RESIDENTIAL DEVELOPMENT ON COASTAL CLIFFLINES

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SUMMARY

The threshold criteria for determining when a detailed stability analysis is required is dependent on a number of factors but primarily the geology of the site. Where previous studies have been undertaken guidelines may be present to determine when detailed studies are required. A case study of a detailed stability analysis of a redevelopment located on a cliiffline is given.

INTRODUCTION

The principle geotechnical hazard affecting residential development on coastal clifflines is stability of the cliffline. Properties located on coastlines are a finite resource within any locality and usually command premium real estate values. With increased demand new development and redevelopment of existing properties on clifflines becomes more profitable.

Increased demand for coastal properties has resulted in pressure to develop marginal sites and intensively redevelop existing residential properties, by demolition of existing structures and replacement with a number of townhouses. Such redevelopment can result in a demand to locate structures as close to the cliff edge as possible to maximise usage of a site. Over the last 20 years well publicised cliffline failures affecting existing residential development has resulted in an increased awareness of the risks associated with cliffline development. These factors have lead to increased demand for stability assessments of sites located on clifflines to satisfy the requirements of the Local Authority and the New Zealand Building Code.

In the past Local Authorities have accepted stability assessments from registered engineers. Recently some Local Authorities have introduced a policy of having a vetted list of professionals from whom stability assessments will be accepted. The issue of who should be undertaking such assessments is clearly, suitably qualified and experienced professionals. For the purposes of this paper it is assumed that the person undertaking a stability assessment has the requisite qualifications and experience.

STABILITY ASSESSMENT THRESHOLD CRITERIA

The initial stability assessment is to determine if stability is an issue at the site and therefore the need for a detailed stability assessment. The criteria used to determine if stability is an issue are dependent on the site geology, land form, slope geometry, any existing slope failures and history of slope failures in the area. The assessor can from a desk study of geological maps and published data, existing site data, coupled with an examination of aerial photos and a walkover of the site determine whether further investigation of stability issues is required. In areas where previous studies have been undertaken threshold guidelines may be present for when a detailed stability assessment is required. Threshold guidelines can be in the form of projected regressed slopes or setbacks from cliff edges.

Safe building areas on sites located on clifflines comprised of soil or soil strength materials can be defined in terms of a regressed slope profile projected from the toe of the cliff with an allowance for toe erosion during the design life of the structure. A regressed profile that yields a factor of safety of 1.5 under fully saturated conditions will give a worst case regression profile for the assumed soil parameters.

Tauranga Area

In the Tauranga area of the Bay of Plenty, a study of slope failures in the developed areas concluded that coastal slopes with a slope of greater than 1V:2H were potentially at risk to slope failure. The projected regression line of 1V:2H is therefore used as the guideline as to when it is considered necessary to undertake a detailed stability assessment of the site. Failure modes can be either deep seated semi circular or planar failure surfaces.

CASE STUDY

INTRODUCTION

In July 1995 a geotechnical investigation was undertaken for a proposed redevelopment of an existing residential site located on a coastal cliffline. The proposal was to remove the existing building and construct four new residential buildings on the site. The geotechnical investigation was undertaken to determine the subsurface conditions at the site to detail any identifiable geotechnical hazards affecting the site. It was determined that the coastal cliffline was influencing the site, therefore a detailed stability assessment was undertaken.

Based on a visual inspection of the site and the data provided by the developer it was apparent that a significant part of the area to be redeveloped was located seaward of the nominal 2H:1V profile projected from the toe of the slope. It was therefore determined that the stability of the coastal slope required assessment to determine its influence on the proposed development at the site. The coastal slope geometry and a stratigraphic profile, through the cliff is shown on Figure 1.

