drawdown was not feasible. Permeability may be inferred to be significantly higher than values determined for BH2 above.			2.1m – Fill 0.65m – T.G 0.25m-V. Weak W.G

6.2 Victory Christian Church/Victoria Park CH1450 to CH1950

6.2.1 Description of Profile

Reclamation works in the Freemans Bay area (Victoria Park) were undertaken around the turn of the last century and included filling over an old infilled gully and stream inlet (Beca, 1997a). The gully extended inland along the area between Franklin Rd and College Hill. The depth of the fill and Tauranga Group sediments over the Waitemata Group Rocks varies from 4.5m near Beaumont Street to in excess of 10m near the centre of Victoria Park.

Fill materials encountered beneath Victoria Park appear to be uncontrolled and variable containing rocks and building debris. During investigations in 2001, 15 CPT tests were unable to penetrate the fill and were considered to have reached refusal due to striking such debris.

The VPV was constructed during the 1960's. As built drawings (Victoria Park Viaduct, 1964) show the viaduct piers and north abutment are supported on a combination of bored and driven piles founding in sandstone. Records do not provide information on founding depth though it is known that borings were carried out in advance of the works.

Table 6.2.1 presents a summary of the sub-surface profile through this section.

Table 6.2.1 – Sub-surface Profile – Victoria Park

Name	Description	Depth to Top (m)	Thickness (m)
Fill	Soft to hard gravelly silts and loosely packed organics, gravel, building waste.	0	2.95 – 4.7
Recent Marine Sediments	Very soft silt and loose silty fine to medium sand, occasional shells.	2.95 – 4.7	0.4 – 4.15
Tauranga Group Sediments	Firm to very stiff silt-clay, sandy silt, shells and peat.	3.7 – 8.85	0.65 – 3.25
Waitemata Group (Ex.Wk)	Highly to moderately weathered sandstone and siltstone.	4.1 - 10	1.2 – 4.25
Waitemata Group (V.Wk)	Fresh to slightly weathered sandstone and siltstone.	4.35 – 12.95	N/A

6.2.2 Groundwater Levels and Permeability Testing

Groundwater is generally encountered at about 1.5 to 3.0 m depth (RL 2.0 to 2.5 m). The groundwater table falls gently to the north-northeast i.e. consistent with the approximate location of a buried paleo valley.

Reverse slug (rising head or recovery) permeability testing was attempted during the 2001 investigation using a manual bailer in standpipe piezometers installed within machine bores BH06 and BH08 (Beca, 2001e). Permeability testing has also been undertaken in the vicinity of Beaumont Quarter by other consultants and made available through ARC records. Results are summarised in Table 6.2.2.

Table 6.2.2 - Permeability Test Results – Victoria Park

Borehole No. / Reference		Analyses Method	Hydraulic Conductivity (m/s)		Screened Aquifer Description
			Early Time	Late Time	
Beca, 2001e	BH06	Hvorslev	4E-8	N/A	1.0m – Fill 0.95m – Marine Sediments 0.55m – Ex. Weak W.G
	BH08	Permeability Testing performed however testing abandoned, as measurable drawdown was not feasible. Permeability may be inferred to be significantly higher than values determined for BH 6 above.			1.2m – Fill 0.7m – Marine Sediments 0.6m – Ex. Weak W.G
Tonkin and Taylor, 2001		Not specified	1.3E-6	-	Fill/Residual Soils
			1.1E-6	-	Waitemata Group
			2.4E-7	-	Fill/Residual Soils
			6.5E-6	-	Waitemata Group
			2.7E-7	-	Fill/Residual Soils
			9.7E-7	-	Fill/Residual Soils
			1.6E-6	-	Waitemata Group
Woodward Clyde, 1997			1.9E-6	1.2E-7	Fill
			6.3E-8	4.6E-8	Fill/Residual W Grp
			2.1E-7	3.7E-9	Fill/Residual W Grp
			1.9E-6	8.3E-7	Residual W Grp?
			8.0E-7	2.6E-7	Residual W Grp?

6.3 Victoria Park to Wellington Street CH1950 to CH2400

The section of motorway south of Victoria Street West/ Drake Street appears to have been constructed partially in cut (SE side) and partially on fill (NW side) over weathered Waitemata Group Formation and hence is discussed separately below.

6.3.1 Conditions on SE side

Ground conditions in this area can be expected to have originally comprised a variable depth of residual soils overlying weathered Waitemata Group rocks. Formation of the existing SH1 corridor appears to have involved the excavation of a cut progressively deepening to the south as the depth of the cut below Union Street increases. Local fill deposits are observed. Table 6.3.1 presents a summary of the sub-surface profile through this section.

Table 6.3.1 – Sub-surface Profile – Victoria Park to Wellington St (SE side)

Name	Description	Depth to Top (m)	Thickness (m)
Fill	Soft to hard gravelly silt.	0	0.2 – 1.85
Waitemata Group (Res. Soil)	Residual soil derived from weathered sandstone and siltstone.	0.2 – 1.65	0.25 – 2.4
Waitemata Group (Ex.Wk)	Highly to moderately weathered sandstone and siltstone.	0.75 – 3.5	0.55 – 6.5
Waitemata Group (V.Wk)	Fresh to slightly weathered sandstone and siltstone.	1.3 - 10	N/A

6.3.2 Conditions on NW side

Ground conditions are expected to comprise a variable depth of fill and residual soils overlying weathered Waitemata Group rocks. Formation of the existing SH1 corridor involved the construction of a sidling fill that is thought to extend from beneath the pavement downslope over a significant extent (and depth) to the northwest.

The current 2005 investigation at borehole MB06 (CH2120) encountered some 7.5m of fill overlying 3m of colluvium and weathered Waitemata Group, before penetrating competent rock at approximately 10.5m depth.

