



Western Ring Route – Waterview Connection



Coastal Works Engineering Report



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1. Introduction

The 'Waterview Connection Project' is the final key project to complete the Western Ring Route (WRR). In 2009 the NZ Transport Agency (NZTA) confirmed that the Waterview Connection Project (Project) would be lodged with the Environmental Protection Authority as a Proposal of National Significance (RoNS). The Project will be the largest roading Project undertaken in New Zealand, and due to its size and complexity has been divided into nine Project Sectors. These Sectors broadly define the different planning and construction requirements of the Project. A diagram of these Sectors is presented in Figure 1-1.



Figure 1-1 - Waterview Connection Project Sector Diagram

The improvements to SH16 provided as part of the Waterview Connection Project are approximately 8km in length, extending from the St Lukes Road Interchange to Henderson Creek, and will primarily consist of widening the existing motorway with additional lanes to accommodate the increased traffic demand from SH20.

The SH16 alignment between the Great North Road and Te Atatu Road Interchanges passes through an estuarine area, crossing parts of the central Waitemata Harbour. From the Great North Road Interchange to the Rosebank Road Interchange, and between the Patiki Road Interchange and Whau River, sections of the carriageway are formed on low man-made embankments and the improvement works will require reclamation of the Coastal Marine Area (CMA).

The following project sectors have works that will encroach into the CMA:

- **Sector 1** – Te Atatu Interchange;
- **Sector 2** – Whau River;
- **Sector 4** – Causeway Reclamation;
- **Sector 5** – Great North Road Interchange.

The section of SH16 between Great North Road and Rosebank Road is commonly referred to as the ‘Causeway’ (see Figure 1-2). The Causeway has been suffering from ongoing long-term settlement and has required periodic shape corrections to alleviate differential settlements since its construction in 1952. At present parts of the existing Causeway are occasionally inundated by extreme high tides and taking into consideration the anticipated sea level rise projections more frequent closures of the motorway are likely. As a result the majority of the coastal works required to accommodate the SH16 improvements will be adjacent to the northern and southern edges of the Causeway (Sector 4).



Figure 1-2 - Photograph of the Causeway taken in 2008

1.1 Report Purpose and Scope

NZTA has confirmed the following Project Objectives for the Waterview Connection Project:

1. To contribute to the region's critical transport infrastructure and its land use and transport strategies:
 - by connecting SH16 and SH20 and completing the Western Ring Route;
 - by improving the capacity and resilience of SH16.
2. To improve accessibility for individuals and businesses and support regional economic growth and productivity:
 - by improving access to and between centres of future economic development.
3. To improve resilience and reliability of the State Highway network:
 - by providing an alternative to the existing SH1 corridor through Auckland that links the northern, western and southern parts of Auckland;
 - by securing the SH16 Causeway against inundation.
4. To support mobility and modal choices within the wider Auckland Region:
 - by providing opportunities for improved public transport, cycling and walking;
 - by protecting opportunities for future passenger transport development (e.g. rail).
5. To improve the connectivity and efficiency of the transport network:
 - by separating through traffic from local traffic within the wider SH20 corridor.

For the Project to comply with the Project Objectives, SH16 must be widened to improve capacity and provide opportunities for improved public transport, cycling and walking. The elevation of the motorway Causeway between Great North Road and Rosebank Road will also need to be increased to prevent inundation and therefore improve resilience.

Where reclamation is required, the philosophy has been to carefully define the extent needed to accommodate the reclamation, permanent occupation and any necessary temporary works. Therefore all design elements that might affect the overall footprint of the enlarged reclamation have to be fully assessed.

The report provides information on the proposed Project works and how these relate to reclamation and occupation of the adjacent CMA. The report describes the locations of reclamation, permanent occupation and temporary occupation in relation to the particular works. It also provides details of the likely activities, construction methodology and timing of the works in order for the environmental effects to be assessed.

The Coastal Works Report is supported by three supporting documents which provide further detail and engineering justification for the Project works. A brief outline of the three supporting documents is summarised below:

- **Causeway Options Report (20.1.11-3-R-J-304)** – investigates and assesses the engineering solutions to provide a motorway connection between the Great North Road and Rosebank Road Interchanges and recommends a preferred solution;
- **Interpretation of Hydrodynamic Design Conditions Report (20.1.11-3-R-J-305)** - defines the design performance requirements the Causeway engineering must achieve taking into consideration the effects of climate change. It also establishes the elevation and coastal protection measures for the motorway embankment to secure future operation of the motorway;
- **Coastal Works Engineering Report (20.1.11-3-R-J-306)** (this report) – summarises the proposed engineering works in the CMA to ensure that the design meets the performance requirements concluded within the Interpretation of Hydrodynamic Design Conditions report. The report demonstrates the extent of works required in the CMA for the permanent and temporary occupation. It also details activities, methodology and timing of the construction works.

The Coastal Works Engineering Report Volume 1 is supported by a sister document (Coastal Works Engineering Report Volume 2) which presents all A3 drawings referred to in Volume 1.

The Coastal Works Engineering Report also references to the following Reports:

Ground Improvement Options Report: Reclamation of the Coastal Marine Area (CMA) adjacent to the existing Causeway will settle. The existing Causeway will also settle further due to the proposed increase in elevation. In addition these new areas of land will be susceptible to slope instability. To address these shortcomings ground improvements will be required to address both instability and reduce settlement effects.

This report assesses potential ground improvement options and the geotechnical implications of widening and raising the existing Causeway, using geotechnical information presented in the Geotechnical Interpretative Report (Aurecon, February 2010a). This report presents the following:

- Range of possible ground improvement options;
- Environmental and constructability issues;
- Stability analysis of preferred ground improvement options;
- Overbuild required to meet 3m RL in 2100 as recommended in the hydrodynamic interpretation;
- Settlement during construction and through operation to 2100;
- Construction methodology of preferred ground improvement options.

Fill Options Report: This report outlines potential fill options for the widened and raised Causeway. The fill will comprise materials of various specifications to suit different locations of the embankment such as bulk fill, rock armouring, fill below the embankment shoulders and material used to construct the new pavement. This report outlines the different types of material that may be available and discusses the potential use of these materials for the different fill situations. The report then recommends preferred fill materials.

Coastal Protection Report: This report considers possible coastal protection options required for the widened Causeway. The coastal protection is required to protect fill material from scour and erosion due to coastal hydrodynamics and wave attack.

This report presents the following:

- Further interpretation and analysis of the hydrodynamic conditions and setting of design parameters;
- Assessment of possible coastal protection options with recommendation of a preferred option;
- Environmental and constructability issues;
- Preliminary design of preferred coastal protection option.

1.2 Report Structure

This report provides a summary of the Coastal Works related engineering activities proposed between the Great North Road Interchange and Henderson Creek. The report is presented in 2 volumes, Volume 1 being the text and Volume 2, the Drawings referred to in the text.

Volume 1 is structured as follows:

- **Existing Ground Conditions** – provides an overview of the typical ground and groundwater conditions underlying the SH16 alignment between the Great North Road Interchange and Henderson Creek;
- **Design Methodology** – summarises the design methodology adopted for the ground improvement works;
- **Preliminary Geotechnical Assessment** – presents the findings of slope stability, bearing capacity and settlement analysis without implementing ground improvement measures;
- **Ground Improvement Options** – provides an overview of the proposed ground improvement techniques adopted;
- **Geotechnical Assessment with Ground Improvements** – presents the findings of slope stability, bearing capacity and settlement analysis, which incorporate ground improvement measures;
- **Ground Improvement Works** – discusses the type and location of the ground improvement techniques that are proposed to be undertaken;
- **Embankment Construction Materials** – provides an overview of the types and volume of construction materials that may be used during the construction works;
- **Coastal Protection** – discusses the types and locations of different coastal protection measures proposed to be used;
- **Bridge Improvement/ Construction** – discusses the proposed construction phasing and temporary staging platforms required at the Whau River and Causeway Bridges;
- **Channel Relocation Works** – provides the proposed construction methodology for the relocation of two channels located in Oakley Inlet and one within the Inner Harbour;
- **Construction Methodology** – provides an overview of the construction methodology for the ground improvement works and embankment construction;
- **Reclamation Construction Programme** – provides a summary of the anticipated timeframes required to undertake preparation works, ground improvement, embankment construction, temporary drainage and erosion and sediment control;

- **Operational Considerations** –provides a brief outline of the anticipated operation considerations, including ongoing settlement, monitoring, pavement maintenance, drainage maintenance and coastal protection maintenance.

2. SH16 Reclamation

The SH16 improvements require the reclamation of seabed adjacent to the existing footprint. This section discusses the definition of reclamation (in the context of this project), the existing embankment profile, the requirement for reclamation and the general philosophy adopted.

2.1 Occupation Definition

This section discusses the definition of reclamation (in the context of the Project) and the philosophy adopted. Typically, in engineering terms, reclamation is the process of converting ground that is permanently or intermittently inundated by water into land that is permanently above sea or flood level. For the SH16 improvements this would encompass areas adjacent to the existing motorway embankment that are currently occupied by intertidal mudflats.

For this project the area of **Reclamation** has been given a strict definition of the creation of land from the existing to the proposed CMA boundary that is anticipated following completion of construction. The existing and proposed CMA boundary has been defined as Mean High Water Springs (MHWS). The MHWS for the Project is 1.63mRL (Relative Level - Auckland Vertical Datum 1946).

Therefore, the permanent elements related to the improvement works which lie below MHWS are considered as **Permanent Occupation** of the CMA. The permanent work covers man-made structures that support the new motorway infrastructure and include embankments below MHWS, pier locations for new bridge structures and ground improvements to the founding soils.

In addition to the Permanent Occupation there is also the need to temporarily occupy intertidal or subtidal zones. These areas will extend beyond the boundaries of the permanent footprint and are necessary in order to accommodate the requirements and activities to allow for safe construction and environmental compliance. Therefore this area is referred to in the report as **Temporary Occupation** of the CMA. The duration that the CMA will be occupied will depend on the work activity required with an approximate duration of these works provided within this report.

A diagrammatic representation of the areas of Reclamation, Permanent Occupation, Temporary Occupation and Marine Habitat Remediation is shown in Figure 2-1:

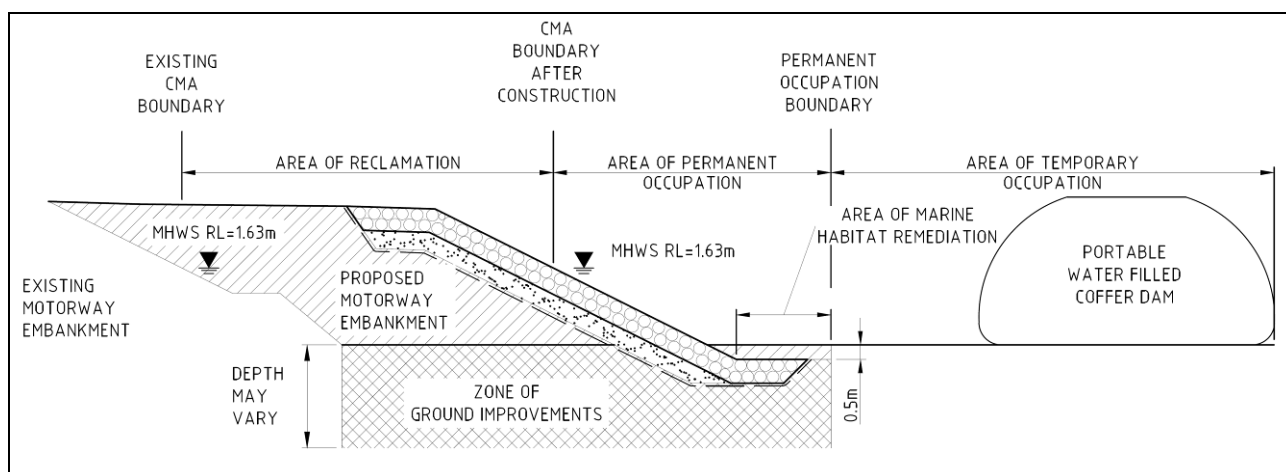


Figure 2-1 – Indicative diagram showing Reclamation, Permanent, Temporary Occupation and Marine Habitat Remediation (not to scale)

2.1.1 Existing Causeway Embankment

The area referred to as the existing Causeway embankment incorporates all the material that has been artificially placed between the northern revetment toe and the southern revetment toe. However, as the existing revetment toes have partially merged with the mudflats it is likely that some of the boulders that form the revetment have migrated outward and downward. Furthermore, it appears that siltation has occurred where mud has been deposited in and around the lowermost boulders of the revetment.

The existing embankment extends from the existing Causeway Bridge abutments to where it merges with the natural ground on the Rosebank Peninsula to the west and in the Waterview Interchange area to the east. The construction materials used in the existing embankment are discussed in the Fill Options Report and typically comprise:

- Bulk fill - main body of the embankment;
- Revetment - outer surfaces of the embankment providing coastal protection;
- Pavement layers – trafficked area and shoulders.

From north to south, the surface of the existing embankment comprises the following elements:

- northern (seaward) revetment;
- northern verge;
- pavement of traffic lanes and shoulders;
- southern verge, incorporating a cycleway;
- southern (landward) revetment.

The limited historical records and existing publications indicate that the existing embankment was placed directly upon the mudflats. There is no evidence to suggest that any material was excavated for the lowermost layers of the embankment to be placed in an undercut zone.

Interpretation of the Stage 1 and Stage 2 ground investigations (Aurecon, 2010a) shows that the basal surface of the existing embankment lies significantly below the level of the mudflats on either side. In the worst case at approximately Chainage 1600, the overall thickness of construction fill is 7.0m, with about 50% lying below the level of the mudflats.

This could suggest that the marine mud has deformed considerably following the placement of embankment fill. However it is believed that a large amount of embankment fill was also used to fill up previous local depressions and channels in the marine mud. It is indicated in the historical records and existing publications that the existing embankment was placed directly onto the previous marine mud surface by end-tipping.

Before the construction of the embankment, tidal water could freely drain out via drainage channels. The tidal water flow was concentrated in drainage channels that were formerly orientated across the alignment but following construction became re-orientated to join the major tidal channel below the Causeway Bridges. Reviewing the contemporary and historical alignment of the channels, it is evident that the embankment construction crossed several of these waterways which must have been infilled during the original construction, resulting in locally thicker and wider embankment fill along the Causeway.

2.1.2 Existing Profile

The horizontal profile of the existing embankment has a high point at the central median of the highway. Despite reports of settlement having taken place since construction, the highway profile has maintained this lateral fall from the median towards the verges (this is likely to have been assisted by re-surfacing).

The northern grass verge and southern cycleway show local areas where the ground surface forms a dip. Along both the northern and southern crests there are places where the top of the rock revetment stands proud of the grassy verge behind it, either because the rock was placed higher and the ground on the inward side has settled locally.

It is evident when driving along the Causeway that the road surface dips below a high point located at the Causeway bridges to a low between these bridges and Traherne Island. The sections indicate that the lateral fall across the traffic lanes and shoulders is about 0.5m and the lateral fall between the median and the grass verge on the inside of the slope crest is generally 0.7m to 1.0m.

2.2 Requirement for Reclamation

The NZTA has given a number of objectives that the SH16 improvements must meet. These include:

- Improving capacity;
- Providing improved public transport, cycling and walking facilities;
- Securing SH16 against inundation.

In order to comply with these objectives to improve capacity and provide public transport facilities and upgrade the existing pedestrian/cycleway, the highway must be widened. The elevation of the Causeway section of the highway will also need to be raised to prevent inundation.

In order to accommodate the additional lanes, the Causeway (including bus lanes and cycle paths) will need to be around 70m wide, which is significantly wider than the existing Causeway section. However, the footprint will be greater than this and will be governed by the angles of the side slopes of the new embankment. Depending on the height of the Causeway and the slope angle selected this is likely to increase the footprint by around 5 to 10m on either side.

The ground improvement options discussed in Section 8 will also affect the width of the permanent occupation footprint.

2.3 Reclamation Philosophy

During the design of the improvement works, one of the philosophies was to minimise the extent of reclamation. The majority of the planned reclamation is within the Motu Manawa Marine Reserve (MMMR or Pollen Island) and requires a re-designation of the boundary between the NZTA highway land and the marine reserve. It has been accepted that reclamation is necessary and it has been agreed that consent will be sought just once. This means that the extent of reclamation needs to be carefully defined and that it should include the maximum extent needed to accommodate the works. Therefore all design elements that might affect the overall footprint of the reclamation have to be fully assessed and the designation defined accordingly.

During the design process, various reclamation options were considered. A staged reclamation process has been developed to support the optimum traffic management philosophy during construction; reclamation of the seaward side of the existing embankment will be completed first, then the landward side. Refer to the Causeway Options Report for more details.

3. Existing Conditions

3.1 Existing Environment

As discussed in Section 2, part of SH16 crosses the Central Waitemata Harbour on a man-made Causeway, which was initially constructed during the 1950s. During the 1990s much of this area was designated as Motu Manawa Marine Reserve (Pollen Island). This reserve covers around 500 hectares of the Central Waitemata Harbour, encompassing Pollen Island and Traherne Island, intertidal mudflats, tidal channels, mangroves, saltmarsh and shellbanks. This reserve is now considered to be one of the best examples of mangrove and saltmarsh habitat in the Central Waitemata Harbour, and is a rich feeding ground for a wide range of wildlife and plant life. The majority of the notable habitat is around Motu Manawa Marine Reserve (Pollen Island) including the mudflats, mangroves and saltmarsh habitat present along the seaward extent of the island.

It is understood that the habitat (both marine and terrestrial) present along both sides of SH16 (particularly along the Causeway) is of poor quality due to the growth of weed species and accumulation of flotsam and litter. These areas (in particular the landward side) are down-stream from numerous industrial and commercial properties within the Rosebank Peninsula and Oakley Inlet areas creating an accumulation of contaminants within the marine sediments. Because there is currently limited surface water treatment system installed on the Causeway, the sediments immediately adjacent to the Causeway may also have been contaminated through surface run-off directly from the highway.

Preliminary ground investigation undertaken as part of the design process concluded that significant amounts of contamination are not present within the soils or the marine sediments. However, it is possible that 'hotspots' of contamination may exist and so a management plan will need to be implemented before any works are undertaken.

3.2 Geology

The area of SH16 that runs between Henderson Creek and Great North Road is generally underlain by four geological strata of different ages, discussed in summary below.

3.2.1 East Coast Bays Formation (ECBF)

This is the oldest geological formation present in the project area; it was formed around 25 million years ago and is over 500m thick. These deposits generally comprise interbedded sandstones and siltstones. The surface of the ECBF is over 50m below ground below the central Causeway area but is much shallower to the east of the Causeway bridges and to the west at the Whau River. The ECBF is at surface or near surface between the Whau River and Henderson Creek.

3.2.2 Tauranga Group Alluvium

This variable assemblage of generally terrestrial sediments overlies the ECBF and was largely deposited over the Pliocene and Pleistocene between 3.6 million and 10,000 years ago. These deposits generally comprise layers of clay, silt, sand, gravel and highly compressible organic clays / peats. The thickness of the unit is highly variable. It is greater than 50m thick underneath the existing SH16 Causeway and less than 2m in other areas (such as the area of the proposed wetland in Jack Colvin Park).

3.2.3 Holocene Alluvium

This is the most recent of the natural geological strata with placement beginning around 8,000 to 14,000 years ago (i.e. in the current marine transgression, since the last Ice Age). Deposits are still being laid today. These deposits are littoral and so only encountered within intertidal areas (such as beside the Causeway). The thickness of this material is between 2m and 12m. These deposits are generally very soft to soft muddy silts with shell fragments.

3.2.4 Fill

For much of its length, this section of SH16 is immediately underlain by a layer of construction fill. The most significant body of fill forms the embankment traversing the intertidal mudflats (the Causeway) first constructed some 60 years ago. The material that forms the fill layer is not well documented. It appears to be variable and there is some discussion as to what it comprises.

3.2.5 Surface Water and Groundwater Conditions

3.2.5.1 Surface Water

The motorway crosses mud flats associated with the Central Waitemata Harbour and Whau River which are regularly flooded by the high tide so that surface water laps against the sides of the embankments. At low tide surface water is restricted to tidal channels and the permanent streams of Oakley Inlet and the Whau River.

Oakley Inlet meanders into the Central Waitemata Harbour from the Waterview Estuary area to the east. The meander of the Inlet channel encroaches on the toe of the Causeway at two locations. A number of tidal channels are also present within the Central Waitemata Harbour with the most defined channel draining the sheltered Estuary area running generally parallel with Traherne Island before deflecting along the Causeway to convey the ebb tide to exit beneath the Causeway Bridges. For about 120m this channel runs immediately alongside the toe of the existing embankment.

Water level monitoring at the site is currently ongoing, however a summary of the data collected to date is presented below.

A tidal gauge has been installed onto a pier of the Causeway Bridges; over the period of monitoring this has measured tidal levels that vary between -1.7m RL (low tide) and 2.15m RL (high tide). The results (plotted in the following

Groundwater Figures) have been compared with published results from the Ports of Auckland (Ports of Auckland 2003) and indicate that high and low tides at the Causeway have a lag time of approximately 15 minutes. The highest and lowest tides shown in the record to date relate to king (perigean-spring) tides.

3.2.5.2 Groundwater

Groundwater monitoring is currently being undertaken along SH16 as part of the Stage 3 Geotechnical Ground Investigation, and is presented and discussed here in advance of production of the Stage 3 Geotechnical Interpretative Report. Eight standpipes have been installed within the central median of the motorway; they contain automatic ground level monitors (“divers”) that are either sealed within the Causeway fill material or the underlying Recent Alluvium.

Of the 8 drillholes with automatic piezometer monitoring, drillholes DH418 and DH424 are located on the Causeway, DH426 is located on the transition between the Causeway and Traherne Island, while DH428 and DH432 are located on Traherne Island. Drill holes DH424, DH426 and DH432 are configured as double arrays, with an additional hole being drilled approximately 1m away from the main borehole. This additional borehole is denoted with a suffix ‘a’ and allowed the groundwater to be monitored in two strata at the same location, with one monitoring device being installed within the Causeway fill material and the other in the Recent Alluvium. The purpose of this double array is to determine differences in hydrogeological conditions between the two strata. Layout plans showing the locations of the groundwater monitoring stations are presented in drawings 20.1.11-3-D-J-200-170 to 171 in Volume 2 of this report.

3.2.5.3 Causeway

A total of three monitoring points (DH418, DH424 and DH424a) are located along the Causeway. Each instrument records similar groundwater levels / cycles, the results of DH424 and DH424a being presented in Figure 3-1.

The results indicate that the groundwater within the Recent Alluvium is greatly affected by tidal variations. However, there is a lag period of between 30 minutes (within the Fill) and up to 4 hours (in the Recent Alluvium) between high tide and the highest groundwater level being recorded.

The groundwater level does not mimic the amplitude of tidal water levels, with a range of approximately 0.8m between the highest and lowest water levels. The groundwater level ranges between 1.7 and 2.45m below pavement level, with the groundwater level in the Recent Alluvium being around 0.25 to 0.5m deeper than the fill.

One notable difference in the groundwater levels recorded in the standpipes installed in the Fill and Recent Alluvium is that groundwater levels within the Fill experience ‘spikes’ where the groundwater level quickly rises and falls by around 10 to 20cm. These ‘spikes’ have been compared to rainfall data collected from three locations within the vicinity of SH16 (presented in Figure 3-1, Figure 3-2 and Figure 3-3 as Oratia, Harmel and Te Pai Rainfall data Lines). The ‘spikes’ in groundwater levels and the corrected rainfall data appear to correlate. This suggests that the groundwater level in the fill is affected by rainfall.

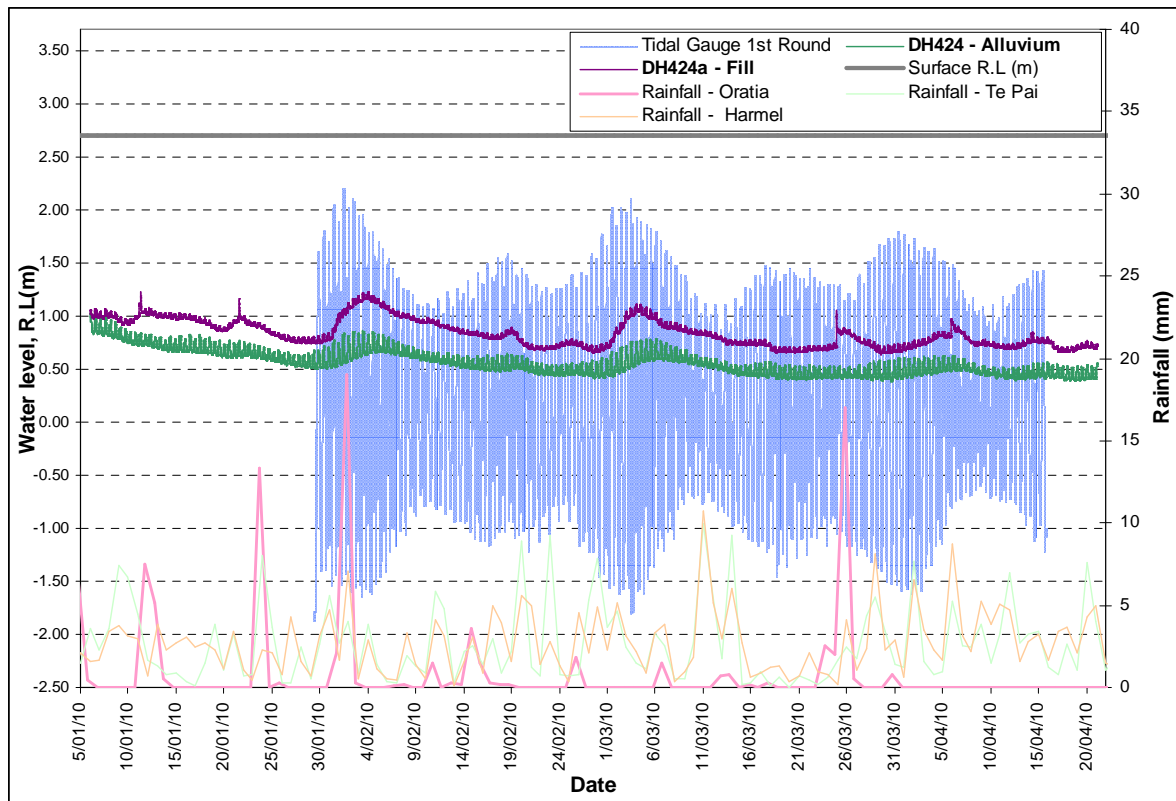


Figure 3-1 - Groundwater results DH424 and DH424a

3.2.5.4 Traherne Island

Standpipes located at three locations along Traherne Island and at the transition between the Causeway and Traherne Island (DH424 and DH426 have been monitored; the groundwater results are presented in Figure 3-2 and Figure 3-3 respectively).

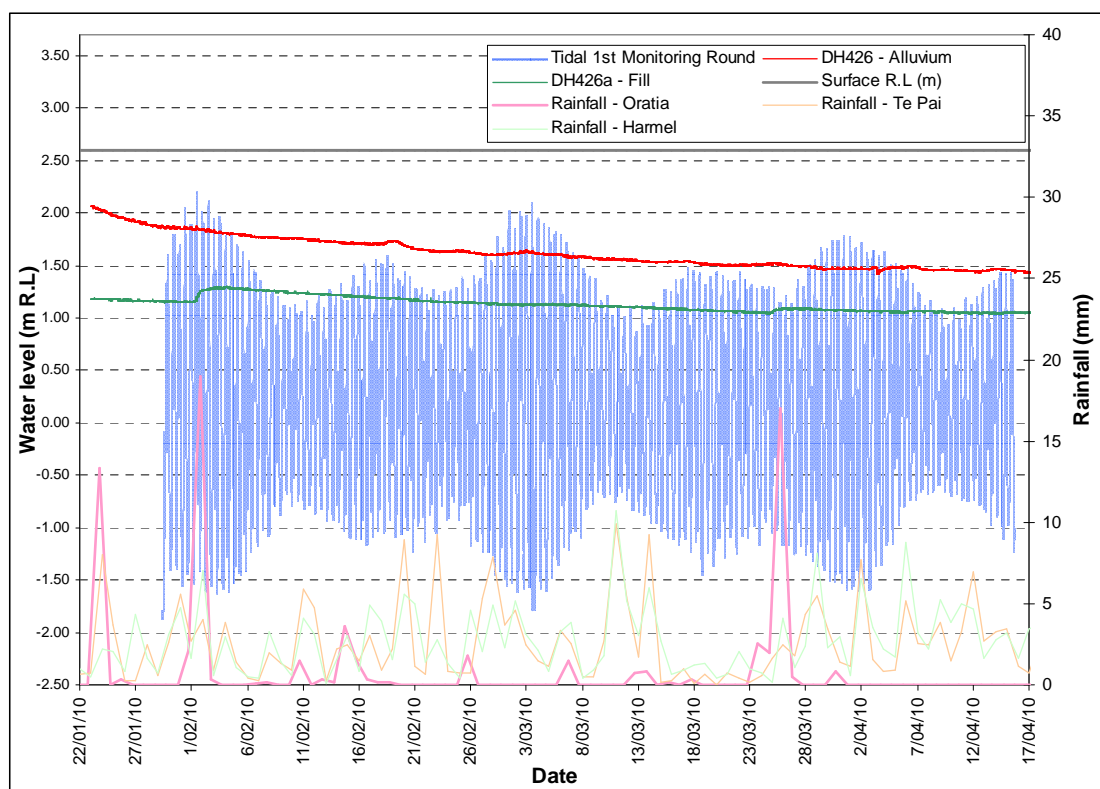


Figure 3-2 - Groundwater results from DH426

Figure 3-2 presents the groundwater monitoring information collected from DH426 and DH426a. The groundwater level at this location is only very slightly affected by tidal variations with a daily range of around 0.05m encountered in both boreholes. Unlike the other borehole clusters that were monitored, groundwater levels encountered within the Fill layer were consistently 40 to 50cm lower than those encountered in the underlying alluvium deposits.

Other than the differences in water level between these two boreholes, they show a very similar trend with a consistent fall in groundwater depths of between 0.3 and 0.5m over the 4 month monitoring period. This monitoring period is not long enough to determine the precise cause of this reduction. It is postulated that this fall is probably caused by a general reduction in groundwater depths encountered during the summer months. As the groundwater monitoring is still in progress, it is expected that additional data may shed more light on the perceived anomaly.

The water level within the fill material appears to be only affected by significant rainfall events as noted at the beginning of February and end of March with a distinct rise in groundwater level. However, unlike the groundwater level on the Causeway, the water level takes a significant amount of time to return to the water level prior to the rainstorm. The average groundwater level in DH432 and DH432a is approximately 1.5m R.L.

Figure 3-3 presents the groundwater monitoring data obtained from two standpipes installed at the location of DH432, which is located at the western side of Traherne Island. The groundwater levels encountered within both of the boreholes

are very similar (within 10cm) and also measure a similar downward trend (as encountered in the other bores located on Traherne Island).

Groundwater levels at this location are typically in the region of 1.8m below ground level. This level is marginally affected by rainfall with a rise of 3 to 5cm being recorded following a rainfall event.