SITE DESCRIPTION

The site was located in a developed residential area located on the cliffline on the western side of the Maungatapu Peninsula within Tauranga Harbour. The site was generally flat to gently sloping up to the crest of the steep coastal cliff which was about 27m high. The cliff has a slope angle of about 54° with a 3m to 5m subvertical face at the toe of the slope, which is coincident with the harbour foreshore. Immediately north of the site is a large gully. The gully side slope falls from the sites northern boundary via moderate to steep slopes ranging from 26° to 50°.

GEOLOGY

The regional geological map[1], indicated that the area was underlain by fluviatile sands and silts of Pleistocene age. These alluvial sediments are referred to as "Tauranga Beds" in this paper. Late Quaternary volcanic ash deposits mantle the region and are indicated to overlie the Maungatapu Peninsula [1]. The inferred stratigraphy at the site is of Tauranga Beds, older rhyolitic ashes, younger ashes, and recent ashes [2]. The results of the investigations confirmed that volcanic ashes overlie alluvial sands and silts at the site.

Generally within the Tauranga area landslip failures have occurred in the older ashes or within clay rich layers within the Tauranga Beds [2]. The map accompanying Houghton and Hegans report[2] indicates that the gully feature immediately north of the site is a possible deep seated landslide based on aerial photo interpretation. Deep seated failures are confined to areas adjacent to the sea cliff where the weathered, clay rich older volcanic ashes are very thick or the Tauranga Beds are unusually clay rich [2].

FIELD INVESTIGATIONS

Two machine boreholes MB1 and MB2 were drilled during the investigation to determine the subsoil conditions at the site. Due to restricted access to the seaward side of the existing house a trailer mounted Gemco HPC7 drill rig was lifted over the house using a mobile crane. Boreholes MB1 and MB2 were drilled to a depth of approximately 24m and 14m respectively. Plasticity Index test and particle size grading curves were determined on samples obtained from Borehole MB1.

