REPORT

TO
BMT WBM PTY LTD

ON
GEOTECHNICAL ASSESSMENT

OF
EROSION HAZARD LINES

ALONG THE
WYONG SHORE COUNCIL FORESHORE AREAS
BETWEEN CRACKNECK POINT AND
HARGRAVES BEACH, NSW

5 April 2016
Ref: 28424WRrpt

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FIGURE 1: SITE LOCATION PLAN

FIGURE 2: DETERMINATION OF SCE RECESSION RATES

TABLE A: COMPARISON OF RECESSION RATES

REPORT EXPLANATION NOTES

1 INTRODUCTION

This report presents the results of a geotechnical assessment of selected portions of the foreshore area between Crackneck Point and Hargraves Beach, within the Wyong Shire Council Local Government Area (LGA). The assessment was commissioned by Paul Donaldson (BMT WBM Pty Ltd [BMT]) in emails dated 7 May 2015 and 13 October 2015. The commission was on the basis of our fee proposals (Ref: P40299ZRlet, dated 9 April 2015 and P40299ZRletrev1, dated 1 July 2015).

The purpose of the geotechnical assessment was to:

- Review the Immediate, 2050 and 2100 coastal planning hazard lines presented in the current Wyong Shire Council Coastal Zone Management Plan (CZMP) for current sea levels and assuming sea level rise (SLR):
    - At the transition zones between sections of foreshore predominantly comprising bedrock cliff lines and sand dunes.
    - Over the remaining lengths of foreshore comprising bedrock cliff lines and steep foreshore slopes comprising indurated sand deposits.
- Based on the review, provide the methodology for BMT to define new Immediate, 2050 and 2100 coastal planning hazard lines.

We note that a review of risk levels associated with the coastal planning hazard lines was not required as part of this assessment.

The role of the coastal engineers (BMT) was to undertake a similar review of the foreshore areas comprising sand dunes and collate the results of the geotechnical and coastal engineering assessments and prepare new Immediate, 2050 and 2100 coastal planning hazard lines.

The geotechnical assessment included site inspections at various locations along the foreshore and a desk top review of the provided information and our own project database. Based on the results of our assessment, we have prepared this report which includes our site observations, a summary of our desk top review, our recommended methodology for defining the extent of coastal erosion impacting the foreshore cliff lines and steep indurated sand slopes for the Immediate, 2050 and 2100 coastal planning hazard lines and additional geotechnical advice pertinent to the formation of the coastal planning hazard lines.

2 PROVIDED BACKGROUND INFORMATION AND REFEENCES

We have been provided with the following information to assist with our assessment:

Information provided by BMT

- Draft aerial photography (including surface level contours) for the study area.
- Draft LiDAR plans (including surface level contours) for the study area.
- Old aerial photographs for selected sections of the study area; 'Gosford-Norahville' dated 25 November 1941, 'Lake Macquarie' (Ref. NSW 128-5103, dated March 1954, 'Shellharbour-Newcastle (Ref. NSW2418 64, 66, 68, 70, 83, 84 dated 17 November 1976, and two unreferenced aerial photographs of the Soldiers Point area, dated 1978.
- Draft version of the 'Review of the Wyong Shire Coastal Hazard Study' report (Ref. R.N20435.002.00.HazrdStudy review, dated 31 March 2016) prepared by BMT.

Information provided by Wyong Shire Council:

- CZMP for the Wyong Coastline (Report No. 1869/RO4, dated December 2011) prepared for Council by Umwelt (Australia) Pty Ltd (Umwelt).
- 'Report on the Geotechnical Issues associated with the Coastline Management Study for the Wyong Shire Council' (Ref. G:\Jobbm\BM001\BM001-5\RN20100406\RN20100406_final.doc, dated 31 May 2010) prepared by Shirley Consulting Engineers Pty Ltd (SCE) as subconsultants to Umwelt and incorporated into the CZMP as Appendix 4 (SCE 2010).
- 'Draft Report on Geotechnical Studies and Identified Hazards for the Wyong Shire Coastline' (Ref. G:\Jobbf\BM001\bf030-5\m20040825\m20040825l.doc, dated 11 October 2004) prepared by SCE as subconsultants to Umwelt.
- 'Report on Geotechnical Investigation, Proposed Residence, 62 Werrina Parade, Blue Bay' (Project 75353.00, dated December 2011) prepared by Douglas Partners Pty Ltd (DP).
- 'Report on Geotechnical Investigation, Proposed Residence, 60 Werrina Parade, Blue Bay' (Project 75433.01, dated March 2014) prepared by DP.
- Letter report presenting coastal engineering advice relating to a proposed pool at 45 Hutton Road, The Entrance North (Ref. 8A0512, dated 3 February 2015) prepared by prepared by Royal HaskoningDHV (RH).
- Letter report presenting coastal engineering advice in relation to Development Application at 45 Hutton Road, The Entrance North (Ref. 8A0503, dated 7 November 2014) prepared by RH.
- 'Report on Geotechnical Investigation, Proposed Additions and Alterations, 44B Werrina Parade, Blue Bay' (Project 41612, dated March 2008) prepared by DP.
- 'Coastal Engineering Report for Proposed Development 64 Werrina Parade, Blue Bay' (Ref. 301015-03655-CA-REP-001, dated 17 April 2015) prepared by WorleyParsons (WP).

- ‘Coastal Engineering Report, 73 Budgewoi Road, Noraville’ (Ref. 59914110/R001, dated 9 July 2014) prepared by Cardno (NSW/ACT) Pty Ltd (Cardno).
- ‘Report on Geotechnical Investigation, 73 Budgewoi Road, Noraville’ (Ref. CGS2153-002.01, dated 30 June 2014) prepared by Cardno.
- Geotechnical Investigation 43 Werrina Parade, Blue Bay (Report No. 06/0653, dated July 2006) prepare by SMEC Testing Services Pty Ltd (STSPL).
- ‘Report on Geotechnical Investigation, Proposed Bank Stabilisation Works, 121 Toowoon Bay Road, Toowoon Bay’ (Project 75616, dated October 2013) prepared by DP.
- ‘Geotechnical Advice, Proposed Piled Retaining Wall, 121 Toowoon Bay Road, Toowoon Bay’ (Project 75616, dated 3 June 2014) prepared by DP.
- ‘Geotechnical Investigation-Slope Stability Assessment, Proposed Residential Extension, 28 Bungary Road, Norah Head’ (Ref. E14 088-AR1, dated 3 September 2014) prepared by Sanko Excavation Environmental & Civil Services Pty Ltd (Sanko).
- ‘Report on Geotechnical Investigation, Proposed Extensions To Existing Residence, 2 Rolls Avenue, Toowoon Bay’ (Project 41952, dated December 2009) prepared by DP.
- ‘Report on Geotechnical Investigation, Proposed Additions and Alterations, 4 Rolls Avenue, Toowoon Bay’ (Project 75570, dated May 2013).prepared by DP.
- Geotechnical assessment report for a proposed residence at 7 Roslyn Place, Noraville (Ref. G09/321-A RS:RS, dated 31 August 2010) prepared by Network Geotechnics Pty Ltd (NGPL).
- ‘Geotechnical Report On Elizabeth Drive, Noraville, NSW’ (Ref. R14.010.1, dated 27 December 2013 and Ref. R14.010.2, dated 18 February 2014) prepared by David Mehan Geotechnical Consultant.
- ‘Report On Geotechnical Investigation, Proposed Dual Occupancy, 43 Elizabeth Drive, Noraville’ (Project 41483, dated 5 May 2007) prepared by DP.
- Geotechnical investigation report for proposed apartments at 88 Ocean Parade, The Entrance (Ref. G09/391-A DS:VdS, dated 7 October 2010) prepared by NGPL.
- Engineering reports for a proposed development at 114 Towoon Bay Road, Toowoon Bay (Ref. 365-09/SS/MS, dated 17 March and 2 July 2010) prepared by CSG Engineers Pty Ltd.
- ‘Geotechnical Assessment, Proposed Residential Development, 114 Towoon Bay Road, Toowoon Bay’ (Ref. 94182/SHS, dated 10 January 1995) prepared by Stephen H Savage.

3 ASSESSMENT PROCEDURE

3.1 Site Inspections

The foreshore study area comprised the crest and toe areas of cliff faces and steep indurated sand slopes within publicly accessible areas. The attached Figure 1 presents a Site Location Plan and is based on an extract from 'Google Earth'. Figure 1 indicates the following locations of the various sections of the study area that are included in this report:

- Crackneck Point
- Bateau Bay
- Shelly Beach
- Little Bay
- Towoon Bay
- Blue Bay
- The Entrance
- Soldiers Beach
- Pebbly Beach
- Norah Head
- Lighthouse Beach
- Cabbage Tree Harbour
- Jenny Dixon Beach
- Hargraves Beach.

Cliff lines and outcrops within adjacent private properties were also included where a visual assessment could be made from publicly accessible areas but access on to private property did not form part of the scope of work.

JK Geotechnics completed inspections on 11 and 12 June 2015 and 27 January 2016. The assessment was completed by a Senior Associate level engineering geologist, from safe vantage points and where access was possible along the crest and toe of the cliff faces and steep indurated sand slopes. The assessment comprised a detailed inspection of the topographic, surface drainage and geological conditions of the specific foreshore areas and their immediate environs. We note that the date and time of the inspections were selected to optimise low tidal water levels in order to allow access, where possible, along the toes of the cliff lines. Inspections using boat access and industrial rope access abseiling techniques were beyond the agreed scope of this assessment.

The features described in Section 5, below were based on hand held tape measure, inclinometer and compass techniques, where access was possible. The slope angles of vegetated slopes were also approximated based on hand held inclinometer readings. The orientations of joint planes presented in Section 5, below have been measured using a hand held compass and presented as strike angles (i.e. perpendicular to the maximum dip angle). Otherwise, the dimensions of features that were inaccessible were estimated using observations made from safe vantage points at the crest or toe of the cliff lines. Our observations also compared current conditions to those assessed from observations carried out as part of previous geotechnical investigations and/or assessments of areas of the cliff lines within the study area (by JK Geotechnics or other consultants) and provided historical photographs.

3.2 Desk Top Study

The desk-top study included a review of the provided information listed in Section 2 and the following additional sources of information:

- The 1:100,000 Geological Map of Gosford-Lake Macquarie, Sheet 9131, 9231 dated 2008.
- Historical aerial photography sourced form ‘Google Earth’.
- Our project database, which included the previous reports listed below:
  - ‘Report on Geotechnical Investigation, Proposed Residence, 62 Werrina Parade, Blue Bay’ (Project 75353.1, dated April 2012) prepared by DP.
  - Letter report presenting coastal engineering advice relating to 62 Werrina Parade, Blue Bay (Ref. 8A0046, dated 24 September 2012) prepared by Royal HaskoningDHV (RH).
  - ‘Report on Geotechnical Investigation, Proposed Residence, 60 Werrina Parade, Blue Bay’ (Project 75433, dated July 2012).prepared by DP.
  - ‘Protection of Foreshore Embankment Investigation and Coastal Hazard Study’ (Ref. J3312/R2096, Issue 1, dated May 1999) prepared by Patterson Britton & Partners Pty Ltd (PB).
  - Geotechnical review and opinion report for the proposed residential development at 62 Werrina Parade, Blue Bay (Ref. 26585W rpt Rev2.doc, dated 17 September 2013) prepared by JK Geotechnics (JKG).
  - ‘Further Clarification of Geotechnical Review, Proposed Residential Development, 62 Werrina Parade, Blue Bay (Ref. 26585W Let2.doc, dated 30 October 2013) prepared by JKG.
  - Geotechnical report for coastal management options (Part A) at Bungary Rioad, Norah Head (Ref. 14634WPrpt, dated 17 December 1999) prepared for Gary Blumberg & Associates Pty Ltd (GBA) by Jeffery and Katauskas Pty Ltd (J&K), now trading as JK Geotechnics (JKG).

- ‘Coastal management Options Part A: Provisional Slope Stabilisation Management Strategy prepared for Wyong Shire Council by GBA.
- Geotechnical investigation report for proposed drainage for slope stabilisation at Bungary Road, Norah Head (Ref. 14634WP2rpt, dated 30 January 2002) prepared for Wyong Shire Council by J&K (now trading as JKG).
- Numerous reports prepared during the installation of the slope drainage works (including on-going groundwater level monitoring) at Bungary Road, Norah Head and responses to Council in relation to perceived geotechnical issues and concerns raised by SCE. The letter and fax reports were prepared between early 2002 and mid 2007.

- The references listed below:
  - Crozier, PJ and Braybrooke, JC (1992) “The morphology of Northern Sydney’s rocky headlands, their rates and styles of regression and implications for coastal development” pub proc 26th Newcastle Symposium.
  - Delaney, MG (2005) “Coastal Cliff line Stability and Regression in the Newcastle Region” Australian Geomechanics Vol 40, No1 March 2005 pp29-38.
  - Nielsen, AF, Lord, DB and Poulos, HG (1992) “Dune Stability Considerations for Building Foundations” Australian Civil Engineering Transactions, Institution of Engineers Australia, Volume CE34, No. 2, June, pp. 167-173.

4 SUBSURFACE CONDITIONS

Reference to the 1:100,000 Geological Map of Gosford-Lake Macquarie, Sheet 9131, 9231 dated 2008 and SCE 2010 indicates that the study area subsurface profile includes:

- Quaternary age “marine” sands with podsols and quartz sand. Both soil profiles include sections of indurated/ cemented sands, and
- Triassic age Patonga Claystone, Tuggreah Formation and Munmorah Conglomerate.
- In addition, dykes (sub-vertical igneous intrusions) are present within the bedrock.