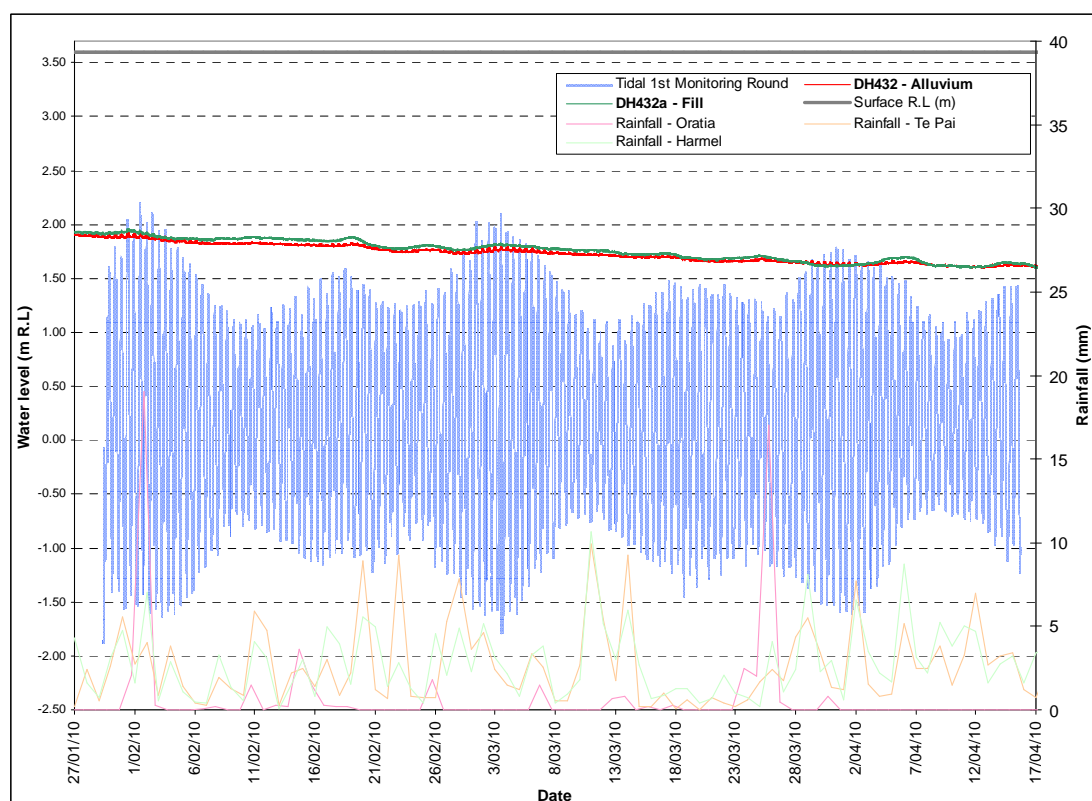


Figure 3-3 - Groundwater monitoring results from DH432

3.2.5.5 Summary of Groundwater Regime

The groundwater regime across the Causeway differs significantly from Traherne Island with the groundwater along the Causeway being significantly affected by tidal cycles, while the groundwater along Traherne Island is only marginally affected. This reflects the proximity of the sea along the Causeway and its distance at Traherne Island.

Rainfall appears to affect the groundwater level within the fill materials but not the underlying alluvium.

Generally groundwater levels along the Causeway vary between 1.7 and 2.45m below the surface of the pavement, while groundwater levels underlying Traherne Island vary between 1.2 and 1.8m below ground level. This means that under normal conditions the pavement layers and subgrade of the existing motorway are not submerged and the fill of the carriageway is being actively drained.

3.3 The Existing Causeway

The existing Causeway was constructed by end-tipping and compacting fill over very soft unconsolidated Holocene Alluvium (Marine mud).

A technical paper originally presented at the first conference held in New Zealand on Soil Mechanics and Foundation Engineering (Newland and Allely, 1952) provides some insight into the Causeway composition. The authors reported on the construction of a 4-lane highway embankment on a tidal flat near Waterview, which presumably refers to the original construction of the Causeway:

“The methodwas to build two parallel bunds of scoria with cross-bunds spaced at five chain intervals, with the intention of filling the space between these at a later date with selected fill which was obtainable near at hand. The bunds were placed approximately 80 ft. apart and each one was about 15 ft. wide at the top with batter slopes of about 35 degrees to the horizontal.”

Based on as-built records, which imply that the existing embankment fill was formed by cohesive clay bunds with a central granular core fills made of scoria. Preliminary results from recent ground investigation describe the density and consistency of the existing scoria and cohesive fill as medium dense to dense granular fill (SPT 'N' > 15) and stiff to very stiff clay (Undrained shear strength (S_u) > 60kPa) respectively.

Assumptions have been made for the long-term (drained) parameters as Cohesion (c') = 1kPa and Friction Angle (Φ') = 32 degrees in accordance with the limited tri-axial testing results carried out within the sidling fills as part of the Stage 1 & 2 investigation. These parameters compare well with the Cohesion (c') = 0kPa and Friction Angle (Φ') = 35 degrees adopted in the analysis of a previous slip which is believed to have occurred in the 1950's along the Causeway (Newland & Allely, 1952).

The investigations that were undertaken at Stage 3 need to be fully interrogated to see whether it is possible to determine where the different fill types were placed. Meanwhile the interpretation has to be that the existing Causeway is formed in different areas from different original materials.

3.4 Settlement Susceptibility

Settlement is controlled by:

- The load (increased stress) imparted to the ground (e.g. from weight of fill)
- The thickness of the layers affected by that load
- The depth to the layers (due to load spreading stresses are reduced with depth)
- The susceptibility of each layer to settlement

This section addresses the susceptibility to settlement, i.e. how prone a particular layer is to the vertical deformation that gives rise to settlement at the surface. There are two components to this susceptibility, compressibility (how much it can deform) and time behaviour (how long it will take).

3.4.1 Settlement Susceptibility of the Marine Mud (Holocene Alluvium)

The uppermost layer known as the Marine Mud (or Holocene Alluvium) is generally considered to be the most susceptible to settlement. It is also this layer that experiences the greatest stress due to its proximity to the surface. Settlements within this layer contribute significantly to the overall deformation determined by calculation.

The recently deposited Marine Mud is water deposited, having accumulated in a low energy environment. With little opportunity to dry out, the Marine Mud has a high natural water content which reflects the naturally high void ratio. Being normally consolidated, the marine mud is prone to virgin consolidation and hence has a very high compressibility.

The predominance of silt and clay sized material and the high proportion of clay minerals in the marine mud means that it has a low permeability therefore it will take a long time for excess pore pressures to dissipate. To date reliable measurements of this permeability have not been achieved.

Any difference between the horizontal and vertical coefficients of permeability is also yet to be established. Observation suggests that the material has a horizontal fabric (layering) which may provide a higher horizontal than vertical permeability. This is reflected in the Stage 1 and Stage 2 Interpretative Report that recommends adopting a horizontal permeability ten times greater than the vertical permeability.

3.4.2 Settlement Susceptibility of the Older Alluvium (Tauranga Group Alluvium)

Having been deposited over the period 3.6 million to 10,000 years ago, the Older Alluvium has experienced a series of geological events that will affect its susceptibility to settlement. During each glacial period, the sea level fell by as much as 100m from the present day level. This would have exposed the surface to erosion and led to ground water lowering. Consequently the ground has undergone some degree of consolidation and the susceptibility to further settlement is reduced. This is reinforced by the lower values of m_v determined by oedometer tests on cohesive layers within the Older Alluvium.

Although generally less susceptible to settlement, the Older Alluvium is still significant on two counts:

- It is a thick deposit, 40 to 50 m thick, about four times the thickness of the Marine Mud;
- It contains organic and peaty layers.

The organic materials may be manifested as clays and silts with some dark mottles, streaks or partings of organic matter. It is generally in a fully decomposed state but represents the range from organic clays to clayey peats and fibrous peats. It

is the high variability of the organic matter that contributes so significantly to the settlement susceptibility of these layers. Even highly decomposed organic matter is still highly compressible. Where the organic material is still in a fibrous form, its propensity to release water and then deform under stress (i.e. primary consolidation and secondary compression) is acute because the vegetable fibres are largely hollow and filled with water.

In contrast with the highly susceptible organic and peaty layers, the Older Alluvium also contains lenses and bodies of sandy material. Although not clean sands, these are expected to have permeability significantly higher than the regular silts and clays and therefore act as drainage layers for the dissipation of excess pore pressure.

3.4.3 Settlement Susceptibility of the Fill

In comparison to the natural soils, it is assumed that the Fill has a low susceptibility to further settlement.

3.5 Preliminary Geotechnical Evaluation of the existing Causeway

3.5.1 Review of Historical Settlement Data

Since construction in the 1950s the Causeway has undergone significant settlement. The exact amount is unclear with a maximum of 2.5m of the Carriageways being reported in NZTA's contract document (Contract No., TNZ PA 2703 (2004)). It is unclear whether there is any data to support this value but it is possible that the figure has been partially developed from an appreciation of differences in level along the motorway. A recent survey revealed that the present elevation of the Causeway Bridges (founded on ECBF Rock) were at about 4.6mRL and that the lowest point along the centre line was at about 2.6mRL. By contrast, an original typical cross section drawing, obtained from archives, indicates that the majority of the Causeway was constructed to an elevation of 3.5mRL (11.5 foot) only. Hence the actual settlement since construction is more likely to be in the order of 0.5m to 1.0m rather than the speculated 2.5m.

Ground penetrating radar (GPR) carried out as part of the Stage 2 Ground Investigation (Aurecon, 2008) on the Causeway for NZTA, combined with the geological interpretation from the intrusive investigations, has shown that it has been repaired at several locations (i.e. several layers of asphalt encountered). Preliminary assessment of the Stage 3 ground investigations within the centre of the Causeway indicate that none of the boreholes passed through different layers of asphalt. This would suggest that any remedial works in the past would have started by removing the asphalt layers then backfilling (topping-up) to make up the elevation difference.

3.5.2 Review of Historical Slips during Embankment Construction

A paper by Newland and Allely (1952) refers to failures of the embankment that were observed:

“Failures were first observed when the bunds had reached a height of approximately 8 ft. and they appeared to be due to plastic flow of the foundation soil. They were characterized by a gradual subsidence in the surface of the bunds accompanied by a heave in the surface of the tidal flat adjacent to the toe of the bunds. In several instances the toe of the bunds moved out for a considerable distance laterally.

The exact location those failures happened is not clearly defined, but the geological descriptions presented in the paper are consistent with their occurrence along the Causeway

The back analysis presented in Newland's paper modeled the lateral spread failure of a 9 ft high embankment with a 1 ft deep embedment in recent alluvium. He calculated the active lateral earth pressure P_a to the vertical section through the embankment fill that caused the lateral spread failure as well as the total shearing resistance R against this active pressure as follows:

$$R = S_u^* L + P_p$$

Where L = distance from toe of slope (in feet)

P_p = passive resistance from the embedment (in lbs/ft run)

S_u^* = disturbed undrained shear strength along the interface of the recent alluvium and the length L of the embankment base (in lbs/sq.ft.)

Newland and Allely obtained a disturbed S_u^* of 3.5kPa (70lb/sq.ft) by assuming a P_p of 3.5kN/m (235lbs/ft) from the embedment (see Figure 5 of Newland & Allely's paper) and concluded that failure was imminent when $R = P_a$ at a distance $L = 11$ ft. from the embankment toe They remarked that this S_u^* was lying in-between the peak and the residual undrained shear strengths of the recent alluvium.

The above S_u^* value is different from the peak field measurements in 1952 of 5kPa (increasing at 1.6kPa/m) which were measurements on soil that had not gained any strength from consolidation. Therefore, the reported 3.5kPa would be more appropriate for a disturbed strength and not its true peak virgin strength. Most likely, Newland & Allely had overestimated the passive resistance from the embedment and underestimated the disturbed undrained shear strength of the recent alluvium. The proof of this is a simple back analysis of slope stability on a typical geometric cross section in keeping with Newland and Allely's paper but adopting the geological profile along chainage 1600. The analytical results presented in Figure 3-4 and Figure 3-5 indicate that a peak undrained shear strength of 5kPa (minimum) would be more realistic as it models the failure threshold for an embankment at 2.4m (8ft) high above the mudflat and that a disturbed undrained shear strength value would most likely have been in the order of 4.6kPa. Residual strength parameters of 3.5kPa (as per Newland's paper) or 0.8kPa (as per field measurements) give unrealistically low factors of safety to be representative of the type of failure described in the paper. Hence the reduction in undrained shear strength due to construction disturbance which is described by Newland as "large rocks of scoria ... caused considerable remoulding of the soil" would only be in the order of 8% of the peak value.

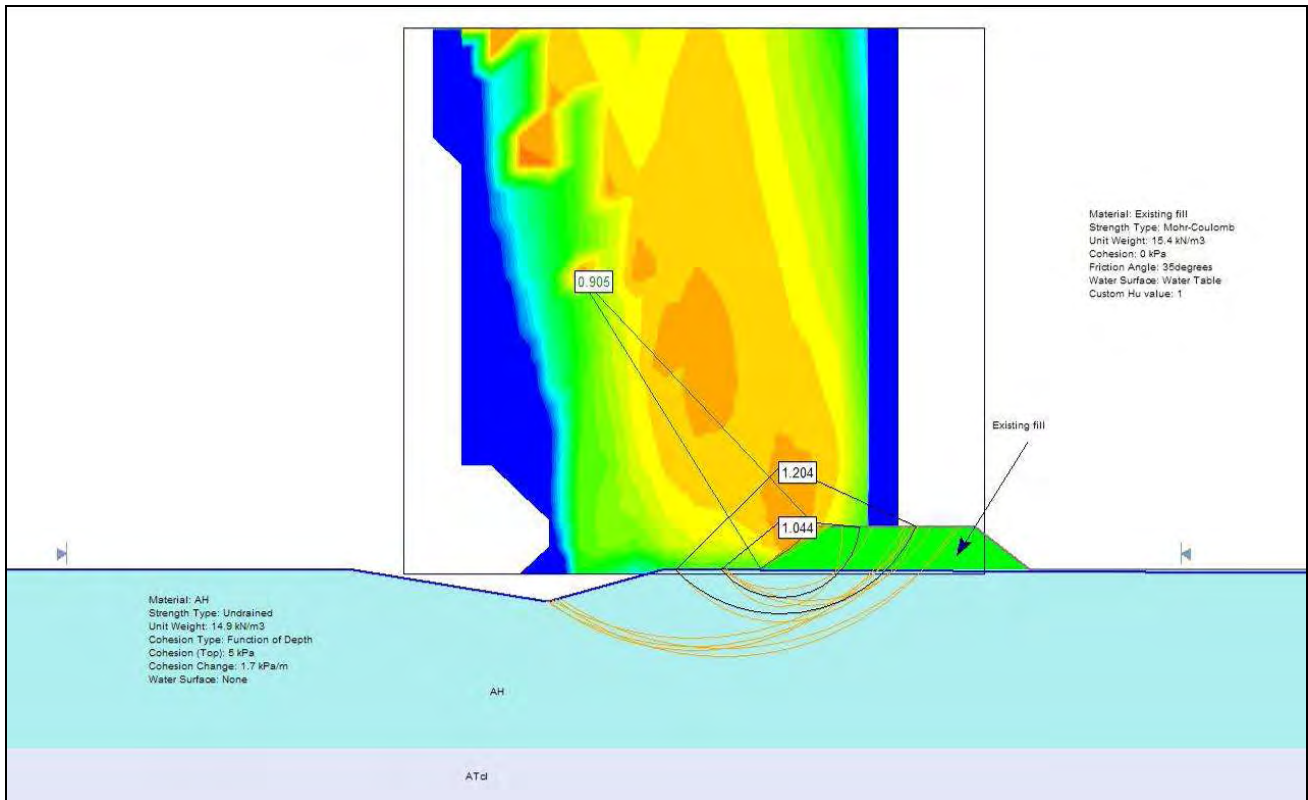


Figure 3-4- Back analysis of previous slip for AH peak undrained shear strength in accordance with Newland and Allely, 1952

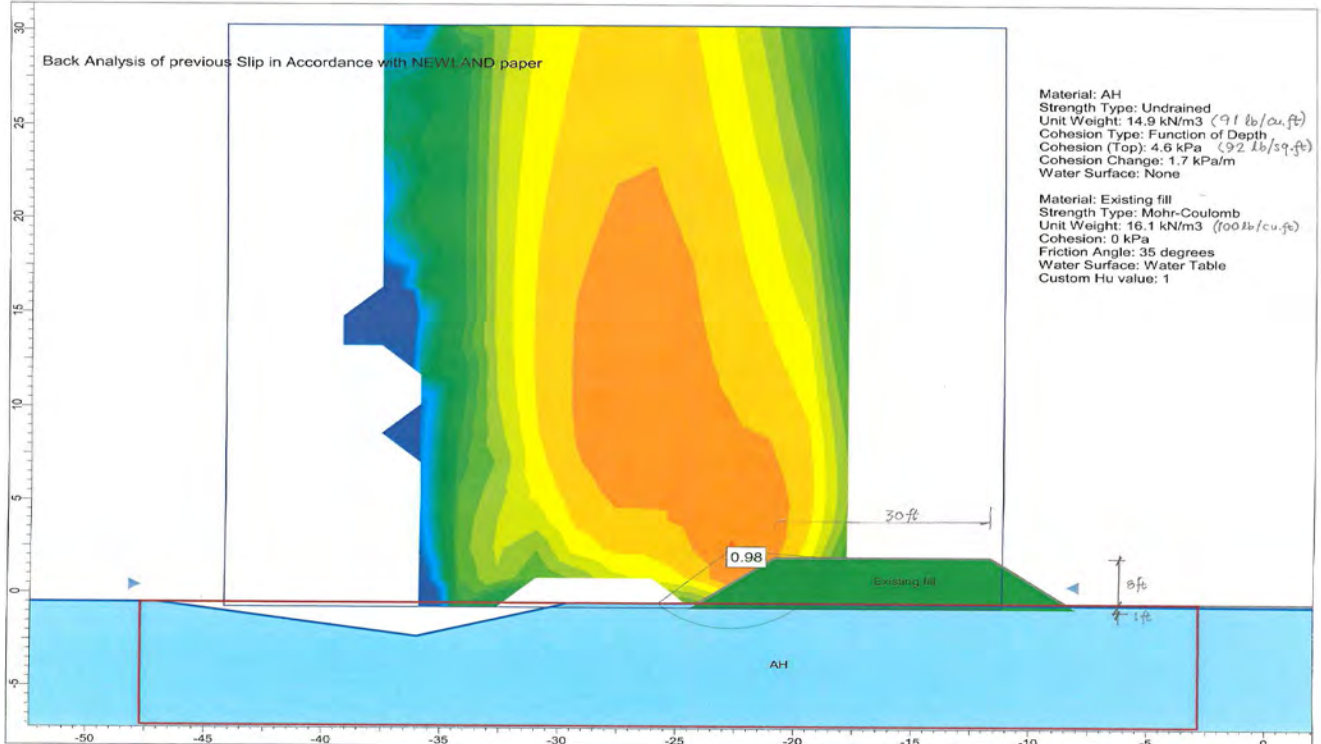


Figure 3-5 -Back analysis of previous slip for disturbed undrained shear strength of AH based on Newland & Allely, 1952

4. Design Methodology

When undertaking analysis and developing a methodology for the design of the Waterview Connection Project, there are several essential project requirements and constraints to be considered. These include:

- Operation of the motorway must be maintained;
- Impact onto the marine reserve must be minimised;
- The improvement works must have a design life until 2100.

These requirements are discussed in more detail in Table 4-1:

Design Consideration	
Motorway Operations	It is a requirement that the motorway continues to operate throughout the period of the works. The minimum acceptable traffic flow to be accommodated is 3 lanes of general traffic in each direction. Furthermore the existing Cycleway standard should be maintained throughout the period of the works.
Impact on Marine Reserve	It is desirable that all encroachments into the Marine Reserve are minimised.
Erosion and Sediment Control	Adequate erosion and sediment control measures must be implemented during and after the construction phase of the project to minimise the amount of sediment that enters the marine area.
Design Life	The design adopts the recommended design life for all permanent earthworks as 100 years, although for practical purposes some elements of the design are only considered up to the year 2100 due to this being the workable limit of projections for global and local climate-change impacts including sea-level rise.

Table 4-1 - Design Considerations

4.1 Design Profile

As SH16 is a Road of National Significance and a key element of the national highway infrastructure, it has been determined that the highway needs to be open to traffic at all times throughout its design life; flooding is unacceptable. This means that the elevation of the embankment has to be maintained at a high enough level to prevent seawater inundation and falls across the embankment need to be maintained to convey stormwater off the embankment.

The geometric design of the Causeway maintains a constant level longitudinally at each element (median, traffic lanes, shoulders and verges). In this design stormwater is only conveyed off the carriageway laterally. To do so efficiently a gradient across the pavement of 3% is required. Stormwater is treated by means of 7m wide lateral Bio-filters with a lateral gradient of 1%.

4.1.1 Design Level

As discussed in the Hydrodynamic Interpretation of Design Conditions, the factors that can cause sea water inundation of the Causeway are:

- **Astronomical tide** - The accepted Mean High Water Spring (MHWS) level at the Causeway is +1.63 mRL (compared to +1.56 mRL at the Port of Auckland). The highest astronomical tide (excluding storm effects) at the Port of Auckland (Ports of Auckland, 2003) is +1.91 mRL, which would amplify to around 2.0 mRL at the Causeway. During recent ground investigation works, a tide gauge was affixed to a pier of the Causeway Bridges and, with 2010 experiencing king tides in January/February and August. Initial monitoring data is presented in Section 3.2.5.
- **Storm surge**- Storm surge causes an additional rise in sea level due to the passage of atmospheric low pressure systems and accompanying winds. Data presented by National Institute of Water and Atmospheric Research (NIWA) (Annex B) indicates a rise in storm-tide water levels to +2.31 mRL (without future sea-level rise) for a storm-tide Average Recurrence Interval (ARI) of 100 years (or an Annual Exceedance Probability of 1%).
- **Wave action and sea level rise** - Due to the risks of inundation or wave overtopping affecting this critical piece of infrastructure, climate-change impacts through to 2100 were incorporated into the design following the climate change recommendations of the Ministry for the Environment (2008) which are based on projections published by the Intergovernmental Panel on Climate Change (IPCC) in 2007. NIWA have interpreted these recommendations (NIWA, 2009) to understand the impact that climate change would have on the project. Climate change effects are two-fold:
 - Possible increased intensity of severe storms, generating more extreme storm surges and higher waves;
 - Sea-level rise.

A sea level rise of 0.8m by 2100 has been adopted for design purposes.

Settlement of the embankment – The materials directly underlying the Causeway area mainly comprise soft to very soft marine sediments and therefore any additional loads imposed by the proposed embankment will cause settlement. Although some of this settlement will occur during the construction phase, a significant component will occur during the operational phase of the highway as discussed in the Ground Improvement Options Report (Aurecon 2010c). An allowance of 500mm residual settlement across the Causeway to account for ongoing settlement up to 2100 is

recommended in the Design Considerations Report (Aurecon, 2009). Subsequent detailed settlement analyses have supported an allowance of this amount.

To account for the compounding effects of climate change and ongoing settlement, the design recommendation is to set the elevation of the northern and southern crests at a minimum of +3.0 mRL at year 2100, with an immediate post-construction level of +3.5 mRL to account for the 500mm residual settlement allowance. These design levels apply at the time of completion of the widened motorway, which is presumed to be at about the year 2016 (Aurecon, 2009). This translates to a design level along the centreline of the highway of +4.44 mRL.

4.1.2 Formation of the Design Profile

A comparison of the design profile with the existing profile indicates the extent of the works required to form the design profile. The sections (Drawing References 20.1.11-3-D-C-150-351, 352, 356 – Ground Improvement Options Report – Aurecon, 2010c) show the design surface levels together with the existing surface levels. The proposed modification works comprise symmetrical widening with space being created adjacent to the existing eastbound and westbound carriageways to accommodate additional traffic lanes, bus shoulders, surface water treatment and cycleway.

In the lowest part of the Causeway (Ch1550 to Ch2400) typical net changes in level between the existing embankment and the proposed embankment surface are 1.5m to 2.0m. Where the proposed embankment overlies side-slopes of the existing embankment or tidal mud-flats beyond the toe of the existing embankment, the changes in level are in excess of 3m. The greatest difference is at the location of the tidal channel to the south side of the embankment where the new profile reaches a height difference of 5.8m above existing ground.

The changes in level applied over the relevant widths and lengths of the alignment have been used to determine the total volumes of fill to be placed as discussed in Section 9. The reality is that placement of fill will cause settlement and more fill will be needed to compensate for this settlement in order to achieve the design level. The design and the fill volume estimations take into account this expected settlement as well as the compaction of the fill material itself.

4.2 Design Standards

Design manuals, standards and publications have been used to guide the design of the SH16 Improvements (i.e. slope stability, ground displacement criteria). These are discussed in more detail within the Ground Improvement Options Report, the Fill Options Report and the Coastal Protection Report.

4.3 Earthworks Design Criteria

All works are to be carried out in accordance with Transit F/1 specification.

4.3.1 Slope Stability Criteria

The slope stability design criteria for the earthworks analysis are summarised in Table 4-2:

Slope Stability Criteria	Limiting Criteria
Static Factor of Safety	Case (a) ≥ 1.5 under design operating conditions – Long term Stability
	Case (b) ≥ 1.2 under extreme groundwater conditions. Input parameters same as Case (a).
	Case (c) ≥ 1.0 for the lowest (25 percentile) effective stress soil parameters.
	Case (d): Factor of Safety ≥ 1.2 – Short term stability (Short term refers to either of the following; (i) construction stage working conditions (ii) a period not exceeding 3 months or one earthworks season (iii) Immediately after construction
Seismic Factor of Safety	Case (e): Factor of Safety ≥ 1.0 Input parameters same as case (d) + seismic coefficient. Where calculated factors of safety are less than 1, foundation displacements are to be assessed to be less than 150mm and displacement is not to adversely affect the serviceability of structures, property and roadway shoulder or barrier.

Table 4-2 – Slope Stability Design Criteria

4.3.2 Ground Displacement Criteria

It is deemed that ground displacements, both vertical and horizontal, shall not result in structural damage, reduced serviceability or reduced design life of the motorway including its structures, embankments, cut slopes, adjacent properties and utility services. The criteria presented in Table 4-3 have been adopted to control ground displacements (Aurecon and Tonkin & Taylor, 2010).

Loading & Ground Displacement Type		Limiting Criteria
Static Load	Vertical and horizontal displacement during construction and operation of the motorway (i.e. within 25 years after completion of pavement construction).	Not greater than 400mm total vertical displacement and
		Not greater than 1% transverse differential vertical displacement measured over the formation width either side of the median and longitudinally measured over successive 50m intervals and
		Not greater than 0.25% longitudinal differential vertical displacement measured over 10 m from all structures.
25 to 100 years		Maintain crest elevation of motorway above +3.0RL throughout the 100year design period

Loading & Ground Displacement Type		Limiting Criteria
Seismic Load	Liquefaction induced vertical, horizontal and lateral displacement	Not greater than 150mm and displacement are not to adversely affect the serviceability of structures, property and roadway shoulder or barrier.

Table 4-3 – Ground Displacement Criteria

5. Preliminary Geotechnical Assessment

5.1 Ground Model

The ground model used for the geotechnical analysis is based on topographic and geological models. The topographic model superimposes the design profile (for the end of construction) upon the existing topography determined by survey and is illustrated by means of geometric cross sections generated using the MX Roads software.

The geological model is essentially presented as a geological long section derived from the centreline of the highway. This long section is an output from the Stage 1 and Stage 2 Geotechnical Interpretative Report (Aurecon, 2010a) and is an interpolation between the exploratory holes from Stage 1 and Stage 2 ground investigations. The geological model transverse to the motorway presumes that the geological strata are extrapolated horizontally from the line of section along the highway median.

An example of a geological model (ref: chainage CH1600) displaying idealised soil layering is presented in Figure 5-1. This Figure shows how the Tauranga Group Alluvium is not uniform in composition, having layers of clay (highlighted in yellow), sand, organic material, peats and volcanoclastic deposits. The thickness of these constituent layers varies from a few centimetres to several metres. In this example, data from more recent ground investigation have also been incorporated to validate the existing hydro-geological model.

It should be noted that the idealised “layer cake” style geological model is the best interpretation available, given the density of ground exploration locations. In reality, the thickness of the individual layers varies, their boundaries are not perfectly horizontal (the sands in the Tauranga Group are known to be typically in lenses) and they do not have a constant composition. Different locations in the same stratum on the geological model could be for instance more sandy, but due to this highly variable nature of the alluvial sediments they are included in the same layer in the model.

For each analytical section a geological model was generated by interpolation of the geology at that particular chainage along the geological long section. This was extrapolated in the “layer cake” style and then the topographic profile added. The main lateral variation in the geology is provided by the presence of the embankment fill underneath the existing Causeway; this is absent to the north and south.

To convert the geological models at each chainage into analytical ground models design parameters derived from the Stage 1 and 2 Interpretative Report were applied to the relevant strata present and then strata were characterised as either consolidating layers or drainage layers. The sand lenses within the Tauranga Group (ATs), the volcanoclastic layers (ATv) and the non-cohesive embankment fill were classified as drainage layers, the rest were presumed to consolidate.

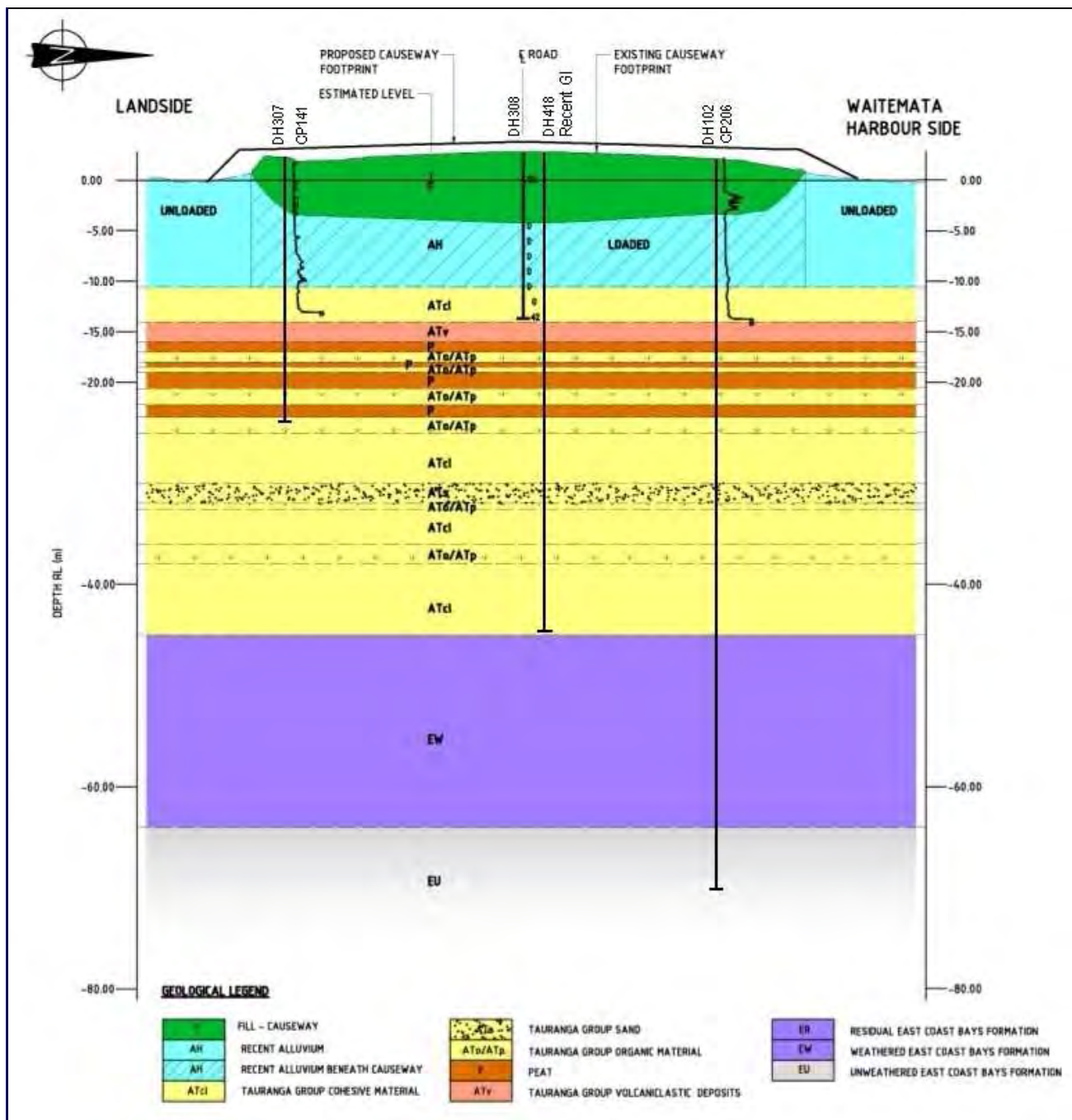


Figure 5-1 - Example geological model at chainage CH1600

5.2 Hydrogeology

5.2.1 Hydrogeological Background

It is recognised that the hydrogeological model for the route is complex. The hydrogeology regime along the main section of the Causeway differs from where the motorway embankment runs across Traherne Island the groundwater along the Causeway is significantly affected by tidal cycles, whilst Traherne Island is only marginally affected. The low lying surface elevation of the Causeway is consistent with a shallow groundwater level being encountered. Groundwater monitoring results from instrumentation placed along the Causeway into Traherne Island shows a clear relationship between incoming tide and shallower water levels. As the alignment passes onto Traherne Island the relationship is no longer so pronounced. The groundwater monitoring also indicated that beneath the Causeway there could be a freshwater lens overlying the more saline groundwater.