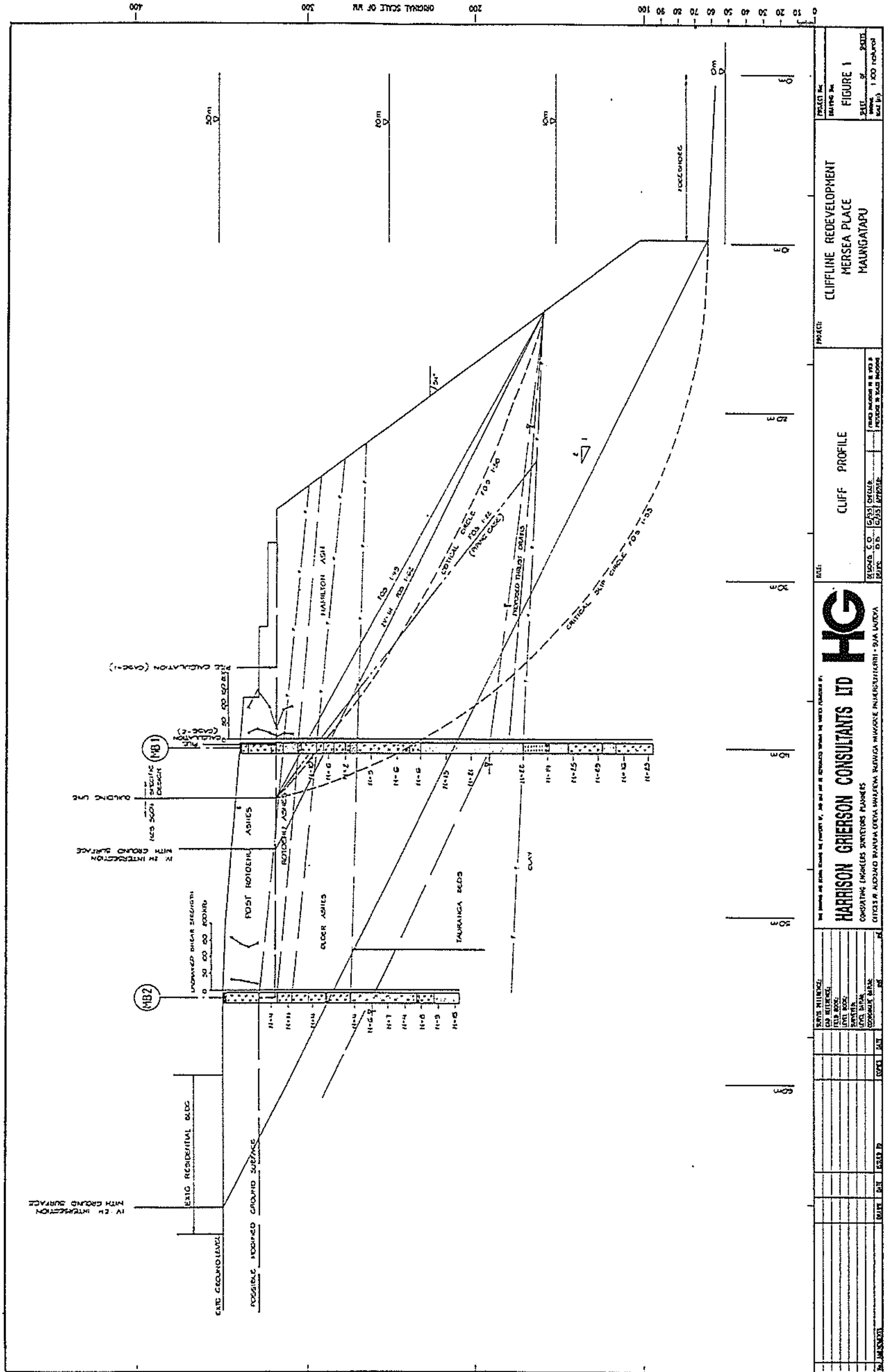

Figure 1: Cliff profile

SUBSURFACE CONDITIONS

The soils encountered in the boreholes indicate that the site is underlain by a stratification of silts with some sand layers, overlying interbedded silts and sands. Good correlation of strata between boreholes was obtained although some variation in colour was noted which may be due to weathering effects.

Volcanic Ashes

The surface soils at the site as encountered in the boreholes consisted of pumiceous, silts and sands about 7m thick inferred to be volcanic ashes. The silts ranged in shear strength from 57 kPa to 157 kPa and were sensitive to disturbance. SPT N values ranged from 2 to 14 indicating very loose to medium dense material. The inferred sequence of volcanic ashes was between 6.8m and 7.5m thick in boreholes MB1 and MB2 respectively. The volcanic ashes are underlain by sands and silts.

Alluvial Sediments

The materials encountered consisted of sands, interbedded silty sands and sandy silts, with some silt layers and fine gravel. The silts and sands were thinly interbedded, with banding and faint bedding observed in the sand horizons. In Borehole MB1 a clay layer 1.4m thick was encountered at a depth of 16.8m. SPT N values from approximately 7m to 13m ranged from 4 to 9 indicating loose material. SPT N values increased from a depth of about 13m to the end of the boreholes with N values ranging from 13 to 29 indicating medium dense material.

SLOPE STABILITY

Effective Strength Parameters

The inferred profile of the surficial ashes was gently dipping towards the coastal slope. A shallow planar failure mode within the surficial ashes at the crest of the coastal slope was considered to be unlikely due to the low groundwater levels measured at the site in combination with the height and geometry of the slope profile. Based on the slope profile and geometry it was determined that the critical mode of slope failure was likely to be a deep seated circular failure. The laboratory data indicated that some soils may be close to their liquid limit, and it was inferred that an increase in pore water pressures due to a increase in ground water levels may result in a deep seated failure occurring.

In order to assess the stability of the coastal slope effective stress parameters for each inferred soil horizon were assumed. Based on the assumed parameters and the measured groundwater levels a stability assessment for circular failure using Bishops simplified method was undertaken.