For the purposes of this report, the pertinent geological units along the study area are:

Indurated/Cemented Sands

Cemented sands, which have formed under high soil overburden pressures where the high pressure leads to cementation of individual grains.

Indurated sands which have formed within past dune sand deposits under high soil overburden pressures combined with surface water (containing iron and organic minerals) leaching through the sands and precipitating iron and organic minerals. Typically the leaching process forms indurated bands which are not laterally or vertically continuous due to the complex range of processes that have led to their formation. Indurated sand lenses are formed by cementation above fluctuating water tables.

The degree of cementation of the cemented and indurated sands is controlled by a range of factors including the presence and extent of overlying soils, water table fluctuations, sand properties, groundwater chemistry the presence of older underlying low permeability soils.

Patonga Claystone

The Patonga Claystone comprises interbedded lithic sandstone, siltstone and claystone which are typically thinly and very thinly bedded. The proportion of sandstone beds varies and are typically more prevalent north of Bateau Bay. The Patonga Claystone overlies the Tuggerah Formation.

Tuggerah Formation

The Tuggerah Formation comprises lithic sandstone with siltstone, claystone and conglomerate beds. The Tuggerah Formation overlies the Munmorah Conglomerate.

Munmorah Conglomerate

The Munmorah Conglomerate essentially comprises conglomerate and lithic sandstone with some sandstone, siltstone and claystone bands.

The principal joint sets within the above bedrock units are generally orientated between approximately 020^o and 040^o, 290^o and 305^o and 325^o and 330^o.

Igneous Dykes

Sub-vertical igneous intrusions that have penetrated bedrock along the pre-existing joint planes in the bedrock and therefore have similar orientations.

5 SUMMARY OF SITE OBSERVATIONS

We provide below a brief summary of the pertinent geotechnical, geological and topographical features of the various sections of the study area which are the subject of this report. Where applicable, additional comments are provided from the results of the desk top study.

5.1 Crackneck Point

From the southern end of the study area, the cliff line was initially a total maximum height of about 50m, reducing to a maximum of about 25m to 30m height to the north at the southern end of Bateau Bay (see Plate 1).

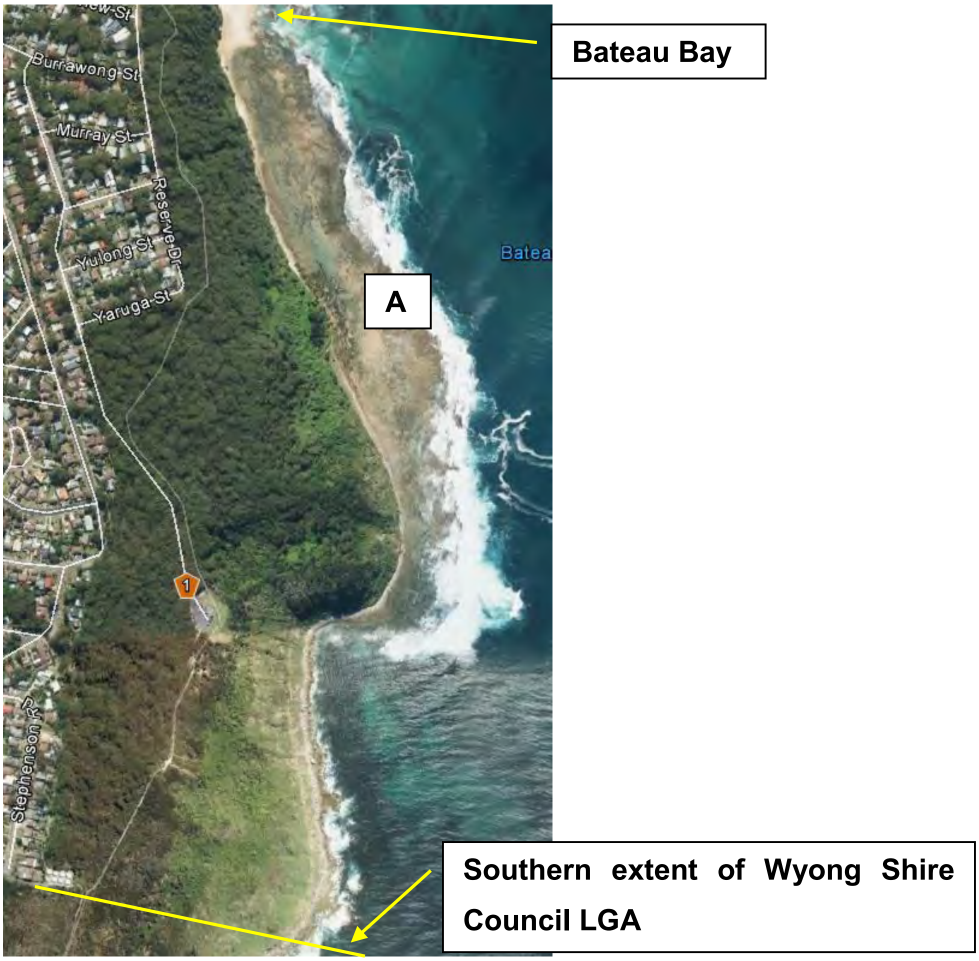


Plate 1: Crackneck Point Site Area

The base of the cliff line exposed an interbedded sequence of sandstone and claystone corresponding to the Patonga Claystone Formation. An essentially flat wave cut platform extended seaward from the base of the cliff and typically exposed conglomerates corresponding to the Tuggerah Formation.

The base of the cliff comprised a sub-vertical rock face ranging from between about 1m to 15m height. The base of the cliff was typically covered with gravel, cobble and boulder sized blocks of sandstone (see Plate 2). A drape of soil with a similar size range of sandstone inclusions was also evident over much of the basal portion of the cliff face (see Plate 2). However, to the north (towards Bateau Bay), the frequency of sandstone blocks significantly reduced.

Plate 2: Southern Portion of Crackneck Point Cliff Line

Above the basal portion of the cliff, the foreshore slope profile was characterised by a steeply sloping vegetated area (typically 45^{o}) that extended up to a wooded bushland area. Over the southern portion of this section if the study area, the steep slopes typically comprised a stepped profile with sub-vertical bedrock outcrop faces and a variable thickness of soil cover with numerous cobbles and boulders of sandstone evident (see Plate 2).

At location '**A**' (see Plates 1 and 3) the cliff line was absent at the landward edge of the wave cut platform. This area of the foreshore comprised a low lying, gently sloping vegetated area, with what appeared to be a cliff line set-back at least 50m landward of the toe of the foreshore slope. This portion of the foreshore may well represent an area of previous landslide activity.

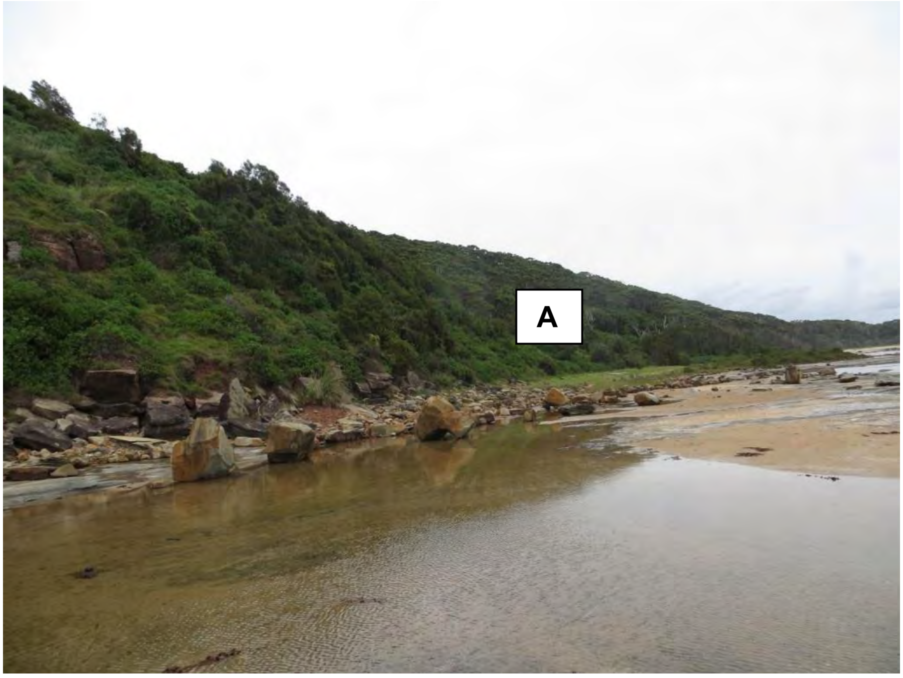


Plate 3: Looking North To Location 'A' (low lying area where cliff face absent)

North of location '**A**' the base of cliff was typified by a 1m to 8m (typically less than 5m) high rock face, with what has been interpreted to be a moderately steep (maximum about 45^{o}) residual/colluvial bushland slope above.

The sub-vertical planar jointing within the rock mass was typically orientated at approximately 330^{o}, 290^{o} and 040^{o}.

Where exposed, immediately above the wave cut platform, the rock face at the base of the cliff typically comprised a claystone band (at least about 0.5m thick) with sandstone above. The claystone had been eroded and undercut the sandstone above. The overlying sandstone 'roof' had clearly collapsed in places and detached on sub-horizontal bedding planes to form 'platy' cobble and boulder sized sandstone blocks (see Plate 4).

Plate 4: Claystone Eroding and Undercutting Sandstone Above

5.2 <u>Bateau Bay</u>

This area of the foreshore was typified by a crescent shaped bay feature (see Plate 5) immediately to the north of the Crackneck Point site area. The crest of a steep curved slope defined the seaward margin of the vegetated reserve areas and car park which lined the crest of the slope.

Plate 5: Bateau Bay Site Area

Adjacent to selected walkways down the steep slope, sub-vertical outcrop faces exposed cemented/indurated sand which was generally assessed to be of very low strength (based on striking with a geopick).

Plate 6: Indurated/Cemented Sand

The overall foreshore slope was thickly vegetated with an uneven surface and had a concave cross sectional profile. Small to medium sized trees on the slope had considerable downslope leans (see Plate 7). The upper portion was typically sloping down to the south-east at a maximum of about 60^o then reducing to less than 10^o at the toe of the slope. The toe of the slope defined the landward margin of the beach. The steep section of the slope occasionally exposed what appeared to be the indurated /cemented sand.

The foreshore slope has been interpreted to represent a landslide area (based on the cross sectional slope profile, the uneven slope surface and downslope lean of threes) which may well comprise several smaller landslides over this site area.

Plate 7: Upper Steep Section of Slope (Looking North-East)

The northern margin of the beach comprised a sub-vertical sandstone cliff face typically about 3m high with a steep (maximum about 45°) vegetated slope above with what appeared to be traces of sub-vertical sandstone outcrop faces (see Plate 8).

Plate 8: Cliff Face at Northern End of the Beach

5.3 Blue Lagoon

This area of the foreshore was typified by sub-vertical cliff face which extended north from the northern end of Bateau Bay to a crescent shaped low lying area occupied by the Blue Lagoon Beach Resort (see Plate 9).

Plate 9: Blue Lagoon Site Area

The northern margin of the Bateau Bay beach comprised a sub-vertical sandstone cliff face with claystone interbeds maximum about 5m high with a steep stepped profile (see Plate 10). The claystone had been eroded and was undercutting the sandstone above. The sandstone blocks scattered along the toe of the cliff have been derived from localised collapse of the undercut sandstone beds with the block sizes controlled by the spacing and orientation of the sub-vertical joint planes; typically orientated at approximately 330°, 290° and 040°. The approximately east-west joints typically had a lateral spacing of about 0.5m (see Plate 10).

Plate 10: Cliff face Southern End of Blue Lagoon

Plate 11: Sub-Vertical Soil Face North of Location 'B'

North from the southern end of Blue Lagoon, above the cliff face a steep (maximum about 45^{o}) vegetated slope was present. North from location '**B**' (see Plate 9), the cliff face was absent and the vegetated slope extended down to a 1m to 2m sub-vertical face which lined the landward margin of the beach surface. At location '**B**' what appeared to be a localised small scale rock fall deposit was observed. North of location '**B**', to the southern boundary of the Blue Lagoon Beach Resort, the sub-vertical soil face exposed what appeared to be colluvial or residual clay soils (see Plate

11). The seaward margin of the Blue Lagoon Beach Resort comprised a vegetated moderately steep sand slope (maximum 1.5m high).

5.4 Shelly Beach to Little Bay

This area of the foreshore was located over the northern end of Shelley Beach and extended north to the Surf Life Saving Club (SLSC) at Little Bay (see Plate 12).

Plate 12: Shelley Beach Site Area

South from the SLSC, the foreshore slope initially comprised a moderately steep (25º to 30º) vegetated colluvial clay soil slope which extended south to a sub-vertical sandstone bedrock face about 2m to 3m maximum height. The vegetated colluvial soil slope extended landward from the crest of the rock face (see Plate 13).

Plate 13: South of SLSC

The sandstone bedrock face was generally overgrown and only intermittently exposed southwards to location '**C**' where a stormwater culvert discharged down the stepped bedrock profile (see Plate 14). Thin claystone bands were evident in the sandstone bedrock. Sub-vertical joint planes were evident in the bedrock and typically orientated at approximately 020°, 110° and 045°.

To the north and south of location '**C**' the crest of the slope was lined by Towoon Bay Holiday Park (see Plate 12).

Plate 14: Below Towoon Bay Holiday Park at Location 'C'

A moderately steep (15° to 20°) vegetated sand dune slope extended south from location '**C**' with no bedrock outcrops evident.