The present day depositional environment of the Causeway area is one of low energy. Historically, the Central Waitemata Harbour had drainage channels that passed directly across what is now the motorway alignment. A higher energy depositional flow in the historic channels would have led to the deposition of sand; these channels were subsequently diverted and in-filled during the construction of the Causeway in the 1950's. The construction technique of directly tipping material onto the marine mud would have simply buried the lenses of sand which must still be there and retain the potential to serve as drainage paths below the embankment fill.

The palaeotopography of the underlying ECBF rock is not likely to play a major role in groundwater along the alignment. For the majority of the site, except immediately west of Whau River, the thick veneer of low permeability, cohesive soils prevents significant groundwater migration in or out of the ECBF rock. Where groundwater does migrate through ECBF rock it is likely to be controlled by persistent bedding and structural features, such as fault zones. Typically joints in ECBF rock are tight and unlikely to provide significant pathways for groundwater flow.

5.2.2 Hydrogeological Implications for Project

It is expected that groundwater recharge within the Causeway area will be primarily governed by surface infiltration from rainwater and lateral infiltration as a result of tidal inflow into the embankment fill. Vertical recharge can then take place into the underlying alluvial materials via seepage from the fill.

The alignment is not expected to be significantly affected by perched groundwater. Geological conditions between the Causeway and Rosebank Peninsula largely minimise the risk due to the low lying environment of this section of the motorway, resulting in shallow groundwater in the heterogeneity, layering of the recent marine sediments and the Tauranga Group alluvium. Moreover no confined aquifers are expected. Hence it has been assumed that groundwater pressures will largely be hydrostatic.

Generally groundwater levels along the Causeway vary between 1.7 and 2.45m below the surface of the pavement, while groundwater levels underlying Traherne Island vary between 1.2 and 1.8m below ground level.

The heterogeneity, layering of the alluvial deposits and contrasting permeability across the site, will play a role in controlling settlement rates and local hydrostatic pressures at the toe of the embankment.

Both horizontal and vertical permeability through the alluvial deposits is anticipated to be variable but low except where more laterally continuous higher permeability sands (ATs) and to a lesser extent, the volcanoclastic layers (ATv) which are described as Rhyolitic silt and sand. The layering continuity of these materials have been established from existing ground investigation data and their ability to provide a conduit for groundwater flow is established from the lesser of the in-situ and laboratory permeability test data as obtained from the testing below and summarised in the subsequent table.

1. Particle Size Distribution (PSD) correlations (applicable to ATs materials only),
2. In-situ CPT dissipation tests,
3. Oedometer and Consolidated Undrained (CU) tri-axial testing

The reliability of the hydrogeological model as it impacts on settlement and rate has been interrogated by carrying out a back analysis on existing historical settlement monitoring records, as Section 3.5.1.

Table 5-1 summarises the field and the laboratory test results and the assumptions made in terms of vertical permeability and coefficient of consolidation.

Soil Type	Vertical Permeability Kv (m/s)			Coefficient of consolidation (m ² /yr)		
	From PSD correlation ¹	From oedometer CU test ¹	From CPT ¹	Horizontal ch from CPT ¹	Vertical cv from CPT ¹	Vertical cv from oedometer ¹
Existing Cohesive Fill	NA ¹	5x10 ⁻¹⁰	3.6x10 ⁻¹¹	5	2.5	5-0.8
AH	NA	3x10 ⁻¹⁰	1.6x10 ⁻⁸ ~ 2x10 ⁻¹¹	3.1-645 ⁽²⁾ mean = 155 (5 no. of data)	1.5-322 ⁽²⁾ mean = 78 (5 no. of data)	0.8-0.5
ATcl	NA	10 ⁻⁹ ~3.5x10 ⁻¹¹	9.1x10 ⁻¹²	1.7-243 ⁽²⁾ mean = 122 (2 no. of data)	0.8-122 ⁽²⁾ mean = 61 (2 no. of data)	22-12
ATo/ATp	NA	10 ⁻⁹ ~ 3x10 ⁻¹¹	3.9x10 ⁻¹¹	5.4	2.7	10-1.8
ATs ⁽³⁾	1.8-6.3x10 ⁻⁷	6-9x10 ⁻⁹	2.4x10 ⁻⁸	243.2-900	121.6-450	10-6.5
ATv ⁽⁴⁾	NA	NA	⁽⁵⁾ 8x10 ⁻⁸	360 ⁽⁵⁾	180 ⁽⁵⁾	NA
ER	NA	1-2x10 ⁻⁹	4.5x10 ⁻⁸ ~4.1x10 ⁻⁹	NA	NA	NA

Table 5-1 - Permeability and consolidation rate test results and Hydrogeological assumption table

Notes:

1. Values taken from Tables 6-19, 6-20, 6-26 & 6-27 in the Stage 1 and 2 Geotechnical Interpretative Report. NA = not applicable (tests not carried out on this material). CPT=determinations made from pore pressure dissipation test undertaken as part of CPT.
2. High values of coefficient of consolidation may be due to the presence of isolated lenses of sand.
3. ATs is described as sand or silty sand.
4. ATv is described as Rhyolitic silt and sand (volcaniclastic).
5. Derived from CPT609, stage 3 ground investigation.
6. It can be inferred from Table 5.1 that ATs and ATv are about 10 to 100 times more permeable than the existing Fill, ATcl, ATo/ATp and ECBF residual soils. If the data outliers of the coefficients of consolidation c_h and c_v of AH, ATcl & ATo/ATp are disregarded, the values of c_h and c_v of ATs and ATv are also 10 to 100 times higher than those of the existing Fill, AH, ATcl, ATo/ATp and ECBF residual soils. Therefore ATs and ATv layers are assumed to be drainage layers in the settlement analyses model.

5.3 Geotechnical Parameters

The geotechnical design parameters used in the stability and settlement analyses presented in the Ground Improvement Options Report are tabulated in Tables 3-1 and 3-2 of that report (Aurecon, 2010c). These geotechnical parameters were generally based on information presented in the Geotechnical Interpretative Report (Aurecon 2010a) for the Stage 1 & Stage 2 investigations (Connell Wagner, 2007 and 2008). Parameters for the new embankment fill have been developed from recent experience on other projects in similar materials.

In terms of primary consolidation, the time for settlement is governed by the parameter, c_v (the coefficient of consolidation) itself directly related to permeability (k = coefficient of hydraulic conductivity (permeability)); permeable soils deform quickly. Compressibility is expressed by the parameter m_v (the coefficient of compressibility). This is not an intrinsic property of a soil but depends on the void ratio, stress history, degree of consolidation, current stress and increased stress. These parameters are generally obtained in the first place from the laboratory oedometer (one-dimensional) consolidation test but it is recognised that the field values may differ due to scale effects and sample disturbance, particularly the time dependent parameters. In the Stage 1 & 2 Geotechnical Interpretative Report, a value of m_v is derived for each change in stress level and the value of m_v appropriate for the problem being resolved is selected.

The parameter c_v has been determined from both CPT dissipation tests and oedometer tests, whichever gives the lesser value (Refer to Section 6.1.3 of the Stage 1 & 2 Geotechnical Interpretative Report Rev3).

Preliminary review of the recent ground investigation data suggest that the parameter m_v for Recent Alluvium is about 10% lower than what has been adopted in the current analysis, and for Tauranga Group organics/peat, it is about 60% lower. The new investigation data also shows that the values of the stiffness parameter m_v for the Tauranga Group silt and clay are the same order as those adopted in the preliminary settlement analyses presented in the Ground Improvement

Report. The reduction in permeability over time has been allowed for in developing compressibility design parameters by back calculating the permeability at a particular effective overburden pressure and time from values of c_v and m_v determined from oedometer tests.

5.4 Approach to Assessment

The design of earthworks and ground improvements (where required) considers the external stability of the new embankment under both static and seismic conditions. The preliminary design of the side slopes of the embankment uses a slope of 1V to 2H which is consistent with the side slopes used in the Hydrodynamic analysis and the Coastal Protection Report. The potential slope failure modes are displayed in Figure 5-2 and discussed below.

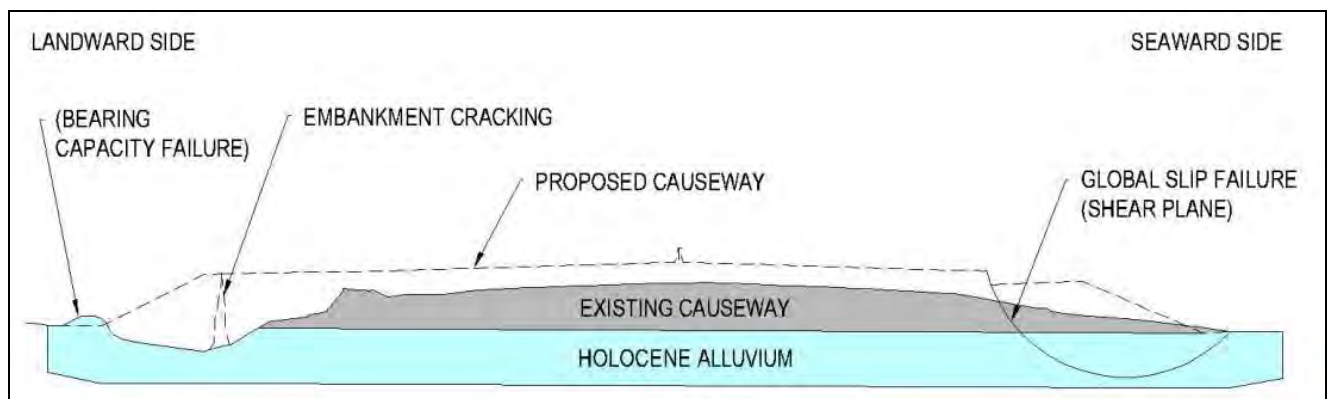


Figure 5-2 - Typical slope failure modes of new Causeway without ground improvement works

5.4.1 Excessive settlement and lateral deformation under both static load and seismic liquefaction

Settlement analysis under a static embankment load has been analysed by hand calculations and finite element software. The two methods are not expected to match numerically. However, by comparing the results from different approaches, the reliability of the outputs can be better assured. The selection and preliminary detailing of all ground improvement measures are based on results of the one dimensional (hand calculation) estimates.

Where sustained ground shaking gives rise to an increase in pore pressures between soil particles and ground conditions not favouring a rapid dissipation of pore water pressure, inter-particle shear resistance is exceeded and liquefaction occurs. Seismic liquefaction is assessed based on an earthquake magnitude of 6.5 and return period of 1500 years and an ultimate limit state (ULS) loading generating a peak ground acceleration of about 0.15g. The acceptable displacement criterion is defined as lateral deformation not in excess of 150mm.

5.4.2 Local bearing capacity failure

Construction of the new Causeway embankment shoulders will lie directly on top of the low strength Holocene alluvium in its unimproved (virgin) condition. The ability of this material to provide adequate foundation bearing during construction and to provide post-construction restraint against embankment cracking and global shear failure have both been assessed. Embankment cracking occurs as tensile strains develop at the base of the new embankment due to the contrast in stiffness between foundation soils and embankment materials. This strain initiates a crack at the base of the embankment which propagates to the top. The analytical guidelines for embankment cracking are in keeping with the recommendations of Sinclair (2003).

5.4.3 Shallow and deep seated global slope stability failure

The embankment shoulders have been analysed for shallow and deep seated global instability using finite element software (Plaxis). For preliminary design, circular analysis has been considered for both long and short-term stability. Seismic analysis has been undertaken adopting non-circular block sliding analysis.

5.5 Settlement Analysis

Settlement analysis has been carried out under three categories in keeping with conventional soil mechanics theory. These three modes of settlement are explained in further detail in the Ground Improvement Options Report (Aurecon, 2010c) and are summarised below:

- **Immediate (or distortion) Settlement** – Is due to elastic deformation (distortion in shape without change in volume for undrained soils and change in volume for drained soils) of the foundation soils;
- **Primary Consolidation** – Occurs when water present within the soil is squeezed out of the voids within the soil mass due to additional load (i.e. construction of the Causeway embankment);
- **Secondary (Creep) Settlement** – This is caused by adjustments in the internal structure of the soil mass under a sustained load and is said to occur when most (around 95%) of primary consolidation is complete.

The sum of all of the vertical settlements equates to the total settlement experienced at a particular point. However, it is unlikely that the amount of total settlement at two points will be the same; the difference between these two values is referred to as differential settlement. There are a number of factors that cause differential settlement, including:

- **Geology & Geotechnical parameters** - The settlement susceptible layers at one point may be thicker or lie closer to the surface or be more compressible than at another;
- **Engineering stresses** – These may differ at two locations (e.g. the embankment may be thicker) causing settlement to vary;
- **Drainage** - In primary consolidation, differential settlements may occur as excess pore pressures dissipate faster at one point than another due to differences in permeability;

- Construction phases – the staged construction process will result in different degrees of settlement in the underlying strata;
- The existing embankment has 'matured' in terms of its settlement while the lateral additions to form the widened embankment would be at an early stage of the settlement process.

The settlement analysis has accounted for the construction staging of the Causeway, with the seaward side being constructed first and taking around 24 months to complete. This would be followed by the construction of the landward side that would take around 22 months. Finally the centre of the motorway would be constructed and this would take around 14 months to complete.

Settlement analysis was only undertaken at a number of chainages along SH16 with the amounts of settlement between these being inferred. The locations analysed were at chainages 1100, 1350, 1450, 1600, 1800, 2150, 2300, 2550, 2800, 3025, 3200, 4100, 4700 and 6490.

5.5.1 Settlement Estimates

This section presents the findings of the preliminary settlement analysis of the proposed Causeway if ground improvement works were not implemented. The analysis estimated the amount of settlement immediately after construction (assumed 5 years), at the end of 30 years (25 years operational period) and at the end of 100 years (the intended design life). Table 5-2 below summarises the results of the one dimensional settlement analysis performed before ground improvement works.

Chainage	Location	³ Average Fill Height from MX Design Section (m)	Total Settlement ²		
			end of T ₁ Yrs ⁵	end of 30 Yrs ⁶	(0 to 100 years)
			(mm)	(mm)	(mm)
1100	South ¹	2.6	T ₁ =4 Yrs = 305	532	630
	Centre ¹	0.8	T ₁ =5 Yrs = 56	117	166
	North ¹	2.5	T ₁ =2 Yrs = 300	508	605
1350	South	4.2	T ₁ =4Yrs = 540	1015	1175
	Centre	No Fill	NA	NA	NA
	North	3.3	T ₁ =2 Yrs = 430	859	1030
1600	South	5.8	T ₁ =4 Yrs = 680	1413	1695
	Centre	1.6	T ₁ =5 Yrs = 190	421	555
	North	3.3	T ₁ =2 Yrs = 510	1136	1435
2150	South	2	T ₁ =4 Yrs = 475	1032	1325

Chainage	Location	Average Fill Height from MX Design Section (m)	Total Settlement ²		
			end of T ₁ Yrs ⁵	end of 30 Yrs ⁶	(0 to 100 years)
			(mm)	(mm)	(mm)
	Centre	1.9	T ₁ =5 Yrs = 339	700	874
	North	2	T ₁ =2 Yrs = 455	966	1210

Table 5-2 - Summary of one dimensional settlement analysis (Before ground Improvement Works)

Notes:

1. South: Southern Shoulder North: Northern Shoulder. The settlement figures shown are the average settlement values across the shoulders.
2. Settlement analyses for North and South adopt the 'most credible likely' case parameters for coefficient of volume compressibility m_v and coefficient of consolidation c_v . Settlement analyses for Centre adopt the 'best probable case' parameters for m_v and c_v . The analyses also adopt a combined c_v where drainage paths cross multiple soil layers. ATs/ATv layers have been assumed to be drainage layers.
3. Based on cross sections generated by the Mx road software
4. Secondary consolidation is assumed to begin at time T₉₅ when 95% primary consolidation has taken place. T₉₅ is assumed to have occurred about 50 years ago at the centre of the Causeway.
5. Total settlement at the end of each embankment shoulder construction phase, which approximately corresponds to the 2nd yr of the entire construction programme for the Northern Shoulder, the 4th yr for Southern Shoulder and the 5th yr for the Centre of the Motorway.
6. Total settlement at the end of 30th year from the commencement of the construction programme.

5.5.2 Back Analysis of Historical Settlement Monitoring Record

Back analysis of existing historical settlement data has been carried out in order to calibrate the compressibility parameters adopted in analytical settlement model. Detailed assessment is presented in the Ground Improvement Options Report – Aurecon, 2010c).

The approach to the back analysis has been, to use the current settlement models to see what the likely settlement magnitudes and rates they would have estimated for the 1950's along section chainages CH1100, CH1600 and CH2150 based on the following:

1. As-built records; the only historical settlement record available is a drawing titled "Auckland Kumeu Motorway Observation of Settlement: Graph of Settlement against time" prepared by NZ Ministry of Works dated 1957. Settlements were monitored at 3 stations on the existing Causeway from June 1952 to Sept 1954, discontinued for construction works from Sept 1954 to June 1956 and then resumed from June 1956 to December 1957. A total

settlement of 1.6 feet was recorded. Locations of these monitoring stations are not known. (Reference: New Zealand Works Ministry as-built settlement monitoring A.D.O. 22109)

2. Current ground model, fill profile (thickness) as interpreted from completed ground investigation,
3. Geometric profile of existing causeway and mudflat as produced by topographic survey and interpreted by the Road software (Mx-design cross sections).

The estimated settlement and rates are then compared with the historical monitoring records as a reliability check on the current analytical model and results.

Predicted settlement magnitudes and rates based on the back analysis are compared with the historical settlement monitoring data in Table 5-3 as follows:

Chainage (m)	Predicted Primary Settlements			Predicted Rate of Primary Settlement		Settlement Figures from As-built records from New Zealand Works Ministry			
	At the 27 th month	At the 48 th month	At the 66 th month	over the first 27months	From the 48 th to the 66 th month	at the 27 th month		From the 48 th to the 66 th month	
						Settlement	Rate	Settlement	Rate
	(m)	(m)	(m)	(mm/day)	(mm/day)	(m)	(mm/day)	(m)	(mm/day)
CH1100(one staged load)	0.31	0.36	0.38	0.37	0.04	0.303	0.37	0.183	0.34
CH1600(one staged load)	0.57	0.75	0.87	0.70	0.23				
CH1600(two staged load)	0.39	0.81	0.97	0.47	0.32				
CH2150(one staged load)	0.38	0.48	0.55	0.47	0.13				

Table 5-3 –Comparison between Back Analysis Results and Historical Settlement Monitoring Data

Discussions and Conclusions on Back Analysis Results:

- From Table 5-3, the initial settlement amounts and rates predicted by the settlement model are seen to be in the same order as those inferred from the historical records, except for the rate from the 48th to the 66th month for chainage 1100 which is much slower than the historical records. One of the reasons could be the recent alluvium layer is much thinner at this location and primary consolidation may have been completed by the 66th month. The staged loading settlement analysis for CH1600 gives settlement rates more closely matching with the historical data. These values endorse the model of an embankment partially raised, left for 2 years and then ultimately topped up by another 3m.

- Notwithstanding uncertainties and reliability of historical records, it can be concluded that the settlement model and geotechnical parameters presented in the Ground Improvement Options Report are consistent with actual ground conditions, as-built records and historical monitoring data.
- It is not considered inconsistent for settlement to have been higher for the initial embankment construction than they are estimated to be for the planned upgrading works. In the first place the original embankment was placed directly upon virgin marine mud and loaded this with a broad footprint 50m wide. For the upgrading works where the embankment is raised in the central part, the marine mud is in its improved state relative to its condition in the 1950s and therefore will have a lower compressibility. Only below the new shoulders will the upgrading works lie directly over virgin marine mud and the loaded width is restricted to about 15-20m width each side compared to the full width of 50m loaded in the 1950s.

5.6 Bearing Capacity Analysis

To assess the potential of embankment bearing failure or cracking of the embankment during and post construction of the motorway three representative analytical sections have been assessed. Results of the analysis indicate that the undrained shear strength of the Recent Alluvium is inadequate and embankment cracking is likely without some form of foundation improvement works. It can therefore be concluded that construction of the new embankment shoulders directly on top of the Holocene alluvium will not be viable in most locations due to inadequate foundation bearing capacity and susceptibility to shear failure.

5.7 Slope Stability Analysis

The new embankment shoulders have been analysed for both short and long-term global instability. Short-term assessment is defined as the stability of the embankment during construction and immediately after it or in the event of an earthquake.

Results of slope analysis based on five representative and critical sections (Ch1350, Ch1600, Ch2150, Ch2550 and Ch4700) indicate that in general, slope stability under long term operational conditions is adequate. However, there is a potential for deep seated global failure immediately after construction if no ground improvement measures are provided. Even during a seismic event, the factor of safety of the slope falls well below unity.

5.8 Summary of Geotechnical Assessment Findings

The findings indicate that the shear strength of the Holocene Alluvium is not adequate to resist embankment bearing shear failure. Global stability of the embankment during and immediately after construction or during an earthquake would not be adequate. The highly compressible Holocene Alluvium means total and differential settlement will be significant and will continue throughout the design life of the embankment, with possible adverse implications on carriageway drainage and road rideability. Therefore, ground improvement works will be required.

6. Ground Improvement Options

Ground improvement techniques have been considered to bring ground performance to within design tolerance. The main concerns that the techniques seek to address are slope instability, settlement and low bearing capacity of the underlying foundation soils.

6.1 Ground Improvement Techniques

A comprehensive appraisal of the various improvement techniques commonly used for weak foundation soils such as soft marine mud or sediments of clay, silts and organic deposits in New Zealand and world wide is presented in the Ground Improvement Options Report and hence not repeated here.

The chosen ground improvement options have been based on a number of factors (attributes) as they relate to the design philosophy. These are namely the ability to provide stability, reduce settlement, potential adverse impact on the environment, their constructability, construction time and physical works cost. To quantify the relative merits of each option, an attribute score is assigned to each ground improvement option. An attribute score of 5 is considered "exceedingly advantageous" while a score of 1 is considered "Not recommended". To reflect the priority of each attribute on the appraisal process, a relative weight of '3' is given to stability, environmental impact, construction cost and time attributes, '2' is given to the 'constructability' and '1' is given to 'reducing settlement' attribute. The total weighted score of each ground improvement option is then calculated as shown in the score matrix table below. The ground improvement option(s) with the highest weighted score is considered the preferred option(s).

Ground Improvement Options	Attributes						Total Weighted Score $\Sigma(\lambda S)/\Sigma\lambda$
	Eliminate Settlement Score S ₁	Stability Score S ₂	Constructability Score S ₃	Construction Cost Score S ₄	Construction Time Score S ₅	Environmental Impact Score S ₆	
Foundation Undercut	3	4	3	4	3	3	3
Staged Construction	4	4	4	5	2	3	4
Surcharging	5	3	3	3	2	4	3
Marine Deposit Displacement	2	3	5	5	5	3	4
Dynamic Replacement	3	4	4	3	5	1	3
Wick Drains	4	2	5	4	3	3	3
Lightweight Fill	4	4	4	2	5	2	3
Mudcrete	3	4	5	4	4	3	4
Geogrid Reinforced Raft	1	3	5	5	5	3	4
Deep Soil Mixing	5	5	3	2	3	3	3
Jet/Compaction Grouting	4	5	2	2	3	3	3
Stone/sand column	5	5	3	2	3	3	3
Geogrid Reinforced Piled Embankment	5	5	2	1	2	3	3
Notes : 1	Attribute Scores:						
	Exceedingly Advantageous		5				
	Moderately Advantageous		4				
	Satisfactory		3				
	Not Satisfactory		2				
	Not Recommended		1				
2	Weight Attribute to S ₁ (λ ₁) =		1				
3	Weight Attribute to S ₂ (λ ₂) =		3				
4	Weight Attribute to S ₃ (λ ₃) =		2				
5	Weight Attribute to S ₄ (λ ₄) =		3				
6	Weight Attribute to S ₅ (λ ₅) =		3				
7	Weight Attribute to S ₆ (λ ₆) =		3				
8	Total weighted score S =		$\frac{\lambda_1 S_1 + \lambda_2 S_2 + \lambda_3 S_3 + \lambda_4 S_4 + \lambda_5 S_5 + \lambda_6 S_6}{\lambda_1 + \lambda_2 + \lambda_3 + \lambda_4 + \lambda_5 + \lambda_6}$				

Table 6-1 - Score Matrix for Ground Improvement Options

Considering the general needs of ground improvement and the results of the appraisal of the various options, the most viable solutions being carried forward are either one or a combination of the options presented below.

- In-situ Soil Stabilisation (Mudcrete or similar)
- Marine Deposit Displacement (MDD)
- Foundation undercut,
- Geogrid reinforced raft,
- Light weight fill

A schematic diagram showing how some of the above solutions are being applied to the proposed Causeway and analytical model is presented in Figure 6-1:

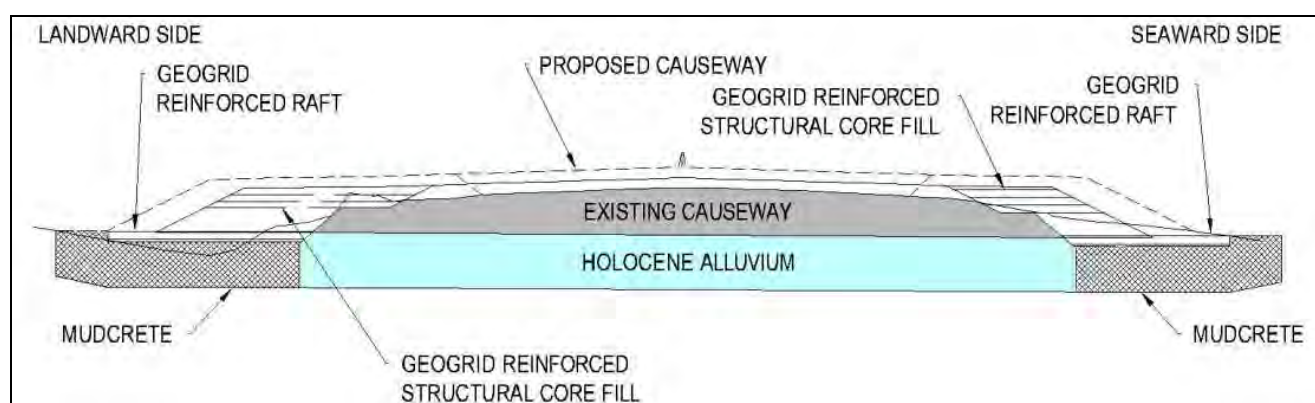


Figure 6-1 - Sketch showing details of ground improvement of new Causeway

6.1.1 In-situ Soil Stabilisation

The preliminary design evaluation indicates that in-situ soil stabilisation is the most feasible solution to improve the strength of the Holocene Alluvium. This is to be achieved by either lime or cement soil mixing which are both considered environmentally friendly and economically efficient (due to the small quantities of cement or lime used).

Mixing of the in-situ soils with lime or cement binders (or both), works well in a high moisture environment such as the Causeway area. The available soil moisture would be used during the hydration process and strength gains would be achieved. The amount of binder required depends on the strength increase needed and would need to be determined. In general, binder contents range from 60kg/m³ for a soft soil site to an upper value of around 300kg/m³ for a very soft soil site with high organic content. However, typical ranges are between 80 to 130kg/m³.

Soil stabilisation can be carried out within the mudflat as both in-situ and ex-situ. However, *in-situ* is the preferred option and is discussed further below. For details on the *ex-situ* technique, refer to the Ground Improvement Options Report (Aurecon, 2010c).

In New Zealand the material formed as a mixture of soil and cement by the mass stabilisation process when it is undertaken over a shallow zone (up to about 5m) is termed “mudcrete”. It is considered that this is a viable process for the project and hence the term “mudcrete” has been widely used in reference to the ground improvement works planned. If the shallow mixing technique is insufficiently robust and soil stabilisation is needed to be taken to greater depths then alternative deeper methods, such as Deep Soil Mixing may be appropriate but the basic premise is the same. A diagram of this technique is shown in Figure 6-2 and Figure 6-3.