No other bore information is available for this area, however it is likely that the significant thickness of fill continues to the north, terminating at Weld Street (emergency vehicle access point to VPV), and to the south towards Wellington St.

6.3.3 Groundwater Levels and Permeability Testing

The groundwater levels of two bores at CH2100 are known, and suggest a shallow (c 5.5m) perched groundwater level within the Fill and a deeper “regional” groundwater level within the Waitemata Group (c 7.7 m). Further monitoring is required to establish if the perched groundwater level is stable year round or seasonal.

Reverse slug (rising head or recovery) permeability testing was attempted during the 2001 investigation using a manual bailer in a standpipe piezometer installed within machine bore BH18 (Beca, 2001e). Results are summarised in Table 6.3.2.

Table 6.3.2 - Permeability Test Results				
Borehole No.	Analyses Method	Hydraulic Conductivity (m/s)		Screened Aquifer Description
		Early Time	Late Time	
BH18	Hvorslev	5E-8	2E-8	4.45m – Ex. Weak W.G 5.65m – V. Weak W.G
	Bouwer - Rice	5E-8	2E-8	

6.4 Preliminary Design Parameters

Table 6.4 presents a summary of typical design parameters for the materials encountered along the HBTC corridor. Parameters are based on test results from the previous HBTC investigations (Beca, 2001d, e) as well as information contained within the Beca database for similar materials.

Table 6.4 – Preliminary Design Parameters

Name	Unit Weight γ (kN/ m^3)	Undrained Shear Strength S_u (kPa)	Effective Friction Angle ϕ' (deg)	Effective Cohesion c' (kPa)	Compression Ratio $C_C/1+e_0$	Recompression Ratio $C_r/1+e_0$	Preconsolidation Pressure p'_C (KPa)	Coefficient of Consolidation C_V (m2/ yr)	Drained Modulus E' (MPa)	Shallow Foundation Bearing Capacity (kPa)[1]	Pile End Capacity (MPa)[1]	Pile Skin Friction Capacity (kPa)[1]
Rolled Clay Fill	18	50-100	26-30	2-5	5-10	1-2	100	0.5	15-25	225-450[4]	-	15-30
Fill	15-19	20-200	26-32	0-5	5-25	1-5	0	0.5->5	5-30	150-450[4]	-	5-25
Hydraulic Fill	15-17	-	26-28	0	5-25	-5	0	0.5->5	5-20	150-300[4]	-	5-10
Recent Marine Sediments	14.5 – 19	<30	15-30	5-25	15- 40	2-8	0-20	0.5	2-15	150[4]	-	5-10
Tauranga Group Sediments	15-18	25-150	25-30	0-10	10-25	2-5	10-100	0.4-4	10-50	150-450[4]	-	10-30
Waitemata Group (Res. Soil)	18	20-100 (soft-stiff)	26-30	4-6	0.3-2	0.05-0.5	-	-	15-50	200-500[4]	-	25-30
Waitemata Group (Ex.Wk)	19	100+ (stiff-hard)	30-34	0-10	-	-	-	-	30-80		-	
Waitemata Group (V.Wk)	20	-	30-40	15-100	-	-	-	-	100-500	-	6-16[2]	200-700[3]

Notes:

1. Ultimate geotechnical capacities.
2. Lower bound for bored pile embedded minimum 3 pile diameters; upper bound for driven piles.
3. Lower bound for driven piles; upper bound for grooved bored pile (non-grooved 500kPa).
4. Lower bound is practical design and construction limit for settlement – actual ground conditions may indicate lower ultimate values, however such conditions would not normally be considered reasonable for foundation bearing.

7 Geotechnical Issues

The following sections present a summary of the principal geotechnical issues likely to be encountered along the proposed alignment, as well as indicative solutions which will require to be confirmed and developed further during later design.

7.1 St Marys Bay to Victory Christian Church CH150 to CH1450

7.1.1 Scheme Description

The NBT option will retain the existing pavement alignment and levels with widening to maximise the available roadway generally within the existing designation by:

- Replacing the median retaining wall with a narrower design that allows a reduced separation between northbound and southbound lanes.
- Replacing the existing edge-line retaining walls with new walls built closer to the edge of the designation.
- Widening the existing roadway within the designation by extension of existing cuts/fills up to about 2m in depth / thickness.
- Construction of a pedestrian overbridge near Jacob's Ladder.
- The Shelly Beach overpass bridge southern span to be extended to accommodate the extra northbound lane.

7.1.2 Foundation Issues

Proposed alignment works along this section will require new foundations for the pedestrian overbridge, signage gantries, noise walls, lighting poles etc. Shallow spread footings may be utilised for lightly loaded elements, subject to settlement limitations (see Section 7.1.4). Where settlements may be considered excessive or where the depth to Waitemata Group rock is shallow (refer Table 6.1.1), piled foundations may be expected, including where wind loading may be significant.

Shallow retaining structures along the medium strip and at pavement edges, particularly on the northern boundary will be required. Reinforced concrete walls supported on shallow spread footings are currently proposed and may be subject to moderate settlements (see Section 7.1.4) for which appropriate articulation and/or reinforcing will be required. Settlement effects may also lead to construction delays and resurfacing of supported pavements.

Significant foundation modification works will also be required for the Shelly Beach Rd overbridge with relocation of one existing pier and rebuilding of the southern abutment. Required works will include new piled foundations for the relocated pier and excavation of the natural rock and installation of anchors into competent (very weak or better) Waitemata Group at the abutment.

7.1.3 Stability Issues

a. Stability of Relict Sea Cliffs on Western Side of Motorway.

Beca has previously carried out slope stability investigations at residential properties above the cliff line in St Marys Bay (Beca, 1993, 1987, 1985). Slips were reported to have occurred in the weaker residual soils on the upper part of the cliffs and were considered likely to occur again. The failure mechanism was generally attributed to saturation of the residual Waitemata soils during periods of prolonged rainfall and slabbing/spalling of the Waitemata rock below.