An essentially flat wave cut platform extended seaward from the base of the foreshore cliff and typically exposed sandstone assessed to correspond to the Tuggerah Formation.

The toe of the northern portion of the foreshore slope was covered with numerous cobble and boulder sized sandstone blocks. To the south, the number of sandstone cobbles and boulders reduced and the wave cut platform had a thin sand covering (see Plate 14).

5.5 Towoon Bay to Blue Bay

North from the Little Bay SLSC, this portion of the foreshore generally comprised moderately steep vegetated sand dunes. The rear yards of private residences typically occupied the upper portions of the sand dune slopes. Two bays formed this portion of the foreshore; Towoon Bay and Blue Bay (see Plate 15) and a small headland feature separated the two bays.

Plate 15: Towoon Bay and Blue Bay Site Area

The following features were observed along this section of the foreshore:

- Location '**D**': this sandy foreshore slope was steep and, where observations were possible through the vegetation, the slope surface was typically concave and has been interpreted to represent an area of previous slope instability. This foreshore area corresponded to the area covered by the No. 2 and No. 4 Rolls Avenue DP geotechnical reports provided by Council (see Section 2, above).

Plate 16: Location 'D' Below Rolls Avenue

- Location '**E**': a short length of sandstone bedrock outcrop (maximum height about 5m) was present at the landward margin of the beach this sandy foreshore slope (see Plate 17). Sub-vertical joint planes orientated at about 040^o and 325^o were recorded. This foreshore area corresponded to the area covered by the JK and DP geotechnical reports and WP coastal engineering reports along Werrina Parade (60, 62 and 64). Buried cliff lines below the sandy slopes have been interpreted to influence the formation of the small headland feature defining the boundary between Towoon Bay and Blue Bay. In this regard, we note that a bedrock wave cut platform was evident on the aerial photograph for this area (see Plate 15).

Plate 17: Location 'E' Below Werrina Parade

- Location '**F**': the northern end of Blue Bay was defined by an approximately east-west trending gully feature. The northern slope of the gully extended east to the south-eastern section of the headland feature. The headland was characterised by a 0.5m to 2m high sub-vertical sandstone bedrock face with claystone interbeds. A steep (maximum 45^{o}) vegetated slope extended landward from the crest of the rock face.

- A stepped bedrock wave cut platform (with a thin sand cover) extended seaward from the toe of the headland rock face. The sandstone blocks scattered along the toe of the rock face were derived from localised collapse of undercut sandstone beds formed due to the preferential erosion of the weaker claystone beds (see Plate 18).

Plate 18: Blue Bay Headland

5.6 <u>The Entrance</u>

The site area extended north from the rock pool to the Marine Parade foreshore area (see Plate 19).

Plate 19: The Entrance Site Area

A sandstone cliff face was present to the south of the rock pool which was located over the northern end of the wave cut platform (see Plate 19). The cliff face reached a maximum height of about 6m immediately to the south of the rock pool then reduced in height to the north of the rock pool and was not observed north of the SLSC. However, historical photographs from 1974 indicated that a cliff face (approximately 3m to 4m high) was present immediately below and to the north of the SLSC and had been eroded by severe coastal erosion.

South of the rock pool, the sandstone had been eroded to form an overhang (see Plate 20) and, adjacent to the southern end of the rock pool, was supported by a concrete pillar.

Plate 20: The Entrance, cliff face south of the rock pool

Sand dune slopes extended from the northern end of the SLSC to about location '**G**' (see Plate 19) where a maximum 1m high sandstone bedrock face was noted at the toe of the grass surfaced moderately steep concave slope (see Plate 21).

Plate 21: Slope below the eastern end of Marine Parade

To the north-west of location '**G**' the foreshore essentially comprised a sandstone boulder wall (maximum height about 2m) which lined the toe of a moderately steep grass surfaced slope which extended down from the northern side of Marine Parade. The boulder wall was founded on the wave cut platform which exposed sandstone bedrock (see Plate 22).

Plate 22: Sandstone Boulder Wall Below Marine Parade

5.7 <u>Soldiers Beach</u>

The site area included the headland which marks the boundary between Soldiers Beach to the south-west, and Pebbly Beach to the north (see Plate 23).

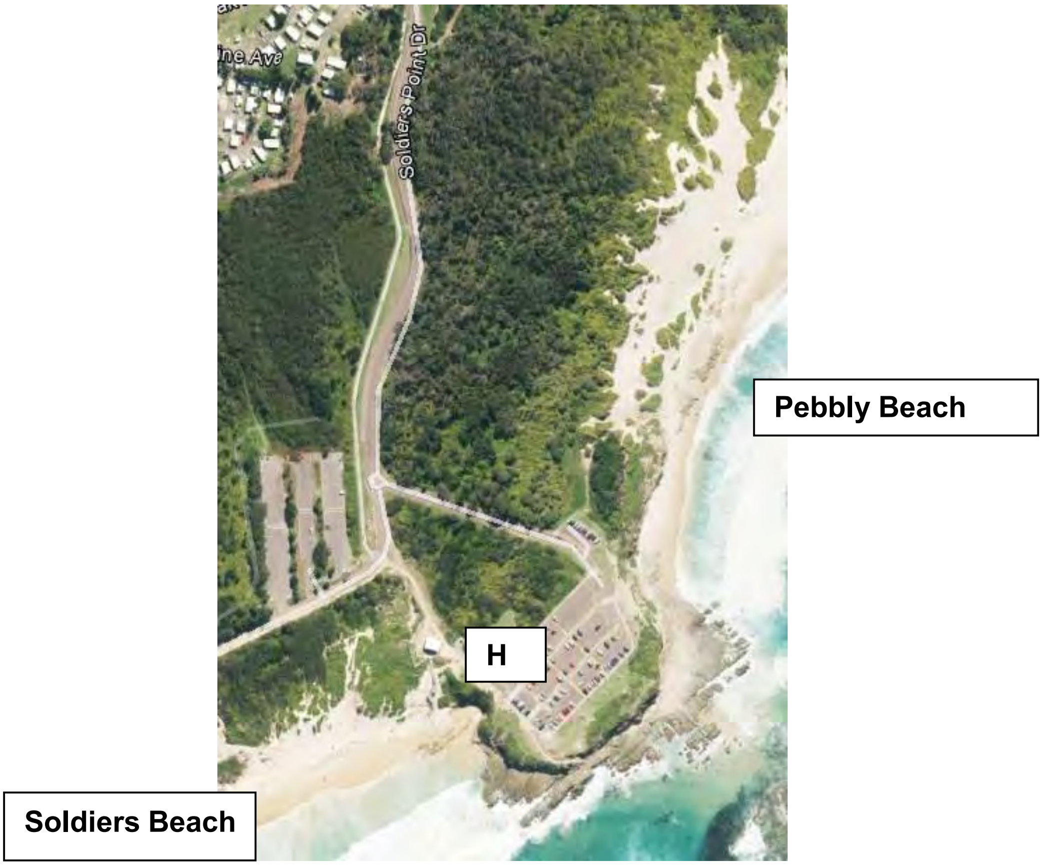


Plate 23: Soldiers Beach site area

The headland cliff face exposed interbedded sandstone, claystone and conglomerates of the Munmorah Conglomerate and ranged between maximum heights of about 6m (south-western side) and 10m (north-eastern side). A wave cut platform lined the base of the cliff face (see Plates 23 and 24).

Plate 24: Soldiers Beach headland (looking north-east)

The cliff face and wave cut platform exposed planar sub-vertical joints orientated approximately 110^{o} and 030^{o}. On the south-western face of the headland, the claystone band had been locally eroded to form an undercut approximately 2m long, 1m high and with a maximum 'depth' of about 1m (see Plate 25).

Plate 25: Undercut in cliff face and colluvium to the west (Location 'H')

To the west of the undercut (location '**H**'), what appeared to be an old rock fall area comprising clayey colluvial soil with cobble and boulder sized inclusions of sandstone and conglomerate was present (see Plate 25). The cliff line (in plan) was characterised by a small arcuate area which probably represents the old back scarp of the landslide (see Plate 23).

From the slope area around the timber stepped access, the bedrock face was typically about 1m high with a moderately steep (maximum 27°) sand dune slope above.

Plate 26: Bedrock face and sand dune slope (looking south-west)

Plate 27: Cliff face, looking north to Pebbly Beach

The cliff face over the northern side of the headland comprised a lower sub-vertical portion and an upper portion with an overall slope of approximately 45^o which included sub-vertical faces, typically at least 1m high (see Plate 27). The upper slope has been interpreted to represent bedrock with a thin residual and/or colluvial soil cover formed by differential weathering and erosion of the interbedded (softer) claystone and the more resistant sandstone and conglomerate bands.

5.8 Pebbly Beach

The site area included the beach between the headlands at the southern end (Soldiers Beach) and north-eastern end of the site (see Plate 28).

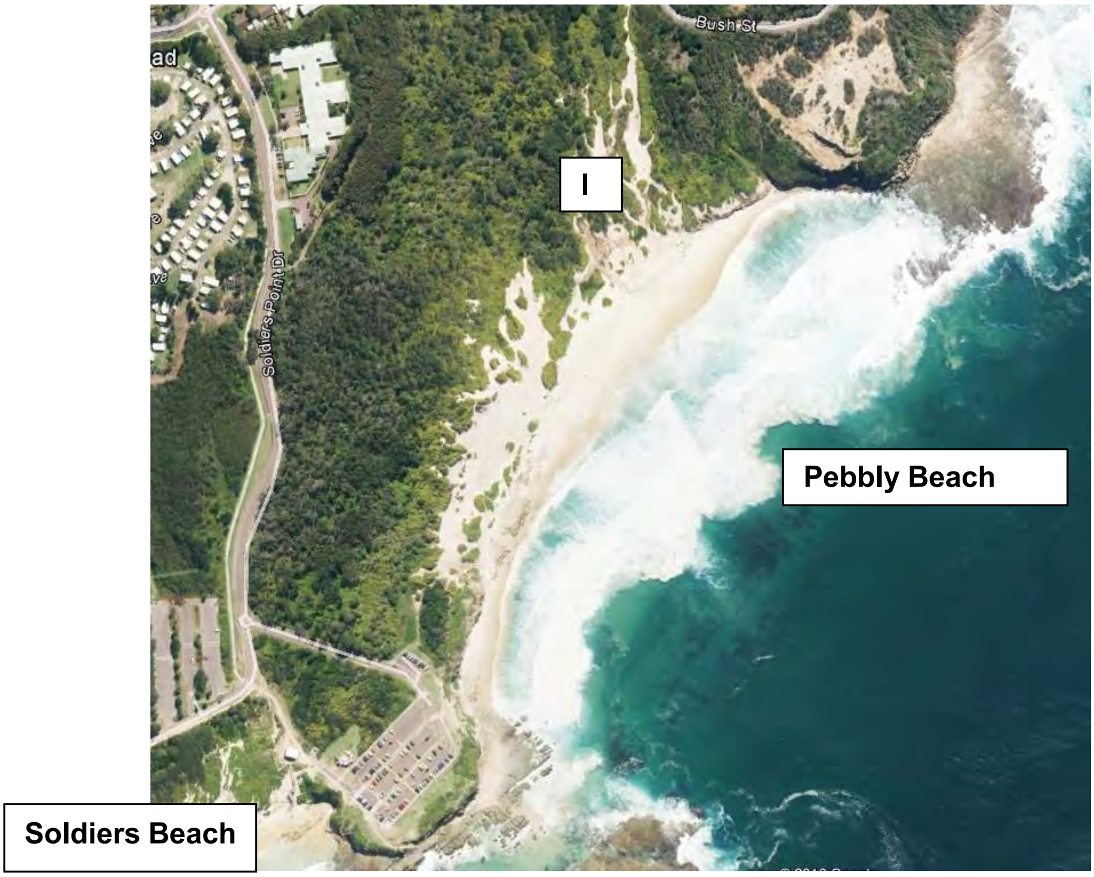


Plate 28: Pebbly Beach site area

The landward (western) side of the beach was lined by a steep (maximum 35^o) sand dune slope which extended to a maximum height of about 30m above the beach surface (see Plate 27). The lower portion of the sand dune slope had a stepped profile with a lower 15^o to sub-vertical section about 2m maximum height at the toe (see Plate 29).

Plate 29: Southern portion of Pebbly Beach

The southern portion of the beach area comprised a stepped sandstone and conglomerate bedrock surface corresponding to the Munmorah Conglomerate, with vertical faces of maximum 2m height (see Plates 29 and 30).

Plate 30: Northern portion of Pebbly Beach

Over the northern portion of the sand dune slope (location '**I**') a creek line discharged at the landward side of the beach (see Plates 28 and 30). The creek sides exposed sandstone and conglomerate which was typically extremely weathered with a thin residual clay cover overlain by the sand dune slope (see Plate 31).

Plate 31: Creek line at Location 'I' exposing extremely weathered Munmorah Conglomerate

The northern end of the beach was defined by a maximum 5m high sub-vertical cliff face which also exposed sandstone and conglomerate of the Munmorah Conglomerate and was controlled by a planar joint orientated at approximately 300^o.

The sandstone beds were preferentially eroding relative to the conglomerate beds and the erosion had formed a series of undercuts (maximum 1.5m height above the beach surface) which extended back a maximum 'depth' of 3m (see Plate 32).

Plate 32: Erosion of Munmorah Conglomerate at northern end of the beach

The cliff face extended round to the north and formed the southern margin of the following Norah Head site area (see Section 4.9, below). The sandstone band at the base of the cliff face had preferentially eroded to form an intermittent undercut (maximum height about 1m, maximum ‘depth’ about 1m). Numerous rounded to angular cobble and boulder sized blocks of sandstone and conglomerate derived from previous rock falls covered the base of the cliff (see Plate 33).