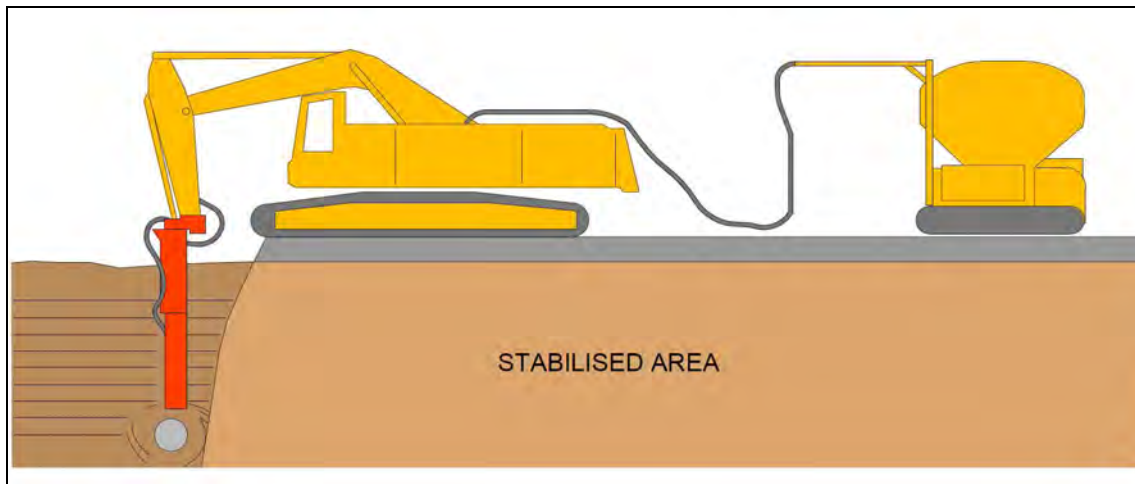


Figure 6-2 – *In-situ* Mass Stabilisation

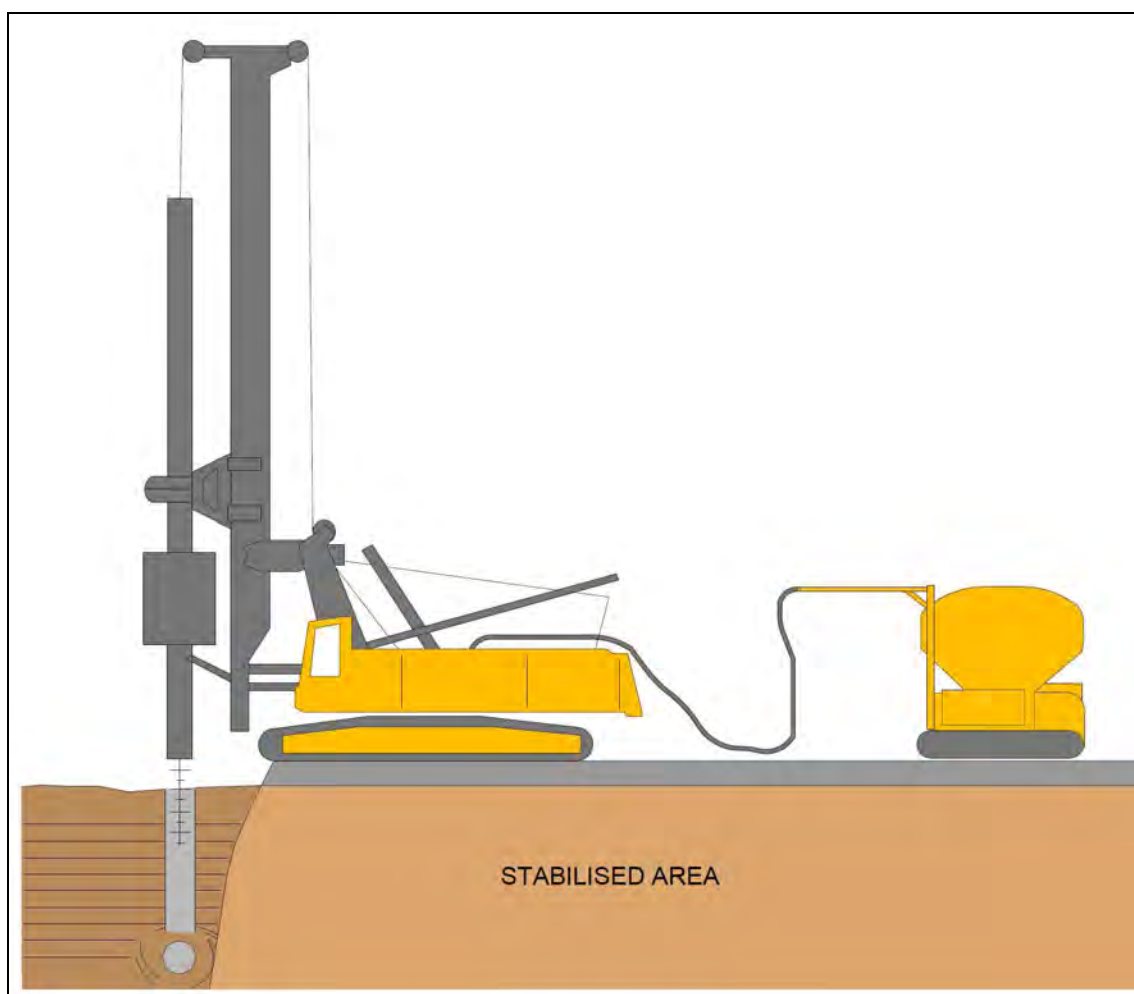


Figure 6-3 - In-situ Mass Deep Stabilisation

In-situ mixing is usually carried out by a paddlewheel or similar plant. A temporary working platform would be locally established on the mudflats to allow plant movement and ease of operation within the designated construction corridor. In-situ mixing can occur on either side of high tide, i.e. once the sea water has retreated sufficiently to allow plant to operate.

Slope stability of the existing Causeway is unlikely to be affected by the stabilisation works, and no removal of the marine mud would be needed as cement and/or lime are the only materials required to be brought from off-site.

6.1.2 Marine Deposit Displacement (MDD)

With this method, a layer of geotextile is first placed on the ground surface prior to placing the hardfill and only lightweight machinery is used to tamp down the hardfill. The principle is to replace the upper layer of marine mud with a blanket of stone. The incorporation of the geotextile fabric between the two materials keeps them from intermixing. By forcing the stone blanket into the marine mud, the technique of Marine Deposit Displacement (MDD) also achieves some

densification of the marine mud itself but, as only lightweight tamping is undertaken, the layer of stone is not densified greatly.

MDD may be used as a substitute for foundation undercut. However, this technique does not sufficiently improve the ground for it to act as a substitute for stabilised soil ("Mudcrete"). The MDD is particularly useful as a foundation support in areas where lightweight fill is to be placed, such as adjacent to the abutments of the Whau River and Causeway Bridges.

6.1.3 Foundation Undercut

The principle of foundation undercut is to remove weak foundation materials (typically 1 to 2m deep) and replace them with less compressible, durable, hard fill. This technique primarily addresses foundation bearing capacity and to a lesser extent, long-term settlement.

Foundation undercut is not preferred along the Causeway where depths of compressible layers extend several metres below formation level and the ground displacement criteria cannot be satisfied. The technique is best suited for foundation treatment works outside the Causeway where thin layers of marine deposits can be easily removed and replaced with hard fill materials. Where excavations are likely to be affected by running water or tidal inundation, MDD is recommended as a suitable alternative to foundation undercut.

6.1.4 Geogrid Reinforced Raft

In this technique one or more layers of geogrid reinforcement are introduced within a controlled granular fill material and placed at the interface between the underlying marine mud and the new fill embankment.

The use of a geogrid reinforced raft addresses differential settlement, shear and bearing capacity failure issues. Hence it works quite well as a safe construction stage working platform which may form part of the permanent work design to resist global shear failure and improve foundation stiffness.

6.1.5 Lightweight Fill

In this technique, filling over settlement prone soils are carried out using a low density material; a favoured material in recent times is polystyrene which strictly speaking is an ultra lightweight material. Alternatively, portions of the underlying compressible soils can be dug out and replaced with lightweight material. Either approach reduces foundation settlement and gravitational loads on existing structures.

Polystyrene fill can be placed quite easily and quickly within the embankment. Due to the cost of such material, it is only recommended to be placed near bridge abutment structures where strict differential settlement criteria apply.

A geotextile membrane and drainage blanket is typically provided at the interface between the lightweight material and the in-situ soils. For installations under elevated ground water conditions, as the case may be for the Causeway, additional measures such as providing a protective reinforced concrete slab may be needed. This should be assessed as part of detail design. Where foundation uplift becomes an issue, an appropriate tension member may be detailed which could tie to the existing Causeway.

6.2 Compensatory Filling

Although the ground improvement options discussed above will reduce the settlement compared to the unmitigated condition, substantial settlement will still take place. Ground improvement has not been planned to eliminate settlement, it being a general philosophy of the project that some settlement is unavoidable and therefore the design needs to accommodate it. The ground displacement criteria Section 4.3.2) recognise this. Even with this approach there still remain two fundamental issues that affect the design that need to be addressed:

- As fill cannot be instantaneously placed, some settlement takes place during the construction phase. In fact settlement rates are at their highest when a load is initially placed and therefore the construction phase settlement is significant.
- Settlement predictions cannot be reliably made to an accuracy of much better than from about 40% below to 50% above the mean. Therefore despite best efforts to achieve certain levels, the actual levels may fluctuate.

The approach adopted to address these fundamental issues is to adopt compensatory filling, which can be undertaken either proactively to place an extra volume of fill to compensate for predicted settlement or reactively to place extra fill as a result of actual settlement that has taken place. For this project, both techniques are proposed.

For each portion of the project, the amount of settlement estimated to take place between the placement of the fill and the design date (i.e. project completion / road opening) is to be compensated by overbuilding through placing a volume of additional general fill equivalent to the amount that is apparently lost through construction phase settlement. This approach will assure that the design levels are achieved.

Compensatory filling is also planned to be used as a reactive measure during the operational phase of the project if the actual rate of settlement in local areas is substantially greater than estimated. In this case the technique would be applied by re-grading the road surface and topping it up to bring it back within tolerance.

7. Geotechnical Assessment with Ground Improvements

The analytical philosophy to the design of earthworks, ground improvement and structures considers the following criteria in the order of priority:

Stability: The new Causeway needs to be safe during construction with adequate bearing capacity and to satisfy overall global slope stability requirements throughout the design life.

Settlement: It has been accepted that it is not economically viable to mitigate all effects of settlement and that settlement will continue over the 100 year design life. The foundation treatment works aim to achieve two results: limit cumulative settlement within 25 years of commissioning to less than 400mm and maintain crest elevation above +3.0RL throughout the 100year design period.

7.1 Settlement Analysis

7.1.1 Accounting for Staging

For a straightforward assessment of settlement, it has been assumed that the Causeway will be raised and widened in a single operation. Practically speaking however, it will be necessary to construct the embankment in stages, as illustrated in Figure 7-1:

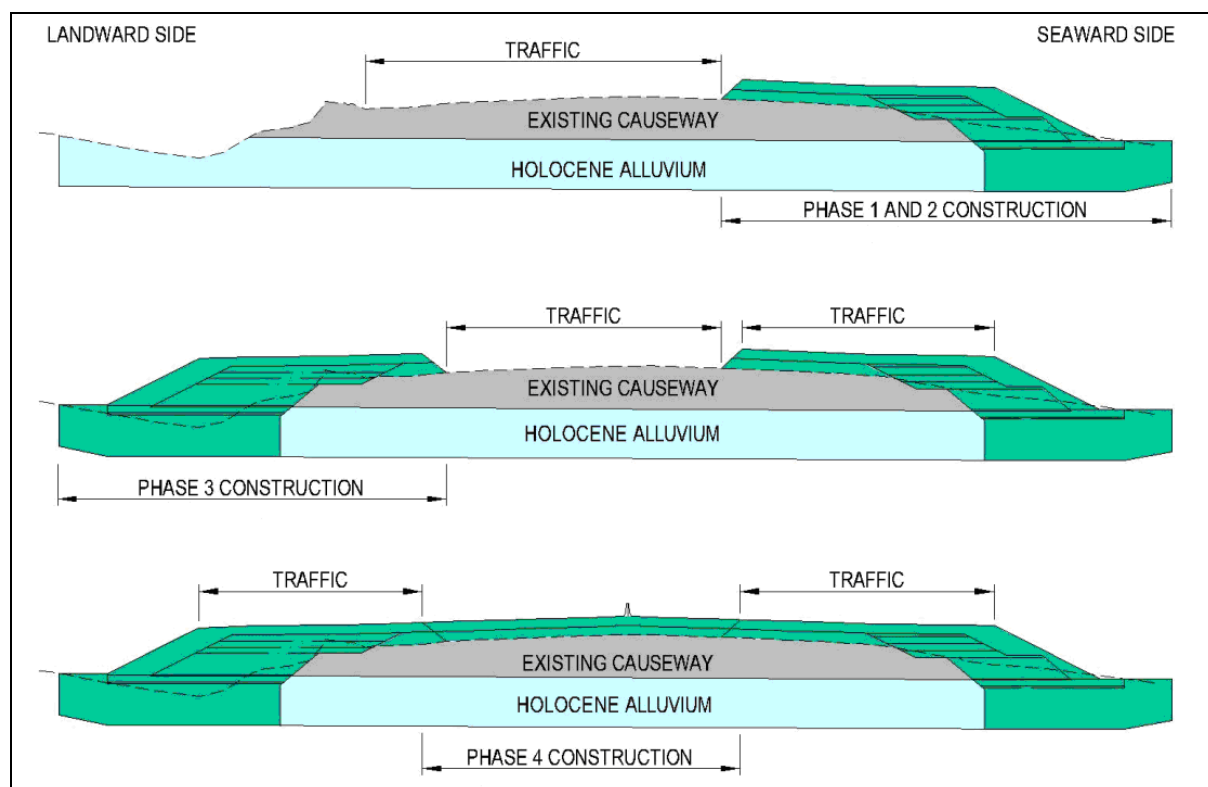


Figure 7-1 - Sketches Showing the Methodology of Staged Causeway Construction used in analysis

Embankment settlement and lateral deformation have been assessed for a 3-stage sequence of filling works along the critical and representative sections as presented in the Ground Improvement Options Report (Aurecon, 2010c).

Each stage of works makes assumptions pertaining to construction time and, where appropriate, strength gain of the underlying soils due to staged loading:

- i) Phases 1 and 2 (Stage 1 works) are assumed to take place with the closure of the northern shoulder which will cover about one-third of the existing Causeway width. It begins with the construction of a temporary working platform which is expected to take about 2 months. Once the temporary working platform is in place, ground improvement work will be carried out. It is assumed that this item of work will take about 7 months. The new embankment will then be constructed once the ground improvement work is complete. Approximately 24 months has been assumed for this phase of works

During the construction of motorway widening the undrained shear strength of Holocene Alluvium under the existing Causeway is assumed to be 20kPa, as it has consolidated and gained strength since its placement in the 1950s.

- ii) The second stage (Phase 3) of filling will form the southern shoulder and also cover about one-third of the final Causeway width. The construction of stage 2 follows a similar timeframe and construction sequence to that in stage

1. It takes approximately 2 months for the construction of the temporary platform, 7 months for ground improvement work and 13 months for the construction of the new embankment.

- iii) The third stage (Phase 4) of filling operations, to close the gap between the shoulders, will cover the remaining middle part of the Causeway. This is expected to take about 14 months to complete.

These time frames outlined above indicate (in part) the likely solutions for ground improvement. For example staged preloading alone will not meet the construction sequence planned if the necessary gains in stability are to be achieved. For the settlement analysis it has been assumed that all of the fill volume required for each stage is placed instantaneously at the commencement of each stage.

7.1.2 Settlement Estimates after Ground Improvement

7.1.2.1 Settlement of the Embankment

Analysis has been carried out for the proposed earthworks to estimate settlement after each stage of works with the specified ground improvement measures. Table 7-1 summarises the results of the one dimensional settlement analysis performed after the ground improvement works.

Chainage	Location	Average Fill Height from MX Design Section (m)	Total Settlement ^{2,4}			Ground Improvement Works Provided	% Reduction in Settlement with Ground Improvement Works
			end of T ₁ Yrs ⁵	end of 30 Yrs ⁶	(0 to 100 years)		
			(mm)	(mm)	(mm)		
1100	South ¹	2.6	T ₁ =4 Yrs = 130mm	195	235	2.5m Mudcrete ⁷	63
	Centre ¹	0.8	T ₁ =5 Yrs = 56mm	116	160	NA	NA
	North ¹	2.5	T ₁ =2 Yrs = 125mm	185	225	2.5m Mudcrete ⁷	63
1350	South	4.2	T ₁ =4Yrs = 375mm	620	740	3.5m Mudcrete ⁷ at CH1350 and then transition to light weight fill near Causeway Bridges	37
	Centre	No Fill	NA	NA	NA	NA	NA
	North	3.3	T ₁ =2 Yrs = 290mm	530	645	3.5m Mudcrete ⁷ at CH1350 and then transition to light weight fill near Causeway Bridges	37
1600	South	5.8	T ₁ =4 Yrs = 485mm	970	1170	3.5m Mudcrete	31
	Centre	1.6	T ₁ =5 Yrs = 191mm	421	553	NA	NA
	North	3.3	T ₁ =2 Yrs = 360mm	885	1065	3.5m Mudcrete	26
2150	South	2	T ₁ =4 Yrs = 411mm	841	1092	3.5m Mudcrete	18
	Centre	1.9	T ₁ =5 Yrs = 339mm	699	870	NA	NA
	North	2	T ₁ =2 Yrs = 380mm	805	1043	3.5m Mudcrete	14

Table 7-1 - Summary of one dimensional settlement analysis (After ground improvement works)

Notes:

1. South: Southern Shoulder, North: Northern Shoulder. The settlement figures shown are the average settlement values across the shoulders.
2. Settlement analyses for North and South adopt the 'most credible likely' case parameters for coefficient of volume compressibility m_v and coefficient of consolidation c_v . Settlement analyses for Centre adopt the 'best probable case' parameters for m_v and c_v . The analyses also adopt a combined c_v where drainage paths cross multiple soil layers. ATs/ATv layers have been assumed to be drainage layers.
3. Based on cross-sections generated by MX road software.
4. Secondary consolidation is assumed to begin at time T95 when 95% primary consolidation has taken place. T95 is assumed to have occurred about 50 years ago at the centre of the Causeway.
5. Total settlement at the end of each embankment shoulder construction phase, which approximately corresponds to the 2nd yr of the entire construction programme for the Northern Shoulder, the 4th yr for Southern Shoulder and the 5th yr for the Centre of the Motorway.
6. Total settlement at the end of 30th year from the commencement of the construction programme.
7. Mudcrete has been assumed to have negligible coefficient of volume compressibility M_v and very high coefficient of consolidation C_v .

A combined settlement profile has been generated along representative Causeway design sections Ch1600, Ch1800 and Ch2150 and indicates settlement estimates at the end of the 5th year from start of construction (i.e. official commissioning year), at the end of the 30th year (25 year operational period assumed) and at the end of the 100th year (the intended design life). A sketch indicating the average amount of settlement that may occur across the Causeway section of SH16 is presented in Figure 7-2. The sketch also presents the amount of settlement anticipated using settlement results of the three critical chainages.

Figure 7-2 presents one of the settlement profiles across the Causeway at critical chainage CH1600. Also provided are the amounts of compensatory filling and/or surface regrading required to make up predetermined road elevation (MX levels) at the end of the 30th year (25 year operational period assumed) and at the end 100th year (the intended design life).

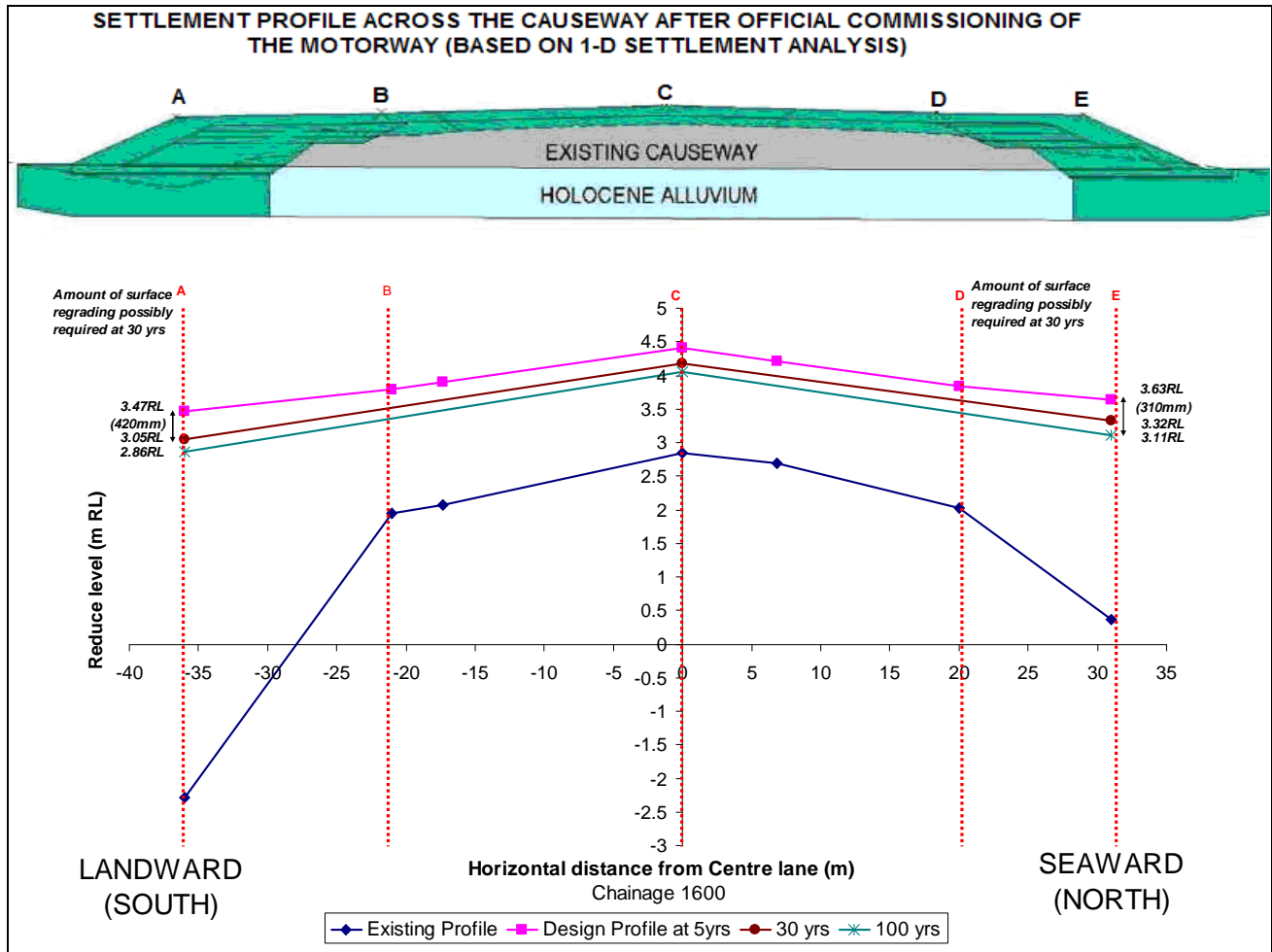


Figure 7-2 - Settlement Profile at Cross Section CH1600 (Based on 1D settlement analyses)

Furthermore, settlement estimates have been provided across several critical and representative sections along the Causeway for the various phases of work and also during the operating period of the motorway.

It can be seen from Figure 7-2 that in general the Causeway section will be above +3.0mRL in 100 years except in the vicinity of point A where the proposed cycleway is located. Therefore at point A surface regrading of about 620mm will be required.

The analytical results tabulated in the Ground Improvement Options Report summarise the resulting immediate, primary and secondary settlements after the implementation of the selected ground improvements works. The settlement estimates show that immediate settlements are generally small and insignificant compared to the consolidation settlements. Immediate settlements also do not affect the post construction settlements. They only affect the fill thickness required to make up the road elevation design level (MX level).

Preliminary settlement estimates indicate that, construction of the south and north shoulders between CH1600 and CH2550 will result in post construction settlements varying from 400mm to 700mm in 30 years if no ground improvement measures are implemented. This exceeds the allowable settlement criterion of 400mm. However, with the implementation of soil stabilisation, the post construction settlements in these areas can be reduced to about 400mm. It must also be appreciated that although post construction settlements from CH1100 to CH1600 and CH2550 to CH2800 before ground improvement are less than 400mm, soil stabilisation is also recommended on the basis of global stability requirements and to limit excessive lateral movement.

7.1.2.2 Settlement at Existing Structures

The effect of newly placed fill adjacent to existing bridge structures at the Causeway Bridges and Whau River Bridges have been briefly assessed based on a limiting 25mm differential settlement criterion measured over a 10m distance from such structures. There are two potential effects: firstly, there could be an increase in drag-down effect on the existing piles as a result of settlement. Secondly, the existing piles will be subjected to an increase in lateral pressures.

Settlement analysis carried out at these locations indicates that the stated differential settlement criterion cannot be achieved without some form of ground improvement work. Lightweight fill is proposed.

Other areas identified to benefit from the application of lightweight fill are chainages 1100 and 1360 near the Causeway Bridges and the cycleway bridge abutments, chainages 2710 and 2900 for the Rosebank Road Ramps and the Cycleway Bridge and CH4700 for the Whau River Bridge.

A transition zone of 40m (minimum) to the lightweight fill material embankment has to be provided to avoid excessive differential settlement between the lightweight fill zone and the rest of the embankment earthworks.

At the Whau River Bridge a technically viable alternative solution is the installation of piles below the abutment; a steadily increasing spacing between piles over the length of treatment will provide a transitional zone. A piled embankment solution is not recommended as this option will be more costly and time consuming than lightweight fill.

It has been noted from existing as-built records for the Causeway Bridges (previously named the Rosebank Bridges) that lightweight fill material was used in the abutment in 1992 when widening works to the existing bridge were carried out. The details adopted for the abutment filling works included the use of Polystyrene lightweight fill with a concrete protective slab. No monitoring records are available on the performance of the lightweight fill material since its placement.

7.2 Bearing Capacity Analysis

Analysis indicates that the Holocene alluvium in its unimproved state does not provide adequate foundation bearing restraint as a working platform. And after the placement of fill, there is a tendency of embankment cracking and general bearing failure.

A suitable working platform consisting of a geogrid reinforced raft has been proposed to satisfy construction stage bearing capacity requirement. This consist of one or more layers of geogrid reinforcement which is to be introduced within a granular fill material and placed on top of or at the interface with the underlying soft clay and the new fill embankment. The platform enables construction plant to be able to track over the soft marine mud and may also form part of the permanent work design to resist global shear failure and improve foundation stiffness.

Other long-term solutions considered include in-situ soil stabilisation (mudcrete or similar), foundation undercut and Marine Deposit Displacement.

shows the double role of the reinforced raft as it adds to foundation bearing capacity and also provides a restraint against construction stage slope failure.

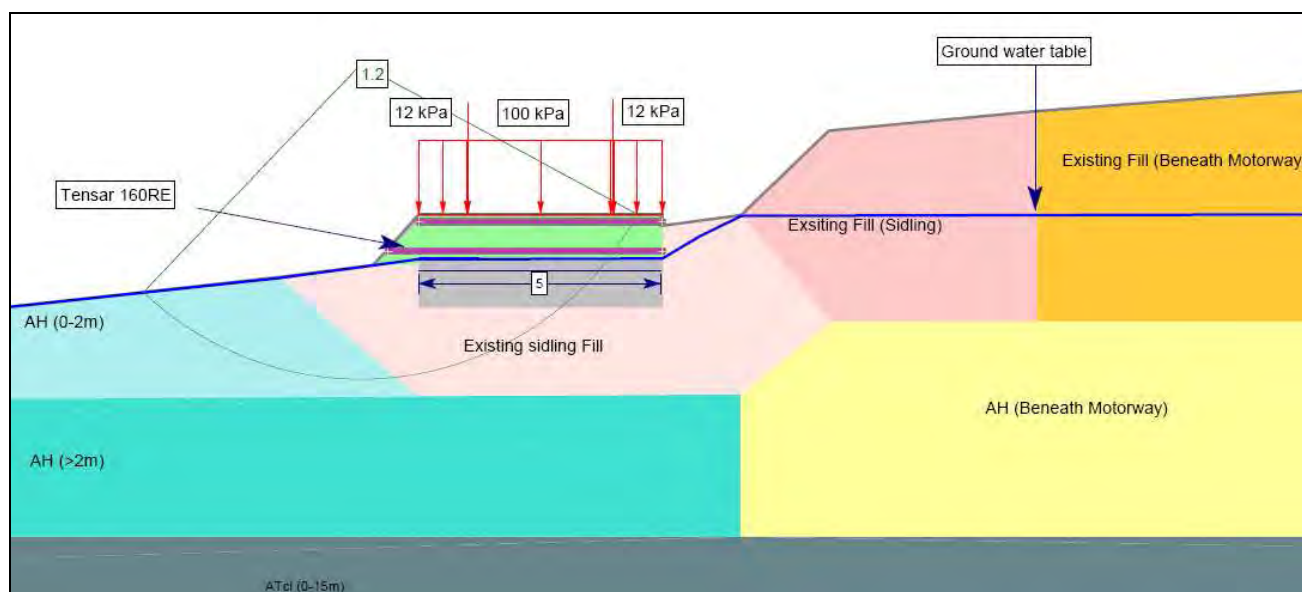


Figure 7-3 - Case (d) –Short-term global factor of safety during construction on a working platform (CH1350)

7.3 Liquefaction Assessment

Liquefaction is said to occur when inter-particle shear resistance (effective stress) within a soil mass drops to zero in response to an increased in pore water pressures which is triggered by a sustained seismic activity (energy), resulting in a loss of strength.

In keeping with New Zealand Standard NZS1170.5, the earthquake intensity for the SH16 motorway liquefaction assessment has adopted a generic earthquake magnitude of 6.5 having a probability less than 10% in 150 years at an epicentral distance¹ 20km from the site. This earthquake is likely to be the largest that could be developed on any local or regional fault near Auckland. For this event the road structures are expected to undergo plastic behaviour but retain sufficient strength and form to prevent significant movements from occurring. For this earthquake event an ultimate limit state (ULS) peak ground acceleration at the site is about 0.15g.

The results of the liquefaction assessment suggest that liquefaction of the proposed foundation stratum for a 1 in 1500 year return period earthquake of magnitude 6.5 is unlikely to occur except for minor liquefaction in discrete thin pockets of Tauranga Group Alluvial Sand (ATs) layer, which occur only infrequently over limited extents and therefore the potential for liquefaction is negligible.

¹ Epicentral distance is the horizontal distance between the site and the point on the ground surface directly above the focus of the earthquake.

The results also show that liquefaction of the proposed foundation stratum for a 1 in 1500 year return period earthquake of magnitude 6.5 will only result in minor settlement (20-80mm) in localised lenses. Hence, liquefaction is not a key issue in the ground improvement design of the embankment earthworks along the Causeway.

7.4 Slope Stability Analysis

7.4.1 Global Slope Stability Analysis

The mitigation solutions best considered to satisfy the global stability requirement are the construction of a shear key at the toe of the embankment formed by in-situ soil stabilisation (i.e. Mudcrete) together with the incorporation of geogrid reinforcement within the core structural fill.

These mitigation details are presented in an analytical model of the embankment shoulders and analysed for global instability using a 2-D Limit Equilibrium computer software Slide version 5.0 (Rocscience, 2009). Circular analysis has been considered for both long and short-term stability together with recommended geotechnical design parameters and the specified Earthworks Stability criteria. Short-term, as defined in this report, refers to slip failures that could occur during and immediately after construction. Adopted short-term parameters for the existing fill are in keeping with the undrained behaviour. Seismic analysis adopts non-circular block sliding analysis but with the same short-term analytical model but with no consideration for motorway live loads.

7.4.2 Results of Global Slope Stability Analysis

The results of slope stability analysis with different loading scenarios and with ground improvement works indicate that the slope stability criteria stated in Section 4.3.1 are satisfied (refer to Table 6-5, Ground Improvement Options Report, Aurecon 2010c).

Figure 7-4, Figure 7-5 and Figure 7-6 present a typical geotechnical model showing ground improvement works and the critical slip surfaces for the slope stability analyses at chainage Ch1350.

