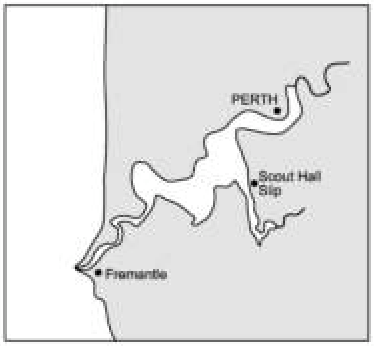
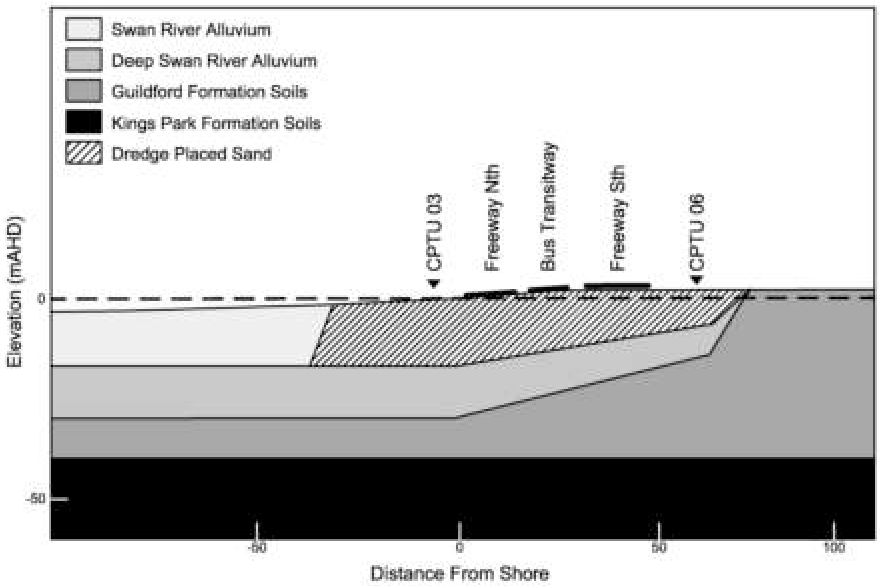
1.0 Introduction

The Scout Hall Slip is on the section of Kwinana Freeway in Perth Western Australia, between Cale Street and Casey Street in Como, just north of Canning Highway. This section of the river shore was originally low lying swampy ground. The natural surface levels at this location required about 1m of additional fill to be placed to raise the site to the freeway design levels. It was estimated at the time that about 100,000 cubic yards (76,000m^3) of sand was required (Marsh).



Locality Plan – Figure 1

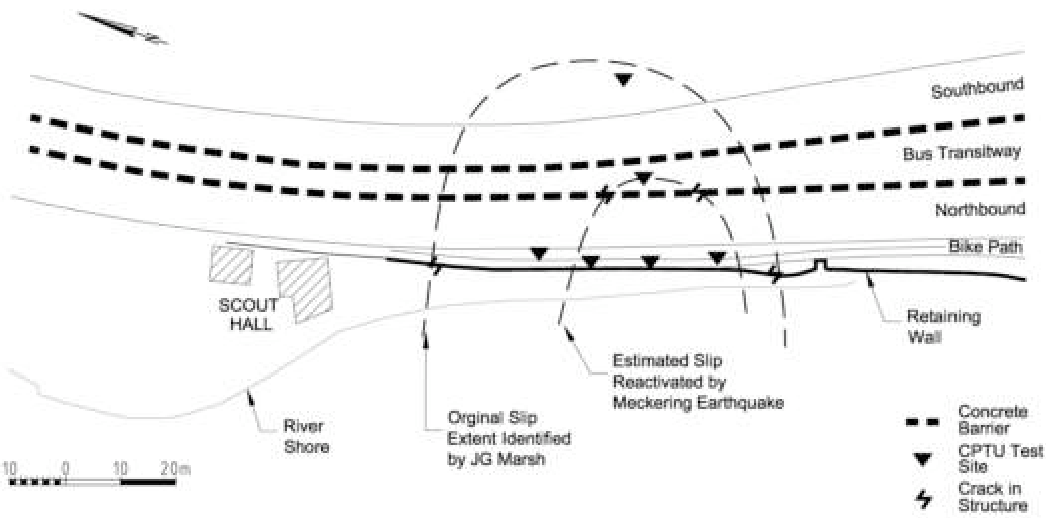
The Public Works Department (PWD) carried out the fill placement using dredged sand from the riverbed. During the placement of the fill a slip formed and was identified by the Main Roads Department Engineers at the time. The sand was placed using dredging pumps and based on pump throughput rates it is estimated about 600,000 cubic yards (460,000 m^3) of sand were placed. This additional 500,000 cubic yards (380,000m^3) of sand displaced the soft alluvial mud in the river bank. These displaced soft soils were observed to form a mound in the river and were removed by the PWD. No records of the volume of material removed have been found by the authors. The PWD appears to have continued to add material until the slip stopped moving, i.e. a Factor of Safety (FOS) of 1 was reached. The freeway was then constructed over the "stabilized" slip zone. The volume of material reported above is reproduced from MRWA report on "Narrows Interchange And Section Of Kwinana Freeway On South Perth Foreshore, Implications Of Construction History", JG (Gilbert) Marsh, June 1993. Figure 2 shows the estimated extents of dredge placed sand fill in a cross-section of the slip zone. This geological model of the slip was developed from CPTU records, observations on site and evidence from G J Marsh (both documentary and anecdotal).