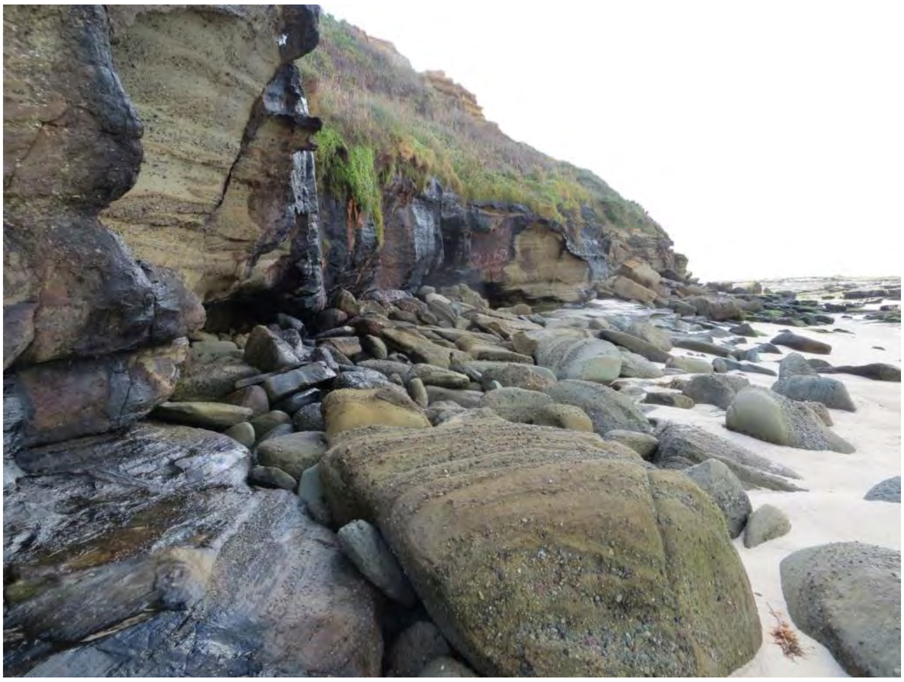


Plate 33: Blocks covering the base of the cliff and undercuts (looking north)

5.9 Norah Head

The site area included the cliff face which extended north-east from Pebbly Beach around the headland and west along Lighthouse Beach to the rock pool at the eastern end of Cabbage Tree Harbour (see Plate 34).

Plate 34: Norah Head and Lighthouse Beach site area

The cliff face exposed interbedded sandstone, claystone and conglomerates of the Tuggerah Formation (with Munmorah Conglomerate possibly exposed in places at the base of the cliff) and ranged between maximum heights of about 10m (south-western side) and 6m (north-eastern end of the headland, below the lighthouse). A wave cut platform assessed to comprise the Munmorah Conglomerate lined the base of the cliff face (see Plate 34).

Northwards from the Pebbly Beach headland, the cliff face was typically vegetated with sub-vertical bedrock faces exposed. Overall slope angles ranged between about 25^{o} and 60^{o} with the less steep areas inferred to comprise an increased cover of residual or colluvial clay soils and dune sands. Sub-vertical igneous dykes crossed the cliff face and wave cut platform and were orientated approximately 020^{o} and 305^{o}, i.e. similar orientations to the sub-vertical planar joints evident in the cliff faces.

At the north-eastern end of the headland, blocks of conglomerate were observed to be detached from the cliff face along sub-vertical joint planes orientated approximately 305° (see Plate 35).

Plate 35: Conglomerate blocks detached from the cliff face

Plate 36: Undercut conglomerate and blocks covering the base of the cliff

A claystone band was noted at the base of the blocks and had been eroded to form undercuts (see Plate 36). Numerous cobble and boulder sized angular blocks of conglomerate were evident along the base of the cliff, presumably derived from collapse of previously detached and undercut blocks of conglomerate. The approximately 020º orientated joints also influenced the size of the blocks (see Plate 36)

The Lighthouse Beach site area extended west from the cliff face headland at Norah Head (see Plate 34). The foreshore area essentially comprised a moderately steep (maximum approximately 30º) vegetated slope which extended down to the beach. Towards the western end of the beach occasional short lengths of sandstone cliff face were exposed along the vegetated slope and a sandstone bedrock wave cut platform was present (see Plate 37). The sandstone has been assessed to correspond to the Tuggerah Formation.

Plate 37: Western end of Lighthouse Beach

5.10 Cabbage Tree Harbour

The site area included the section of foreshore which extended west, then north, from Lighthouse Beach to the headland at the northern end of the bay which marks the southern end of Jenny Dixon Beach (see Plate 38).

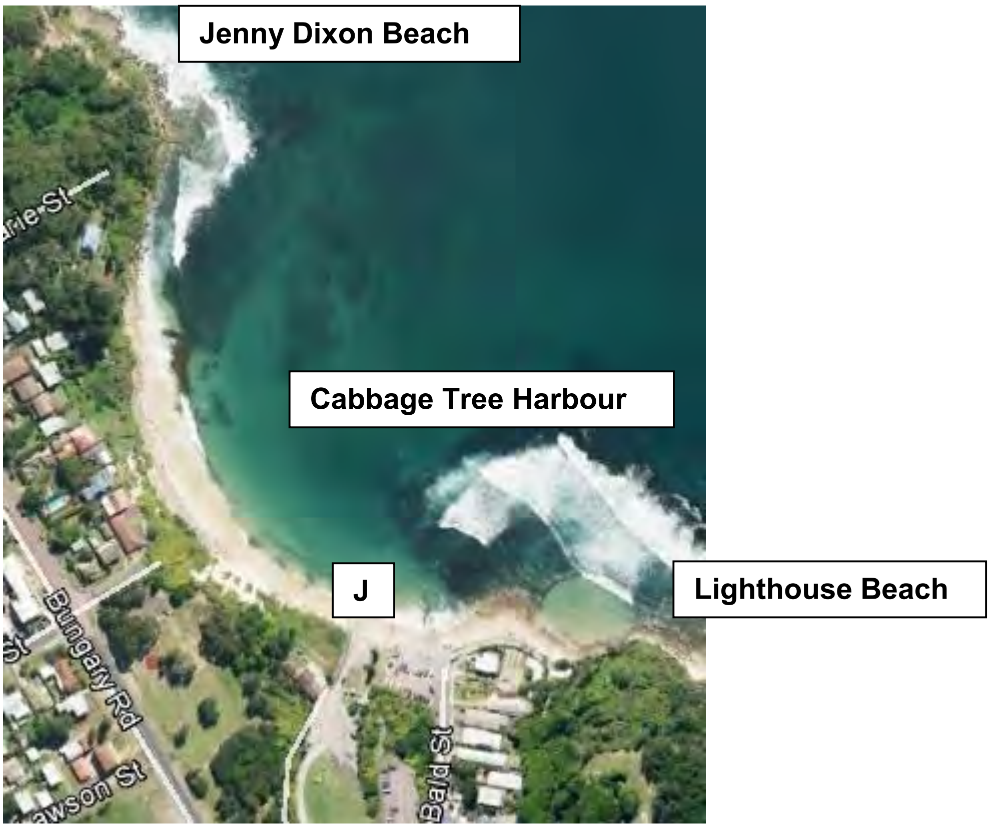


Plate 38: Cabbage Tree Harbour site area

West of Lighthouse Beach the stepped sandstone bedrock wave cut platform was assessed to correspond to the Tuggerah Formation. The wave cut platform extended west towards the concrete boat ramp area which had been provided with igneous boulder 'rip-rap' erosion protection (see Plate 39). The vegetated slope above the wave cut platform adjacent to Lighthouse Beach comprised residual clayey soils which were exposed in the sub-vertical face (1.5m to 3m high) present at the toe of the slope.

West of the boat ramp at Location '**J**' an approximately 18m long landslide feature was noted within the moderately steep (maximum 30°) vegetated slope below the surf club building (see Plate 39). The toe of the landslide (at the landward side of the beach) was sub-vertical, presumably as a response to wave erosion. Traces of what appeared to be residual sandy clay were exposed at the eastern end of the base of the landslide.

Plate 39: Landslide at Location 'J'

To the west of Location '**J**', the foreshore was dominated by a sub-vertical face, approximately 10m high, which exposed silty sand and clayey sand (see Plate 40). The sands were typically moist and wet at the toe of the slope where debris from previous slope instability had collected. Striated and smooth surfaces in the sands sloped down to the north at between about 60^{o} and 80^{o}, and have been interpreted to indicate previous slip planes. A reserve area and car park lined the crest of this section of foreshore.

Plate 40: Unstable sub-vertical sand face

West of the sub-vertical sand face, a grass surfaced concave slope extended down from the rear yards of residences along Bungary Road. Igneous boulder 'rip-rap' erosion protection had been placed along the toe of the slope (see Plates 40 and 41). The upper portion of the slope was typically sub-vertical and exposed light grey and light brown sands.

Plate 41: North-Western end of Cabbage Tree Harbour

The vegetated sand slopes extended north to the sandstone bedrock cliff face forming the southern side of the headland which marked the southern end of Jenny Dixon Beach. The cliff face increased in height to a maximum of about 15m at the headland with a steep vegetated slope above the cliff face. Traces of orange brown sandstone faces were evident over the cliff face (see Plates 42 and 43) which may represent areas of more recent cliff face rock falls.

Plate 42: Cliff Face at the western end of Cabbage Tree Harbour

'Blocky' sandstone boulders were evident on the sandstone bedrock wave cut platform at the toe of the cliff face (see Plate 43).

Plate 43: Cliff Face at the western end of Cabbage Tree Harbour

5.11 Jenny Dixon Beach

The site area included the cliff face which extended north from the headland at the northern end of Cabbage Tree Harbour to the headland separating Jenny Dixon Beach from Hargraves Beach to the north (see Plate 44).

The cliff faces comprised sandstone with claystone and conglomerate bands assessed to be the Tuggerah Formation. At the southern end of the site area the headland cliff face was about 15m high and reduced in height to 4m over the central portion of the cliff face. The cliff faces were defined in places by a sub-vertical planar joints orientated at about 325^{o}. The crest of the sub-vertical portion of cliff face was overhanging in places. A wave cut platform lined the base of the headland cliff face and 'blocky' sandstone boulders were evident on the wave cut platform (see Plate 45).

Above the sub-vertical cliff face the vegetated slope was typically steep (maximum about 45^{o}) with occasional sub-vertical faces of bedrock evident. The steep slope has been interpreted to represent a more deeply weathered bedrock profile with a thin cover of residual and/or colluvial soils.

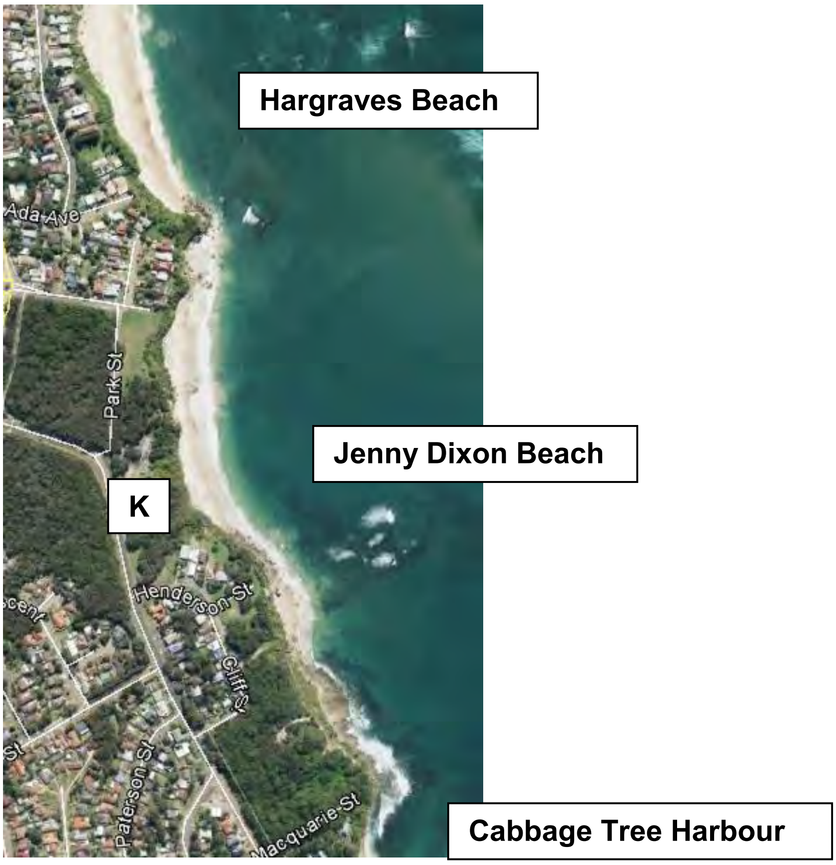


Plate 44: Norah Head and Lighthouse Beach site area

Plate 45: Headland at southern end of Jenny Dixon Beach

Over the central portion of the site area (Location '**K**') the cliff face was absent although the wave cut platform was still evident immediately below the beach surface. The vegetated slope was a maximum of about 30^o and has been interpreted to represent an area of previous cliff face instability (see Plate 46).

Plate 46: Area of previous cliff face instability (Location 'K')

To the south of Location '**K**' the cliff face was eroding back landward to a sub-vertical planar joint and blocks had locally collapsed from the face (released on sub-horizontal bedding partings and the sub-vertical joint planes orientated approximately 325^o and 040^o) leaving a remnant overhang at the crest of the cliff (see Plate 47).