Cliff conditions vary considerably between properties, including vegetation (e.g. Pohutukawa trees), proximity of structures, garden developments, and retaining structures. Movement including creep, slips and rock falls have been observed in the past.

Waitemata Group beds are mapped as dipping out of the cliff along St Marys Bay at around 5° to 25°the north and north east. Potential planar block slides could occur on weak beds (e.g. clay coated bedding planes), particularly if exposed by any future cutting into these cliffs.

The cliff face and slopes are generally outside of the motorway designation and do not involve physical works directly altering the existing cliff line geometry. However, an assessment of potential indirect affects on the stability of these slopes should be carried out during detailed design. Indirect effects may include:

- Changes to water table (rise in level).
- Changes or loss of vegetation cover (particularly reduction in the root-binding effect of Pohutukawa trees).

The works proposed at the Shelly Beach Rd overbridge abutment as described in Section 7.1.2 are an exception and will require an assessment of stabilisation works for the cliff face.

b. Stability of Existing Reclamation

The stability of the reclamation at the Harbour Bridge south abutment was analysed as part of the Auckland Harbour Bridge Seismic Assessment (Beca, 1997b). The report concluded that the sections analysed were reasonably stable under static conditions. It was also concluded that when subjected to seismic shaking, the sandy hydraulic fill may liquefy resulting in both vertical and, potentially, horizontal permanent displacements.

Current proposed works do not involve extension of reclamation into Westhaven Marina. Modifications to the existing roadway are not anticipated to significantly impact the overall stability of the existing reclamation, however the effect will need to be established during detailed design.

7.1.4 Settlements

This length of the motorway is reported to have been constructed by placing hydraulic fill over the foreshore area behind a rock seawall which was then capped by rolled clay fill to the underside of pavement. Where underlain by Tauranga Group sediments (particularly recent marine sediments) and/or hydraulic fill, any significant increase in load will result in renewed settlements.

Previous preliminary work (Beca, 2001e) indicated significant potential settlements could occur as summarised in Table 7.1.4.

Table 7.1.4 - Summary of Settlement Estimates (Beca, 2001e)

Location	Comments	Equivalent surcharge for analysis	Estimated settlement (mm)	Estimated time to 90% consolidation
CH350	Narrow fillet of fill up to 1.6m thick (generally < 0.5m) and up to 3.0m wide.	14 kPa over 3m wide strip	40 - 80 (sandy hydraulic fill) 150 – 250 (silty hydraulic fill)	0.2 years (sandy hydraulic fill) 4 years (silty hydraulic fill)
CH750	Trapezoidal cross-section approximately 5m wide, 2m high.	36 kPa over 5m wide strip	100 – 150 (sandy hydraulic fill) 200 – 300 (silty hydraulic fill)	0.1 years (sandy hydraulic fill) 3 years (silty hydraulic fill)

Settlements of the magnitude indicated above, particularly differential movements at the interface between new and existing fills, will require a high level of maintenance.

Options to mitigate the effects of settlement include:

- Monitor the settlement during construction.
- Flatten interfaces between existing and new reclamation to spread the zone over which differential settlement occurs.
- Remove and replace compressible soils from beneath new reclamation areas.
- Improve the settlement characteristics of the underlying soils by in-situ treatment e.g. stabilisation by lime mixing.

Where time allows, the preferred response is to wait out the bulk of the settlement before proceeding with constructing the seal. However, as noted in Table 7.1.4, the time for 90% consolidation may be several years where finer grained materials exist.

Settlement is also expected to continue in the long term. However if the magnitude of residual movement is small, periodic overlaying of the highway pavement surface could be used to accommodate these residual movements.

Further investigation and analysis of these aspects will be required during detailed design.

7.2 Victory Christian Church/Victoria Park CH1450 to CH1950

7.2.1 Scheme Description

The NBT scheme comprises the following elements through this section:

- Construction of a cut and cover tunnel south of about CH1550 up to a depth of 12m (to underside of floor slab) below existing ground surface.
- Construction of an open roof trench tunnel exit ramp north of about CH1550.
- Excavation into the former shoreline promontory between about CH1460 and CH1620 forming an open cut for the Fanshawe St on-ramp.
- Retrofitting of the existing VPV for southbound traffic.

7.2.2 Cut and Cover Tunnel and Exit Ramp

a. Proposed Design

The proposed construction comprises either secant pile or diaphragm walls founded into competent (very weak or better) Waitemata Group. Through the cut and cover tunnel section, walls are propped by the roof structure near the ground surface. Along the exit ramp, walls are either cantilevered or anchored, with anchors extended down into competent (very weak or better) Waitemata Group. The tunnel floor comprises a reinforced concrete slab which may be drained or undrained (and anchored) dependent on the assessed groundwater responses (refer Section 8).

b. Retention Requirements

Wall design would require structures to resist:

- Appropriate earth pressure plus surcharge loading due to adjacent roadways, structures and construction loading.
- Hydrostatic pressures including short-term inundation levels (external water level at the top of the lowest freeboard point – even if this is above existing ground level).

Adequate retention must be provided for both the short term (during construction) and long term condition. The option of either a secant pile or diaphragm wall system provides both short and long-term support.

It should also be noted that the assessment of wall depth and associated wall response (displacement, moment etc) must consider both global stability and the groundwater drawdown requirements (refer Section 8).

Preliminary wall analyses using the commercial software packages Frew and Stawall from the Oasys Geo suite have been carried out to assess potential embedments, wall displacements and responses. Analyses were carried out for an assumed 1m diameter pile wall having a reduced cracked stiffness and propping without pre-stress for the short-term and long-term conditions. Results of maximum responses are summarised in Table 7.2.2. Figure 5, Appendix A shows the ground profiles at the analysed chainages.