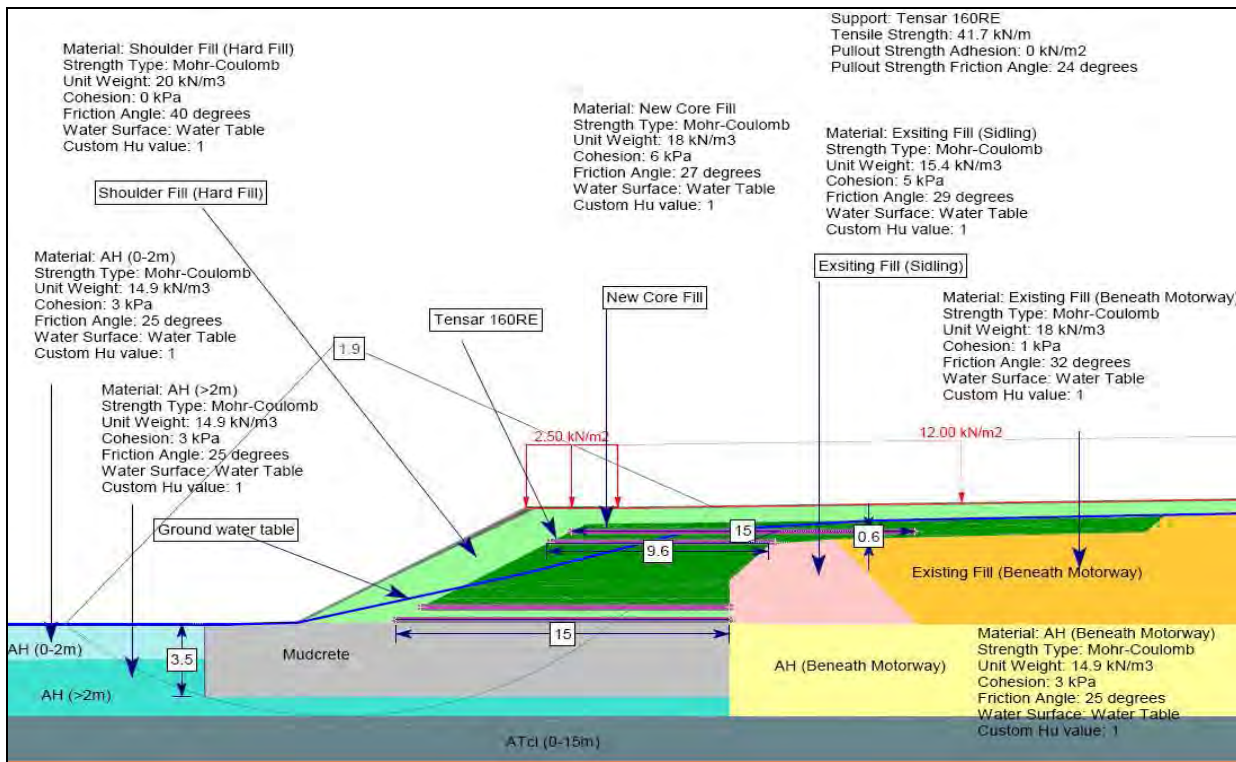


Figure 7-4 - Case (a) – Static long-term global factor of safety under design operating condition (CH1350)

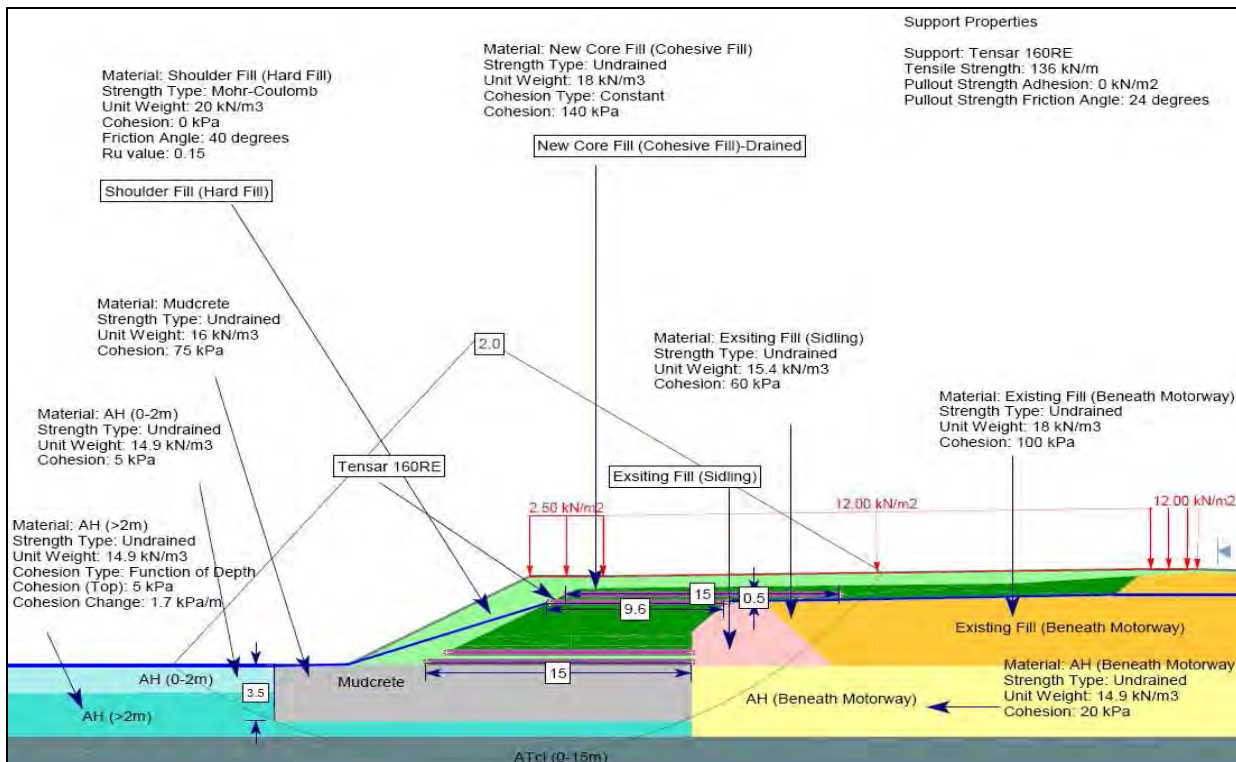


Figure 7-5 - Case (d) – Short-term global factor of safety immediately after construction (CH1350)

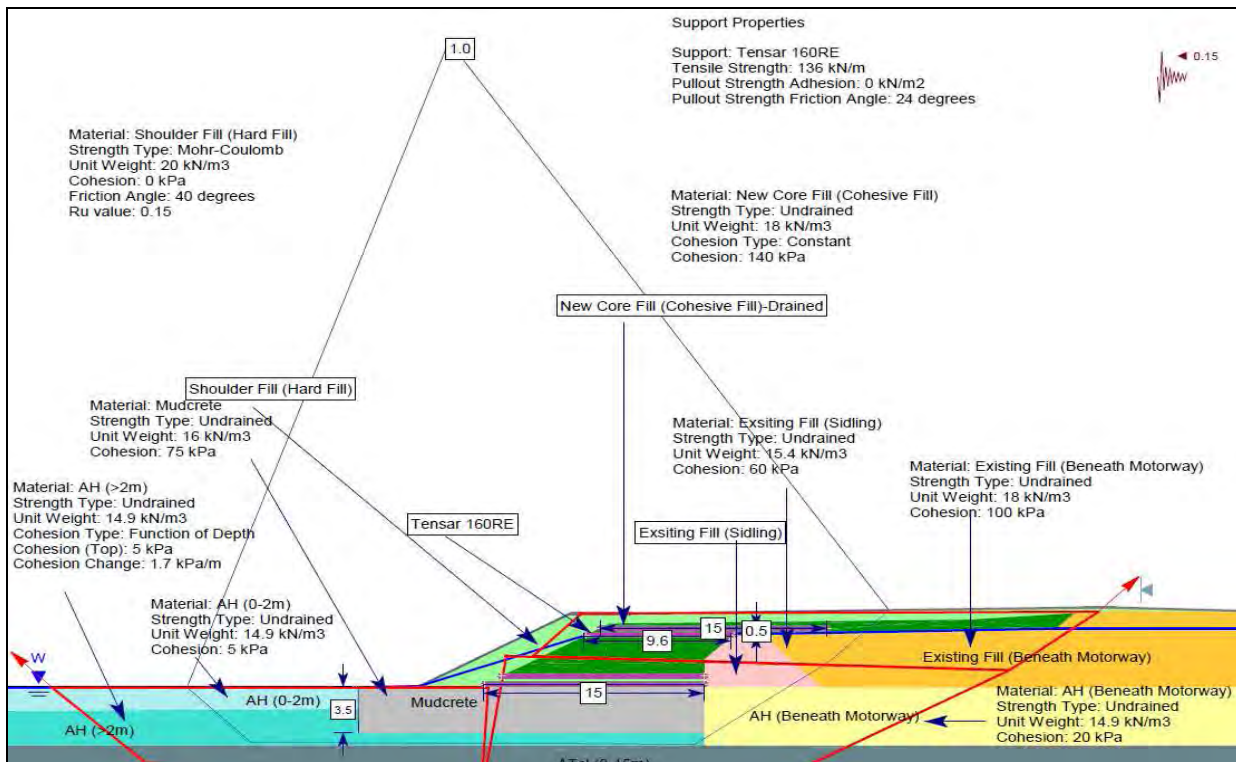


Figure 7-6 – Case (e) – Seismic short-term global factor of safety (CH1350)

7.4.3 Slope Stability Sensitivity Analysis

Because of a lack of information of what constitutes the existing fill and what may be the geotechnical properties of the existing fill materials, a sensitivity analysis has been undertaken which varies the values of the short term parameters of the fill in a slope stability model with traffic load.

The varied properties refer to whether the existing fill behaves in a drained or undrained manner in the short-term stability model, with no gain in strength in the foundation soils. The objective is to establish a conservative model of the existing Causeway using appropriate short-term parameters. Then use it to establish the basis for the preliminary design of the new embankment by carrying out a performance based analysis of the new and existing Causeway.

Detailed assessment is presented in the Ground Improvement Options Report (Aurecon, 2010c) but findings for the critical section chainage Ch 1600 is presented below.

7.4.3.1 Existing Causeway Stability at Ch 1600 for Drained Behaviour of Existing Fill in the Centre

In this model a clay bund 3.5m (10ft) wide, has been adopted as sidling fill which is considered a reasonable engineering solution for the 1950s and conforms with borehole log of DH307 of the Stage 1 and 2 ground investigation and one set of as built records. The undrained shear strength of this clay bund sidling fill is assumed to be 60kPa and the remaining fill is treated as granular material with drained parameters ($c = 1\text{kPa}$ and $\Phi = 32$ degrees).

Through this approach a representative model is created of the existing ground conditions at Ch1600. The resulting parameters are considered to be realistically conservative since they yield a Factor of Safety of 1.2 (Figure 7-7)

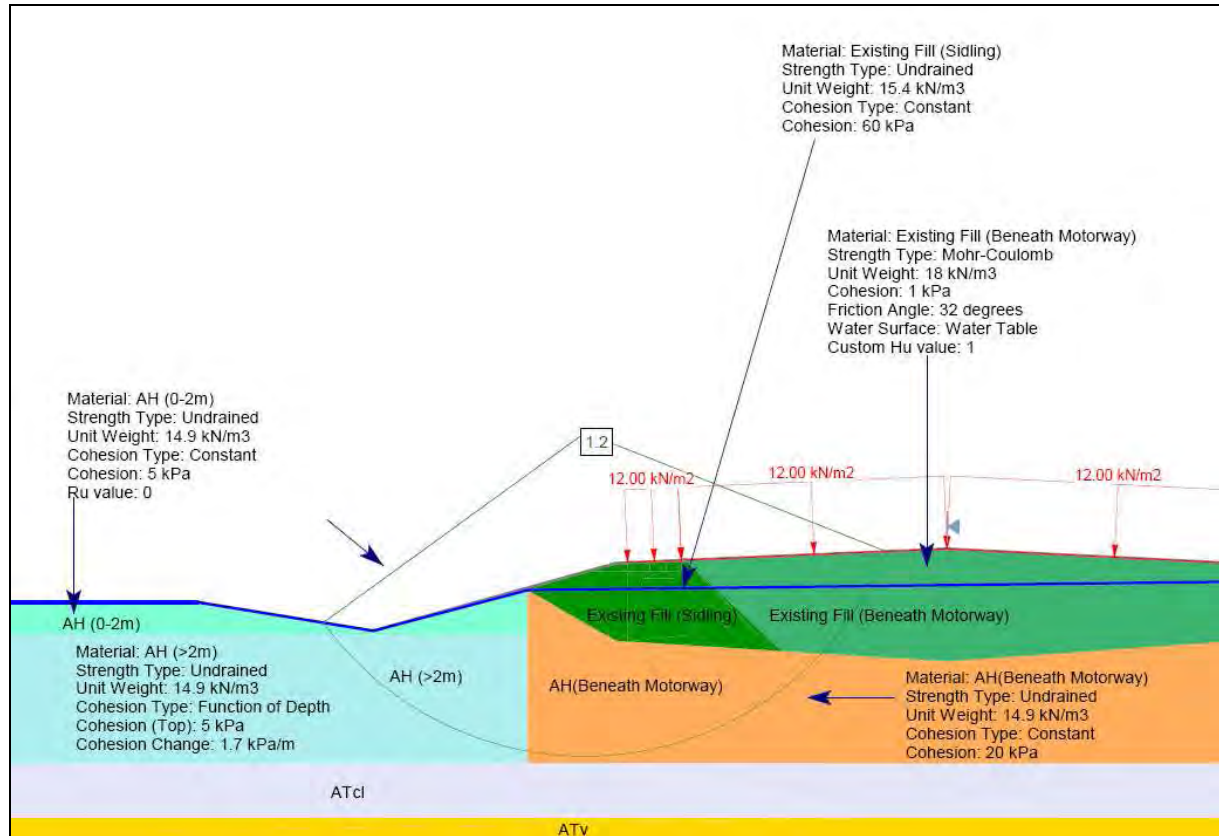


Figure 7-7 - Sensitivity analysis of existing Causeway

Having established a conservative model of the existing Causeway, these values are then used to form the basis upon which the new embankment is to be built. The input parameters for the preliminary design of the new embankment is then interrogated by carrying out a performance based analysis of the new and existing Causeway.

7.4.3.2 New Causeway Stability at Ch 1600 (Accounting for Staged Construction) for Drained Behaviour of Existing Fill in the Centre

Maintaining the same values for the existing Causeway and superimposing the new embankment shoulder with its ground improvement measures and assumed input parameters for the short term condition yields a Factor of Safety of 1.3 (Figure 7-8).

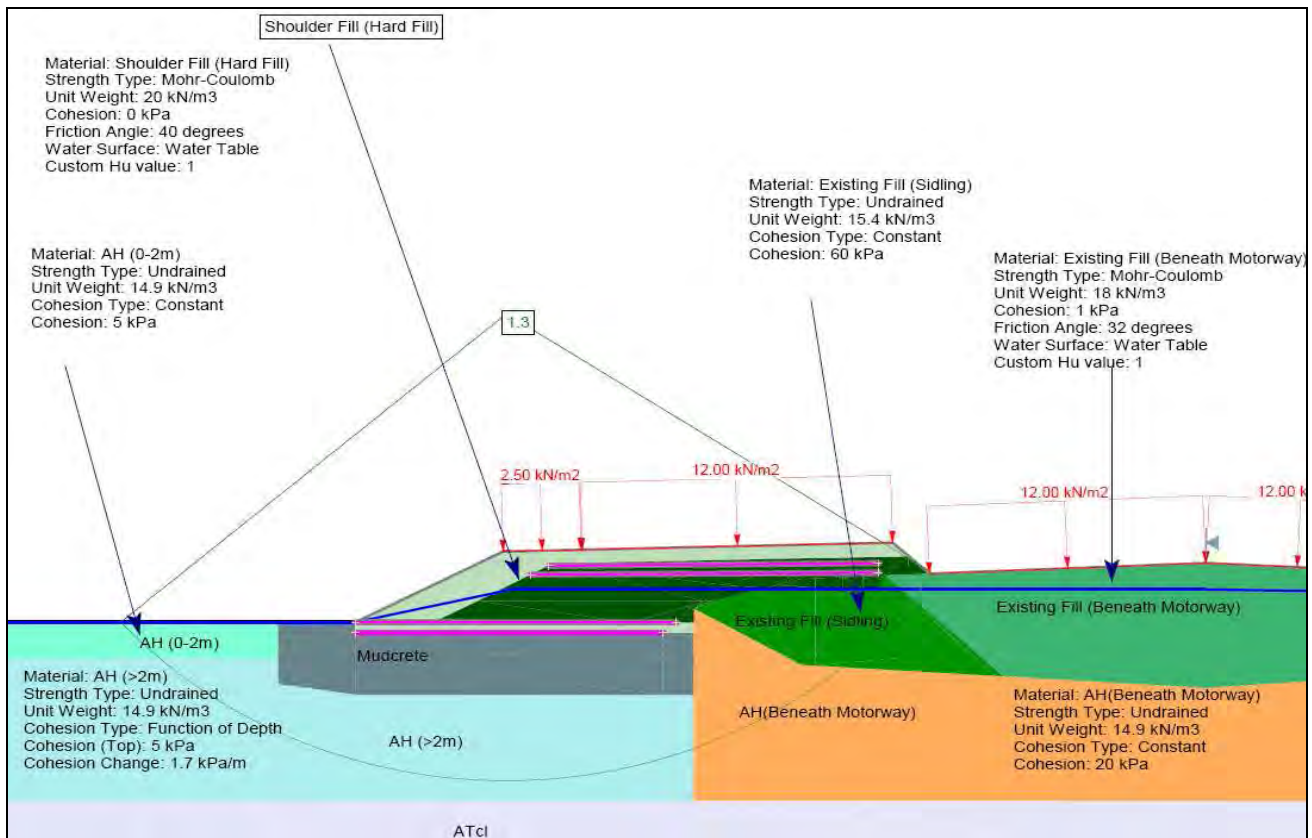


Figure 7-8 - Sensitivity analysis of new Causeway accounting for staging

7.4.3.2.1 New Causeway Stability at Ch 1600 (Not accounting for Staged Construction) for Drained Behaviour of Existing Fill in the Centre

The final sensitivity model considers the extension of the new fill across the width of the embankment which gives a Factor of Safety of 1.3 (Figure 7-9). Hence the sensitivity analysis shows that the global stability of the proposed embankment in the short term is still maintained when the existing fill under the carriageway is exhibiting drained behaviour.

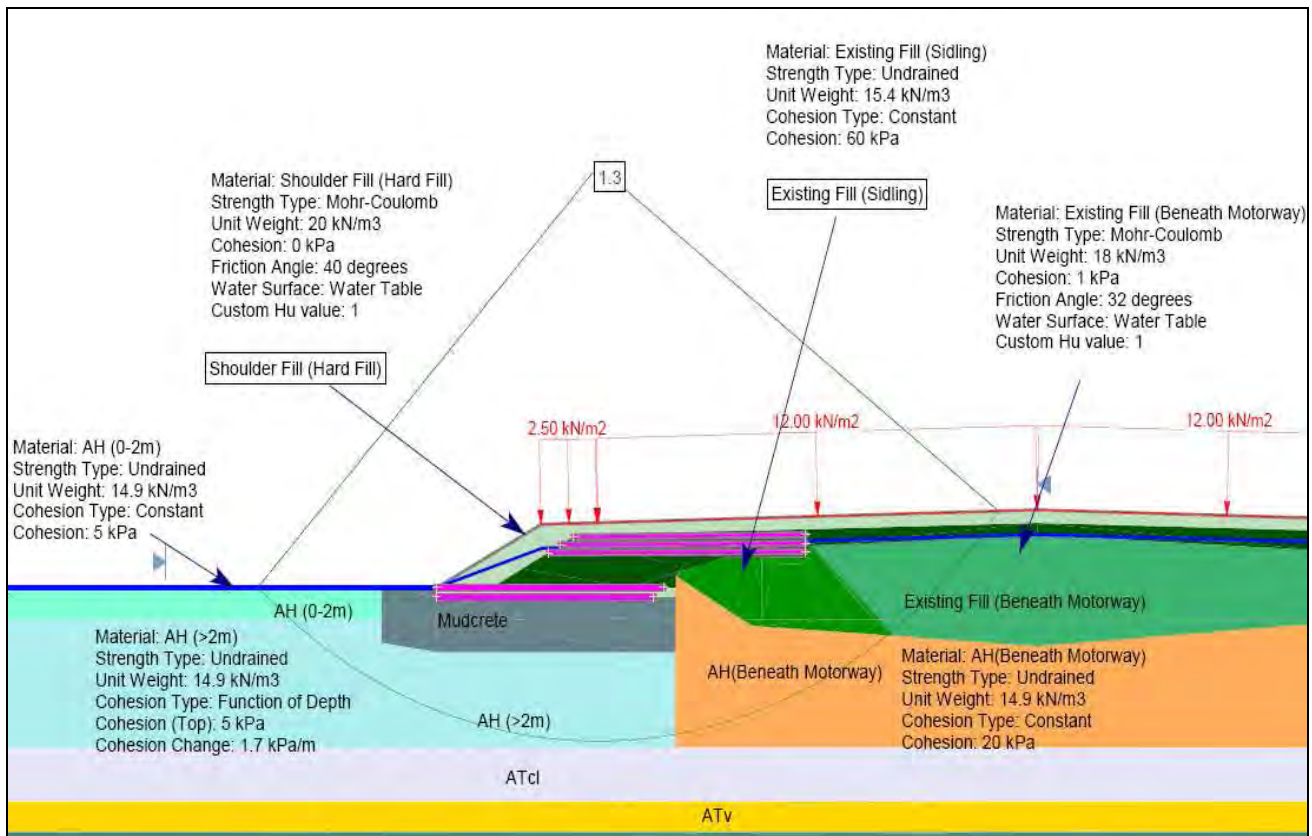


Figure 7-9 - Sensitivity analysis of new Causeway not accounting for staging

7.4.3.3 Discussion on Sensitivity Analysis for Existing Fill Drained Behaviour

It can be concluded that the global slope stability of the new embankment is adequate (with a factor of safety more than 1.2) in the short term conditions when the existing fill is exhibiting undrained or drained behaviour. It should be noted that these factors of safety are achieved with no increase in strength of the underlying soils. The reality is that they will strengthen under load and stability will be enhanced.

7.4.4 Sensitivity Analysis at Ch 1600 using reduced Short-term Undrained Parameters

It is considered that the ground conditions immediately after the construction of the Causeway embankments and just before opening to traffic are the most critical to stability as the embankment is at full height and the materials (recent alluvium) immediately underlying the improved ground have not had sufficient time to gain much shear strength, with the undrained shear strength at a minimum of 5 kPa.

The location considered to be the most critical is the landward section at chainage 1600, because:

- The marine sediments are deepest at this location (around 10m thick);
- The proposed embankment fill height is highest (4.6m);

- A drainage channel is to be relocated close to the toe of the proposed embankment so any failures could significantly affect the integrity of this channel.

A number of different stability analyses have been undertaken for this critical section and the results are discussed as follows:

- Analysis of sliding, overturning and bearing stability of the new Causeway embankment (with geogrid reinforcement) and ground improvement works has been undertaken in accordance with BS8006, where reduction factors are applied to the soil parameters and additional factors are applied to the surcharge (i.e. embankment). Consideration is also given to lateral squeezing stability in accordance with Section 4.1.5 of Transfund New Zealand Research Report No. 239 guidelines. The findings indicate that the sliding, overturning and bearing capacity of the embankment immediately following construction is satisfactory. Moreover, lateral squeezing stability is not critical as the thickness of the marine mud together with the soft soil layers of alluvial peat and organics is much thicker than the height of the fill embankment.
- A sensitivity check has been undertaken, which discounted the passive earth resistance of the surface 2m of marine sediments in front of the improved ground. The results of this analysis also indicate that the stability of the Causeway embankment immediately following completion is still adequate.
- The overall slope stability of the embankment has been calculated for the critical section (chainage 1600). Appropriate short-term parameters and a full traffic load were used for the analysis and a factor of safety against failure > 1.2 (refer to Section 7.4.2), which suggests that it is highly unlikely that slope instability (i.e. slips) will occur.

This sensitivity analysis follows on from what has been assessed in the Ground Improvement Options Report by significantly reducing the soil input parameters. The analysis attempts to determine the likely ground conditions required to cause significant instability (Factor of Safety of approximately 1.0). The reduction factors are presented in Table 7-2.

Soil Input Parameter	Adopted	Reduced (Sensitivity analysis)	Reduction
Fill Beneath the centre of the Causeway	100 kPa	40 kPa	60 %
Fill beneath the shoulders of the Causeway	60 kPa	40 kPa	33 %
Recent Alluvium beneath the existing Causeway	20 kPa	10 kPa	50 %
New engineered shoulder Fill	140 kPa	60 kPa	57 %
Tensar reinforcement	136 kN/m	60 kN/m	56 %
Virgin recent alluvium (<2m)	5 kPa	5 kPa	0 %
Virgin recent alluvium (>2m)	8 kPa	8 kPa	0 %
Mudcrete Ground Improvement	75 kPa	30 kPa	60 %

Table 7-2 – Reduced Input Parameters for Sensitivity Analysis of Embankment Stability

As highlighted by Table 7-2 a significant reduction will be required in all assumed critical natural soil and ground improvement parameters will be required to significantly compromise the embankment stability. A reduction of all of these parameters (at the same time) is required to reduce the Factor of Safety to 1.0 suggesting that a failure is unlikely to occur. The analysis also indicates that shear strength parameters for the virgin recent alluvium originally adopted as 5kPa is already low enough that any further reductions would not be a realistic representation of the actual conditions. The model output using the parameters presented in Table 7-2 is presented as Figure 7-10.

Notwithstanding a reduction in soil parameters the embankment slope can still achieve a Factor of Safety of 1.0, which theoretically indicates that the slope is on the threshold of failure rather than well below that threshold where failures are likely.

The ground investigations to date demonstrated that the natural soils and fills present on site are all variable but these reduced values represent extremes which are statistically unlikely to occur extensively and simultaneously with extreme values of each material.

Furthermore quality assurance and a construction monitoring programme should reduce the likelihood of unfavourable conditions occurring which could compromise stability.

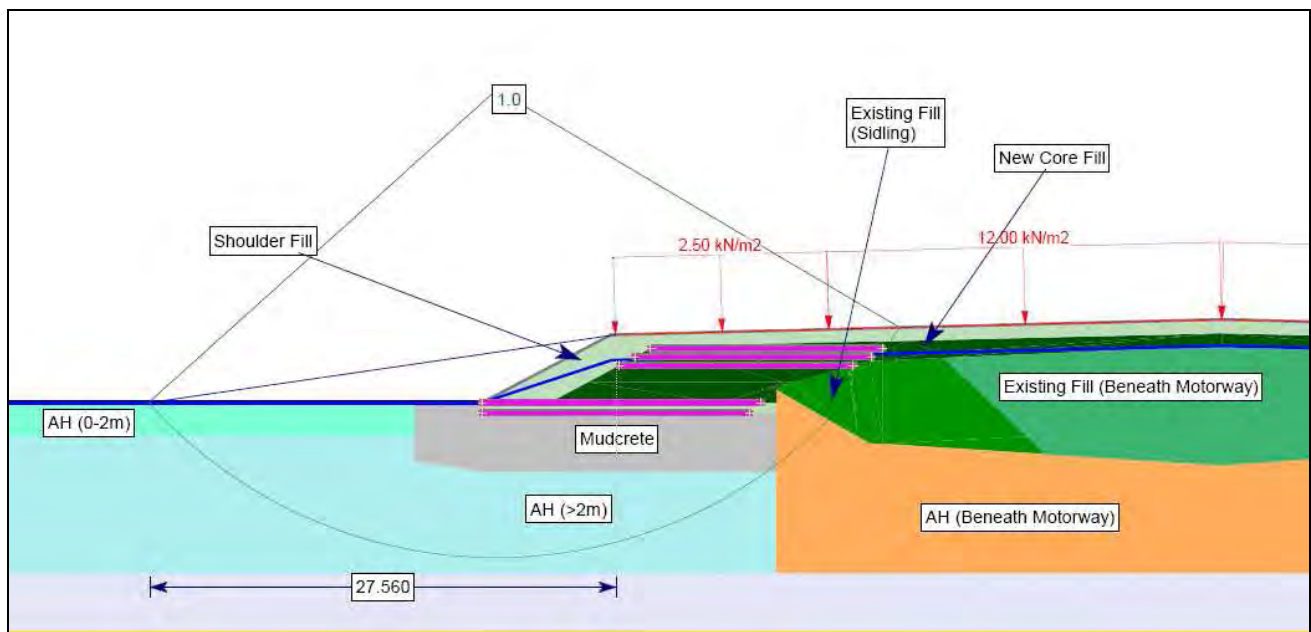


Figure 7-10 - Short-term global factor of safety during construction at critical chainage (1600) with reduced parameters from Table 7-2

7.4.5 Extent of possible embankment slip

In the very unlikely event that significant instability of the embankment were to occur, we have undertaken an assessment of the likely extent of the resulting slip, based on the most critical section (chainage 1600, westbound).

Given the variable nature of the existing Causeway materials and the factors required to induce an embankment slip during construction, it is impossible to define the precise width of slip that may occur. However, experience suggests that a typical dimension of such a slip is likely to be around 20m wide.

We have adopted two analytical approaches to determine the likely distance that any potential slip may migrate from the crest of the proposed Causeway embankment. The first method adopts the circular failure slip approach, which allows for the likely extents of the failure to be determined (Figure 7-10) and the second adopts an empirical method for fill slopes published by Finlay et al (1999) which has been developed using data from over 3000 landslides that have occurred in Hong Kong. It must be understood that as this method is based purely on an empirical formula so the actual travel distance may vary.

According to Finlay, the travel distance of landslips generated by fill slopes over relatively flat ground is empirically related to the geometry of the landslide by the following equation.

$$\log(L) = 0.453 + 0.547 \log H + 0.305 \log (V/W)$$

where H = height of fill slope (m) = 4.6 m

V = volume of landslide (m³)

W = width of landslide (m) = 20 m

L = travel distance of landslide measured from the crest of the slope

As a conservative assumption, the volume of landslide is taken to be the volume of the critical slip surface in global overall slope stability. Hence $V = W \times A$ where A = cross section area of critical slip $\approx 67 \text{ (m}^2\text{)}$ as measured from slope stability analysis using computer software Slide (Version 5).

Hence $V \approx 1340 \text{ m}^3$. And hence $L = 0.453 + 0.547 \log 4.6 + 0.305 \log (1,340/20) = 24\text{m}$ approximately.

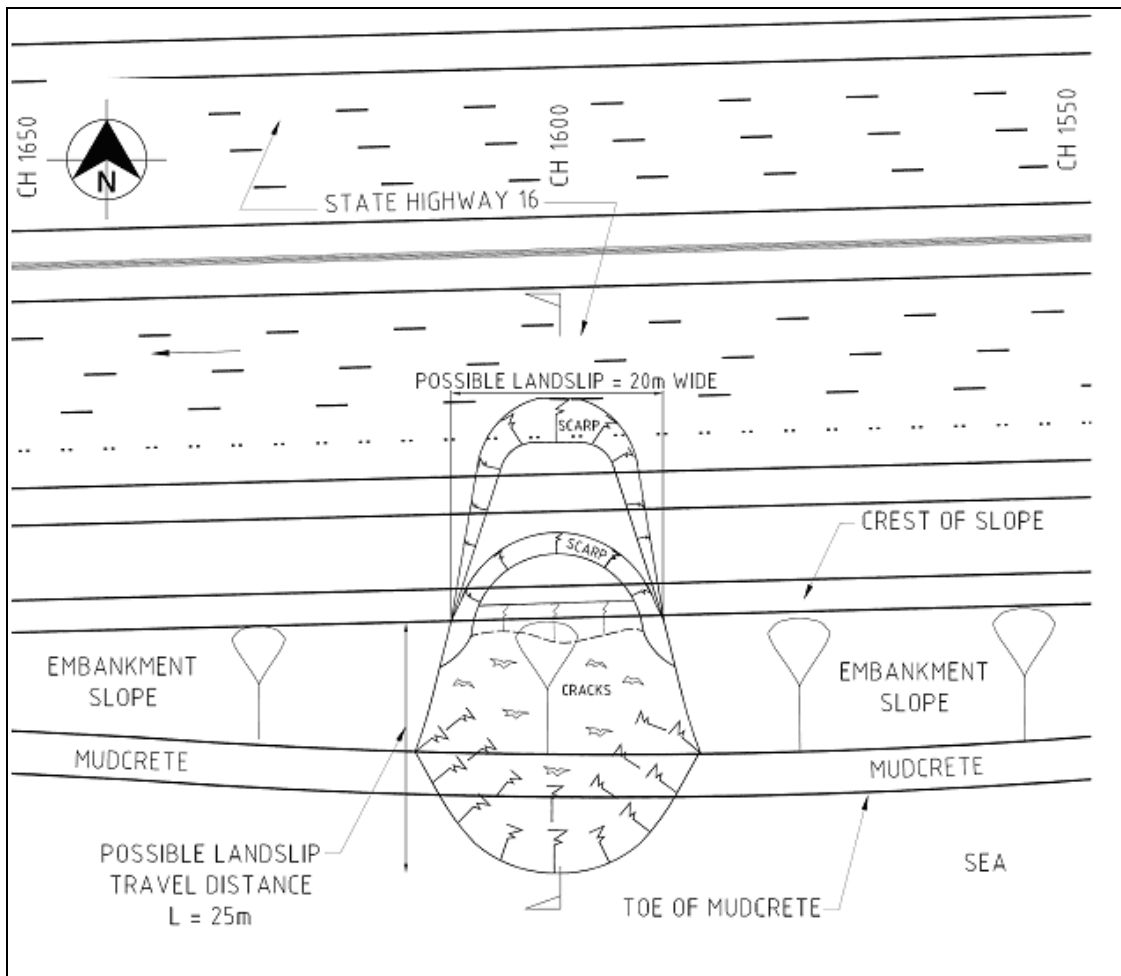


Figure 7-11 - Schematic plan of possible landslide extent based on Finlay et al

The circular failure model (Figure 7-11) indicates that the travel distance of an embankment failure is likely to be in the order of 25m from the crest of the new Causeway embankment. The likely travel distance calculated using the method presented by Finlay et al, was also around 25m. Refer to Figure 7-11 and Figure 7-12 for schematic diagrams.