Plate 47: Area of previous cliff face instability (Location ‘K’)

North from Location ‘**K**’ the cliff face increased in height to a maximum height of approximately 20m at the headland at the northern end of the beach.

5.12 <u>Hargraves Beach</u>

The site area included the cliff face which extended north from the headland at the southern end of Hargraves Beach (see Plate 44). North from the headland, the cliff face reduced in height from 20m to typically 4m to 5m, and was absent at a point approximately opposite 19 Elizabeth Drive (see Plate 48). The cliff face comprised sandstone with claystone assessed to be the Tuggerah Formation.

The cliff face was predominantly overgrown with traces of bedrock faces evident. A localised rock fall (3m x 2m x 0.8m thick) controlled by sub-vertical planar joint orientated at about 305^{o} was noted in a section of the 4m to 5m high portion of cliff face.

Above the sub-vertical cliff face the vegetated slope was typically steep (maximum about 45^{o}) with traces of residual clay soils evident through the vegetation. The southern end of the overgrown sub-vertical face, which has been interpreted to comprise bedrock, was orientated approximately 280^{o} and extended landward. To the north of the sub-vertical face the beach surface extended north.

Plate 48: Looking south to the headland at the southern end of Hargraves Beach

6 EVALUATION OF CLIFF LINE RECESSION RATES

We have re-assessed the recession rates for the foreshore cliff lines along the study area. It is our interpretation that the rates adopted in the CZMP have been conservatively rounded up from two historical rates of recession applicable to the specific geology (lithology or rock types) at the various cliff lines derived by SCE from examination of aerial photographs (see SCE 2010). We have considered other published data, and derived rates of recession based on the existing wave cut platforms.

6.1 Recession Rates Adopted By SCE For CZMP

As part of the SCE study for the CZMP, SCE examined historical rates of recession. The results are summarised in Table B4 of SCE 2010 and have been included in the attached Table A.

SCE identified that the presence of dykes (sub-vertical igneous intrusions) within the bedrock had an effect on the recession mechanism and hence the rate of recession. The underlying geology was also a determining factor for the rate of recession. As noted in Section 4 above, the pertinent geological units along the study area are:

- Indurated/Cemented sands
- Patonga Claystone
- Tuggerah Formation

- Munmorah Conglomerate

With regard to the data presented in Table B4 of SCE 2010, we note the following:

- At the Ocean Parade Headland, adjacent to Location '**F**' on Plate 15, (Patonga Claystone) SCE found there was no measurable recession over the 21 year interval appropriate to the aerial photographs being considered and the 1985 orthophoto map. We consider this is consistent with site observations during our walk over survey as there was no evidence of recent instability other than the presence of small isolated detached blocks at the toe of the cliff face.

- At Crackneck Point, indicated on Figure 1, (Patonga Claystone) the SCE measured recession ranged between 2m and 10m and was based on the 1985 orthophoto map and a comparison of the 21 year interval between the 'Google Earth' image and an historical image. However, we note that our recent observations at the southern end of Bateau Bay did not indicate evidence of recent instability other than the presence of small isolated detached blocks at the toe of the cliff line. The SCE assessed recession rate of 10m over the 21 year interval may be regarded as a conservative assessment as it appeared to be at variance with our site observations.

- At Norah Head, indicated on Figure 1, (Tuggerah Formation) the SCE measured recession ranged between 14m and 20m based on a comparison of the 130 year interval with a village plan from 1878 and a comparison of the Google Earth image and an historical image. However, SCE found there was no measurable recession over the 21 year interval appropriate to the aerial photographs being considered and the 1985 orthophoto map. We consider the no measurable recession is more consistent with site observations during our walk over survey as there was no evidence of recent instability other than the presence of small isolated detached blocks at the toe of the cliff face.

- At Jenny Dixon Beach, indicated on Figure 1, (Tuggerah Formation *capped with* Patonga Claystone) the SCE measured recession ranged between 16m and 26m based on a comparison of the 130 year interval with a village plan from 1878 and a comparison of the 'Google Earth' image and an historical image. The SCE measured recession ranged between 5m and 10m when based on the 1985 orthophoto map and a comparison of the 21 year interval between the Google Earth image and an historical image. However, we note that our recent observations only indicated evidence of localised areas of recent instability and the presence of detached blocks at the toe of the cliff face was more typical. The SCE assessed recession rate of 10m over the 21 year interval may be regarded as a conservative assessment as it appeared to be at variance with our site observations.

- At Henderson Street, Norah Head (Tuggerah Formation *capped with* Patonga Claystone) the SCE measured recession ranged between 10m and 15m based on a comparison of the 130 year interval with a village plan from 1878 and a comparison of the Google Earth image and an historical image. However, SCE found there was for no measurable recession over the 21 year interval appropriate to the aerial photographs being considered and the 1985 orthophoto map. We consider the no measurable recession is more consistent with site observations during our walk over survey as there was no evidence of recent instability other than the presence of small isolated detached blocks at the toe of the cliff face.

- At Cabbage Tree Harbour (Indurated/Cemented sands) the SCE measured recession was 34m based on a comparison of the 130 year interval with a village plan from 1878 and a comparison of the Google Earth image and an historical image. The SCE measured recession ranged between 4m and 7m when based on the 1985 orthophoto map and a comparison of the 21 year interval between the Google Earth image and an historical image. However, we note that this section of the study area represents a complex geological and geotechnical area that has, and continues, to be affected by landslides. Any recession rate for Indurated/Cemented sands for this area would need to be carefully reviewed if it was to be applied to different section of the study area.

We have examined the cliff faces at Crackneck Point, Norah Head, Jenny Dixon Beach and Henderson Street, Norah Head and our site observations have indicated that only discrete sections of these areas suggest increased rates of erosion (e.g. areas of suspected or documented landslides). We note that SCE 2010 provides no further information and we therefore have not had the opportunity to understand the extent and nature of the recession that has been measured from the aerial photograph comparison carried out by SCE. This leads to uncertainty as to the location and extent of the recession referred to by SCE. For example;

- Is the recession confined to particular localised areas of instability (say about 20m length along the coast) or is the recession evident along significant lengths of cliff line (e.g. 1000m)?
- Is the recession confined to the upper crest area of the cliff line or evident only at the toe of the cliff line, or is the recession over the full height of the cliff line?
- Is the recession within predominantly claystone or sandstone strata within the Patonga Claystone beds?
- The data shown on Table B4 of SCE 2010 for Jenny Dixon Beach, Henderson Street, Norah Head and Norah Head indicates that igneous dykes are present within the geological units and that the recession rates are likely to have been increased as a consequence.

- Cabbage Tree Harbour represents a complex geological and geotechnical section of the study area that has, and continues, to be affected by landslides primarily triggered by elevated groundwater levels and exacerbated by coastal erosion at the base of the slopes. The recession rate for this area is likely to be increased as a consequence.

In the absence of such additional data, the results reported in Table B4 of SCE 2010 need to be considered with caution before extrapolation and adoption for assessment of erosion hazard lines. In particular, this applies to:

- The recession rates for the Tuggerah Formation and Patonga Claystone predicted by the assessment of Jenny Dixon Beach, Henderson Street, Norah Head and Norah Head (where igneous dykes are present).
- The recession rates for the Indurated/Cemented sands at Cabbage Tree Harbour, which is impacted by a complex range of geological, hydrogeological and geotechnical factors which manifest as landslides impacting the steep foreshore slopes. Any recession rate for Indurated/Cemented sands for this area would need to be carefully reviewed if it was to be applied to a different section of the study area.

It could be argued that in these instances the recession rates are relatively localised rather than being representative of large sections of the cliff lines and steep foreshore slope areas.

We recognise that determination of historical rates of recession is a difficult task requiring care, detailed field observations and research. Of particular relevance is how observations over a relatively short time period (such as 21 years) can be extrapolated over the longer term taking in to account the episodic nature of such events, the inevitable localised nature of observations made and how they can be extrapolated to a much greater length of coast.

The data in Table B4 of SCE 2010 were also summarised in Table 4 of SCE 2010. It is apparent that Table 4 has involved an element of 'conservative rounding up' of the data from Table B4. For example:

- For Indurated/Cemented sands the "*typical value of Recorded Historical Recession*" is shown on Table 4 in SCE 2010 as "*30 – 33m*" per 100 years. This is slightly different to the data in Table B4 which might be alternatively represented as "*19m - 33m*". An average of this range (say 26m/100 years) appears to have been used for the plot on Figure B5 of SCE 2010 (see attached Figure 2) at about 1900 increasing to about 30m per 100 years by about 1990. However, beyond 1990 the recession rate substantially increases and has been postulated to increase to about 46m per 100 years by 2100 (this includes SCE's assessment of the impact of SLR and climate change on recession rates).

- For Patonga Claystone the "*typical value of Recorded Historical Recession*" is shown as "*10 – 15m*" per 100 years on Table 4 of SCE 2010. This is slightly different to the data in Table B4 of SCE 2010 (say about 12m/100 years) which might be more reasonably summarised as "*0m (?) to 10m*". The average value from Table 4 of SCE 2010 appears to have been used for the plot on Figure B5 of SCE 2010 (see attached Figure 2) at about 2007. However, we note that the shaded portion on Figure B5 does not fully reflect the range of recession rates for Patonga Claystone from Table 4 of SCE 2010, or the wider range from Table B4 of SCE 2010.

Nonetheless, we consider that if the historical rates of recession are as high as suggested in Table 4 of SCE 2010 then it would be reasonable to expect more widespread evidence of recession than appears to have been identified by SCE or by our study. This is discussed further in the following Section 6.2.

6.2 <u>Modified Recession Rates For This Study</u>

6.2.1 Field Evidence

As noted in the preceding Section 6.1, if the historical rates of recession are as high as suggested by SCE then it would be reasonable to expect more widespread evidence of recession than appears to have been identified by SCE. For example:

- Comparison of the 1941 and 1976 aerial photograph and the 2015 'Google Earth' imagery at the Swadling Reserve and Ocean Parade headlands at Towoon Bay (see Plate 15) indicates little if any discernible recession. There also appears to have been an appreciable increase in vegetative cover at Swadling Reserve.
- Comparison of the 1941 and 1976 aerial photograph and the 2015 'Google Earth' imagery at the northern end of Bateau Bay indicates little if any discernible recession although there now appears to be a greater number of blocks of sandstone along the toe of the cliff face compared to 1976.
- Comparison of the 1941 and 1976 aerial photograph and the 2015 'Google Earth' imagery at Crackneck Point indicates little if any discernible recession with the exception of some signs of loss of vegetative cover over the southern end of the cliff face crest area since 1976 and erosion/removal of blocks of bedrock from the toe of the cliff face. There also appears to have been an overall increase in vegetative cover since 1941.
- Comparison of the 1941, 1976 and 1978 aerial photograph and the 2015 'Google Earth' imagery at Norah Head, Pebbly Beach and Soldiers Beach indicates little if any discernible recession

with the exception of some signs of beach erosion and possible localised cliff face recession over the south facing section of headland at the northern end of Pebbly Beach. There also appears to have been an overall increase in vegetative cover since 1941.

- Comparison of the 1976 aerial photograph and the 'Google Earth' imagery between 2005 and 2015 at Bateau Bay indicates little if any discernible recession. However, buildings located at the northern end of the beach in 1976 were not present by 2005. There also appears to have been an overall increase in vegetative cover since 1976.

The above indicates that in the main negligible recession over at least a 39 year period is typical. This is supported by the headland profiles along the study area which typically show a convex slope from higher elevations terminating in a steeper toe slope at the base. The presence of slopes flatter than at the toe are consistent with a rate of recession at the toe which is sufficiently slow to allow the slope forming processes in soils at higher elevations to dominate, rather than 'faster recession at the toe' which would be expected to form a more extensive/higher steep cliff line.

By adopting the SCE CZMP typical recession rates presented in (Table 4 of SCE 2010) we note the following:

- For the Indurated/Cemented Sands, a recession rate of 30m to 33m per 100 years, which is considered to be conservative and impacted by localised land slips (i.e. Cabbage tree Harbour), a period of about 8 years would typically be required for the assumed 2.5m loss presented on Drawing No. BM001G21 of SCE 2010 and assuming the recession is at a 'uniform rate' over time.
- For the Patonga Claystone, a recession rate of 10m to 15m per 100 years, which is considered to be conservative, a period of between about 20 years and 33 years would typically be required for the assumed 3m loss Drawing No. BM001G22 of SCE 2010 and assuming the recession is at a 'uniform rate' over time
- For the Tuggerah Formation, an upper recession rate of 8m per 100 years, which is considered to be conservative, a period of about 50 years would typically be required for the assumed 4m loss Drawing No. BM001G23 of SCE 2010 and assuming the recession is at a 'uniform rate' over time
- For the Tuggerah Formation where igneous dykes are present, a recession rate of 10m to 16m per 100 years, which is considered to be conservative, a period of between about 25 years and 40 years would typically be required for the assumed 4m loss Drawing No. BM001G23 of SCE 2010 and assuming the recession is at a 'uniform rate' over time

6.2.2 Cyclical Recession Episodes

It is recognised that recession would not occur at a uniform rate with time, but is cyclical, occurring in 'bites' that only occur infrequently and typically influenced by:

- The defect spacing (joints and bedding planes) within the rock mass
- Erosion of the softer siltstone/claystone beds leading to undercutting.
- Eventual toppling of the overlying sandstone with detachment along sub-vertical joint planes.