Slip Cross-Section – Figure 2

During the 1968 Meckering Earthquake a portion of the Scout Hall Slip remobilised and formed a 300mm to 600mm scarp on the north bound freeway carriageway opposite Henley Street. The earthquake had a magnitude of 6.9 on the Richter Scale with an epicentre at about 140km from the slip location.

The site was visited August 2003 by one of the authors in the company of Mr J G Marsh, who was an Engineer with Main Roads WA at the time of the filling and at the time of the Meckering Earthquake. Discussions with Mr Marsh indicate that the original slip extended about 20m east of the southbound carriageway and affected about 150m of what would become freeway pavement. During the 1968 Meckering Earthquake only part of the original slip was remobilised and only the north bound carriageway was affected. Figure 3 attached shows the estimated extents of the original and the 1968 slip crest lines.



Plan of Slip Areas – Figure 3

Main Roads WA (November 1990) identified a section of the northbound carriageway that had developed cracks in the pavement. The testing carried out indicated that there was no defect with the subgrade. It was concluded that the observed cracking was probably due to settlement and or movement of the foundation and that the cracking was localised and had little influence on pavement performance.

The pavement has been reconstructed since that time. The authors have observed cracking in the adjacent cycleway and in the concrete barrier that separates the Freeway from the transitway and a recent repair in the pavement (northbound carriageway) in the area of the slip.