As indicated by Figure 7-12, if a circular slip of the anticipated dimensions was to occur at the critical chainage (Ch1600 westbound), the toe of the slip would not occur forward of the cofferdam and therefore it would not seriously compromise the cofferdam and the drainage channel would be unaffected.

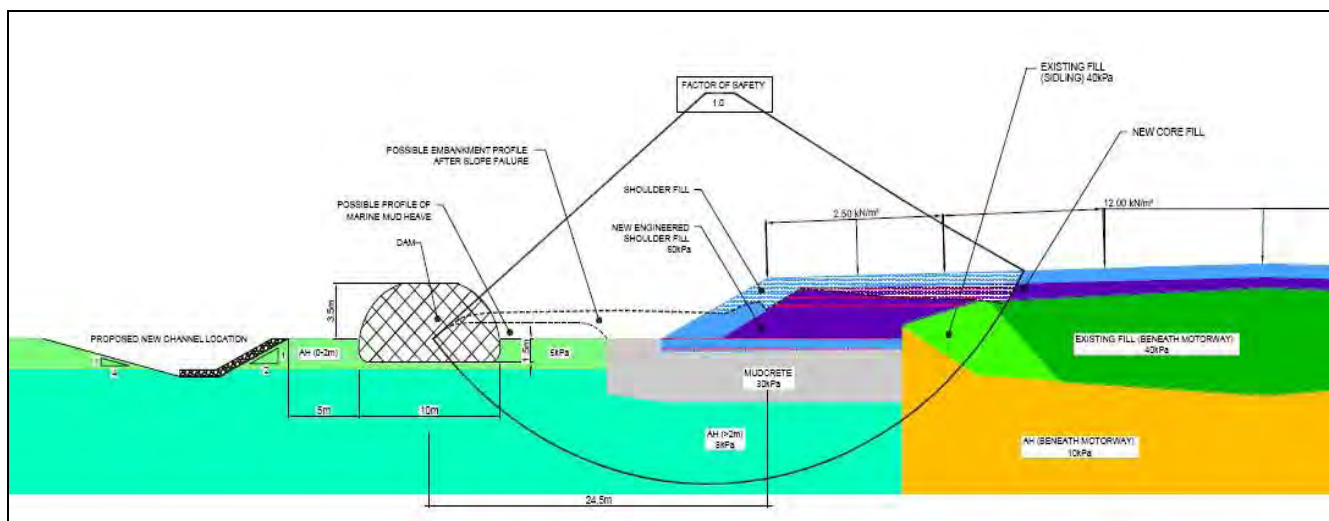


Figure 7-12 - Schematic of possible landslide extent based on circular stability analysis

7.4.6 Assessment of the Likelihood of Embankment failure during Construction

When an embankment is built on top of soft ground, there is a potential for failure of the founding soil. This potential increases with the height of the embankment being raised and thus the stress placed on the ground increases. Ideally an embankment should be constructed slowly to allow sufficient time for the excess pore water pressure which has developed by the placement of the embankment load to dissipate so that the underlying soil can gain shear strength. However, in many cases the time required is unacceptably long and conflicts with the construction program and therefore other measures are adopted to achieve a similar benefit.

For the Western Ring Route Project, Aurecon has designed an increase in the shear strength of the upper part of the founding soil through in-situ soil stabilisation (mixing cement with the existing marine mud). The increased shear strength and increased mass stiffness of the treated zone effectively depresses any potential instability shear planes to where the natural soil has adequate strength.

The design adopts a Factor of Safety (FoS) Approach to stability. Applying a higher FoS has the effect of lowering the probability of failure, however this conservative approach will have significant impacts on the construction costs. The minimum FoS for this design is 1.2 during and immediately after construction, increasing to 1.5 for the long-term operation. This is in line with industry practice.

The likelihood of an embankment failure occurring during the construction phase of the Causeway embankment is calculated based on probabilities which are related to the adopted FoS. This likelihood and number of failures that may occur is significantly reduced due to the proposal to undertake significant ground improvements either side of the Causeway.

For the embankment to fail, the proposed mudcrete would need to collapse through the development of complete shrinkage cracks through the full mudcrete thickness and complete loss of tensile and shear strength. Such total loss of

strength is considered remote given the extent of mudcrete stabilisation. Moreover, a geogrid reinforced earth fill raft is proposed which will be constructed over the mudcrete to reduce the tendency of shear strains to propagate through the embankment and initiate failure. This will provide tensile strength across the width of the mudcrete and resist shearing through the improved zone.

For temporary works (construction phase) a typical FoS of 1.2 is used in accordance with the design philosophy. A FoS of 1.2 equates to a probability of around 1% - 5% that evidence of a failure will occur (eg tension cracks or heave). However, it is estimated that around 75% of these potential failures will be identified (through site monitoring) and therefore local mitigation measures can be implemented before full failure occurs.

The assessment of potential failure extent (reference Section 7.4.5) does not account for such mitigation measures and therefore it would be reasonable to anticipate that the travel distance would be reduced below the 2.5m likely travel distance indicated and therefore it would be contained within the construction footprint.

Construction Phasing will permit the design FoS (1.2) against instability to be achieved in a controlled manner thereby further reducing the probability of edge failure. Analysis indicates that once the embankment is constructed to its full height, the long-term operational FoS will be more than 1.5. This indicates that a probability of around 1% is appropriate. Construction will include a robust and comprehensive monitoring programme that will detect any distressing effects of embankment filling over the stabilised zone like excessive lateral deformations. A contingency plan will be developed before any embankment construction commences to ensure that there are appropriate actions available to be taken in response to any abnormal monitoring data.

In conclusion, slip failure is statistically possible, though a significant failure is unlikely given the level of ground improvements and the application of geogrid within the embankment layers. Furthermore, specific monitoring equipment will be used throughout the construction of the causeway to alert the contractor and designer of any immediate threat of slope failure. Action plans will be put in place to remediate the embankment in the event of a failure. It is also noted, that if failure were to occur, it would be localised within a 20m area, limiting/containing the effects within the construction footprint.

8. Ground Improvement Works

With the results of the geotechnical analysis and an understanding of environmental constraints, it is considered that the use of a suite of ground improvement measures each selected on merit for treating particular areas is the most appropriate design. The depth of ground improvement indicated in the following Sectors may change pending the findings from and interpretation of additional ground investigation, laboratory testing and further design. If the depth of in-situ soil stabilisation is to exceed 5m, deep in-situ stabilisation technique may be considered. The type and location of these options are discussed in more detail in the Ground Improvement Options Report (Aurecon, 2010c) but summarised herein.

8.1 Ground Improvement - Sector 1

The only ground improvement works required in Sector 1 relates to the construction of a Wetland at Jack Colvin Park. In order to construct the Wetland, an embankment approximately 2 to 3m high is required along the northern boundary of the Wetland.

Preliminary geotechnical investigations undertaken in this area indicate approximately 2m of Recent Alluvium (marine sediments) overlying East Coast Bays Formation Rock (ECBF).

In areas with around 2m of Recent Alluvium overlying ECBF, such as this, foundation undercut is considered to be the most appropriate option.

8.2 Ground Improvement - Sector 2

In Sector 2 the highway runs along an embankment on mudflats between chainage 4400 and 4700 and so ground improvement works will be required as the proposed widening works will encroach into the intertidal area. Whau River Bridge is also to be widened, which will require additional spans together with a new cycleway bridge to be constructed on its south side, therefore widening of the abutments will be required.

Sector 2 also includes the section between Patiki Road On-ramp and the Rosebank Domain Raceway where the proposed widening works will encroach into the intertidal area between chainage 3970 and 4100. Hence, ground improvement will also be required here.

At the eastern abutment of Whau River Bridge ground conditions are assumed to comprise around 4 to 6m of Recent Alluvium overlying 1 to 2m of Tauranga Group Alluvium, which are underlain by ECBF. From chainage 4640 (eastwards) to 4400 the Recent Alluvium thins to less than a metre but the Tauranga Group Alluvium thickens to a maximum of about 6m; the ECBF surface steadily rises to the east.

Geotechnical analysis indicates that wherever ECBF rock is present at a relatively shallow depth, ground improvement is only required to prevent embankment instability.

Both engineering aspects and environmental considerations have been considered when selecting the most suitable ground improvement technique. The preferred options are:

- ***In-situ* Soil Stabilisation** – stabilising the in-situ soil by mixing it with cement to form mudcrete will be undertaken between chainage 4400 and 4600 (approx.) on both the north and the southern side of the motorway. Due to the height of the proposed embankment and depth of underlying sediments it is considered that shallow ground improvement to a depth of about 2 to 3m (below ground level) will be sufficient.
- **Lightweight fill** - The widening of Whau River Bridge will require broader abutments that will stress the ground and may cause settlement around the piles causing a drag down effect and/ or increased lateral pressures onto piles. A 90m wide area of lightweight fill has therefore been designed to be placed to form a transition zone between the Causeway embankment (which is likely to settle) and the bridge supports (which will not settle) in order to prevent any potential negative effects and ensure that the differential settlement criterion is achieved. The average existing ground level is 1.6mRL and hence potential buoyancy uplift from seawater during high tides (which have a Mean High Water Spring level currently at 1.63mRL) is negligible. The lightweight fill will be tied securely to resist additional buoyancy forces due to sea level rise in 100 years. Further details will be developed in the detail design stage.

A technically viable alternative solution is the installation of piles below the abutment, the spacing of these would need to increase to provide a transitional zone. This option is not recommended as it would be more costly and time consuming.

- **Marine Deposit Displacement** – This will be undertaken in areas where lightweight fill materials are used (bridge abutments only) to form a foundation to support the lightweight fill (this will be around 1.5m thick).
- **Foundation undercut** – of up to 1.0m thick is recommended between chainage 3970 and 4100 and between approximately chainage 3790 and 3820 of the Rosebank Domain Access. This treatment is appropriate as the embankment height at this location is not very high and the marine alluvium is quite thin (<2m).

8.3 Ground Improvement – Sector 4

Sector 4 extends along the northern side from CH900 to CH4400 and along the southern side from CH900 to CH2975.

The highway improvement works for this sector are quite extensive and include:

- Widening of the Causeway into an intertidal area along both sides;
- Raising of the surface of the Causeway embankment;
- Widening and elevating the abutments around the Causeway Bridges;
- Construction of a new bridge to accommodate the cycleway to the south of the existing Causeway Bridges.

The general ground conditions within Sector 4 comprise around 2 to 12m of Recent Alluvium (marine mud) overlying up to 30 to 40m of Tauranga Group Alluvium, which is in turn underlain by ECBF. The thickness of the marine mud increases from east to west; from the western abutment of the Causeway Bridges across Traherne Island (to chainage 2600) it is 7 to 12m thick but is not present west of about chainage 2800. Similarly, the Tauranga Group Alluvium is much thinner to the east of the Causeway bridges and thickens sharply (>10m) to the west of the bridges. The ECBF rises on the western side of Patiki Road Interchange and its surface is between 10 and 20m below ground level.

As discussed in the previous sections, the marine sediments are very soft and the geotechnical analysis indicates that slope instability, bearing capacity failure and excessive settlement all need to be addressed. The Tauranga Group Alluvium underlying this is highly organic and very soft in areas and analysis demonstrates that the materials are prone to excessive settlement.

Ground improvement will be required to mitigate the above concerns, with several methods (depending on the geological conditions/ proposed improvements) being proposed. These include:

- **In-situ Soil Stabilisation** – Stabilising the in-situ soil by mixing it with cement to form mudcrete is planned to improve the shear strength of the marine sediments adjacent to the existing Causeway between chainage 900 and 2850 (excluding an area around the Causeway Bridges and the eastbound Rosebank Road onramp). The depth requiring treatment varies along the Causeway and is primarily dependent on the thickness of marine alluvium and the elevations of the existing causeway and adjacent mudflats. The varying thickness of settlement susceptible material and varying thickness of fill to be placed (and hence load) causes the treatment depth to vary along the highway and in places the thickness of the improvement zone differs between the northern and southern sides. The preliminary design indicates the following depth of treatment which may change pending the findings from additional investigation and further design:
 - Along the northern side, a treatment depth of 2.5m will be required between chainage 900 and 1250, this increases to 3.5m between chainage 1250 and 2400 and then is reduced to 1.5m between chainage 2400 and 2850.
 - Along the southern side a treatment depth of 2.5m will be required between chainage 900 and 1250, 3.5m between 1250 and 2750 and 2m between chainage 2750 and 2850.
- **Foundation undercut** – of up to 1.5m and 2.5m thick is recommended for the cycleway retaining structures at chainage 3275 to 3340 (approximately) and the cycleway bridge abutment at chainage 3730 to 3760 (approximately), respectively. This treatment is appropriate because the structures only require shallow footing, requiring low to medium ground bearing capacity and the geology at those locations indicates significant amounts of compressible clays, organic clays and peats of the Tauranga Group Alluvium (ATcl, ATo & ATp) are present.

Foundation undercut up to 1m thick is also recommended along the northern shoulder at chainages 2850 to 2950, 3150 to 3225, 3415 to 3475, 3850 to 4020, 4050 to 4400 and the southern shoulder at chainage 2940 to 2975. Foundation undercut is appropriate at these locations because only the surface 1m requires improvement

for the proposed fill heights. As the proposed fill heights at these locations are not high enough to cause any excessive lateral deformation or post construction vertical settlements in excess of 400 mm, settlement is not anticipated to be a concern.

- **Lightweight fill** - The effect of newly placed fill adjacent to the existing Causeway Bridges has been briefly assessed based on a limiting 25mm differential settlement criterion measured over a 10m distance from such structures. There are two potential effects: firstly, there could be an increase in drag-down effect on the existing piles as a result of settlement. Secondly, the existing piles will be subjected to an increase in lateral pressures.

Settlement analysis carried out at these locations indicates that the desired differential settlement criterion cannot be achieved without some form of ground improvement work. Lightweight fill is proposed.

Areas identified as being prospective locations for the beneficial application of lightweight fill are chainages 1100 and 1360 near the Causeway Bridges and the proposed cycleway bridge abutments, chainages 2710 and 2900 for the Rosebank Road Ramps structures.

- A transition zone of 40m to the lightweight fill material embankment has to be provided to avoid excessive differential settlement between the lightweight fill zone and the rest of the embankment earthworks. However due to the high variability of the thickness of the recent alluvium in the Causeway Bridge (formerly known as the Rosebank Bridge) area, a longer transition zone may be necessary pending the findings from additional investigation and further design.
- **Marine Deposit Displacement** – This will be undertaken at areas where lightweight fill materials are used (bridge abutments only), with a thickness of about 1.0m below ground level requiring treatment. Marine Deposit Displacement will also be used to form a temporary working platform (15m by 5m) to carry out the ground improvement works on the mudflats prior to placement of the Geogrid Reinforced Raft. However, the platform will be removed once the ground improvement works are complete.
- **Geogrid Reinforced Raft** – A geogrid reinforced raft will be installed above the Marine Deposit Displacement, along the area of ground improvement works to be undertaken on the mudflats. The role of this raft will be to provide a temporary working platform to allow the ground improvement plant to operate. It will remain in place following completion of the works to improve the bearing capacity and provide a restraint against global shear failure.

8.4 Ground Improvement - Sector 5

The reclamation required in Sector 5 is similar to Sector 4, with the current Causeway embankment being widened on both the northern and southern sides. However, the geological conditions underlying this sector differ from the majority of Sector 4 since ECBF rock is present within 10m of the surface.

Due to the relatively shallow depth to competent rock (ECBF) and the fact that this area doesn't interact with any structures, it is considered that one ground improvement technique will be suitable, as discussed below.

- ***In-situ* Soil Stabilisation** – Cement will be mixed into the marine sediments adjacent to the existing Causeway area to improve their strength by formation of mudcrete. The depth requiring improvement is 2.5m below ground level for the northern and the southern sides.
 - The area along the northern side to be treated is between chainage 710 and 900.
 - The area along the southern side to be treated is between chainage 675 and 900.

9. Embankment Construction Materials

The Fill Options Report (Aurecon, 2010b) assesses the needs for fill materials and evaluates potential fill sources for use on the project, and should be read in conjunction with this report. Specific details of the volumes and types of reclamation fill material types are presented in drawing no. 20.1.11-3-D-C-150-357 (Volume 2).

9.1 Construction Material Requirements

As discussed in the previous sections, the proposed Coastal Works scheme requires a relatively large amount of fill material, in particular within the main reclamation area (Sector 4). Table 9-1 outlines an approximation of the likely amounts and types of fill required within the reclamation area. Please refer to Figure 9-1 for a typical section detailing the distribution of the different fill materials in the construction profile.

9.1.1 Economic Opportunities for Fill Optimisation

Conventional construction would try to match cut and fill volumes to create an earthworks balance within a site, as this is the most economic technique. Import and export of materials offsite is generally expensive as the haulage cost may outweigh the purchase cost of the fill materials themselves.

The opportunity potentially exists to utilise fill from the proposed tunnels and approach cuttings (Sectors 7, 8 and 9 of this Project) since the purchase of offsite materials, the disposal costs of spoil and the haulage cost for all these materials are minimised. These proposed works are close to the Causeway and use of tunnel spoil could add significant economic and environmental benefits to both elements of the Project. To be useable for Causeway widening, the tunnel spoil would need to be stockpiled to manage the differences in timing and productivity rate for the two elements. A suitable location for a stockpile appears to be the Great North Road Interchange, located in Sector 5.

9.2 Proposed Fill Types and Volumes

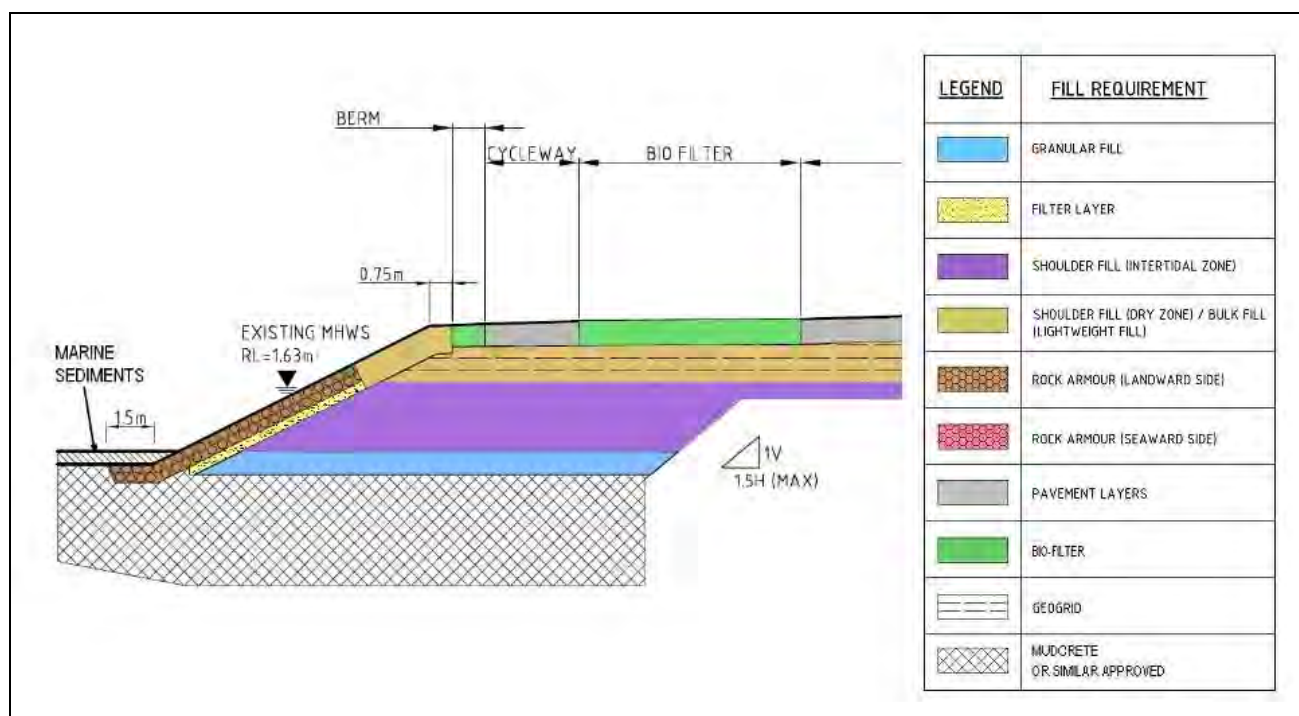


Figure 9-1 - Typical cross section of type and location of proposed fill types (not to scale)

Fill Requirement	Volume (m ³) to be filled	Bulking Factor after Look (2007) and Settlement factor*	Total Volume accounting for compaction and settlement (m ³)
Granular Fill	130,700	1.4	182,980
Shoulder Fill (Intertidal)	73,200	1.4	102,480
Shoulder Fill (Dry Zone)	29,200	1.4	40,880
Filter Layer	13,400	1.1	14,740
Rock Armour	28,100	N/A	28,100
Bio-filter	20,800	1.3	27,040
Pavement Layers	77,100	1.1 then 1.3	110,253
Total	372,500		506,473

Table 9-1 - Fill requirements for Reclamation Works along the Causeway embankment accounting for the compaction and the settlement during the construction period

Note * The settlement factor is a contingency of 30% extra over the compensatory fill volume (Table 6-2, Ground Improvement Options Report) for settlement during the embankment construction period in order to keep the carriageway road elevation to design level before official commissioning of the motorway

As indicated above, a variety of different fills will be required during the reclamation works. For each fill type in the reclamation works, the volume to be filled is estimated from the elevation differences between the existing and the new ground profiles and is presented in Table 9-1. Table 9-1 also takes into consideration the compaction and settlement factors that result in the total volume of the different types of fill materials being greater than the volumes to be filled. Although it is understood that there are a number of different sources of fill material, it is recommended that where possible excess material won from the construction of the Waterview tunnels, which is planned to be running concurrently shall be used. At this stage, the tunnelling programme and precise type and nature of the spoil arising is not known and therefore, several assumptions have had to be made based on professional experience and knowledge of typical Auckland soils. The tunnelling project will generate indicative volumes of material as shown in Table 9-2.

Material Type	Estimated volume
Tauranga Group Alluvium and Residual East Coast Bays Formation	590,000m ³
East Coast Bays Formation Rock (<i>due to its durability and drilling process it is likely that this material will not be classified as a granular fill</i>)	700,000m ³
Parnell Grit	10,000m ³
Vesicular Basalt	70,000m ³

Table 9-2 – Anticipated Excavation Volumes from the SH20 Tunnels and Approaches by material type.

It is very likely that in order for the material arising from the tunnels to be used for the reclamation of SH16, it will need to be excavated in a manner that considers the proposed end use. It is also envisaged that the material will require processing (sorting and grading) to satisfy the required specification.

9.2.1 Shoulder Fill – Dry Zone (Bulk Fill)

This material will be required to raise the shoulders of the proposed Causeway to the design levels for the underside of the filter strips. The type of fill to be used in this area can either be cohesionless (i.e. granular) or cohesive (i.e. clay), or the two may be used in different areas of the embankment. Volumes of additional fill needed to be placed to form compensatory fill will need to be added to the volumes of bulk fill.

The selection of the chosen fill will be governed by engineering placement criteria (grading, moisture content and compaction). As this material is to be placed above sea level its selection has not been based on potential environmental effects (such as erosion of sediment into the Central Waitemata Harbour).

9.2.2 Shoulder Fill – Intertidal Zone

Materials to be placed below the Mean High Water Spring (MHWS) tidal level are referred to as intertidal zone shoulder fill. Unlike the selection of the fill material to be placed above sea level which was based purely on engineering factors, other factors were also considered when selecting this material, in particular the potential environmental effects of the selected material. Consequently, a cohesionless (i.e. granular) material that is not susceptible to erosion in a marine environment was selected.

9.2.3 Granular Fill

Granular fill refers to material required to create a working platform above the ground improvements (refer to Figure 7.3) and also to material that is placed between the existing embankment surface and the pavement layer of the proposed carriageway. It is assumed that only a single material type is placed in this zone and the more stringent requirements have been applied. When the volume of compensatory fill needed for the central section is added, there may be some merit in substituting bulk fill for the lower part of the layer but currently the entire volume is assumed to be granular. As with material discussed in Section 9.2.2 a more stringent selection criterion that satisfies engineering and environmental requirements has been adopted for materials to be placed below sea level. Therefore, a granular material that will not erode under plant operating on the working platform and will not be eroded by coastal action is preferred.

9.2.4 Rock Armour

Rock armour forms the outer layer of coastal protection along the perimeter of the entire reclamation area where it is subjected to wave or current action. Armour will be required on both sides of the Causeway, with a more robust protection required on the seaward side. The chosen rock armour must exhibit requirements set out in the adopted design criteria and, and its durability must satisfy the required design life criterion (year 2100).

Therefore, an unweathered basalt material (similar to what is used along the existing Causeway) is the preferred option. Details on the sizes of this material are discussed in Section 10 of this report.

9.2.5 Filter Layers

Filter layers will be required to protect the shoulder fill from tidal action and subsequent erosion through fines migration, which may cause destabilisation to the embankment. Unlike other fill materials, where construction methods can change to accommodate material with different properties, filter layer material has very little flexibility in design and so the physical properties of a filter layer will also be similar. The precise size and thickness of the filter layer has been calculated and is presented in the Coastal Protection Report (Aurecon, 2010).

9.2.6 Bio-filter

Bio-filters will be installed along both sides of the proposed Causeway to treat surface water runoff. This material does not have any engineering requirements and so the selection has been based purely on stormwater treatment requirements, which is for a material that can biologically support the required vegetation. Typical details are presented in the Stormwater and Streamworks Report (Aurecon, 2010).

9.2.7 Pavement Layers

It is expected that the proposed pavement structure along the Causeway will adopt similar flexible design criteria to those used along the majority of the existing SH16. The general design comprises three layers:

- Sub-base

- Base course
- Surface course

The materials used within the layers must be in accordance with New Zealand Standards. The materials used to construct the pavement will be high quality aggregates imported to site.

9.2.8 Geogrids and Geotextiles

The very soft and compressible nature of the Recent Alluvium (AH) indicates that there is a possibility of local bearing failure if heavy construction plant were to directly track over it. A working platform is required at most locations along the Causeway which will be formed by placing durable hard rock onto the mudflats. Such working platforms may be over 1m thick depending on the local soil conditions and the plant proposed. A Tensar Triax geogrid or similar material will then be placed on top of the rock fill. To minimise the thickness of the working platform, one or two layers of additional reinforcing geogrid may be subsequently placed to complete its construction. The geogrid reinforced raft has a twin role of providing a safe working platform and contributing towards the basal reinforcement and/or global shear restraint to the permanent slope in keeping with the manufacturers guidelines. The Tensar Triax geogrid is not relied on for long-term stability of the embankment.

The mitigation solution considered to satisfy global stability requirements incorporates Tensar 160RE or similar reinforcing geogrid within the structural fill and at the interface between the in-situ soil stabilisation (mudcrete or similar) and the new embankment, as shown in Figure 9-1.

The geogrid reinforcement will provide adequate factor of safety against shear failure of the embankment over the soft ground by preventing lateral spreading of the fill as well as shear failure in the foundation soil by extrusion (squeezing) and overall rotational failure. These failure modes under the short term conditions are most critical because of the relatively low permeability of the foundation soils which do not permit complete settlement in the normal time scale of construction. At the end of construction when the embankment loading is fully applied, there is increase in foundation shear resistance due to on-going consolidation but that may not be adequate to ensure stability. Hence shear stresses are transmitted from the foundation soil and the fill which generates tension in the geogrid and restores stability in the embankment. Once settlement is complete, there will be improvement in the foundation shear resistance. This means that the embankment will rely less on the geogrid reinforcement for stability. Analysis has also been carried out to assess whether the shear strains that will develop within the geogrids as a result of differential settlement, will exceed the capacity of the geogrid. A finite element analysis (Plaxis Version 8.6) along critical section chainage Ch 1600 was used, and considered no gain in shearing resistance of the soft marine mud during the staged construction. The analysis indicates that a maximum strain of up to 2% is developed in the basal geogrid reinforcements by the end of the construction. This is well within the ultimate strain of about 11.5% in accordance with Tensar product specification (Certificate No. RF5/03, Tensar International Limited).

Construction of the embankment fill slope on top of the Mudcrete, MDD or Foundation undercut ground improvement work, will require a geotextile fabric or separator (BIDIM A39 or similar) to be provided between the foundation material

and the new embankment. This will act as a sediment control to prevent the migration of fines from the structural fill into either the foundation or the sea.

10. Coastal Protection

10.1 Coastal Protection Design Philosophy

Coastal protection is needed to prevent embankments and associated elements that support the highway from being eroded by tidal flows, stream flows or wave action. In combination with the selection of design level for the embankment, the coastal protection measures serve to protect the highway from wave overtopping. Some 6 km of coastline requires coastal protection measures, of which 2.8 km lies on the seaward (northern) side of the motorway embankment (Causeway). The coastal protection design is developed and presented in the Coastal Protection Report (Aurecon, 2010), which:

- Interprets and analyses hydrodynamic conditions and sets design parameters
- Assesses possible coastal protection options
- Designs preferred coastal protection measures
- Accounts for environmental and constructability issues

In developing the design, three key factors were taken into account:

- **Hydrodynamic Conditions** – protection needs to be sufficient for both the existing hydrodynamic conditions and those predicted to occur during the design life of the facility.
- **Environmental Impact** – the protection measures adopted should have a minimal impact on the environmental landforms and their aesthetics particularly of the Coastal Marine Area (CMA).
- **Sustainability** – the design solution should be environmentally sustainable where possible.

Besides including natural sources of protecting material, re-using existing materials and considering environmentally friendly design options in order to address the key factors of environmental impact and sustainability, the coastal protection design philosophy adopts the following principles:

- **Probabilistic design principles:**
Probabilistic design principles are commonly used for coastal design works (e.g. assessing the joint probability of extreme wave conditions combined with high water levels), to optimise the design process. The design of the coastal protection works does allow for some degree of damage to occur, which will need to be managed through an appropriate maintenance programme. Designing for no damage would substantially increase construction costs and, as a result, increase whole life time costs of the project.
- **Preference for soft engineering solutions:**
Soft engineering solutions (i.e. design options that are able to adjust to changes in the environment such as

settlements) are preferred within a coastal environment. Coastal environments are typically dynamic environments with changes in conditions over time due to coastal sedimentation and erosion processes. It is important that the design is able to adapt to such changes.