Looking at the loss of bedrock in another way, it would require an 'unusual' storm event to trigger a loss given the above mechanism. Our site observations have indicated that toppling over extensive lengths of cliff face does not occur immediately undercutting occurs, but more typically manifests as blocks falling from the undercut section or localised sections of collapse, such as indicated on the following plates presented in Section 5, above:

- Plate 4: Crackneck Point.
- Plate 8: Bateau Bay.
- Plate 10: Blue Lagoon.
- Plate 18: Blue Bay.
- Plate 20: The Entrance.
- Plate 25: Soldiers Beach.
- Plates 32 and 33: Pebbly Beach.
- Plates 35 and 36, Norah Head,
- Plate 47: Jenny Dixon Beach.

Therefore, there needs to be a process of progressive undercutting combined with a trigger event to cause the failure.

However, the scale of the failure needs to be sufficiently extensive along the cliff face so that it extends upslope as well as along the cliff face in order to affect a considerable portion of the foreshore. That is, in other words, a localised failure is less likely to affect a site landward of the existing cliff face than a larger scale failure. There has been no study reported in SCE 2010 of the scale of likely undercutting and failure.

We also note that for the SCE lower bound 10m/100year recession rate for the Patonga Claystone currently presented in the CZMP, since the 1941 aerial photographs (74 years), at least 7.4m of erosion would have been expected to be evident along the cliff lines within the study area. On this basis it would be reasonable to assume that clear evidence of such recession would be apparent when comparing current and historical aerial photography. However, as we have outlined above, this is not the case.

6.2.3 Other Data

Other data on recession rates have been published in relation to other areas in Crozier and Braybrooke 1992 and Delaney 2005. These data are presented on the attached Table A together with notes on the appropriate geology/lithology (rock types) as these have an impact on the rates of recession that occur.

Comparable rates might be obtained from Crozier and Braybrooke 1992 for the mudstone/siltstone of Sydney Northern Beaches which have recession rates determined from rock platform widths of:

- An average 6.2mm/year (0.6m/100 years).
- A maximum of 18.1mm/year (1.8m/100 years).
- At Long Reef, the recession rate was determined as 57mm/year (5.7m/100 years).

Data in Crozier and Braybrooke 1992 from the sandstone areas would be expected to be at the low end of values for the predominantly interbedded shales and sandstone of the Tuggerah Formation and Patonga Claystone geological units within the study area. However, we note that the expectation/experience is that the Northern Beaches sandstone is more resistant to weathering than the siltstone and claystone.

Data from Table 1 in Delaney 2005 (first section of part 3 of the attached Table A) covers a variety of lithology and probably also a variety of geological ages for the strata. The data for the Newcastle area (as shown in the second section of part 3 of Table A) does not give any details of the specific lithology applicable. In general terms, the Newcastle rocks are of Permian age comprising interbedded sedimentary rocks of the Newcastle Coal Measures. Delaney 2005 also considered the rates of recession that could be determined from the rock platforms present at many headlands. Resulting recession rates were 10mm/year (1m/100 years) assuming no recession of the rock platform to 15mm/year (1.5m/100 years) assuming the rock platform also recessed at 2mm/year (0.2m/100 years).

Delaney 2005 concluded "*In general the correlation between the various methods is good with a long term cliff regression rate in the order of 10mm to 25mm per year (1m to 2.5m per 100 years) applicable for the higher interbedded cliff sections of the Newcastle coastline*."

6.2.4 Wave Cut Platform Recession Rates

Crozier and Braybrooke 1992 also used the width of the wave cut rock platforms as an indication of the rate of recession since the last sea level rise about 6000 years ago. The same approach can be used in the study area for wave cut rock platforms at Ocean Parade Headland, Blue Bay/Toowoon Bay reef, Swadling Reserve, Toowoon Bay south tombolo reef / platform, Norah Head, the north end of Bateau Bay and Crackneck Point. The relevant distances measured from 'Google Earth' aerial photographs are detailed on Table A, part 4 together with the directly derived average recession rate over 6000 years assuming there is no recession at the edge of the rock platform.

However, it would also be logical to expect that the seaward edge of the wave cut platform has also been subject to recession over the same period. The likely rate of recession of the platform edge would be expected to be similar to, or less than, the rates derived from existing cliff lines above sea level, which are assumed to form the basis of Table B4 data and hence Table 4 of SCE 2010. Based on Table 4 of SCE 2010, we have adopted edge recession rates of 1m/100years to 4m/100years for the wave cut platforms in the Tuggerah Formation. A similar range has also been adopted for the Munmorah Conglomerate, which are the appropriate rock type/group for the wave cut platforms. This assumed rate of platform recession can have a significant effect on the derived recession rate for the Patonga Claystone as indicated in part 4 of Table A.

If the Tuggerah Formation is accepted as being more resistant to recession, then the platform recession rate would be expected to be no more than the recession rate for the cliffs comprised of Patonga Claystone, particularly where they are predominantly comprise sandstone beds as revealed in the cliff face exposures over the study area.

6.2.5 Adopted Recession Rates for Bedrock

Based on the data in Table A, we consider it reasonable to adopt an average historical recession rate for design purposes of:

- 4m/100 years for Patonga Claystone (sandstone dominant), based on an average of ten minimum and maximum recession rates and with a platform recession rate of 1m/100 years.
- 3m/100 years for Tuggerah Formation, based on the Norah Head data with a platform recession rate of 1m/100 years.
- 3m/100 years for Munmorah Conglomerate, based on generally similar rock types to the Tuggerah Formation (i.e. sandstone and conglomerate, although conglomerates are more

prevalent in the Munmorrah Conglomerate) and the Norah Head data with a platform recession rate of 1m/100 years. The platform comprises Munmorrah Conglomerate at Norah head.

These recession rates make no allowance for climate change and sea level rise effects, which are addressed in Section 6.2.7, below.

The available evidence in Table A suggests these recession rates may be conservative when applied to a length of coastline rather than to specific areas of known recession, (that is, when the episodic events are averaged out over time over a length of coastline rather than taken at a specific location). The limited evidence of significant cliff face recession indicated by the comparison of aerial photographs between 1941 and 2015 supports this assessment.

Design Recession Rate For Patonga Claystone

- Over a period of 35 years (2015 to 2050), extrapolation of the above rate implies about 1.4m of recession of the cliff face comprising Patonga Claystone (with sandstone dominant).
- Over a period of 85 years (2015 to 2100), extrapolation of the above rate implies about 3.4m of recession of the cliff face comprising Patonga Claystone (with sandstone dominant).

We note that the typical joint spacing for Patonga Claystone (with sandstone dominant), is 2m which indicates that the 2050 and 2100 figures should be rounded up to 2m and 4m, respectively.

Design Recession Rate For Tuggerah Formation and Munmorah Conglomerate

- Over a period of 35 years (2015 to 2050), extrapolation of the above rate implies about 1.1m of recession of the cliff face comprising Tuggerah Formation and Munmorah Conglomerate.
- Over a period of 85 years (2015 to 2100), extrapolation of the above rate implies about 2.6m of recession of the cliff face comprising Tuggerah Formation and Munmorah Conglomerate.

We note that the typical joint spacing for Tuggerah Formation and Munmorah Conglomerate is 4m, which indicates that the 2050 figure may be conservatively rounded up to 4m and the 2100 figure should be rounded up to 4m.

Based on our site observations and review of historical aerial; photographs, there was no readily discernible increase in cliff erosion rates where igneous dykes were present. We note that the igneous dykes were recorded in the sandstones of the Tuggerah Formation. The conservative adoption of a 4m erosion rate for the Tuggerah Formation for the 2015 to 2050 and 2050 to 2100 year when compared to the calculated erosion rates for the 35 year and 85 year periods outlined above, is considered appropriate in capturing any localised increased erosion rates due to the presence of igneous dykes.

6.2.6 Adopted Recession Rates for Indurated/Cemented Sands

A review of available technical information on the engineering behaviour of indurated/cemented sands, our past experience at Cabbage Tree Harbour and information presented in SCE 2010 we note the following:

- The cementation of the sands increases their shear strength parameters (effective cohesion and internal friction angle). However, when exposed to water (wave action, surface water run-off, seepage and/or elevated groundwater levels in the subsurface profile etc.) then the cementation minerals dissolve and the shear strength parameters reduce to more typical values for sand.
- The indurated bands typically form discrete bands within past dune sand deposits and are not laterally or vertically continuous due to the complex range of processes that have led to their formation. Indurated sand lenses are formed by cementation above fluctuating water tables. The degree of cementation of the cemented and indurated sands is controlled by a range of factors including the presence and extent of overlying topsoil, water table fluctuations, sand properties, groundwater chemistry the presence of older underlying low permeability soils.

Based on the above, to account for the loss of shear strength of the cemented/indurated sands, we consider that these areas should be assessed in a similar manner as the other coastal sand areas within the study area. However, with regard to the preceding discussion of available recession rates, we note the following:

- For Cabbage Tree Harbour, whilst a recession rate of about 0.3m per year is believed to be conservative, as an acknowledgement of the complex issues affecting this portion of the coastline, we consider that this erosion rate is appropriate for this area. When applying the Nielsen et al 1992 methodology for dune stability to determine the immediate erosion hazard line, the base of the 'Zone of Slope Adjustment' should be defined by a reduced angle of repose of 30^{o}.
- For Bateau Bay there is no historical data on recession rates. When applying the Nielsen et al 1992 methodology for dune stability to determine the immediate erosion hazard line, the base of the 'Zone of Slope Adjustment' should be defined by a 'typical' angle of repose value of 35^{o}.

6.2.7 Recession Rates Assuming Sea Level Rise

The SCE 2010 report has discussed the combined effect of climate change in terms of sea level rise and increase in storminess, and that is not repeated here.

SCE 2010 has made an assessment of the likely effect of climate change on rates of bluff recession for rock areas as presented in Table 5 of SCE 2010. We have adopted the SCE assessment as appropriate for design purposes. SCE has adopted the following recession rates:

- At 2050 110% of the current (2007) base value
- At 2100 120% of the current (2007) base value.

Based on our modified recession rates, this equates to the following:

Design Recession Rate For Patonga Claystone

- Over a period of 35 years (2015 to 2050), extrapolation of the above rate implies about 1.5m of recession of the cliff face comprising Patonga Claystone (with sandstone dominant).
- Over a period of 85 years (2015 to 2100), extrapolation of the above rate implies about 4.1m of recession of the cliff face comprising Patonga Claystone (with sandstone dominant).

We note that the typical joint spacing for Patonga Claystone (with sandstone dominant) is 2m which indicates that the 2050 figure should be rounded up to 2m and that 4m for 2100 is reasonable.

Design Recession Rate For Tuggerah Formation and Munmorah Conglomerate

- Over a period of 35 years (2015 to 2050), extrapolation of the above rate implies about 1.2m of recession of the cliff face comprising Tuggerah Formation Munmorah Conglomerate.
- Over a period of 35 years (2015 to 2100), extrapolation of the above rate implies about 3.1m of recession of the cliff face comprising Tuggerah Formation and Munmorah Conglomerate.

We note that the typical joint spacing for Tuggerah Formation and Munmorah Conglomerate is 4m, which indicates that the 2050 figure may be conservatively rounded up to 4m and the 2100 figure should be rounded up to 4m.

7 GEOTECHNICAL HAZARDS

Based on the results of our inspections, the potential geotechnical hazards for the foreshore areas are summarised and outlined below.

1. Instability of bedrock cliff faces:
2. Instability of previous landslide areas.
3. Instability of soil slopes.

7.2.1 Cliff Face Instability

Current sea levels are believed to have been reached around 6,000 years before present (ybp). A glacial period between about 17,000 and 25,000 ybp is believed to have caused a sea level fall of around 130m below present day levels. At the end of this glacial period ice melted and sea levels rose to their current levels. This cycle of varying sea levels is believed to have occurred several times during the Quaternary (about 1.6 million years ago to present day). The wave cut platforms observed along the bases of many of the cliff faces are likely to have developed during inter-glacial sea level highs (including the present day). About 70 million years ago it is believed that the current cliff faces were located approximately 90kms east of their present location. Recession over the last 70 million years has resulted in the retreat of the cliff faces to the present coast line. Such recession caused the release of horizontal stress within the rock mass of the cliffs. This resulted in horizontal expansion of the rock along sub-horizontal bedding partings assisted by detachment along sub-vertical defects.

Over significant sections of the cliff faces, a wave cut platform was present which was covered by varying quantities of detached blocks sourced from previous rock falls. The blocks were either elongated or "cubic" which suggests that they were derived from collapse of cliff face overhangs and wedges of sandstone bedrock within the cliff face. The shape and size of the blocks appeared to be controlled by the two principal orthogonal joint sets. The principal joint sets were generally orientated between approximately 020° and 040°, 290° and 305° and 325° and 330°.

Based on our observations, sections of the cliff faces contained claystone bands which were evidently preferentially eroding compared to the more competent sandstone or conglomerate, resulting in collapse of blocks and form overhangs in the cliff face. In some instances, sandstones were preferentially eroding compared to the more competent conglomerate. Typically, a sub-vertical defect was present at the rear, or close to the rear of the overhang/undercut. Weathering and erosion by wave and wind action together with surface run-off would be the primary mechanisms resulting in deterioration of the rock mass. This was particularly evident at the following locations:

- Plate 10: Blue Lagoon.
- Plate 18: Blue Bay.

- Plate 20: The Entrance.
- Plate 25: Soldiers Beach.
- Plates 32 and 33: Pebbly Beach.
- Plates 35 and 36, Norah Head,
- Plate 47: Jenny Dixon Beach.

We note that all sections of bedrock cliff faces within the study area would be affected by the above mechanisms although the frequency of landslips would vary.

7.2.2 Instability of Previous Landslide Areas

At selected locations along the study area there was evidence of previous landslides and/or previous landslides were inferred from topographical features over the foreshore area:

- Crackneck Point.
- Bateau Bay.
- Cabbage Tree Harbour.
- Hargraves Beach.