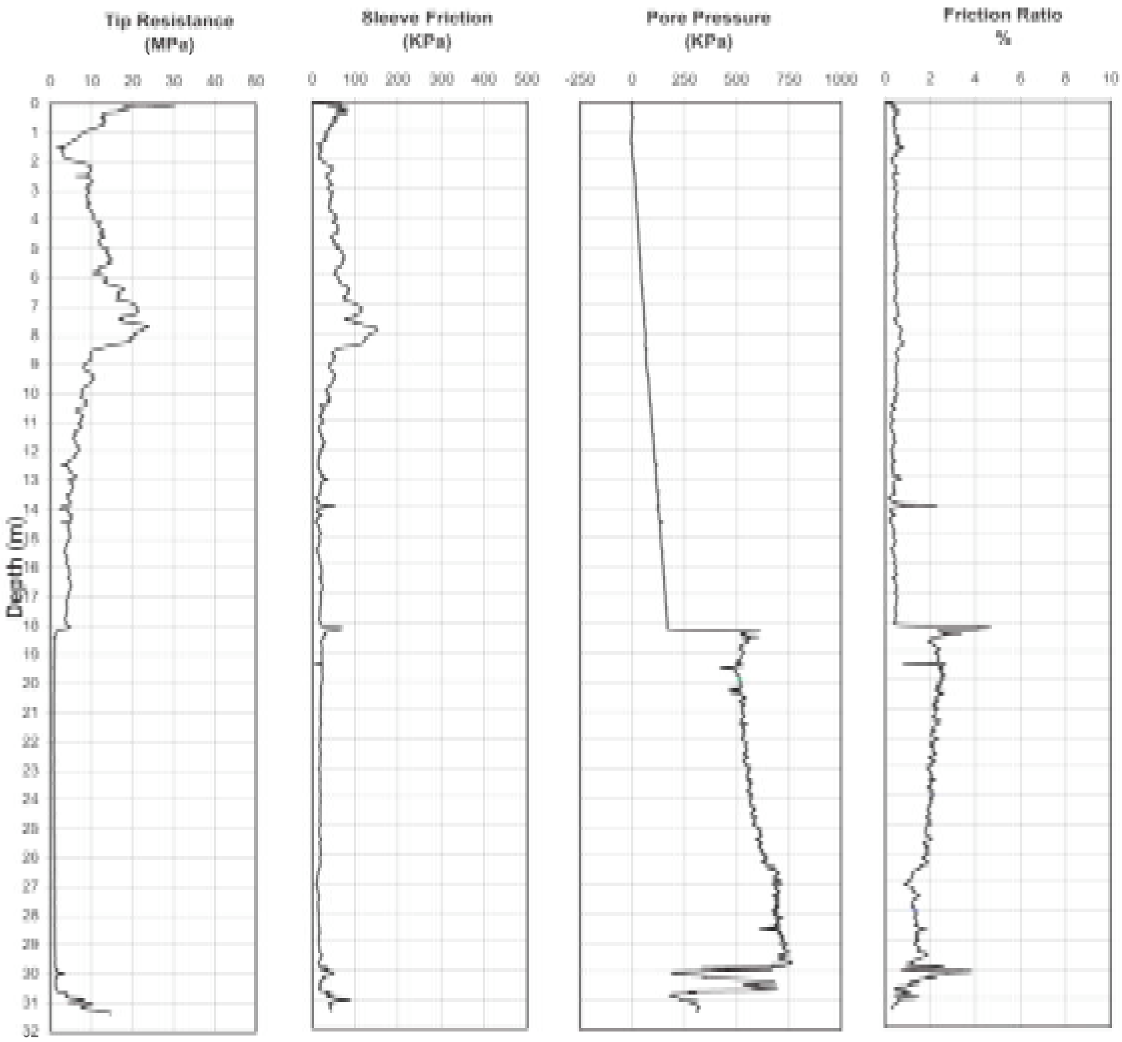
2.0 Recent Investigations

During tender design phase (2003) for the Perth to Mandurah railway the authors, in conjunction with BGC Contracting, investigated the slip and its potential to re-mobilise.

The investigation was targeted at developing a geological model of the slip and assessing its current stability under static and earthquake loading scenarios.

Cracks identified in both the existing retaining wall below the cycle way and of the concrete barrier between the northbound carriageway and the bus transit way provided indicators to the longitudinal extent of the area that is either still undergoing creep settlement or creep slippage. The cycle way pavement was observed to contain diagonal cracking near the estimated northern extent of the slip. These cracks appear to be similar to those described in Main Roads WA (1990). It is likely that these cracks are the result of ongoing slip movement and/or secondary consolidation of alluvial sediments below the dredge placed sand fill.

Six cone penetrometer peizometric tests (CPTU) in the vicinity of the scout hall slip were carried out as part of the 2003 investigations. The CPTU tests generally encountered dredge placed sand fill in a medium dense to dense condition overlying soft to firm clay soils. Coarse-grained materials were encountered underlying the soft clayey soils. Figure 4 shows one of the CPTU records.



CPTU Probe Record Near Centre of Slip – Figure 4

3.0 Geological Model at the Time of the Original Slip Formation (1955)

The shear strength of the soft clay at the time of the original slip occurred was derived by consideration that the alluvial sediment would have been normally consolidated at that time.

Assuming a buoyant density of $5kN/m^3$, and a ratio of shear strength to vertical effective stress of 0.2, then the shear strength profile of the pre-slip soils would have been about 1kPa per meter depth. Therefore at the base of the original slip (18m deep) the shear strength at the time of original formation would have been about 18kPa, increasing to a shear strength of 30kPa at 30m depth (the base of the Swan River Alluvium sediment as recorded in recent CPTU tests).

Using the SLOPEW software package with a Morgenstern Price analysis, the calculated factor of safety under static conditions at the time of slip formation was calculated to be about 1.3. As this factor of safety exceeds 1.0 there must have been some other factor(s) for a slip to have occurred. Possibly the elevation of the dredge placed fill was higher than the current ground level. This would have reduced the factor of safety. Once the slip mobilised, the shear strength of the clay would have been reduced by remoulding.