10.2 Hydrodynamic Assessment

The key input for the coastal protection design is the assessment of the hydrodynamic conditions. This was undertaken for this project by NIWA (2009) and generated the following key outputs:

- High-tide levels
- Storm frequency and the effects of storm surge on sea level
- Tidal currents and stream flows
- Wind speeds
- Design wave heights
- Climate change effects and how these affect all of the above

Under the adopted projections for climate change, peak wind velocities are likely to increase and sea level will continue to rise. Rising sea levels will increase water depths which, in combination with increased wind speeds, will lead to greater wave heights. The coastal protection measures have been designed assuming a 0.8m increase in sea level and a 20% increase in wind speed due to climate change out to 2100. These factors have been input into the assessment of the joint-occurrence probabilities of wave heights and water levels. The significant wave height (average of highest 33% of waves) for exposed sections of the Causeway shoreline has been determined as 0.86m. This wave height in combination with a water level of 1.96m RL by 2100 (and in approximately 2m water depth) would occur jointly with an Average Recurrence Interval (ARI) of 100 years by 2100. As the extreme water levels increase above 1.96m RL, the significant wave heights would decrease to have the same joint ARI of 100 years, or alternatively the joint ARI increases substantially above 100 years for a fixed significant wave height of 0.86m (NIWA, 2009).

10.3 Wave Exposure Zones

The maximum wave height applies to the section of the shoreline which faces the harbour where water depths can attain the depths required to permit the wave to strike the embankment revetment. For Waterview Estuary on the landward (south) side of the Causeway, the wind fetch across the Estuary is much shorter and consequently lower wave heights are anticipated. As the embankment traverses from the intertidal mudflats onto Traherne Island and around the Rosebank Peninsula, surface elevations are much higher and lower wave heights also apply. Taking these factors into account the coastline has been subdivided into four major zones for defining the coastal protection measures. These zones are described in Table 10-1.

Wave Exposure Zone	Description	Criteria	Design Wave Height (Hs)
Zone A	Fully exposed	<ul style="list-style-type: none"> Northern Side of Causeway, and Revetment toe level below 1.9 mRL 	0.86 m
Zone B	Fully exposed with shallow foreshore	<ul style="list-style-type: none"> Northern Side of Causeway, and Revetment toe level above 1.9 mRL 	0.50 m
Zone C	Sheltered	<ul style="list-style-type: none"> Southern side of Causeway, or Northern Side of Causeway behind Pollen Island 	0.50 m *
Zone D	Land based	<ul style="list-style-type: none"> In fully exposed areas toe level above 3.5 mRL In sheltered areas toe level above 3.0 mRL 	N.A.
<p>Note: It is noted that the significant wave height in Zone C is not expected to exceed 0.3m. However a conservative 0.5m design wave height has been adopted in order to reduce the number of design details.</p>			

Table 10-1 - Wave Exposure Zones and Design Wave Height

10.4 Overtopping

For the performance of the embankment it is necessary to protect it from overtopping. Using the CIRIA/CUR (1991) guidelines, a maximum mean overtopping discharge of 0.02 l/m/s has been selected for the highway; discharge rates of higher than this are deemed to be unsafe for vehicles at any speed. Besides the water depth and wave height, the other parameters that affect the overtopping rate are:

- **Slope angle** of the revetment – waves can run up slopes at shallow angles and when waves break against steep slopes water splashes over the top; analyses have been undertaken with a slope of 1V:2H.
- **Roughness** of the slope surface – this depends on the nature of the revetment facing; a roughness factor of 0.55 is assumed.
- **Crest width** – water that has overtopped the crest may be dissipated in the zone where the revetment continues beyond the crest on the top surface; a crest width of 3m has typically been modelled on the seaward side.

10.5 Selected Coastal Protection Measures

A number of different types of coastal protections measures have been considered and the preferred option is to provide a steep sloped, two layer, rock revetment (1V:2H), which is consistent with the current design of the existing coastal protection.

10.6 Revetment Design

The revetment design incorporates three components:

- **Primary armour structure**
- **Filter layers**
- **Toe Protection**

The primary armour structure will comprise two layers of rock boulders. These are separated from the shoulder fill by the filter layer. The primary armour structure initially dissipates the wave energy but the filter layer is a crucial element of the revetment as it further dissipates energy, permits water to be absorbed and then drained away and prevents water from penetrating the body of the embankment.

Two sizes of primary rock armour will be used with boulders of a nominal diameter of 0.45m (D_{50}) to be placed in areas that will be impacted by significant wave height (such as the seaward side of the Causeway) and 0.3m diameter boulders to be placed in areas not affected by the significant wave height.

Given the two boulder sizes, the coastal subdivision and the variation in crest level for the proposed embankment, five coastal protection details (Refer to drawing 20.1.11-3-D-C-510-110) have been developed:

- **Detail 1:** Uses 450mm diameter boulders and applies to the most exposed sections.
- **Detail 2:** Is to be used where the coastal protection is close to structures, where the embankment crest lies above +3.8m RL. Therefore, the rock armour does not need to continue into the crest zone.
- **Detail 3:** Uses 300mm diameter boulders and applies to areas with lower wave heights.
- **Detail 4:** Is to be used in lower energy environments (i.e. inshore), therefore 300mm diameter boulders are to be used, without any crest protection (if the embankment crest lies below +3.2m RL).
- **Detail 5:** This is a combined revetment / retaining wall option and applies to just one particular area.

The distribution of the five different coastal protection details is shown in drawings 20.1.11-3-D-C-510-101 to 108.

11. Bridge Improvement/ Construction

11.1 Whau River Bridge Coastal Works (Sector 2)

New permanent occupation of the CMA is required in Sector 2, which includes the proposed bridge piles and abutments, which are to be located in the Whau River.

In order to reduce differential settlements between the existing bridge structures and widened bridge structures, appropriate measures are needed to support the new abutments with transition zones extending back into the approach embankments. Refer to Section 8.2 for a discussion on ground improvement here. Bridge piers constructed near or in the water are subject to aggressive corrosion, erosion and scour environments; therefore it is recommended that the piers are circular in section to reduce the scouring action.

The recommended foundation solution for the bridge piers comprises *in-situ* concrete cast piles inside a sacrificial metal casing. The metal casing can be installed by bottom driving to the pre-determined founding soil stratum. The thickness of this casing has been determined by assessing corrosion rates throughout the design life of the structure.

Key Issues to be considered are:

- Construction Phasing including traffic management
- Temporary Staging platforms
- Relocation of services
- Construction over the waterway

11.1.1 Whau River Bridge Construction Phasing

The typical staging for the bridge construction is shown on drawing 20.1.11-3-D-C-150-324 and summarised as follows:

- **Phase 1** - widen the eastbound motorway bridge and construct the Cycleway Bridge on the westbound side of the carriageway from temporary staging platforms. Details of the proposed staging platforms are discussed below
- **Phase 2** - will be the stitch joint between the existing eastbound bridge and the widened section of the bridge.
- **Phase 3** - is to construct the westbound widening from temporary staging platforms.
- **Phase 4** - will be to complete strengthening of the existing structures and improvements to the far side barriers on each of the existing bridges.
- **Phase 5** - will involve the final surfacing and lane marking to the bridges and approaches.

The existing number of traffic lanes will be maintained throughout construction.

11.1.2 Whau River Bridge Temporary Staging platforms

The construction of all bridges will be completed from temporary platforms constructed adjacent to the bridges to be widened. This allows each bridge to be constructed without the need for construction plant to be placed on the existing bridge decks. The use of temporary platforms ensures that the existing structures are not overloaded and that the existing motorway traffic lanes can be maintained during construction.

Allowance for a 7m wide temporary platform has been provided with 0.5m clearance to the permanent bridges for formwork to be placed. The temporary platform will be supported by (temporary) driven piles, driven into the underlying material until sufficient bearing capacity is achieved. The temporary piles are typically arranged in pairs at 9m centres longitudinally and 5 to 6m centres laterally. The temporary piles will support a steel superstructure with a thick timber decking to form the surface of the temporary platform.

The temporary deck and supporting beams will be set at a level high enough to not reduce the vertical clearance (freeboard) to the water. The westbound staging platform will be located between the motorway extension and the proposed Cycleway Bridge to enable construction of both bridges from the same platform.

Following construction, the platform will be removed using cranes positioned on the platform. The piles will be removed using vibration equipment and cranes.

Whau River is currently navigable between the 25m spans. The temporary platforms will be designed to allow construction access out from each river bank and one span will be left clear for continued river navigation. This span may need to be changed during construction, by reducing the extent of one platform followed by increasing the length of the opposite platform. This will increase the complexity, time and cost of the construction but navigation of the Whau River, however, will be possible throughout construction.

11.2 Causeway Bridges Coastal Works (Sector 4)

The new permanent CMA occupation in Sector 4 consists of the proposed bridge piles and abutments for the Causeway Bridges. These are to be located in the drainage channel through the existing Causeway (at the location of the bridges).

The abutment fill, bridge piers and key issues related to this bridge are the same as discussed for the Whau River Bridge in Section 11.1.

11.2.1 Causeway Bridge Construction Phasing

Construction of the widening of the Causeway Bridges and construction of Sector 4 earthworks must be phased to coincide with each other. The designation has been defined to allow a construction method which minimises the disruption

to the traffic during the construction period. The existing number of traffic lanes is proposed to be maintained throughout construction.

The bridge construction sequence will follow a similar methodology to the construction sequence for the Whau River Bridge as discussed in Section 11.1.1.

11.2.2 Causeway Bridges Temporary Staging Platforms

Widening of the Causeway Bridges and construction of the separate Cycleway Bridge is assumed to be completed from temporary platforms constructed adjacent to the widening in a similar manner to the proposed temporary staging platforms at the Whau River. The inlet channel is not navigable and therefore no provision for navigation during construction will be provided.

The temporary deck and supporting beams will be set at a level high enough to not reduce the vertical clearance (freeboard) to the water. The westbound staging platform will be located between the motorway extension and the proposed Cycleway Bridge to enable construction of both bridges from the same platform.

Refer to drawings 20.1.11-3-D-C-150-313 for typical construction staging.

11.3 SH20 Ramp Viaducts Coastal Works (Sector 5)

For details of the proposed viaducts for the Great North Road Interchange refer to drawings 201113-D-S-917-430 and 431. The new permanent occupation in Sector 5 consists of proposed viaduct piles / piers for the new Northbound Motorway to Motorway link from SH20 to Westbound SH16 (Ramp 2), and the new Southbound Motorway to Motorway link from SH16 to SH20 (Ramp 3). Viaduct piles constructed near to the sea are subjected to an aggressive environment under corrosion and erosion from scour actions. The shape of the piers is recommended to be circular to reduce scouring action. Viaduct piles will be concreted inside a sacrificial metal casing that is driven down to the required founding level. The casing extends up to the top of the normal tidal range. Above this point the circular pier is constructed with temporary formwork that is removed after concreting, exposing the finished concrete surface.

Key Issues to be considered are:

- Construction Phasing
- Temporary Staging platforms
- Construction over the waterway

11.3.1 SH20 Ramp Viaducts Construction Phasing

The first stage of the works involves the installation of the temporary piling in the CMA. It is envisaged that the Contractor will progressively install all the required temporary piles in one continuous sequence to avoid additional mobilisation and

demobilisation costs. This would include the piles for the temporary platforms required for crane access during the installation of the Super-T deck units.

The temporary access platforms supported by the piles will then be installed in locations to suit the Contractor's construction sequence. With a project of this size, the Contractor is likely to have more than one piling rig and construction team so piling and pier construction works will most probably be undertaken at multiple locations at any one time.

Platforms around the piers are required for the construction of the piers and the pier crosshead beams, and are also required for access during the placement of the Super-T deck units. Once this work is complete the temporary platforms and their support piles can be removed.

The estimated construction period for the Great North Road Interchange is approximately 2 years so the temporary platforms could be in place for up to 18 months in some locations, depending on the Contractor's construction sequence.

11.3.2 SH20 Ramp Viaduct Temporary Staging Platforms

The construction of the viaducts in the CMA is assumed to be completed from temporary platforms built adjacent to the viaduct alignment. It is envisaged that temporary platforms will be built from both the northern and southern sides of the CMA allowing the central river channel to remain undisturbed. Refer to drawings 20113-D-S-610-500 and 501 for proposed locations of the temporary platforms.

The temporary platforms provide the required access for pile boring equipment to allow for 360 degree construction access around the pile / pier and also provide a working platform to protect from spoil dropping from the auger into the CMA. Temporary platforms are also required at specific locations to provide a working platform for the crane required to lift into place the precast Super-T deck beams.

All temporary platforms will be supported by temporary driven piles that are driven into the underlying material until a sufficient bearing capacity is achieved. The temporary piles will support a steel superstructure with a thick timber decking to form the surface of the temporary platform. The temporary piles for the temporary platforms are shown on drawings 20113-D-S-610-500 and 501.

Following completion of the construction works the temporary platforms will be dismantled in the reverse order to how they were constructed with the temporary piles removed completely from the CMA, using cranes and vibration equipment.

12. Channel Relocation Works

12.1 Location of Channels

The widening of the Causeway embankment into intertidal areas will generally only encroach onto the mudflats, however there are three locations where the widening works will extend into tidal inlets or marine drainage channels. It has been determined that these channels require relocation in order to allow the Causeway improvement works to be completed.

The locations of the existing channels affected are:

- Waterview Estuary Channel – Chainage 1550 to 1710 (south);
- Oakley Inlet Channel No.1 (lower) – Chainage 810 to 870 (south);
- Oakley Inlet Channel No.2 (upper) – Chainage 660 to 700 (south).

A discussion of the proposed excavation/construction methodology is presented below and shown on Drawings 20.1.11-3-D-C-150-227 to 229.

12.2 Channel Realignment – Waterview Estuary

To provide a new tidal drainage channel where the existing Waterview Estuary is affected by the improvement works the following channel relocation methodology is proposed. These works will be completed before the main Causeway widening works commence.

i. Phase 1 and 2 – Preparation Works

- a. A barge (approximately 10m wide by 20m long) will be floated under the existing Causeway Bridges to the landward side of the Causeway and temporarily moored against the existing coastal protection. A long reach excavator will then be tracked onto the barge from the existing Causeway;
- b. A sediment pump will also be loaded onto the barge.

It is estimated that this preparation work would take approximately 1 week to complete.

ii. Phase 3 and 4 – Excavation of new Channel

- a. At high tide the barge will be floated onto the area between the proposed and existing channels;
- b. Excavation of the proposed channel will be undertaken at low tides. During high tides the barge will move progressively along the proposed channel, with excavation taking place at low tides. Excavation will proceed from the middle progressing towards the ends. To prevent significant sediment release during excavation a bund of marine mud will be left in place at either end of the proposed channel;

- c. Excavated marine material will be placed in the inlet of a pump located on the barge and pumped via a temporary surface pipe.
- d. The pumped material (in a residual strength state) will be mixed with cement in order to achieve approximately 15kPa in undrained shear strength (easier to store and to carry). The mixing will be done by a small pug mill located on the existing Causeway.
- e. The mixed material will then be conveyed by trucks to a suitable temporary storage location (to be determined by the Contractor).
- f. Scour protection will be placed progressively along the northern slope of the newly excavated channel at low tide and at the end of every work shift. The proposed channel will not remain unprotected for more than 1 tidal cycle.
- g. The final part of the excavation works will require excavation of the two bunds at either end of the proposed channel to connect into the existing channel. This will be done over two progressive low tide cycles. These connection works will be undertaken at favourable tidal conditions (i.e. the connection excavation at the downstream end will be undertaken as the tide is falling before low tide so the new channel breakthrough can be achieved in a more controllable manner and with less bed scour).

It is estimated that the excavation works will take approximately 2 to 4 weeks to complete.

iii. Phase 5 and 6 – Infilling of Existing Channel

- a. Prior to infilling the existing channel, bunds of durable rock material (AP 150–300 basalt, greywacke or similar) will be placed at both ends to prevent significant flow of seawater through the existing channel. Scour protection can then be placed onto the northern slope of the rock bunds.
- b. Once the new channel is functional, infilling of the existing channel behind the rock bunds can commence. During low tides, utilising the barge and long reach excavator, the existing channel will be infilled with the stored 15kPa material. After infilling is complete the top layer of the 15 kPa material will be levelled to provided a suitable area for placement of the temporary water filled coffer dam.
- c. Following installation of the temporary water filled cofferdam the *in situ* soil stabilisation (as part of the Causeway widening works) can then continue as normal through the existing channel location.

It is estimated that the infilling works will take approximately 2 to 4 weeks to complete.

12.3 Channel Realignment – Oakley Inlet

The same methodology used for the Waterview Estuary Channel will be adopted for the excavation of the two channels at Oakley Inlet.

The excavation works of these areas are smaller in size than the Waterview Estuary work and it is estimated that the works at each channel will take approximately 1 to 3 weeks complete.

12.4 Environmental Considerations

A number of considerations have been adopted when designing the channel excavation works in order to minimise potential environmental impacts. These include:

- **Contamination Testing** – Due to historical industrial activity that has taken place within areas adjacent to the Waterview Estuary and Oakley Inlet, it is considered possible that elevated concentrations of particular contaminants may be present. Therefore, a detailed investigation has been undertaken prior to the commencement of the excavation works to determine contaminant concentrations and assess whether or not they pose a significant risk to the surrounding environment.

If a significant risk is considered plausible then the excavated material shall be either stabilised or removed from site and 'clean fill' imported. The results of this contamination testing have been presented in a separate report.

- **Site Management Plan** – A site management plan will be prepared prior to undertaking the works. This management plan will set out working times (i.e. tidal conditions) and appropriate mitigation measures if unexpected ground conditions are encountered.
- **Site Monitoring** – Monitoring of sediment volumes and water quality will be undertaken throughout the works. If any significant increase in sediment or decrease in water quality is recorded, works will be stopped and appropriate mitigation measures implemented.

13. Construction Methodology

13.1 Preparation Works

Before constructing the main Causeway embankments (in Sectors 2, 4 and 5), preparation works will be completed to facilitate the undertaking of the main construction works. These will entail the formation of a dry working area by the installation of a dam and the formation of working platforms for the operation of plant to carry out ground improvement works, as described below.

13.1.1 Installation of dam around working area

It is envisaged that the construction works for the main Causeway embankment will be undertaken within three individual work phases that will proceed along the embankment (further discussed in section 14.4).

Prior to starting any intrusive works where *in situ* soil stabilisation is proposed, a dam must be constructed to form a dry working area (an alternative technique is discussed below). A suitable form of dam is made from inflatable fabric which is filled with water. It is envisaged that these portable water filled dams will be installed to define each individual work area. It is anticipated that each work area will be around 100m long, depending on the Contractor's preference. A suitable approach is to install linear segments of dam parallel to the Causeway with shorter lengths connecting via a tee-connection, forming the sides between the seaward dam and existing Causeway. The existing coastal protection will have to be removed in areas where the dam joins the existing Causeway to form an impermeable termination. Even once the dam is installed, it is expected that some water will enter the work area, primarily from seepage through the existing embankment; this water will be diverted into sumps and removed from the work area via a suction truck. If the water is sediment or contaminant laden, it will be treated and disposed of.

The dam will be filled using seawater pumped from various channels within the harbour (the location of this may vary depending on the location of the dam and tidal stage (i.e. high tide or low tide). Once the works in a particular area are complete, the dam will be emptied, with the water either being pumped into an adjacent dam or out into a drainage channel (water will not be allowed to flow directly over the mudflats). It is envisaged that the dam segments will be leapfrogged past one another to create the next 100m long work area.

A portable water filled dam is not the most suitable method at several locations due to space constrictions, vegetation cover and topographic undulations. Therefore, it is proposed that a sheet pile wall is used as a dam in the following locations:

- Chainage 650 to 1200
- Chainage 4400 to 4650

The sheet pile wall will typically extend about 10m into the underlying sediments and will be installed using an excavator operating from the existing Causeway embankment.

This dam will be required for the following reasons:

- a. **Protection of the existing Causeway embankment** - The existing coastal protection present along the Causeway will need to be removed before any ground improvement / embankment construction works can be undertaken. Therefore, there is a potential risk that if no mitigation measures are employed then there is the potential for inundation and consequent erosion of the existing embankment. The dam will act as temporary coastal protection and so prevent this from occurring.
- b. **Increase in intertidal working time** – Some of the proposed Coastal Works activities in the intertidal zone (such as *in-situ* soil stabilisation) can only be undertaken when there is minimal surface water present. Therefore, if no mitigation measures are implemented the works can only be undertaken during a short period of time either side of low tide.

The water filled dam is designed to control the amount of water inundation and minimise the amount of surface water present. A dam in the order of 2 to 5m high is proposed in order to prevent overtopping from a 1 in 5 year storm event.

- c. **Sediment Control** – As discussed above, the dam will be designed to not only prevent tidal water inundating the working area but it will also prevent significant flushing of the works by ebbing tides and thus minimise the erosion and redistribution of sediment into the harbour.

This has the advantage that the surrounding environment should not be significantly affected by water emerging from the works either being sediment laden or cement contaminated (with cement arising from the soil stabilisation activity) as it can either be allowed to stand to encourage suspended solids to settle out of solution or pumped into storage containers and removed from site.

13.1.2 Working platforms

Working platforms will need to be installed on top of the marine sediments in order to allow the ground improvement works to be undertaken. Details of the proposed working platform are discussed in Section 9.2.8.

The working platform will have a twin role of providing a safe working platform (able to support loads of up to 100kPa along the tracks and 12kPa between the tracks of plant) and provide basal reinforcement to improve global stability of the new embankment.

13.2 Construction of the Ground Improvement Works

The construction methodology for each of the ground improvement measures to be employed are discussed in detail in the Ground Improvement Options Report (Aurecon, 2010c) but summarised briefly below:

- **Shallow *In-situ* Soil Stabilisation**
 - The *in-situ* mudcrete ground improvement of the marine mud by its stabilisation in-situ to a shallow depth to form mudcrete will be undertaken using an adapted hydraulic excavator with a specially designed rotary mixing head.
 - Works will be undertaken from a temporary working platform sited adjacent to the toe of the existing Causeway embankment.
 - Dry cement will be injected directly into the marine sediments through the base of the mixing head and mixed.
 - The cement will be stored in a pressurised tanker which will be located nearby, either on the existing Causeway or temporary working platform.
 - The mixing head will be moved up and down to thoroughly mix the materials through the full depth of treatment required. This is generally 3.5m but may vary depending on the geology, topography and proposed geometry.
 - The mixing head will be swung to an adjacent area to continue the operation to eventually form a raft of treated soil.
 - A depth of up to 5m below the surface level of the marine sediments will be treated by this method (this varies depending on the precise location). Greater depths require different plant and the technique will alter somewhat.
 - The previous steps are completed until the entire design footprint has been treated.

- **Foundation undercut**
 - Foundation undercut will require between 1 to 2m of soft material (such as marine sediments or organic layers) to be removed using a conventional excavator.
 - Excavated material is likely to require offsite disposal.
 - Precautions need to be taken when undertaking the undercut with a maximum safe slope batter of 1V:1.5H required.
 - The excavation is to be backfilled using compacted AP500 durable rock, or other approved competent engineered fill for use in Sector 1.
 - The surface of the backfilled material is to be capped using a geotextile fabric separation layer prior to constructing the embankment.

- **Marine Deposit Displacement (MDD)**
 - Marine Deposit Displacement will only be undertaken in areas where lightweight fill is to be placed.
 - A geotextile fabric is to initially be placed onto the treatment zone, durable rock is then placed on top of the geotextile to displace the underlying soft marine deposits.

- Light weight tamping of the durable rock may be required to achieve the required treatment zone (1 to 2m below ground level).
- A geotextile will be placed on top of the treated area prior to the placement of the embankment fill.

13.3 Embankment Shoulder Construction

Details of the type of fill materials required to construct the Causeway embankment are discussed in Section 9 and the construction staging is discussed in Section 7.1.1. These elements are therefore not discussed further here.

14. Reclamation Construction Programme

14.1 Key Programme Assumptions

Co-ordination of all the design activity that has led to production of this report is based upon an overall construction programme of 5 years. Shorter construction durations would invalidate some of the analytical assumptions necessitating re-analysis and potentially revision of some of the methodology. Much affected by the construction duration is the settlement analysis, which differentiates between the amount of settlement that occurs in the construction period and that which takes place during the operational phase. Shorter construction periods will have greater operational settlements, leading to the need for other mitigation but, the shorter the period, the less room there is for certain options to be engineered.

Certain construction activities, in particular the soil stabilisation ground improvement, will be time consuming activities that lie on the critical path programme. A 5-year programme was found to be an appropriate balance for the complex range of activities that need to be undertaken. Was this overall programme to be significantly shortened a critical reassessment of the construction methodology will be needed.

14.2 Preliminary Works

14.2.1 Diversionary Works

The channel relocation works (discussed in Section 12) need to be excavated as enabling works. As construction of the landward side of Sector 4 is not envisaged to start until the end of year 2 (of a 5 year construction programme), the excavation of these bypass channels can be undertaken any time within the first two years of construction.

14.2.2 Trial Embankment

It is proposed that a trial embankment should be constructed in advance of the main works. The purpose is discussed in the Stage 1 and 2 Geotechnical Interpretative Report but in summary the construction and monitoring of a small embankment trials some of the methodology proposed for the main construction and therefore this works needs to be undertaken prior to the first construction season of the mains works.

14.3 Programme Drivers (Traffic Management)

The requirement to maintain existing motorway traffic flows throughout the period of the construction works is one of the primary drivers that has defined the phasing of the works. Traffic needs to be relocated across the width of the embankment in phases in order to allow temporary working areas to be formed.

The widened Causeway construction phasing, from a traffic management and construction zone aspect, is summarised in the drawings sheet listed in Table 14-1.

Drawing Title	Drawing Numbers	No. of Sheets
Causeway Construction Phase Typical Sections	201113-D C-150-301 to 303	3
Indicative staging of reclamation and embankment works	201113-D C-150-371 to 375	5

Table 14-1 – Construction and traffic management drawing sheets

These drawings provide a summary of the six main phases of work planned in Sector 4 with further explanation given below.

The initial widening will begin on the eastbound carriageway with three work faces in operation in order to meet the construction programme. Construction will switch to the westbound carriageway only once the entire eastbound widening (Great North Road to Whau River Bridge) is nearing completion. Multiple construction zones will occur along the 5.7km length of the works. The raising of the Causeway and more complex sections of motorway are detailed in the six key phases summarised as follows:

- **Phase 1** concentrates on defining the new revetment and toe extent (eastbound), and providing the necessary working platform to begin ground improvements.
- **Phase 2** will be the placement of bulk fill to achieve the new carriageway design level.
- **Phase 3** concentrates on defining the new revetment and toe extent (westbound), and providing the necessary working platform to begin ground improvements.
- **Phase 4** will see all westbound traffic using the new westbound Causeway profile, whilst the central Causeway is increased in height.

- **Phase 5** will commence when the main engineering components associated with raising the Causeway are complete.
- **Phase 6** will include several night time activities focused on finalising the road markings and completing all signage requirements.

14.4 Programme Timeframe

The critical item for the SH16 programme is the ground improvement and reclamation required along the Causeway (Sector 4), since limited other work can commence until these are done and the durations of these activities are long. To minimise the effect on the motorway operations, initial programming for the works is to construct the eastbound carriageway in the first 2 years (Phases 1 and 2) and the westbound carriageway in years 3 and 4 (Phase 3). The remaining construction (Phases 4, 5 and 6) will take place in year 5. Cycleway bridges have been programmed to commence at the same time as the eastbound works so cyclists are not impacted when the existing cycleway is demolished for the westbound carriageway works.

14.4.1 Sector 1 – Wetland Coastal Works

The wetland and associated reclamation is planned to be constructed during the first 18 months of the construction programme.

14.4.2 Sector 2 – Whau River Bridge Coastal Works

The coastal works in Sector 2 is planned to be undertaken as follows:

- Separate Cycleway Bridge – first 10 months
- Eastbound Widening – first 18 months
- Westbound Widening – 21 months (i.e. Months 19 to 39)

14.4.3 Sector 4 – The Causeway Coastal Works

The coastal works in Sector 4 is planned to be undertaken in the following timeframes:

- Reclamation and coastal occupation - eastbound - 18 months (3 work faces)
- Pavement construction – eastbound - 7 months (3 work faces)
- Reclamation and coastal occupation - westbound - 16 months (3 work faces)
- Pavement construction - westbound - 4 months (3 work faces)

Reclamation of three main work faces is required in order to complete (within 5 years) the Sector 4 ground improvements, embankment construction, coastal protection and other coastal works. Each work face will be undertaken concurrently in the sections of motorway subdivided into set work areas.

14.4.4 Causeway Bridges Coastal Works

Phase 1 to 4 of the Causeway Bridges construction will be implemented during Phase 1 to 4 of the main carriageway construction phasing. Phase 5 of the Causeway Bridges construction will be implemented during both Phases 5 and 6 of the main carriageway construction phasing.

The coastal works will be undertaken as follows:

- Eastbound Widening – first 12 months
- Separate Cycleway Bridge – first 12 months
- Westbound Widening - year 3 (12 months)

14.5 Construction Staging

As discussed in the ground improvement methodology and shown on the drawings, construction works will be phased so that the seaward side is initially completed, followed by the landward side and then finally the centre. This section details the main individual stages of construction work that have to be undertaken in a logical progression to complete the embankment construction. As shown on the drawings, there are 21 individual stages of activity, multiple stages of which can be undertaken during each of the major construction phases dictated by the traffic management plans (ref.14.3). These stages are discussed below.

14.5.1 Ground Improvement – Seaward Side (Phases 1 and 2)

- **Stage 1 – Forming and Accessing the Work Area**

When it is required to form a working area from which the sea is kept out, a dam (either water filled or sheet pile) will first be installed prior to starting any works.

The basalt coastal protection will be removed following installation of the dam (if required) and the slope graded to a 1V:1.5H angle in order to form an access from the existing embankment down to the mudflats. No more than 15m (longitudinally) will be excavated at any one time.

- **Stage 2 – Entrance Platform**

Following the completion of Stage 1, an entrance platform will be created to allow the ground improvement plant to access the work area. The platform will be created by initially placing a layer of geotextile and geogrid onto the marine sediments. The geotextile will act as a separator between the marine mud and working platform, while the geogrid will act as reinforcing layer.

A layer of coarse gravel (AP300) will then be placed onto the geotextile and geogrid and compacted into the marine sediments until a platform with an adequate strength is created (due to the very soft properties of the marine sediments it is possible that this layer will be up to 1m in places). This platform is only required as an

access point for the ground improvement plant equipment and so will be around 5m wide and 15m long. This process will only be required to be completed once per work face.

- **Stage 3 – Preparation of the Ground Improvement Area**

Following completion of the entrance platform, a 0.6m layer of medium coarse gravel (GAP 65) will be placed incorporating two layers of geogrid reinforcement. Finally, to complete the platform and provide protection to the geogrid reinforcement, a 150mm layer of GAP 65 will be placed. This platform will be permanent and will protect the ground improved area, together with providing sufficient subgrade reaction to allow the plant equipment to operate.

- **Stage 4 – Main Ground Improvement**

Following completion of Stage 3, the ground improvement works will begin. Although there are a number of different ground improvement works that may be utilised the installation/ completion process will be similar.

For the bulk of the area where the marine sediments require ground improvement this shall be by soil stabilisation over an area whose width is approximately 12m (from the existing Causeway toe to 4m beyond the proposed toe).

The soil stabilisation shall be taken to the appropriate depth which is initially estimated to typically be a maximum depth of 3.5m but may reach 5m depth. Different plant and technique shall be deployed if the soil stabilisation depth is more than 5m.