Table 7.2.2 – Summary of Preliminary Wall Analyses

Location	Permanent Retained Height (m)	Ground Water (m b.g.l.)	Pile Socket Length[1] (m)	Wall Displ (mm)	Maximum Wall Moment (kN.m/m)	Maximum Wall Shear (kN/m)	Maximum Prop Force (kN/m)
CH1400	6	-2.5	8.2	43	538	177	-
CH1500	8	-0.5	3.6	12	438	198	180
CH1630	8	-0.5	3.0	7	302	169	158
CH1710	8.5	-0.5	4.6	11	377	209	163
CH1780	9	-0.5	8.0	30	808	370	264
CH1810	9	-0.5	7.1	22	676	313	241
CH1910	6.5	-0.5	3.7	8	290	139	128

1. Socket length below finished pavement level.

c. Foundations

As a large open box structure in an area of high groundwater levels, primary foundation loads are typically in uplift. Uplift loads may normally be resisted by structural deadload, tendon anchors or anchor piles:

- Deadload is not cost effective, requiring large volumes of concrete.
- Tendon anchors are unlikely to have sufficient durability/long term reliability.
- Anchor piles are the preferred solution, and would be designed to socket into Waitemata Group.

Decreased design uplift pressures on the tunnel floor and sidewalls may lead to capital cost savings. Consequently, an option to provide drainage beneath the tunnel floor (to a continuously pumped sump) is being assessed. The selection of this option would be contingent on a number of factors:

- Relatively low volumes of seepage.
- Long term operation of pumping for dewatering.
- Development of a mitigation scheme in the event of pump failure.
- Successfully addressing concerns regarding effects on long-term groundwater levels and induced settlements.

A review of the hydrogeological environment, groundwater drawdown, tunnel inflows etc is presented in Section 8.

d. Groundwater Inflow

Relatively low levels of groundwater leakage through construction joints in diaphragm walls and between pile overlaps in secant pile walls would be expected, with larger potential inflow in fill or sandy soils. Remedial grouting may be required if leakage is deemed excessive both in terms of constructability and effects of drawdown beyond the excavation footprint.

Groundwater inflow from the excavation floor during construction, and in the long-term for the drained tunnel option is discussed in Section 8.

e. Settlement

Ground settlements are expected to occur beyond the excavation footprint in response to retention deflection (ground loss) and groundwater drawdown.

For excavations in soft ground such as the fill and Tauranga Group sediments, empirical assessment of ground movements resulting from retention deflection indicates potential maximum settlement approximately equal to the maximum wall deflection. The extent of settlement in such soft materials is observed to be up to about three excavation depths, with, particularly for soft soils, a relatively uniform maximum settlement profile for a distance up to about one to one and half excavation depths.

Based on the results summarised in Table 7.2.2, settlement due to wall deflection up to in the order of 50mm may be anticipated up to about 15m from the excavation, thereafter decreasing to zero at distances of about 30m-40m.

Settlements resulting from groundwater drawdown will occur in addition to movements resulting from wall deflection. They may be significantly greater than those described above, and extend over larger distances. An assessment of groundwater related settlement is considered in Section 8.

Preliminary review of monitoring results of some excavations in Waitemata Group materials indicate a potentially limited lateral extent of settlement effects. However, review of rock excavations elsewhere indicates the potential for ground movement up to distances of three excavation depths. The extent of lateral movement in rock excavations is a function of a number of factors such as in-situ horizontal stress, rock mass quality, weathering etc. Further review will be required, however it is not anticipated that significant movements will occur in the rock itself.

f. Excavatability

Proposed tunnel and tunnel ramp works will involve excavation of the order of 125,000m^3 through Fill, Marine Sediments and Waitemata Group materials. Excavation depths along the majority of the tunnel and tunnel ramps will extend to Waitemata Group materials (Figure 5), however, where thin Fill and Marine Sediments exist below the underside of the tunnel / tunnel ramp floor, over-excavation is likely to be carried out to improve pavement performance.

Obstructions were encountered within fill material through Victoria Park preventing CPT penetration during the 2001 investigation (see Section 6.2.1), however the sizes of these obstructions are not known. One borehole of the 2001 investigation (Beca, 2001e) is reported to have encountered a piece of rubble at least 200mm thick. The absence of larger fragments of rubble or other undesirable materials in the fill cannot be confirmed at this time.

Unconfined compressive strength (UCS) tests carried out on samples of core taken from Waitemata Group sandstone and siltstone in the 2001 investigation (Beca, 2001d, e) indicates a range of strengths generally less than 5MPa UCS. Materials of such strength can generally be excavated by ripping and then digging with conventional excavator. Harder and more resistant Parnell Grit was not encountered during drilling in the area of the VPV, however is known to exist near Pt Erin (see Section 5).

g. Contamination

Excavated materials are potentially contaminated. Issues relating to contaminated soils are presented under separate cover (Beca, 2002, 2005).

7.2.3 Victoria Park Viaduct

a. Existing Situation

The depth to competent sandstone and siltstone varies along the existing viaduct from about 5m near Fanshawe Street to in excess of 10m within the park (refer Section 6).

The existing VPV and most of the approach embankment retaining walls are founded within Waitemata Group sandstone/siltstone on a combination of bored and driven piles as summarised in Table 7.2.3a.