Using the existing ground profile, as measured groundwater levels and the assumed soil parameters the minimum theoretical factor of safety for the cliff slope was less than 1.0. As the slope had not failed it was therefore apparent that the initially assumed parameters were too low. From inspection of the slope profile it was apparent that the parameters assigned to the Tauranga Beds were the major factor affecting stability analysis.

A friction angle of 36° was considered realistic for the materials encountered, however a zero cohesion value was overly conservative for existing conditions. By iteration it was determined that a minimum cohesion value of 22 kPa for the Tauranga Beds was required to yield a minimum theoretical factor of safety of approximately 1.0 for the critical circle for the existing ground water profile. It was concluded that the revised effective cohesion value of 22 kPa and friction angle of 36° were realistic parameters for the Tauranga Beds, which we believed were conservative parameters as a raised ground water condition was not used in the iterative analysis.

In order to check that the revised parameters were reasonable, the adjacent gully profile was analysed assuming that the gully was the result of a deep seated failure of the cliffline. It was assumed that originally the cliffline extended northwards with a similar geometry and soil profile to that encountered at the site and that the existing slope profile, approximated a deep seated failure surface, with the slumped material assumed to have been eroded away by tidal action.

The slope was analysed for a circular slip surface using the revised parameters which yielded theoretical factor of safety values of 2.03 for as measured groundwater conditions and 0.74 for fully saturated conditions. It was therefore assumed that at the time of failure the groundwater condition was raised above that measured within the site. It was therefore concluded that the revised parameters listed in Table 1 are reasonable conservative parameters based on the available data.

Table 1: Revised Effective Strength Parameters

Soil Horizon	Bulk Density (kN/m^3)	Effective Cohesion (kPa)	Internal Friction Angle (°)
Younger Ashes	14	2	32
Rotoehu Ashes	14	4	32
Hamilton Ash	16	4	32
Older Ashes	15	4	32
Tauranga Beds	16	22	36

Cliff Stability

A possible development option was to cut down the ground surface by 1.5m to 3m below existing as shown on Figure 1. Based on the revised parameters, existing groundwater conditions and the proposed modified ground surface it was determined that the existing cliff slope had a theoretical factor of safety of less than the minimum value of 1.5 required by the Local Authority. Various slip circles were analysed to determine the slip circle with a theoretical factor of safety of 1.5, which is plotted on Figure 1. The slip circle was further analysed for the case where cohesion equals zero under existing groundwater conditions which yielded a theoretical factor of safety(FOS) of 1.28.

Based on the results of the investigations at the site and knowledge and previous experience with soils of the type encountered at the site, it was our opinion that the area of the site landward of the point where the FOS = 1.5 slip circle intersected the existing ground surface would have an acceptable stability state in terms of conventionally accepted criteria, for the subsurface conditions assumed to exist or likely to occur at the site.

The intersection of the FOS = 1.5 slip circle with the proposed modified ground surface determined the location of the "Recommended Building Line Limitation" (RBLL) which defined the extent of the area suitable for conventional residential foundations. The area seaward of the RBLL defined the area within which building foundations required specific design to provide a stable building area with a minimum factor of safety of 1.5. It was recommended that the location of the RBLL should be reassessed when the development design was finalised to take into account any modifications to the site.

In the foregoing analysis no allowance for earthquake effects had been made. It was our opinion that the proposed development had the same risk of being affected by an earthquake as most other existing residential developments located on the coastline and elsewhere in the area.

Toe Erosion

The coastline on the eastern boundary of the site varied from steep slopes, up to sub vertical banks. The harbour at the site location is tidal and exposed to limited wave action. It was considered that active erosion of the coastline along the eastern edge of the site was occurring due to the combined erosive action of waves and tidal currents. The erosion rate of the cliffs at the site locality was not known, but in it was considered unlikely to have any significant influence on the area defined as suitable for conventional building construction, located approximately 43m from the toe of the coastal slope.