4.0 Geological Model at the time of Meckering Earthquake (1968)

Following the original slip formation, the underlying alluvial soils would have experienced an increase in overburden pressure equivalent to the difference in buoyant density of the soft clay and the dredge placed sand (an estimated difference of about $4kN/m^3$). At 18m depth the increase in overburden pressure would have been about $14.4kN/m^2$.

The soft clay would have gained strength due to the dissipation of excess pore water pressure. By 1968 the dissipation would have been about 80% complete. Allowing for this dissipation, the shear strength of the clay at the time of the Meckering Earthquake would have been about 11.5kPa greater than at the time of the original slip.

5.0 Stability of the Scout Hall Slip at the time of the Meckering Earthquake

Using the SLOPEW software package with a Morgenstern Price analysis, the calculated factor of safety under static conditions was calculated to be about 1.7. Using the model generated above, it is possible to estimate the ground acceleration required to remobilise the Scout Hall Slip.

Under earthquake conditions a horizontal ground acceleration would develop which could remobilise the slip. Various seismic accelerations were applied to the model to assess that required to remobilise the slip. By a process of trial and error it was found that the required horizontal acceleration to reduce the factor of safety to 1.0 was 0.05g.

6.0 Ground Acceleration at the site from Meckering Earthquake

The authors are not aware of any ground acceleration measurement data being available in Perth for the 1968 Meckering Earthquake. However there are some indirect indicators. Gaull et al (1995) presented evidence that there was significant basin amplification in Perth. The potential for significant basin amplification in Perth is supported in later work by Gaull (2003) and by Jones et al (2005).

Gaull 1995 indicated that the ground intensity (Modified Mercalli) in Perth was as high as VI to VII at some locations. Using correlations in Kramer (1996) this intensity infers a ground acceleration of about 0.09g. This exceeded the value of 0.05g estimated as being necessary to reduce that factor of safety to 1.0 and remobilise the slip.

7.0 Displacement Analysis For the Meckering Earthquake

Ambraseys & Menu (1988) proposed that displacement of a slope under earthquake shaking could be estimated from the equation:

$$\text{Log } u = 0.90 + \log \left[\left(1 - \underline{a_{y/}}\, a_{max} \right)^{2.53} \left(\underline{a_{y/}}\, a_{max} \right)^{-1.09} \right] \qquad (1)$$

u = Vector displacement of slip during historical event (cm)

a_{max} = maximum ground acceleration(0.09g for Meckering Earthquake))

a_y = ground acceleration corresponding to a FOS = 1 (0.05g)

Applying equation 1 the calculated displacement for the Scout Hall Slip under an earthquake with a peak ground acceleration of 0.09g is about 2cm. This compares to the 30cm to 60cm that was observed on site by G J Marsh in 1968. Some possible reasons for the difference are that the shear strength was reduced through remoulding once initial movement commenced and/or there was significant local site amplification that increased the ground acceleration at the Scout Hall site.

References:

Ambraseys N N & Menu J M (1988) Earthquake-induced ground displacements. *Earthquake Engineering and Structural Dynamics Vol 16 pp985-1006*

Gaull B A, Hirosho Kagami, M Eeri and Hitoshi Tanguchi (1995) The Microzonation of Perth, western Australia, Using Microtremor Spectral Ratios. *Earthquake Spectra Vol 11 No 2 pp173-191*

Gaull B A (2003) Seismic characteristics of the sediments of the Perth basin. *Australian Geomechanics Vol 38 Sept 2003 The Engineering Geology of Perth Part 1 pp111- 122*

Jones Middleman & Corby (2005) Natural hazard risk in Perth Western Australia. *Geoscience Australia, Commonwealth of Australia*.

Kramer S L (1996) Geotechnical Earthquake Engineering. *Prentice Hall page 67*

Marsh J G (1993) Narrows Interchange And Section Of Kwinana Freeway On South Perth Foreshore, Implications Of Construction History. *Main Roads Western Australia, Materials Report*

Salem B (1990) Pavement Evaluation of Kwinana Freeway 5.31 to 4.91 SLK *Main Roads Western Australia Materials Report 90/115M*