Table 7.2.3a – Details of Foundations of Existing Viaduct

Pier Numbers	Pile Type	Design Pile Shaft Size	Min. Design Socket	Design Bell Diameter	Comments
North Abutment and embankment wing walls	Circular section bored piles	432mm	648mm	648mm	17 inch piles. Design Load 30 Tonne. Penetration noted as not less than RL–8.0′
$2^{1\&2}$ - $10^{1\&2}$, 11^{1}, $13^{1\&2}$ - $18^{1\&2}$, 21^{1}, $23^{1\&2}$ - $25^{1\&2}$, 30^{2}, 33^{1} Inner Row Southern Abutment	Circular section bored piles	560mm	838mm	838mm	22 inch piles. Design Load 70 Tonne. Majority of these piles at southern abutment raked1:6
Outer Row Southern Abutment	Circular section bored piles	762mm	Unknown	Unknown	30-inch diameter piles. Design Load unknown.
12^{1}, 11^{2}, 12^{2}	Circular section bored piles	1830mm	3962mm	nil	6-foot diameter pile. Design load unknown.
$19^{1\&2}$ to $20^{1\&2}$, 21^{2}, 24^{1}, 25^{1} $26^{1\&2}$ to $29^{1\&2}$, , 30^{1}, $31^{1\&2}$, $32^{1\&2}$, 33^{2*}	Square section driven piles	457mm	N/A	N/A	No information on Set/Rebound

*Pilecap of Pier 33, Bridge 2 has 3 x 560mm bored piles and 1 x 457 mm driven.
12^{1} Denotes pier 12, bridge 1 (southbound)

b. Proposed Retrofitting

Proposed foundation treatment for the VPV retrofit comprises two general arrangements.

i. Box girder sections
   - Existing piles to be jacketed.
   - Existing pilecap to have pilecap overlay with post tensioned reinforcing elements.

ii. Solid I-beam sections
   - Existing piles to be jacketed.
   - New 600mm diameter bored piles to be installed around outside of existing piles.
   - Existing pilecap to have pilecap overlay with post tensioned reinforcing elements, with pilecap extended to encompass new piles.

Enabling works for both foundation treatment options to allow pile jacketing and installation of new piles are expected to involve excavation of an approximately 2.5m deep trench around the existing pile cap supported by sheetpiles.

c. Indicative Pile Depths

Table 7.2.3b provides indicative founding depths at selected locations along the viaduct based on 2001 investigation results (Beca, 2001e).

Table 7.2.3b - Indicative Pile Founding Depths at selected borehole locations VPV

Location	Depth to founding materials (m)	Indicative* depth to pile toe (m)	Overlying soil type	Founding soils/rock type
CH1600	6m	12m	Fill, marine sediments, Tauranga Group sediments, weathered Waitemata Group	V Weak Waitemata Group
CH1800	14m	20m		
CH1950	8m	14m		

*Indicative depth to pile toe based on 10-diameter socket for 600mm diameter bored pile.

d. Negative Skin Friction

Piles that are subjected to settlements such as indicated in Section 7.2.2, are likely to experience down drag or negative skin friction.

The amount of down drag and the frictional/adhesion forces acting on the pile shaft is dependent on the local soil conditions, likely future loading and groundwater conditions and should be assessed during detailed design.

e. Potential Obstructions to Piling

As noted in Section 7.2.2, obstructions of unknown size were encountered within fill material through Victoria Park.

The use of mixed pile types is noted in construction records of the existing viaduct foundations and may indicate the effect of obstructions. It is possible that either bored pile holes encountering obstructions were subsequently bypassed by driven piles or that driven piles held up or deflected by obstructions were subsequently replaced by bored piles and modified pile caps.

f. Permanent Casing

Bored piles constructed through fill, saturated loose sands, soft clays and peats would require the use of temporary casing, and in particularly poor conditions such as high permeabilities, permanent casing to maintain ground support during construction and prevent 'necking' or contamination.

Casing would need to be socketed into stable soils/rock at depth to prevent soil inflow or collapse. With high groundwater tables, concrete pours would be by tremie.

It is not known if any of the bored piles installed during construction of the original viaduct required permanent casing. We are not aware of any integrity testing having been carried out on the existing piles to identify if necking or other similar defects are present.

g. Temporary Excavation

Sheetpiles for temporary excavation support would be required, for global stability, to be driven to depths of the order of 8m-9m for cantilever sections and to depths of the order of 5m-6m where internal propping may be achieved.

As noted above however, obstructions within the existing fill may restrict driving capability. Furthermore, limitations on induced vibrations may restrict driving energies particularly where obstructions are encountered.

Groundwater inflow from sheetpile clutches would be expected, however provided good construction practice is followed, inflow rates should be relatively low. Remedial grouting/sealing of clutch flow points can be carried out if required.

An assessment of groundwater inflow into the excavation base (as well as base heave) will be required and dependent on flow rates and associated potential drawdowns, penetration depths may require extension well beyond those needed for global stability.

7.2.4 Ancillary Structures

Surface structures comprising tanks, services buildings and other ancillary elements are proposed to be located on the north side of the existing SH1 at about CH1500.

Shallow spread footings may be utilised for lightly loaded elements, subject to settlement limitations (see Section 7.1.4). Where settlements may be considered excessive or where the depth to Waitemata Group rock is shallow (refer Table 6.1.1), piled foundations may be expected.

7.3 Victoria Park to Wellington Street CH1950 to CH2400

7.3.1 Scheme Description

The proposed works through this section comprise:

- Construction of an open roof trench tunnel approach ramp.
- Tunnel approach to accommodate the on-ramp from Wellington St.
- The trench west wall excavation to comprise piled walls and cut slopes.

7.3.2 Piled Walls

a. Proposed Design

Proposed construction comprises secant pile or diaphragm walls north of about CH2020. Bored pile walls (approximately 1m diameter at 1.5m centres) are proposed south of CH2020, with shotcrete arches with rear drainage constructed between the piles.

Eastern walls will be supported by anchors located within competent (very weak or better) Waitemata Group. Along the western excavation boundary, a combination of piled walls and cut slopes is proposed:

- Cantilevered piled walls north of CH2040,
- Cut slopes from CH2040 to CH2100 and CH2260 to CH2340,
- Combined cantilevered piled walls (typically <2.5m retained height) with cut slopes above between CH2100 and CH2260.

b. Design Requirements

Design considerations for this section will generally be in accordance with aspects discussed for the Victory Christian Church/Victoria Park cut and cover tunnel and northbound exit ramp (refer Section 7.2.2).