With regard to landslides impacting cliff faces, in addition to the weathering and erosion processes outlined in Section 7.2.1 above, one or more of the following additional triggers could have caused collapse:

- Water pressure developed in the sub-vertical open joints behind potentially unstable features during and following rainfall events.
- Localised tree root 'jacking' where tree roots penetrate sub-vertical open joints landward of potentially unstable features over the cliff faces.
- Water collecting in open defects and decomposing vegetation in the open defects (generating humic acid) resulting in continued weathering and degradation of the bedrock forming the defect face.
- Expansion and contraction of the bedrock can also be expected as a response to temperature variations. This would lead to lateral expansion and contraction of the bedrock surfaces forming the open and possibly infilled defect, with additional soil entering the open defect during periods of expansion. The increased quantity of soil infill within the defect would then inhibit the contraction of the bedrock resulting in a build-up of stress, which would lead to further propagation of the defect.

- Concentrated discharge of stormwater pipe outlets from the crest areas of cliff faces, which could result in localised erosion of the areas below undercut or overhangs and/or increased volumes of water within open defects.

For foreshore soil slopes, previous instability would typically have been governed by one or more of the following factors; over-steep slopes, elevated water pressures within the soils associated with ineffective drainage through the natural soils and/or surface water run-off and erosion of the toe of the slopes by wave action. In addition, concentrated discharge of surface run-off after heavy or prolonged periods of rainfall can cause localised instability.

Selected portions of the steeply sloping upper vegetated portions of slopes above cliff faces over the study area have been assessed to possibly comprise colluvial soils. Colluvial soils represent relic landslide deposits and typically contain numerous boulder sized bedrock inclusions within a sandy or clayey soil matrix which would also be impacted by similar factors as described above.

7.2.3 Instability of Soil Slopes

The areas of foreshore comprising soil slopes have been assessed to typically comprise:

- Sand dunes.
- Indurated/cemented sands.
- Residual clayey soils with bands of weathered bedrock and/or detached sandstone blocks derived from localised collapse of unstable cliff face features but with little, if any, downslope movement.
- Colluvial soil slopes as described in Section 7.2.2, above.

Instability of such slopes is typically governed by one or more of the factors described in Section 7.2.2, above.

On-going creep of soil slopes is also typical over moderate and steeply sloping areas. Creep would be indicated by uneven slope surfaces, leaning or curved trees and/or localised sub-vertical back scarp features.

8 CONSTRUCTION OF CLIFF FACE HAZARD LINES

In order to construct the hazard lines for the immediate, 2050 and 2100 planning horizons, the following sources of information were used:

- Field mapping observations, including typical slope angles for the soil slopes, the rock types exposed in the cliff faces, the defects (spacing and orientation), the responses of the bedrock to erosion, cliff face failure mechanisms and other indications of slope instability,
- The review of cliff face recession rates discussed in Section 6, above.
- Review of previous geotechnical reports prepared by ourselves and other geotechnical consultants.
- The LiDAR imagery, aerial photography and historical photographs.

Initially, we provided BMT with our interpretation of the likely orientation of buried cliff faces at the transition zones between cliff faces and sand dune slopes. BMT then used this information to assess beach erosion lines and identify at which planning horizon the buried cliff faces were exposed (if at all). These exposed cliff lines were then included in the construction of the cliff face hazard lines, as outlined below.

The cliff face has been assumed to erode back to a sub-vertical joint plane orientated parallel to the cliff face. Based on our observations, a typical dip angle of 70^{o} has been adopted for the joint planes within the various bedrock formations. The dip angle is consistent with our field observations and was also adopted by SCE 2010.

We recommended that BMT adopt the following methodology to construct cliff face hazard lines using the bedrock recession rates outlined in Section 6.2.5:

- The Immediate hazard line cliff face was established at a landward set-back distance equivalent to the joint spacing and a new cliff face projected up at 70^{o} from the wave cut platform to the estimated bedrock surface level landward of the slope or cliff face.
- From the bedrock surface level, flatter lines were then projected upward to the ground surface level through the inferred soil profile. The projected angle for the soil profile was assessed based on the following:
  - For a soil profile above bedrock that was assessed to primarily comprise sands, an overall angle of 30^{o} was adopted, based on a typical angle of internal friction for sandy soils. These site areas corresponded to Little Bay, Blue Bay, Towoon Bay, Blue Lagoon, The Entrance and sections of Lighthouse Beach.
  - For the steeply sloping vegetated areas above the cliff faces, where bedrock faces were exposed in places, a relatively thin clayey residual soil profile was assumed to cover the

bedrock surface. An average thickness of 2m of residual clay soil was adopted and has been assumed to include the upper portion of the bedrock surface, which in most instances, would comprise a stepped surface influenced by differing weathering and erosion rates for sandstone, conglomerate and claystone. An overall angle of 45^{o} was adopted for this ‘composite’ soil/buried bedrock surface profile. The assumption has been made that any cliff face collapse impacting the soil profile above would initially form a sub-vertical face which, over time, would recede back to an angle of 45^{o}. These site areas corresponded to Crackneck Point, Soldiers Beach, Pebbly Beach, Norah Head, Jenny Dixon Beach and Hargraves Beach.

This process was the repeated for the 2050 and 2100 hazard lines.

Where there was considered to be additional detrimental impacts on slope stability due to geotechnical processes not directly triggered by coastal erosion, ‘Geotechnical Hazard Zones’ were identified (see Section 9, below).

9 GEOTECHNICAL HAZARD ZONES

We note that SCE 2010 identified a number of ‘Geotechnical Hazard Zones’ situated immediately landward of the coastal erosion hazard lines which were identified as being primarily impacted by landslide and soil erosion and which may have a serious impact on development.

We consider that the designation of ‘Geotechnical Hazard Zone’ areas over the study area is a valid addition to the coastal erosion hazard lines. These ‘Geotechnical Hazard Zones’ identify to Council areas of potential instability in the coastal zone that are not primarily triggered by coastal processes but are triggered by geotechnical processes. These ‘Geotechnical Hazard Zones’ have been identified based on:

- Our review of the available information.
- Our experience of previous instability at selected locations within the study area (e.g. Cabbage Tree Harbour).
- Our own site observations.
- Features evident in the LiDAR plot information; e.g. arcuate areas at breaks in slope which could represent old landslide features not readily discernible at ground surface level.

Our 'Geotechnical Hazard Zones' presented on the Figures ??? in the BMT 2016 report (Reference) have, in some instances, expanded the areas identified in SCE 2010. In some instances they extend landward (upslope) of the 2100 erosion hazard line to the significant break in slope.

10 GENERAL COMMENTS

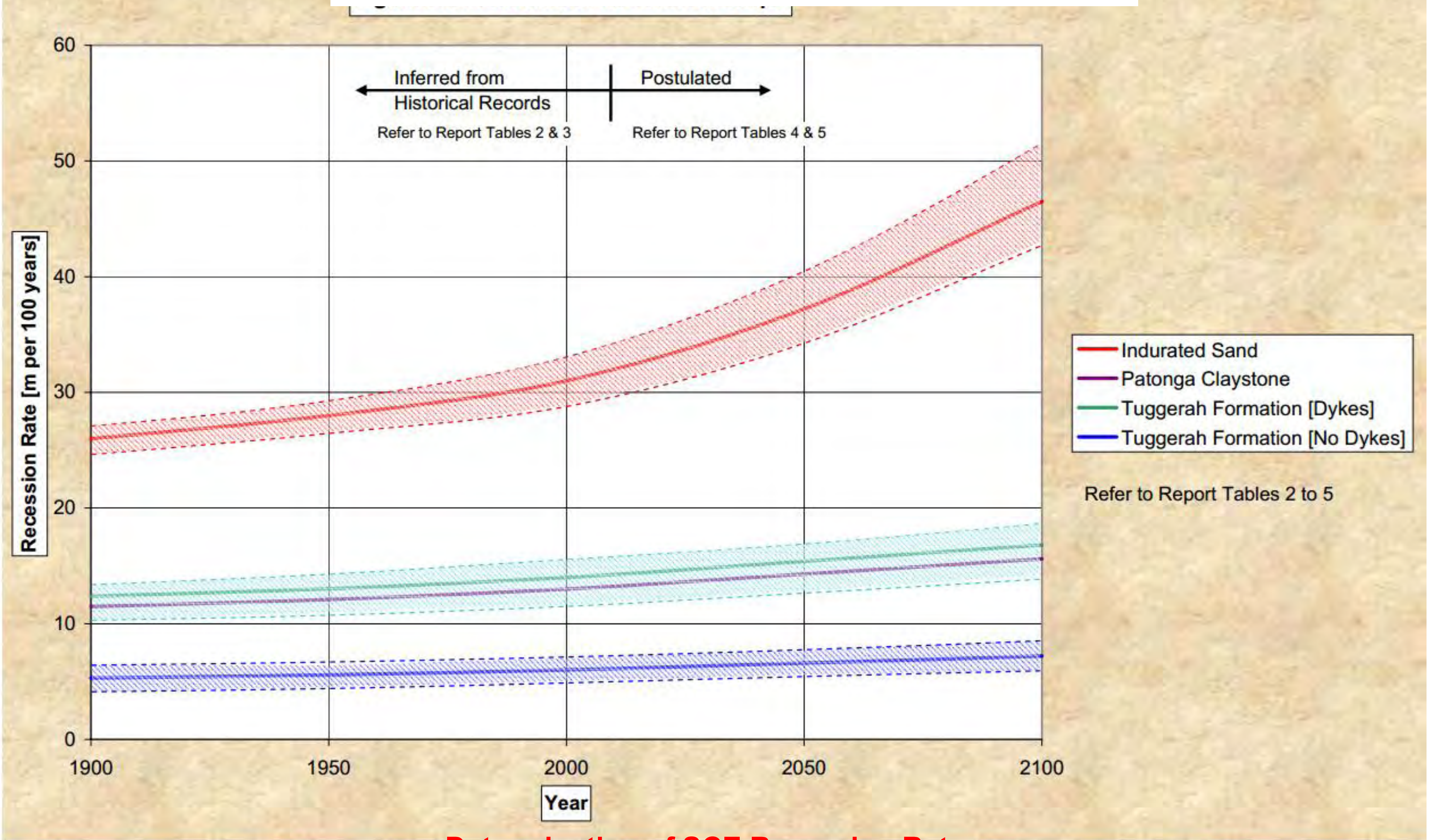
It is possible that the subsurface soil, rock or groundwater conditions encountered during implementation of the landslide risk management measures may be found to be different (or may be interpreted to be different) from those inferred from our surface observations in preparing this report. Also, we have not had the opportunity to observe surface run-off patterns during heavy rainfall and cannot comment directly on this aspect. If conditions appear to be at variance or cause concern for any reason, then we recommend that Council immediately contact this office.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

Site Location Plan

2013

Figure B5: Postulated Recession Group (Extract from SCE 2010)



Determination of SCE Recession Rates

To be read in conjunction with text of report.

TABLE A: COMPARISON OF RECESSION RATES

1: FOR WYONG SHIRE BASED ON TABLE B4 OF SCE 2010						
Location[1]	**Geology**	**Affected by Dyke[2]**	**Map Type[3]**	**Period (Years)[4]**	**Measured Recession[5] (m)**	**Recession Rate[6] (m/100 Years)**
Jenny Dixon Beach	Tuggerah Formation with Patonga Claystone Capping	Yes	1878 survey	130	16 – 26 ± 3	12 – 20
			1985 orthophoto map	21	5 – 10	23 – 48
Henderson Street (Norah Head)	Tuggerah Formation with Patonga Claystone Capping	Yes	1878 survey	130	10 – 15 ± 3	8 – 12
			1985 orthophoto map	21	0	?
Cabbage Tree Harbour	Quaternary Indurated Cemented Sands	No	1878 survey	130	34 ± 3	33
			1985 orthophoto map	21	4 – 7	19 – 33
Norah Head	Tuggerah Formation	Yes	1878 survey	130	14 – 20 ± 3	11 – 15
			1985 orthophoto map	21	0	?
Ocean Parade Headland (Blue Bay)	Patonga Claystone	No	1985 orthophoto map	21	0	?
Crackneck Point (south of Bateau Bay)	Patonga Claystone	No	1985 orthophoto map	21	2 – 10	10

NOTES:

1 Location indicates approximate area of the measured coastal bluff changes. No further details given in Reference 4.

2 'Affected by Dyke' column indicates that the measured area was assessed to be influenced by a dyke structure, and likely increased rate of recession.

3 Details of historical maps used given in Table B4 of SCE 2010.

4 Period determined by comparison with *'Google Earth'* aerial imagery dated 11 January 2007.

5 Measured Recession determined by comparison between *'Google Earth'* image and historical maps.

6 Assessed recession rates provided in Table B4 of SCE 2010.

7 Refer to SCE 2010.

To be read in conjunction with text of report.