- **Stage 5 – Removal of Entrance Platform**

Once the ground improvement works are complete the entrance platform will be removed and the ground under the platform improved to the required depth.

14.5.2 Embankment/ Road Construction – Seaward Side (Phase 2)

- **Stage 6 – Placement of Intertidal Shoulder Fill**

Following completion of the ground improvement works the new Causeway embankments can be constructed. A geotextile will be placed on top of the existing platform and benched slope and the selected intertidal shoulder fill placed to a level above MHWS. The seaward facing slope will have a maximum gradient of 1V:1.5H.

- **Stage 7 – Initial Placement of Coastal Protection**

Following completion of Stage 6, a geotextile layer will be placed on the front slope of the fill and 600mm of the chosen granular filter layer placed on the top. Then a small key will be excavated into the ground improved sediments to act as toe support to aid the placement of the rock armour coastal protection.

Any sediment laden water trapped between the new embankment and dam will be removed. Once this has been completed the dam can also be removed.

- **Stage 8 – Construction of Seaward Dry Shoulder**

Once Stage 7 is completed the remaining shoulder (above sea level) will be constructed using the selected fill material. Several layers of geogrid reinforcement will be installed within this material to provide adequate stability. After the shoulder fill has been brought up to its design level the coastal protection layers shall continue to be placed on the front face.

- **Stage 9 – Eastbound Pavement Construction**

Once the embankment is complete the road base and pavement will be constructed.

14.5.3 Ground Improvement – Landward Side (Phase 3)

The ground improvement works proposed on the landward side will replicate that undertaken at the seaward side as discussed for Stages (Steps) 1 to 5 above and shown on drawing 20.1.11-3-D-C-150-371.

14.5.4 Embankment/ Road Construction – Landward Side (Phase 3)

The embankment/ road construction works proposed on the landward side will replicate that undertaken at the seaward side as discussed for Stages (Steps) 6 to 9 above and shown on drawing 20.1.11-3-D-C-150-372.

14.5.5 Road Construction – Centre (Phases 4 to 6)

Once the embankments to both sides of the new Causeway are complete the road base and pavement can be constructed.

14.6 Temporary Drainage Measures

Temporary drainage measures are discussed in detail in the Stormwater Assessment Report (Aurecon, 2010) which also presents typical details for temporary drainage devices. The following is a summary of the temporary drainage measures applicable to the coastal works.

Stormwater management during the construction phase is a separate and unique stage in the water management of the motorway. It occurs after earthworks activities are complete in an area and erosion and sediment controls are no longer appropriate, but before operational stormwater controls are in place.

The philosophy for stormwater management during the construction phases is as follows:

- Maintain compliance with existing stormwater divert and discharge consents (e.g. those requiring stormwater treatment for section of SH16);

- Provide stormwater quality treatment for impervious areas where there are potential water quality effects such as from construction yards. Stormwater quality treatment to be assessed on a Best Practical Option (BPO) approach;
- Provide stormwater quantity treatment such as detention where there are potential flood or stream erosion effects such as for the surface section of the SH20 motorway in the vicinity of the Oakley Inlet;
- Provide stormwater conveyance and overland flow paths to protect worksites and neighbouring properties from stormwater flooding.

14.6.1 Sector 1 – Temporary Drainage

The coastal works in Sector 1 consist of the reclamation of an area to construct the Jack Colvin Wetland east of the Henderson Creek Bridges. There is currently no stormwater treatment or any consent requiring treatment of stormwater from the existing pavement in this section of Sector 1. Therefore the temporary catchment requiring treatment of stormwater during the construction phase consists of the additional impervious area created by the carriageway widening. Because of construction sequencing, the period of time for which there is impervious area additional to the existing impervious area during the construction phase is small.

Quantity control of stormwater in this area is not beneficial as runoff discharges directly to the CMA. In Sector 1 the proposed solution is to leave Erosion and Sediment Control Devices in place until the permanent stormwater treatment devices for Sector 1 are in place.

14.6.2 Sector 2 – Temporary Drainage

The works in Sector 2 consists of SH16 carriageway widening and the corresponding widening of the Whau River Bridges. There is currently no stormwater treatment or any consent requiring treatment for the existing pavement in Sector 2. As described above for Sector 1, in Sector 2 the proposed solution is to leave Erosion and Sediment Control Devices in place until the permanent stormwater treatment devices are in place.

14.6.3 Sector 3 – Temporary Drainage

Runoff from the bridge structures of the Rosebank Road on and off ramps and the Patiki Road eastbound off ramp will be intercepted, collected and discharged through temporary cartridge vaults located under the first lower span of each bridge as detailed in the Stormwater and Streamworks Report (Aurecon and Tonkin & Taylor, 2010).

The catchment for additional temporary stormwater treatment in Sector 3 during the construction phase consists of the additional impervious area created by the carriageway widening and the approaches to the on-and off ramp bridges, which are currently treated by settlement tanks. Due to construction sequencing, the period of time for which there is impervious

area additional to the existing impervious area during the construction phase is small. Quantity control of stormwater in this area is not beneficial as runoff discharges directly to the CMA.

In Sector 3 the proposed solution is to leave Erosion and Sediment Control Devices in place until the permanent stormwater treatment devices for Sector 3 are in place for works on the approaches and motorway widening.

14.6.4 Sector 4 – Temporary Drainage

For the existing Causeway, between Great North Road Interchange and Traherne Island, the stormwater runoff from the motorway with normal camber passes as sheet flow to the outer edge of the sealed shoulder. It then flows across a narrow strip of grass (and paved cycleway- westbound only) before diffusely discharging onto the tidal mud flats.

Stormwater management of Sector 4 during construction focuses on (1) draining the motorway of surface water and (2) providing stormwater treatment to ensure that the current resource consents are complied with.

During the construction phase, it is proposed that the existing level of treatment for the Causeway be maintained, plus full treatment of any additional impervious areas once they are paved. This presents a challenge, since there will be significant additional impervious areas at times due to the proposed construction sequencing, which involves traffic being shifted onto different lanes while others are raised and widened.

The proposed phases of construction and the catchment areas required for treatment are summarised below. The treatment areas are defined to correspond to an area which can be directed to a particular treatment device, and may be relevant for one or more of the construction phases:

- **Phase 1** – will consist of reclamation works on the eastbound (seaward) side and raising and enclosing approximately a 24m width of the eastbound carriageway. This section will accommodate three traffic lanes, a priority bus lane and a temporary access/haul road for construction machinery. Stormwater conveyance and treatment in Phase 1 will be via a 150mm diameter perforated pipe installed in a 0.6m wide x 0.6m deep trench at a slope of 0.5% and connected to cesspits at 60m intervals. The cesspits will be fitted with screening filter inserts to capture larger sediments and rubble. The trench will be lined with geotextile and filled with drainage material. The cesspits and 225mm diameter outlet pipes at 1% slope will be spaced at 60m intervals to accommodate the 1 in 100 year flow and prevent flooding.
- **Phase 2** – will consist of raising the existing Causeway on the eastbound (seaward) side and enclosing approximately a 9m width of the eastbound carriageway. This section will accommodate two traffic lanes. Construction machinery will utilise the fill area as a temporary access and haul road. Stormwater conveyance and treatment in Phase 2 will be via a 150mm diameter perforated pipe installed in a 0.6m wide x 0.6m deep trench at a slope of 0.5% and connected to cesspits at 120m intervals. The cesspits will be fitted with screening filter inserts and 225mm diameter outlet pipes will be spaced at 120m to accommodate the 1 in 100 year flow and prevent flooding.

- **Phase 3** – will consist of providing two traffic lanes at the existing motorway level approximately 9m wide and three traffic lanes with a priority bus lane on the raised eastbound (seaward) side Causeway approximately 16.5m wide. The works will also consist of reclamation works on the landward side and raising and enclosing approximately a 14m width of the westbound carriageway. The catchment area will consist of one traffic lane, a priority bus lane and the temporary cycleway. Stormwater conveyance and treatment on the seaward side in Phase 3 will be a 3 metre wide grassed filter strip at 1% transverse slope to be installed on the shoulder next to the coastal protection. On the landward side a 150mm diameter perforated pipe will be installed in a 0.6m wide x 0.6m deep trench at a slope of 1:200 and connected to cesspits at 100m intervals. The cesspits will be fitted with screening filter inserts and 225mm diameter outlet pipes at 1% slope will be spaced at 100m to accommodate the 1 in 100 year flow and prevent flooding.
- **Phase 4** – will consist of moving the traffic lanes onto the new widened shoulders of the Causeway while filling in the median area above the existing motorway. Three traffic lanes with a priority bus lane will be provided on the raised seaward side of the Causeway approximately 16.5m wide. Three traffic lanes, a priority bus lane and the temporary cycleway will also be provided on the raised westbound (landward) side of the Causeway approximately 20m wide. Stormwater conveyance and treatment on the seaward side in Phase 4 will be a 3 metre wide grassed filter strip at 1% transverse slope to be installed on the shoulder next to the coastal protection. On the landward side a 150mm diameter perforated pipe will be installed in a 1.5m wide x 0.6m deep trench at a slope of 1:200 and connected to cesspits at 100m intervals. The cesspits will be fitted with screening filter inserts and 150mm diameter outlet pipes at 1% slope will be spaced at 100m to accommodate the 1 in 100 year Average Return Interval (ARI) flow. There are no flooding issues as the excess water will spill directly over the revetment into the CMA.
- **Phase 5** – will involve moving the traffic lanes back onto the middle of the raised and widened Causeway while establishing the 7m wide permanent grassed filter strips on the shoulders.

14.6.5 Sector 5 – Temporary Drainage

The reclamation required in Sector 5 is very similar to Sector 4, with the current Causeway embankment being widened on both the seaward and landward sides. Similar temporary drainage conveyance and treatment will apply to the Causeway section of Sector 5 as described for Sector 4 above.

14.7 Erosion and Sediment Control

Erosion and Sediment Control Devices are discussed in detail in the Western Ring Route – Waterview Connection Erosion and Sediment Control Plan (refer to report 20.1.11-3-R-N-1017). The measures applicable to the coastal works are summarised in this section. Plans of the proposed erosion and sediment control measures are presented in this report.

14.7.1 Sector 1 – Erosion and Sediment Control

A wetland will be constructed at Jack Colvin Park on the northern side of the SH16 motorway between Ch6200 and Ch6350 to provide water quality treatment during the operational phase of the project. The construction of the wetland will require reclamation of the Coastal Marine Area (CMA) to the east of the Henderson Creek. The reclamation area consists of mangrove covered tidal mudflats generally above 1.0m RL. Erosion and sediment control measures during the reclamation and construction of the wetland will consist of a Rock Toe with embedded geotextile to act as silt fence installed at the toe of the fill embankment.

Reclamation will be done by foundation undercut and removal of the in-situ material to a depth approximately 2m and building up the wetland embankments using cohesive material. Due to the elevation of the mudflats, this area will only be inundated for short periods on either side during high tide to a depth of up to 0.6m. With the works programmed to correspond with the tidal cycle, the Rock Toe with embedded geotextile silt fence will provide adequate protection.

This methodology for construction of a **Rock Toe** silt fence is summarised as follows:

- i) Install erosion and sediment control silt fence;
- ii) Strip off and remove existing top soil between motorway and silt fence;
- iii) Clear mangroves within the construction and reclamation zone during low tide;
- iv) Prepare a smooth surface (i.e. remove rocks, mangrove, etc.) for the Rock Toe with embedded geotextile silt fence approximately 3m to 5m clear of the works area;
- v) Hand lay 3m wide geotextile / combgrid directly on the marine mud during low tide along the toe of the embankment;
- vi) Place new AP300 rock armour toe directly upon the geotextile using a long reach excavator in layers and embed geotextile filter blanket (e.g. Bidim A29 or similar approved) into the rockfill to form a silt fence on the tidal mudflats with returns back onto land. The rock toe must have a minimum freeboard of 500mm above MHWS level of 1.63mRL;
- vii) Form pump sumps along the inside of the Rock Toe silt fence to frequently remove seepage water. Contaminated water (i.e. sediment laden runoff) should be pumped to the sediment retention ponds or to tankers and removed from site;
- viii) Excavate and remove the virgin Holocene Alluvium (marine mud) to a depth of approximately 2m deep to reveal the East Coast Bays Formation founding level;
- ix) Commence the construction of the embankment shoulders, then proceed with bulk earthwork filling. Place selected cohesive fill in layers and compact to form pond floor and embankments;
- x) Carry out earthworks to form wetland in accordance with the details of size, type and exact location of the wetland as shown in the Stormwater and Streamworks Report (Aurecon and Tonkin & Taylor, 2010).
- xi) Install a silt fence on top of the embankment fill next to the rock armour once the backfill level is above MHWS;
- xii) As bulk back filling progresses, the Rock Toe can be used to clad the outer surface of the embankment and provide erosion protection.

14.7.2 Sector 2 - Erosion and Sediment Control

Sector 2 is located between the western side of the Rosebank Domain and the western abutment of the Whau River Bridges. Works to the motorway in this Sector mainly consists of widening the Whau River Bridges, construction of a separate new bridge structure accommodating the cycleway/footpath over the river on the southern side of the existing bridges and widening the existing motorway to four traffic lanes plus a bus lane in both directions.

- **Whau River Bridges widening and new cycleway/pedestrian bridge.**

Concrete placement will be carefully controlled using pumps and skips to ensure no contamination of the environment. Designated concrete truck wash out areas will be required near the bridge or at the Contractor's yard and these will contain the water from washing the trucks, drums and chutes; this water can then be treated or disposed of off site.

- **Rosebank Park Domain to Whau River Bridges**

Widening of the existing motorway and the construction of the cycleway/footpath will require reclamation of the CMA. The level of the marine tidal flats in this section of motorway is between 0.9m RL and 1.7m RL. which with the MHWS level at 1.63m RL means that the reclamation area will be inundated by up to 0.73m during high tide. The sediment control measures during the ground improvement and reclamation process are discussed in detail in the subsections dealing with Sectors 1 and 4, and similar methods will apply on this section of the motorway.

The surface works can be easily managed through the use of Rock Toe Silt Fences as described for Sector 1 above.

14.7.3 Sector 4 - Erosion and Sediment Control

Between Great North Road Interchange and Rosebank Road Off-Ramp (westbound carriageway), and between the Whau River and Great North Road Interchange (eastbound carriageway) for the eastbound lanes, the motorway traverses the Motu Manawa Marine Reserve and other areas of the CMA, a sensitive and protected marine environment. The interfaces between these areas and the proposed motorway widening works are typically below high tide level and as such will be subjected to twice daily tidal inundation. This creates some challenges to control and capture any sediment that is generated by the works.

- **Eastbound lanes from Rosebank Park Domain to Rosebank Road On-ramp**

The works from the Rosebank Road eastbound on-ramp between Chainage 3200 and 3850 westwards along the existing State Highway 16 traverse mainly the Rosebank Peninsula, bordering the CMA up to and including, the Patiki Road off ramp. This section of SH16 is also constrained by availability of space for the construction of controls. The works can be managed through the use of Dirty Water Diversion Channels and Super Silt Fences channelling dirty water to Decanting Earth Bunds.

The Decanting Earth Bunds will be chemically treated in accordance with the Chemical Treatment Plan. Dirty Water Runoff Diversion Channels will be established that will direct flows to these devices. In recognition of the values of the receiving environment, Super Silt Fences will also be placed along the toe of the embankments to act as a “back up” in the unexpected event of a sediment control failure.

- **Traherne Island**

The works to the Causeway traverse Traherne Island between Chainage 2100 and 2800. The works are relatively minor in this area and include shoulder widening activities. The interfaces between these areas and the proposed motorway widening are typically above high tide level and as such are only expected to be inundated, if at all, on an infrequent basis. Therefore traditional erosion and sediment control methods, such as Super Silt Fences or Rock Toe Silt Fences, as described, above can be used.

- **Great North Road Interchange to Rosebank Road (Excluding Traherne Island)**

The widening and raising of the carriageway between Great North Road Interchange and Rosebank Road will involve reclamation of the CMA adjacent to each side of the existing motorway. To manage erosion and sediment control in this marine environment the project needs to be constructed sympathetically. Consideration must be given to, firstly, reducing the potential for sediment generation and, secondly, managing any suspended material generated from the earthworks. The proposed primary erosion and sediment control measures for the Causeway section is the use of potable water filled dams, with sheet piles being used at certain locations.

It is anticipated that erosion and sediment can be reasonably controlled using the above methodology because:

- i) Clearing of the mangroves by hand will cause minimal disturbance to the marine mud;
- ii) Placing of the temporary dam by hand (or installation of a sheet pile wall) and working at low tide will eliminate the majority of possible disturbance to the marine mud. Once the dam is in place, it will provide a relatively dry enclosed work environment and protection from further disturbance by subsequent stages of construction;
- iii) Undercutting or excavation into the marine mud will be contained within a cofferdam;
- iv) Careful placement of any fill material, such as placing the rock armour boulders onto the geo-synthetic using a long reach excavator, will reduce disturbance;
- v) Selecting fill material that contains very low or no-fines will reduce sediment yield when it becomes inundated during high tides;
- vi) Selecting clean and non-contaminated fill material will prevent contaminants from being washed into the CMA;
- vii) Stage construction: to limit sediment yield, sites should use only the areas needed for the immediate activity and current stage of the construction so that earthworks are undertaken in small units at a time – having no more than 0.25ha (approximately 100 metre chainages) exposed to erosion during each work phase will minimise sediment yield intensities in the event of a temporary dam failure;
- viii) Timing the works by the lunar cycle in order to take advantage of favourable tidal variations;
- ix) Installing traditional Erosion and Sediment Control (E&SC) devices, such as silt fences, on land above MHWS;
- x) Installing motorway temporary stormwater drainage;

- xi) The process of soil stabilisation bonds finer material together so that it is less susceptible to being eroded to yield sediment;
- xii) Soil stabilisation effectively immobilises many contaminants, including heavy metals and some organics.

14.7.4 Sector 5 - Erosion and Sediment Control

The reclamation required in Sector 5 is similar to Sector 4, with the current Causeway embankment being widened on both the northern and southern sides and therefore similar erosion and sediment control measures apply.

15. Operational Considerations

15.1 Ongoing Settlement

Although the implementation of ground improvement measures will resolve instability risks and limit settlement, due to the soft underlying materials it is inevitable that the highway will experience some continuing settlement during its operational life. These settlements have been calculated and are presented in the Ground Improvement Options Report (20.1.11-3-R-J-304). It is critical that settlement monitoring of the embankment continues throughout the construction and the operational phases to determine actual settlement rates. This analysis will also allow predictions of future settlements so that any maintenance can be planned.

As discussed in previous sections, it is expected that the largest settlements will occur at the shoulder areas of the embankment and not within the trafficked pavement layers. Therefore, significant distortion of the pavement layers is not envisaged but regular monitoring should be undertaken to confirm this.

15.2 Monitoring

It is recommended that survey monitoring of the highway is undertaken during the operational phase to determine levels and calculate actual settlement rates. It is also recommended that regular surveys are carried out of the pavement surface using a towed surveying device to determine surface profiles and identify any areas of differential settlement. The results from this monitoring will indicate when and where (if at all) the highway is becoming distorted and areas out of tolerance can be identified and corrected.

15.3 Pavement Maintenance

15.3.1 Pavement Maintenance

The proposed ground improvement/ embankment construction for the Causeway embankment has been designed so that the road pavement should not experience total or differential settlements in excess of the criteria set out in the Design Philosophy Statement (Aurecon & Tonkin and Taylor (2010)). However, it must be noted that these assumptions are based on preliminary global parameters and it is possible that local settlements may be greater than stated in the design criteria.

It is recommended that an assessment is undertaken following the regular monitoring (discussed above) to determine if, and where, excess settlements are occurring and what corrections (if any) are required.

15.3.2 Drainage Maintenance

As the main proposed stormwater drainage system comprises bio-filter strips running along the verges of the proposed Causeway embankment, the need for significant maintenance of the drainage system is minimal in comparison with buried pipe systems, cartridge filters or similar.

Geotechnical analysis indicates that the greatest amount of settlement is likely to occur along the shoulder areas of the embankment (Ground Improvement Options Report). The bio-filter strips are required to maintain a lateral fall of 1% in order to maintain their treatment efficiency. Therefore, the cross-fall of these filter strips will require regular monitoring and occasional re-grading.

15.3.3 Coastal Protection Maintenance

To maintain protection of the embankment, it is necessary that the integrity of the revetment is maintained. It is unlikely that locally extreme differential settlement will disrupt the revetment in places. Assurance of the integrity of the revetment is best maintained by visual inspection. If rock boulders have become dislodged and the filter behind has become exposed to wave action, it will be necessary to repair the revetment by replacing or adding new boulders to the filter layer and rock boulders to plug any gaps.

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17. Abbreviations

AH	Layer Code for Holocene Alluvium
ARI	Annual Return Incident
ATcl	Layer Code for clayey and silty layers within Tauranga Group Alluvium
ATo	Layer Code for organic layers within Tauranga Group Alluvium
ATp	Layer Code for peat layers within Tauranga Group Alluvium
ATs	Layer Code for sandy layers within Tauranga Group Alluvium
ATv	Layer Code for volcaniclastic layers within Tauranga Group Alluvium
BPO	Best Practical Option
Ch, CH	Chainage
CMA	Coastal Marine Area
CPT	Cone Penetration Test
D ₁₀	Particle size (sieve) at which 10% of a sample will pass, i.e. 10% is smaller than
D ₅₀	Particle size (sieve) at which 50% of a sample will pass, i.e. 50% is smaller than
DOC	Department of Conservation
E & SC	Erosion and Sediment Control
ECBF	East Coast Bays Formation
ER	Layer Code for Residual East Coast Bays Formation
Es	Drained Elastic Modulus
Eu	Undrained Elastic Modulus
FoS	Factor of Safety
GAP	General All Passing
GPR	Ground Penetrating Radar
IGNS	Institute of Geological and Nuclear Sciences
IPCC	Intergovernmental Panel on Climate Change
LHS	Left hand side
Ma	Million years ago
MDD	Marine Deposit Displacement
MHWS	Mean High Water Spring tide level (currently +1.63mRL)
MMMR	Motu Manawa Marine Reserve (Pollen Island)
MPL	Managed Priority Lane
N, 'N', "N"	In the Standard Penetration test the number of blows for 300mm penetration
NIWA	National Institute of Water & Atmospheric Research
NWC	Natural water content (= natural moisture content)
NZTA	New Zealand Transport Agency
PSD	Particle Size Distribution
RHS	Right hand side
RL	Reduced Level
SH16	State Highway 16
SPT	Standard Penetration Test
TG	Tauranga Group
TGA	Tauranga Group Alluvium
WRR	Western Ring Route
12D	Proprietary name for Computer software used to generate alignments
2-D	Two dimensional

18. Glossary

Term	Definition
Alluvium	A suite of geological materials typically deposited by a river system but used to describe the assorted non-lithified sediments that overlie the bedrock. Each distinct material represents different local conditions of depositional energy and source material.
Batter	Slope angle or the process of forming a slope
Bulk Fill	Material used as a general fill to raise the level of an embankment (or other earthworks). Along the Causeway this particularly refers to the material placed on top of the existing embankment in order to raise the level generally.
Bund	An earthen embankment or dyke
Causeway	Typically an embankment that runs across terrain that is periodically or permanently underwater. The SH16 Causeway refers to those sections of the route that pass across mud flats of the Central Waitemata Harbour between Great North Road Interchange and the Rosebank Peninsula and between Rosebank Park Domain and the Whau River (approximately Ch700 to Ch2950 and Ch4400 to Ch4720).
Clay	Soil which is either clay size (<0.002mm) or made up of clay minerals (illite, smectite, kaolinite and the like) or frequently both. The sheet structure of the clay minerals causes them to have a host of characteristic properties including a high propensity for the absorption of water which in turn lead to a tendency to have typical geomechanical characteristics that may be undesirable, such as high shrinkage and swelling potential, low permeability, low shear strength.
Cohesive	A soil, particularly clay, whose particles adhere to each other by means of adhesive and cohesive forces. These materials are plastic when wet.
Compaction	The process of reduction of the proportion of air voids in a soil, usually undertaken in civil engineering practice by the use of mechanical plant. As the proportion of air voids is reduced the soil density and undrained shear strength increase.
Compensatory Filling	Placement of additional fill to make up for predicted settlement so that a design level at a future design date will be achieved.
Compressibility	That property of a soil which indicates its capacity for being consolidated and hence its propensity for settlement. It is measured by the modulus of compressibility.
Consolidation	A settlement process whereby soil decreases in volume through the escape of groundwater from the pores between soil grains (pore water). Whenever a soil is loaded to a higher stress state than currently exists the increase in stress is initially taken up by the pore water; over time this excess pore water pressure dissipates as the increased stress is taken up by the soil skeleton, the soil particles re-pack

Term	Definition
	themselves and the pore water drains out from between the soil particles.
Crest	The break of slope at the margin of the upper surface of the embankment and the forward slope that leads down to the existing terrain.
Differential Settlement	The relative change in settlement between several points. Such differential settlements lead to alterations in gradients between points. Many structures are sensitive to differential settlement when they are not affected by total settlement.
Ebb tide	The outgoing tide, the period between high water and the succeeding low water.
Equilibrium Moisture Content	The water content at which the soil is neither gaining nor losing moisture.
Filter Layer	A material (typically granular soil) whose particle size grading is such that it forms a transition between a coarser soil and a finer soil to permit water to flow from one soil to the other without being able to erode fine particles from the finer soil. The main usage envisaged for the Causeway is, in combination with the rip-rap, to protect the shoulder fill from tidal and wave action. A geosynthetic may be able to substitute for a filter layer.
Fines	Soil particles whose size is less than 0.06mm, the clay and silt size fraction
Geosynthetic	A term used to describe a range of generally polymeric products used in civil engineering applications. These include: geotextiles, geogrids and geomembranes.
Granular	A soil such as sand or gravel with little or no clay content. Granular soil has no cohesive strength.
Ground Improvement	A collective term for a range of different processes that improve the characteristics or geotechnical properties of a soil so that it can serve a particular engineering function.
Heave	Upward movement at a point.
Holocene	A geological epoch that began at the end of the last Ice Age, about 11,700 years ago and is generally considered to continue to the present day. A synonym for Recent.
King Tide	An extreme tide where the high tide is significantly higher than the MHWS.
Left Hand Side	For SH16 this is the side along which traffic travels away from the city. For the embankment this is the southern side above which westbound traffic will travel.
Littoral	The environment between the highest and lowest levels of spring tides.
Mud flats	The areas of the seabed whose surface is formed of estuarine or marine mud that are tidally inundated and are therefore exposed at low tide and are not part of the engineering works (existing or proposed).
Mudcrete	Is a material formed by a soil stabilisation process which mixes mud (commonly marine mud) with an additive such as cement. When the final product has cured it has a lower moisture content with significantly enhanced shear strength and durability and significantly reduced compressibility. Traditionally mudcrete has been produced by mixing excavated natural material with the additive and then replacing

Term	Definition
	the mixture into the ground. An alternative is to introduce the additive into the ground and mix it in-situ with the natural soil.
Optimum Moisture Content	The water content at which a specified compactive force can compact a soil to its maximum dry unit weight.
Organic soil	A soil with a significant component of plant debris which can be in various stages of decomposition. The organic material may be in the form of isolated specks or streaks or be accumulated in lenses, partings and layers. The presence of biological fibres affects the geotechnical properties as these materials can absorb significant quantities of water and are compressible.
Pause Period	That time which is allowed to elapse following a fill operation. When a pause period occurs between embankment lifts, the intention is for an improvement in foundation soil properties to occur. When a pause period takes place at the end of a bulk filling operation, this permits settlement to occur in advance of pavement placement (= preloading phase).
Pavement	The "pavement" layer refers to the sub-base, base course and wearing (surface) course components of a highway.
Perigee	The closest distance in an elliptical orbit. When the moon is at its perigee the moon's effect on the tide is enhanced. When the perigee coincides with the spring tide (ie the moon and the sun are aligned), the spring tide is higher than usual, resulting in a "king tide".
Permanent Occupation	Defined as the permanent elements of the project works which lie below MHWS. The permanent works cover man-made structures that support the new motorway infrastructure and include embankments below MHWS, pier locations for the new bridge structures and ground improvements to the founding soil.
Preload	Ground is loaded in advance of the main construction in order to improve soil properties; in particular early consolidation allows the soil to be drained in advance thus reducing the residual settlement to take place in the main construction and operational phases of a project. Preloading can take a variety of forms including the advance construction of an earthworks embankment so that by the time the pavement is placed much of the settlement has already occurred.
Recent	In geological terms the last period leading up to the Present Day. This generally equates to the Holocene.
Phase	A stage in a process of development.
Reclamation	The process of converting ground that is permanently or intermittently inundated by water into land that is permanently above sea level or flood level. For the SH16 project the reclamation is defined as the land that has been created by the works that lies above the high tide level (MHWS).
Revetment	Slope protection.

Term	Definition
Right Hand Side	Along SH16 this is the side along which city-bound traffic travels. For the embankment this is the northern side, above which eastbound traffic will travel.
Rip-rap	Is rock or other material used to armour the shoreline against erosion by absorbing and deflecting the impact from waves so that wave energy does not penetrate the interior of the embankment.
Seabed	Ground that lies below the level of the high tide (MHWS).
Secondary Compression	A later stage settlement process, also referred to as (long-term) soil creep. A rule of thumb is that secondary compression continues after the completion of primary consolidation. In this form of settlement the soil particles themselves distort under the increased stress that triggered settlement. Secondary compression is particularly pronounced in peats and soils which are high in organic matter since much of the soil is composed of plant fibres which contain significant voids within themselves (high porosity).
Shoulder Fill	The portion of the embankment to be placed in the vicinity of the crest. This is that volume lying above and outward from the existing embankment crest.
Silt	Soil particles between 0.002mm and 0.06mm in diameter. These are the coarser end of the spectrum of fine materials. Particle composition may be variable and can include material that contains a high component of clay mineral.
Soil Stabilisation	A ground improvement technique where an additive (such as cement or lime) is mixed with the soil to form a soil-additive mixture. The geotechnical properties of the mixture are an improvement on the properties of the original soil although this improvement may take some time to develop.
Spoil	The by product from an engineering excavation, especially that material arising from tunnelling or cutting.
Staged Construction	A form of embankment construction that incorporates pause periods. In certain ground conditions, raising an embankment to its full height in one stage could cause slope instability or bearing capacity failure. By staging the construction, advantage is taken of the improvement in soil properties that takes place under loading primarily through the process of consolidation.
Sub-base	The lowermost pavement layer; this is the fill placed on top of the sub-grade.
Sub-grade	The ground surface upon which the pavement layers are placed.
Surcharge	A form of preloading where an additional load of fill is temporarily placed on an embankment in order to accelerate the settlement that will occur. Surcharging permits the ground response to move further along the settlement curve with respect to the standard fill load. After the surcharge is removed the residual settlement remaining is much diminished.
Temporary Occupation Stage	The area of Temporary occupation is the sum of the areas of reclamation and occupation during the construction period. This may form the permanent case.

Term	Definition
Total Settlement	The cumulative settlement from all factors that may affect a point, typically these factors would include immediate settlement, primary consolidation and secondary compression. Detection of total settlement relies on comparison of level with a fixed datum.
Undercut	The excavation of weak soils with poor geotechnical characteristics, which are then replaced with better quality fill.
Vadose	The unsaturated portion of the soil between ground surface and the water table.
Vesicular	Description of a rock that contains many small holes, characteristic of some lavas which contained numerous air bubbles in them when they solidified.

All coastal works engineering drawings referred to in this report are listed in for clarity. All drawings are located in Coastal Works Engineering Report - Volume 2

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