A preliminary wall analysis has been conducted for an eastern combined bored pile wall and cut slope system. Analysis was performed for a profile at approximately CH2120 on the basis of the results of the current (2005) investigation drilling (MB06) which encountered deep fill and disturbed ground along the western boundary (see Section 6.3.2). The analysis assumed a 2.8m high batter at 3H:1V above a 2.5m high piled wall with groundwater at the top of the wall. Results are summarised in Table 7.3.2

Table 7.3.2 – Summary of Preliminary Pile Wall Analysis, Western Wall, CH2120

Profile	Maximum Displacement (mm)	Maximum Moment (kN.m/m)	Maximum Shear (kN/m)
7.5m Fill and 3m Weathered Waitemata Group over V.Wk Waitemata Group.	14	133	79
10.5m Fill over V.Wk Waitemata Group.	46	531	188

Groundwater inflow and drawdown may be expected to be limited through this section. As noted in Section 6.3.3, current data indicates a perched groundwater level at about 5.5m depth, at or near the base of the excavation.

7.3.3 Western Slopes

Cut slopes along the western excavation boundary having batters at 3H:1V are proposed based on typically adopted design profiles. Further work during detailed design will be required to confirm stability requirements including drainage measures etc.

In addition, a review of the stability of the existing slopes (assumed fill) below the proposed excavation above Freemans Bay Primary school will be required. The assessment will review the impacts of the proposed works such as modified groundwater, modified loading conditions etc.

7.4 Vibrations

A preliminary assessment of construction vibration effects has previously been carried out (Beca, 2003) and subjected to external review (MDA, 2003). MDA (2005) have carried out an updated vibration assessment to include proposed design criteria, areas of impact and mitigation procedures. For the purposes of this current report, a number of general comments concerning criteria and potential zones of impact are presented.

7.4.1 Vibration Sources

Construction of the HBTC project is likely to involve the following potential sources of vibration:

- Piling – bored piling is typically proposed, and forms the basis of this assessment.
- Excavation – potentially extending into the Waitemata Formation.
- General construction activities and associated truck traffic.

Typical intensities of vibration from the operation of construction equipment are presented in Wiss (1981) and have been used for this review. Caisson (pile) drilling, operation of a large bulldozer (excavation) and truck operation are all shown to result in similar attenuation relationships between vibration (measured by peak particle velocity, PPV) and distance from the source. Other potential sources (jackhammers, idling cranes, small bulldozers) are shown to result in significantly lower intensities of vibration.

7.4.2 Vibration Criteria

On the basis of a review of council and international criteria, MDA (2003, 2005) recommended HBTC vibration criteria be based on the following Standards:

- Human Response – NS 8176E (1999)
- Protection of Buildings – DIN 4150 Part 3 (1986)

Limiting vibrations indicated by the above Standards have been compared to the attenuation relationship of Wiss (1981). Table 7.4.2 presents a summary of the limiting vibrations and the assessed maximum distance from the source that such vibrations are likely to be experienced considering a likely maximum vibration energy source (assumed to be bored pile drilling).

Table 7.4.2 – Summary of Vibration Criteria and Zones of Impact

Standard	Description	Criteria (PPV)	Maximum Distance from Source
NS 8176E	Vibrations not normally noticeable	0.10mm/s	60m
	Some vibration disturbance	0.15mm/s	45m
	About 8% of people would be highly annoyed by vibrations (typical limit for vibrations in new residential buildings)	0.30mm/s	30m
	About 11% of people would be highly annoyed by vibrations (typical limit for vibrations in existing residential buildings)	0.60mm/s	20m
DIN 4150	Historical, ancient buildings, ruins and monuments	2mm/s	9m
	Buildings visibly damaged and cracked	4mm/s	6m
	Structurally sound buildings	6mm/s	4m
	Industrial buildings, concrete buildings, generally without plaster	10-40mm/s	1m-2.5m

7.5 Pavement Subgrade

Subgrade conditions along the alignment will be expected to vary significantly. Low CBR values of the order of <5% are likely for Fill and Recent Marine/Tauranga Group sediments. Weathered Waitemata Group soils would be associated with higher CBR values of the order of 5% to 7%. Very weak Waitemata Group material is typically associated with design CBR values of >10%.

For areas containing low CBR material, undercutting, reworking, stabilisation (lime or cement), compaction or replacement may be required to provide acceptable and economic subgrade strength.

Further evaluation of subgrade strength along the alignment will be required during detailed design.

8 Hydrogeological Evaluation of Tunnel Options

8.1 Introduction

Preliminary 2D seepage modelling has been undertaken to provide an estimate of drainage requirements and the extent of drawdown associated with short-term construction and long-term performance of the HBTC northbound tunnel.

Results of the current on-going investigation program will be used to refine the 2D modelling, as well as allowing formulation and assessment of 3D models.

8.1.1 Methodology

The preliminary modelling has been undertaken using the commercial software package SEEP/W.

Three hydrogeologic units occur within the study area, the assumed hydrogeologic properties of these units are presented in Table 8.1.1.

Table 8.1.1 – Assumed Hydraulic Conductivities

Unit	Horizontal Hydraulic Conductivity k_x (m/s)	Vertical Hydraulic Conductivity k_y (m/s)	K_y/K_x Ratio
Man-made Fill	$3x10^{-5}$	$1x10^{-6}$	0.033
Tauranga Group	$2x10^{-7}$	$2x10^{-8}$	0.100
Waitemata Group (bedrock)	$5x10^{-7}$	$5x10^{-8}$	0.100

Rainfall (estimated to be approximately 1200 mm/yr from ARC and NIWA statistics) minus evapotranspiration (estimated to be ~60% of rainfall from NIWA statistics) was applied to the model, and resulted in significant ponding and flooding of the surface. This is consistent with observed behaviour on the Victoria Park fields and indicates that the groundwater is predominantly recharged from sources other than localised infiltration.