2: DATA FROM CROZIER & BRAYBROOKE 1992

Cliff Regression Rates Based on Widths of Wave Cut Platforms (Based on Table 1 of Crozier Braybrooke 1992)

Rock Type at Sea Level	Regression Rates (mm/year)		Extrapolated Recession Rate (m/100 year)	
	Average	Maximum	Average	Maximum
Sandstone	4.3	12.1	0.4	1.2
Mudstone/Shale	6.2	18.1	0.6	1.8
Mudstone/Shale At Long Reef	–	57	–	5.7

Field Measured or Estimated Rates of Weathering for Various Sedimentary Rocks (based on Table 3 of Crozier Braybrooke 1992)

Location	Lithology	Defect Type Depth & Facies	Time Period (years)	Maximum Weathering Rate (mm/year)	Extrapolated Recession Rate (m/100 years)
Sassafras, Shoalhaven Valley	Nowra SST	Scarp Retreat (Young & McDougall, 1985)	30 million	0.012 to 0.025	0.001 to 0.002
Liverpool Cemetery	Hawkesbury Sandstone		106	0.24	0.02
Bondi	Hawkesbury Sandstone	Salt weathering of sea cliff (Roy, 1983), limonite cemented SST resistant sandstone 'softer' sandstone	100	 1 1 to 2 up to 5	 0.1 0.1 to 0.2 up to 0.5
Warringah Road, Road Cut, Beacon Hill	Sandstone	Differential weathering rates between retaining wall and weathered fine grained clayey sandstone	15	10 to 17.4	1 to 1.7
Oxford Falls Road, Road Cut at Oxford Falls	Sandstone	Differential weathering rates between sandstone layers within Sheet Facies	13	1 to 4.6	0.1 to 0.5
	Shale	Between sheet sandstone and mudstone		5 to 8.5	0.5 to 0.9
Newcastle Expressway between Berowra and Hawkesbury River. Various road cuts	Shale	Differential weathering rates between sandstone and shale	30	23 to 43	2.3 to 4.3
	Sandstone	Sandstone		1 to 3.3	0.1 to 0.3
Gosford	Narrabeen Sandstone	Salt decay in sandstone caves (Lambert, 1980)		0.1 to 0.2	0.01 to 0.02

NOTE: 8 Refer to Crozier Braybrooke 1992 for further discussion.

To be read in conjunction with text of report.

3: DATA FROM DELANEY 2005					
Recorded Cliff Line Regression Rates in Sedimentary Rock (Sunamura, 1992) (based on Table 1 of Delaney 2005)					
Location	**Rock Type**	**Erosion Rate (mm/year)**	**Interval (years)**	**Method**	**Extrapolated Recession Rate (m/100 years)**
Sturt Point, Victoria	Siltstone	17.5	6,000	Survey	1.8
Sturt Point, Victoria	Sandstone (arkose)	9	6,000	Survey	0.9
Point Peron, Perth	Limestone	0.2 – 1	9	Steel pegs	0.02 to 0.1
Kuji, Japan	Cretaceous Sandstone	10	6,000	Surveys	1
Ngapotiki, New Zealand	Conglomerate	3,500	29	Air photos	350
Yorkshire, England	Shale	9 – 20	1 – 60	Erosion meter, maps	0.9 to 2
Leucadia, California	Claystone and Sandstone	0 – 500	5	Nails	0 to 50
La Jolia, California, USA	Cretaceous Sandstone	0.3 – 0.6	–	Dated inscriptions	0.03 to 0.06
	Sandstone and Shale	10 – 200	39	Photos	1 to 20
Sunsets Cliffs, San Diego, USA	Cretaceous Sandstone Siltstone	12	75	Photos	1.2 to 7.5

Rates of Cliff Line Regression – Newcastle Coastline (based on Table 2 of Delaney 2005)				
Method	**Location**	**Rate (mm/year)**	**Comment**	**Extrapolated Recession Rate (m/100 years)**
Historical	Newcastle Beach, Bogie Hole, Merewether	1 – 40	–	0.1 to 4
	Lloyd Street	30 – 80	Mine subsidence effects?	3 to 8
Survey	Nobbys	10 – 15	Direct survey	1 to 1.5
	Watt Street	20 – 25	Direct survey	2 to 2.5
Geological	Rock platforms	10 – 15	High cliff areas	1 to 1.5
	Rock platforms	Up to 25	Low cliff areas	Up to 2.5
Rock Surface	Newcastle Beach, Dixon Park, Seawalls	1 – 5	Erosion rate of sandstone exposures	0.1 to 0.5

NOTE: 9 Refer to Delaney 2005 for further discussion.

4: EVALUATION OF EXISTING WAVE CUT PLATFORMS NEAR BLUE BAY						
		Assuming No Platform Recession		Assuming Platform Edge Recession		
					Cliff Recession Rate	
Location	Platform Width (m)[11]	Cliff Recession Rate[10] (mm/year)	Cliff Recession Rate[10] (m/100 years)	Platform Recession Rate (m/100 years)	(mm/year)	(m/100 Years)
Ocean Parade Headland	60	10	1	1	20	1
				4	50	5
Blue Bay/ Toowoon Bay Reef	Assumed 300	50	5	1	60	6
				4	90	9
Swadling Reserve Headland	60	10	1	1	20	2
				4	50	5
Toowoon Bay, South Tombolo and Reef	410	68	6.8	1	78	7.8
				4	108	10.8
Crackneck Point (south of Bateau Bay)	180	30	3	1	40	4
				4	70	7

NOTE: 10 Assumes recession over 6,000 years.
11 Measured from Google Earth images

REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, the SAA Site Investigation Code. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached Unified Soil Classification Table qualified by the grading of other particles present (e.g. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2mm
Gravel	2 to 60mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

Cohesive soils are classified on the basis of strength (consistency) either by use of hand penetrometer, laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	Strength not attainable – soil crumbles

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'Shale' is used to describe thinly bedded to laminated siltstone.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All except test pits, hand auger drilling and portable dynamic cone penetrometers require the use of a mechanical drilling rig which is commonly mounted on a truck chassis.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock fragments. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from “feel” and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term ‘mud’ encompasses a range of products ranging from bentonite to polymers such as Revert or Biogel. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, “Methods of Testing Soils for Engineering Purposes” – Test F3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the ‘N’ value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

  N>30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

A modification to the SPT test is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as "N_c” on the borehole logs, together with the number of blows per 150mm penetration.

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between EFCP and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of EFCP values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the sub-surface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will enable the most reliable assessment, but is not always practicable or possible to justify on economic grounds. In any case, the boreholes or test pits represent only a very small sample of the total subsurface conditions.

The attached explanatory notes define the terms and symbols used in preparation of the logs.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than “straight line” variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or ‘reverted’ chemically if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg bricks, steel etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably determine the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soil for Engineering Purposes'*. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg to a twenty storey building). If this happens, the company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.

If these occur, the company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'*, published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. License to use the documents may be revoked without notice if the Client is in breach of any objection to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/ constraints are quite complex, it is prudent to have a joint design review which involves a senior geotechnical engineer.

SITE INSPECTION

The company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

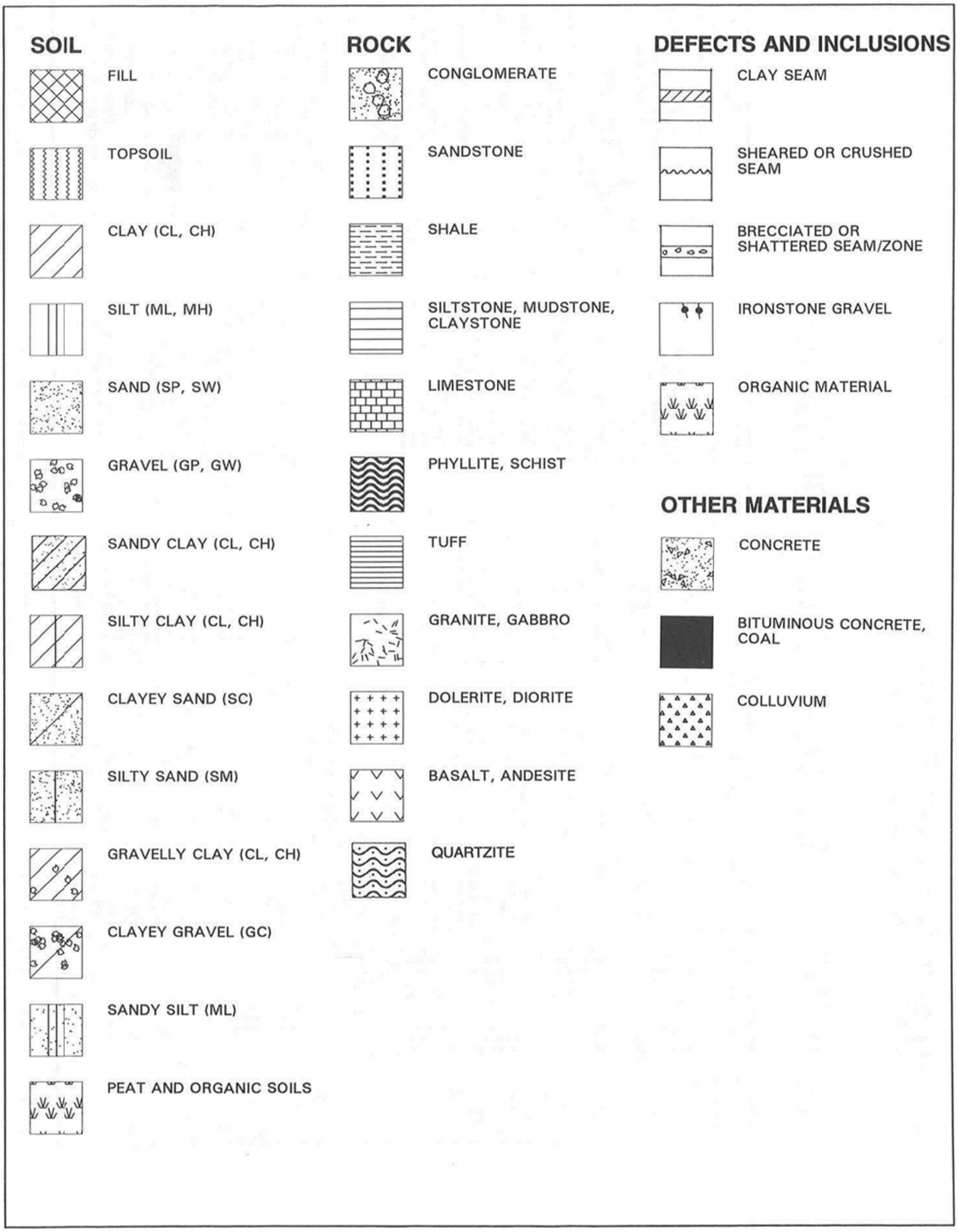
Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

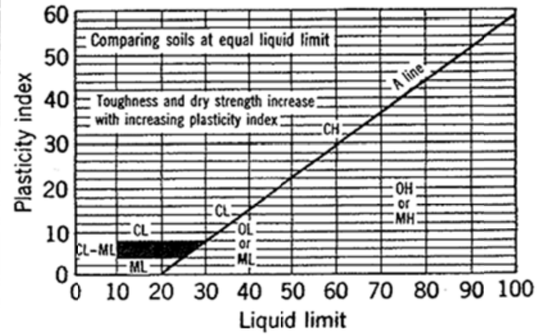
ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or

iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



Major division	Subdivision		Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)			Group Symbols a	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria		
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b] (The 75 μm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes			*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses	Use grain size curve in identifying the fractions as given under field identification	Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP*; More than 12% *GM, GC, SM, SC*; 5% to 12% *Borderline* cases requiring use of dual symbols	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing			*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines				Not meeting all gradation requirements for *GW*
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)			*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures				Atterberg limits below "A" line, or *PI* less than 4 / Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics			Atterberg limits above "A" line, with *PI* greater than 7
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes			*SW*	Well graded sands, gravelly sands, little or no fines	Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)			$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing			*SP*	Poorly graded sands, gravelly sands, little or no fines				Not meeting all gradation requirements for *SW*
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)			*SM*	Silty sands, poorly graded sand-silt mixtures				Atterberg limits below "A" line or *PI* less than 5 / Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*SC*	Clayey sands, poorly graded sand-clay mixtures				Atterberg limits below "A" line with *PI* greater than 7
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Identification Procedures on Fraction Smaller than 380 μm Sieve Size		Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)						
	Silts and clays liquid limit less than 50		None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses			
			Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays				
			Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions			
	Silts and clays liquid limit greater than 50		Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts				
			High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)			
			Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity				
Highly Organic Soils			Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils				



Plasticity chart for laboratory classification of fine grained soils

Note: 1 Soils possessing characteristics of two groups are designated by combinations of group symbols (eg. GW-GC, well graded gravel-sand mixture with clay fines).
2 Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record	▼	Standing water level. Time delay following completion of drilling may be shown.
	C	Extent of borehole collapse shortly after drilling.
	▶	Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos screening.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'R' as noted below.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60 degree solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of Undrained Shear Strength.
	PID = 100	Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless Soils)	D	DRY – Runs freely through fingers.
	M	MOIST – Does not run freely but no free water visible on soil surface.
	W	WET – Free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – Unconfined compressive strength less than 25kPa
	S	SOFT – Unconfined compressive strength 25-50kPa
	F	FIRM – Unconfined compressive strength 50-100kPa
	St	STIFF – Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF – Unconfined compressive strength 200-400kPa
	H	HARD – Unconfined compressive strength greater than 400kPa
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** — **SPT 'N' Value Range (Blows/300mm)**
	VL	Very Loose <15 — 0-4
	L	Loose 15-35 — 4-10
	MD	Medium Dense 35-65 — 10-30
	D	Dense 65-85 — 30-50
	VD	Very Dense >85 — >50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

LOG SYMBOLS continued

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		Easily remoulded by hand to a material with soil properties.
-----------------------	--------------	0.03	
Very Low:	VL		May be crumbled in the hand. Sandstone is "sugary" and friable.
-----------------------	--------------	0.1	
Low:	L		A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
-----------------------	--------------	0.3	
Medium Strength:	M		A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
-----------------------	--------------	1	
High:	H		A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
-----------------------	---------------	3	
Very High:	VH		A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
-----------------------	---------------	10	
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	