Given the highly developed nature of this area, recharge to the groundwater system through localised infiltration is expected to be negligible (with the majority of recharge diverted by surface runoff) and has not been included in the analyses.

Modelling of the tunnel itself was carried out by removing mesh elements to create the required geometry.

8.1.2 Boundary Conditions

To determine potential drainage requirements, constant head boundaries were set at either end of the 2D section representing the known groundwater level as determined from static water level records for existing wells in the area (Figure 6). This scenario provides a conservative estimate of drainage discharge volumes as it assumes an unlimited supply of water is available to the system through all geologic units.

To determine potential drawdowns, it was assumed the groundwater was only being recharged through the Waitemata Group. In this case, the groundwater presently stored in the Fill and Tauranga Group can be pumped dry.

Both these scenarios are considered to be conservative and will need to be reassessed as additional information becomes available. Anecdotal evidence from shallow and deep basements within the Freemans Bay – Central City area indicate that generally, drawdowns and inflows are significantly less than modelled.

8.1.3 Model Sensitivity

The majority of groundwater flow into the tunnel has to pass through Waitemata Group rocks. Groundwater flow within this unit will be dominantly from within fractures. The hydraulic conductivity of this unit is therefore dependent on localised fracture density and connectivity. In order to check the sensitivity of discharge to hydraulic conductivity of the Waitemata Group, a range was evaluated and the results summarised in Table 8.1.3 for several tunnel configurations used in the analyses.

Table 8.1.3 – Model Sensitivity to Waitemata Group Hydraulic Conductivity

Tunnel Configuration	Daily Discharge Into Tunnel (m^3/d)	
	Waitemata Group $k_h = 5x10^{-7}$ m/s $k_v = 5x10^{-8}$ m/s	Waitemata Group $k_h = 1x10^{-7}$ m/s $k_v = 1x10^{-8}$ m/s
Drainage only, no cut-off walls	41	13
Drainage, 5m cut-off wall on both sides	26	6
Drainage, 6m cut-off wall on both sides	24	5

This relatively small change in conductivity has a significant impact on the volume of groundwater to be drained from the tunnel. Therefore, it is appropriate for each drainage / cut-off scenario to consider a range of inflows.

8.1.4 Tunnel Options

Undrained and drained tunnel conditions have been considered.

a. No drainage

The tunnel was initially modelled as a submerged box without drainage. As expected, this results in damming of the groundwater up gradient (west) of the tunnel raising the groundwater level to the ground surface (Figure 7).

Should an undrained option be employed, damming of the groundwater is unlikely to be permissible and would likely be dealt with using a passive collector/recharge trench system.

b. Drained Tunnel

A drained tunnel condition with pumping assumed to be at the base of the tunnel has been modelled (Figure 8). This option would require the capture, treatment and disposal of pumped groundwater and would result in drawdown on either side of the tunnel. It is important to note that the down gradient groundwater flow is reversed, so flow enters the tunnel from the east side as well as the west.

The volume of groundwater inflow and drawdown could be reduced with the installation of cut-off walls (also required for tunnel structural stability). The results of various cut-off wall combinations are discussed in Sections 8.2 to 8.4.

8.1.5 Short Term Construction vs. Long-Term Permanent Dewatering

Modelling indicates that the majority of the predicted long-term steady state drawdown and inflows occur within an 18 to 36 month period. Consequently, depending on construction programs, construction staging etc, there will be a requirement to consider potential drawdown regardless of final tunnel design (drained or tanked).

8.2 St Marys Bay to Victoria Park CH1200 to CH1550

8.2.1 Extent of Drawdown Due To Drainage

The extent of drawdown associated with temporary (construction) or long-term (permanent) drainage of the tunnel will vary on either side of the tunnel due to the contrast in geologic profile. Drawdown on the down-gradient side may be significant (2m to 3m at 100m distance from the tunnel) and therefore the potential for saline intrusion and/or migration of contamination associated with the Western Reclamation due to flow reversal exists. Drawdown on the up gradient side may be less extensive due to the significant thickness and relatively shallow depth of Waitemata Group.

8.2.2 Likely Groundwater Inflows

At the northern portal, with no cut-off walls a groundwater inflow of 0.02 to 0.07m^3/day per m length of tunnel is predicted (dependent upon the hydraulic conductivity of the Waitemata Group), equivalent to 2 to 7m^3/day of groundwater for the 100m length of tunnel at this section. Modelling indicates that the installation of 6m long cut-off walls could reduce the inflows by 40% to 60%.

8.3 Victoria Park CH1550 to CH1950

8.3.1 Extent of Drawdown Due To Drainage

Modelling of a section through CH1850 indicates that drawdown associated with temporary (construction) or long-term (permanent) drainage of the tunnel could extend some 200m to 300m either side of the tunnel. Drawdown immediately adjacent to the tunnel could be reduced by up to 50% to 75% through the installation of 6m long cut-off walls (Figures 9 and 10). The modelling indicates that the cut-off walls do not have a significant impact on the drawdown at distance, predicted to be up to 4m at 100m distance (down gradient) from the tunnel.

Some potential for migration of contamination associated with Western Reclamation due to flow reversal exists. Drawdown on the up gradient side may be slightly less extensive due to the lesser thickness of Fill and shallower depth to Waitemata Group.

8.3.2 Likely Groundwater Inflows

With no cut-off walls a groundwater inflow of 0.02 to 0.07m^3/day per m length of tunnel is predicted (dependent upon the hydraulic conductivity of the Waitemata Group) – this would be equivalent to 8 to 28m^3/day of groundwater for the 400m length of tunnel at this section.

Modelling indicates that with 6m long cut-off walls on either side of the tunnel, groundwater inflows could reduce to 4 to 16m^3/day.

8.4 Victoria Park to Wellington Street CH1950 to CH2400

8.4.1 Extent of Drawdown Due To Drainage

Modelling of a section through CH2000 indicates that drawdown associated with temporary (construction) or long-term (permanent) drainage of the tunnel will be comparable to the Victoria Park section.

8.4.2 Likely Groundwater Inflows

Groundwater inflows of 0.015 to 0.050m^3/day per m length of tunnel are predicted (dependent upon the hydraulic conductivity of the Waitemata Group) for this section where Waitemata Group is encountered at a shallower depth and the thickness of Fill/Tauranga Group is significantly less. This would be equivalent to 1.5 to 5m^3/day of groundwater for the 100m length of tunnel at this section.

Modelling indicates that with 6m long cut-off walls on either side of the tunnel, groundwater inflows could reduce to 0.4 to 2m^3/day.

8.5 Anecdotal Evidence - Britomart – Queen Street Station

This project required excavation of an approximately 300m (L) x 50m (W) x 10m (D) basement through Fill, Tauranga Group and Waitemata Group materials comparable to those in the Freemans Bay area.

During the excavation period, a sump of approximately 8m^3 required pumping 1 to 2 times per day. The sump drained not only the excavation faces but also rainfall and a "significant volume of water leaking from a nearby tunnel". Therefore the actual seepage from the excavation was considered to be significantly less than 8m^3/day. Modelling of the excavation with assumed groundwater inflows of 1 to 8m^3/day, the hydraulic conductivity of the Waitemata Group is assessed to be in the order of $5x10^{-8}$ to $5x10^{-7}$m/s (horizontal) and $5x10^{-9}$ to $5x10^{-8}$m/s (vertical). On this basis, the range of hydraulic conductivities assumed for the Waitemata Group for the HBTC modelling appears reasonable.

Groundwater monitoring of the area surrounding Britomart indicates that long-term drawdown was not significant (< 1m drawdown within 10m of the site boundary reducing to less than 0.2m drawdown at 100m distance) suggesting likely recharge of the surface sediments. This has not been taken into account during the current preliminary modelling of the HBTC tunnel.

8.6 Settlements

Drawdown of the extent indicated in the preceding sections will result in significant settlement of the Fill and marine (Tauranga Group) sediments due to their soft/loose in-situ condition. Table 8.6 presents estimates of settlement of the Fill and marine sediment for varying levels of drawdown and varying profile thicknesses.

Table 8.6 – Estimate of Drawdown Settlement

Profile	Drawdown (m)	Fill Settlement (mm)	Marine Settlement (mm)	Total Profile Settlement (mm -50% to +100%)
5m Fill, 5m Marine	2	75	200	275
	3	75	250	325
	4.5	100	350	450
5m Fill, 2.5m Marine	2	75	100	175
	3	75	150	225
	4.5	100	200	300
2.5m Fill	2	25	-	25
	3	25	-	25
	4.5	25	-	25

Estimates in Table 8.6 are based on an assumed existing groundwater level at 1.0m depth and compression ratio ($C_C/1+e_O$) values of 0.1 and 0.25 for the fill and marine sediments respectively. Potentially greater settlement could occur in response to a higher groundwater level, greater drawdown and greater compressibility.

Settlements of the order indicated would impact on surface elements (shallow foundations, pavements, services including the existing stormwater pipe across Victoria Park etc). Whilst such settlements are indicated to occur over broad areas and consequently typical average rotation profiles may not be severe, localised differential settlements may occur due in particular to the variable nature of the fill deposits.

The extent of settlement would result in full mobilisation of negative skin friction (NSF) on embedded elements within the drawdown footprint, including the HBTC tunnel wall and existing piles such as the VPV.

It should also be noted that above settlement estimates relate to long-term consolidation of the Fill and marine sediments, and may take 10+ years to occur. These estimates do not include 'immediate' excavation related movements that are a function of system stiffness, staging etc. Such movements could be of the order of 50mm decreasing to zero at a horizontal distance about three times the excavation depth, and can be expected to occur relatively quickly (see Section 7.2.2e).

8.7 Possible Mitigations

In order to lessen the effects of short or long term drainage on groundwater and associated potential migration of contaminants, the following options could be considered:

- Surface recharge system to allow replenishment of the Fill and Tauranga Group; this could include reuse of water drained from the tunnel (however this would require a separate resource consent); or
- A bentonite curtain parallel to the tunnel that penetrates to the Waitemata Group; this would lessen the drawdown at distance assuming that some recharge of the Fill and Tauranga Group occurs. If no recharge occurs then in the long term the units will still drain completely.
- Recharge trenches to collect and then disperse surface water to improve infiltration.

9 Additional Investigation and Testing

Further investigation and testing will be required for subsequent design and will need to be refined to suit the final scheme arrangement. It is likely that additional work will be required to address the following issues:

9.1 St Marys Bay

- Investigations of specific locations where extensions to fill embankments are proposed to provide data for an assessment of post construction differential settlements and to confirm foundation conditions for retaining walls, gantries and other structures.
- Investigation of the Shelly Beach Road southern abutment and at the proposed relocated southern pier location to confirm founding and stability conditions.
- Investigations to assess pavement subgrade.
- Investigations to assess the stability of the existing reclamation.

9.2 Victoria Park

- The current borehole spacing may not adequately define the interface between Waitemata Group rock and overlying fill/sediments. Additional investigation in the form of further drilling, cone penetration testing or geophysical surveys should be considered so that depth to rock is verified, in particular the presence locally deepened drainage channels.
- Further investigation and analysis is required to address retention and groundwater issues for detailed consideration of the proposed tunnel.
- Investigation for ancillary structures outside the tunnel footprint.

9.3 Wellington Street

- Investigation along the western side of the proposed approach ramp alignment to provide data for retention system design and assess the stability of the fill embankment and retaining structures above Freemans Bay primary school.
- Investigations to assess pavement subgrade.

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- Appendix A

Figures