REVIEW

Shortly after the investigation of the above site on the western edge of Maungatapu Peninsula, a landslip occurred on the cliffline at the northern end of the peninsula about 150m north of the site.

Detailed investigation of the slip was undertaken by another consultant on behalf of the Local Authority. The failure surface of the slip at the north end of the peninsula was understood to have exited the slope above the toe at the upper level of a clay horizon. A clay horizon was encountered in Borehole MB1 at our site, which was assumed to be similar to the clay layer encountered at the north end of the peninsula. We therefore reassessed the stability of the cliff at the site taking into account the analysis of the recent slips at the north end of the peninsula.

STABILITY APPRAISAL

Taking into account the recent slips and the results of our investigations, it was concluded that the likely slip failure surface would not be deep seated exiting through the slope toe. A more probable failure mechanism was for a slip surface to develop which exits the slope immediately above the upper surface of the clay horizon within the overlying silts and sands. The existing vegetation cover and steepness of the slopes prevented an inspection of the cliff face being made.

It was assumed that the clay layer was laterally continuous at the site and acted as an aquiclude to groundwater movement. The clay layer was inferred to be exposed in the cliff face. The existing cliff slope was back analysed for a circular mode of failure assuming a groundwater level and failure surface controlled by the clay layer. It was determined that for the Tauranga Beds layer effective stress parameters of 35° and 10 kPa cohesion were required to approximate unity for a shallow circular failure at the cliff face. The soil parameters were revised and the cliff analysed for a circular failure exiting the cliff face at the top of the clay layer.

It was determined that the critical slip circle with a back scarp located at the previous Recommended Building Line Limitation (RBLL) had a factor of safety of approximately 1.58. It was further determined that a 2H:1V line extending from the top of the clay layer at the cliff face intersected the proposed ground surface approximately 3m behind the RBLL, which had a factor of safety value of 1.62 for a planar faliure and the existing ground water conditions. A planar failure surface from the clay layer at the cliff face extending to the RBLL location yielded a theoretical factor of safety value of 1.49 for the existing groundwater conditions.

The cliff was further analysed for a non circular failure mode assuming that a failure plane develops immediately above the upper clay surface. It was assumed that groundwater seepage at the cliff face along the clay surface resulted in piping erosion of the overlying sands and silts. The piping was modelled by assigning residual parameters to the slip surface along the level of the layer. The residual parameters used were 18° internal friction angle and zero cohesion. The residual internal friction angle was considered to be a conservative value for volcanic soils based on test results reported by Wesley [3].

By iteration the minimum theoretical factor of safety for a non circular failure was 1.22 for existing groundwater conditions assuming that a piping layer developed along the clay interface for a horizontal distance of 9m into the slope. The back scarp of the failure was located to be coincident with the RBLL determined in the original analysis. This value was considered to be satisfactory provided that groundwater levels are maintained at existing levels. It was therefore recommended that thrust drainage be installed at the site immediately above the clay layer to intercept any ground water seepages and to maintain the existing groundwater conditions. The various failure planes and the proposed thrust drains are shown on Figure 1.

CONCLUSIONS

Based on the above results it was considered that the original RBLL on the site had a satisfactory stability state provided that the groundwater level does not rise above the inferred existing levels and that piping erosion does not occur. The installation of appropriate drainage measures should maintain the ground water levels at the levels inferred to exist at the site in the long term. It was our opinion that horizontal drains thrust from the cliff face coincident with the top of the clay layer as shown on Figure 1, should provide sufficient drainage to maintain the existing ground water conditions. The critical slip failure mechanism assumes that piping develops along the clay surface. The installation of thrust drainage with suitable filter cloth should minimise the potential for piping erosion to occur.

References:

1 Healy, J, Schofield, J C and Thompson, B N 1964. *Sheet 5 - Rotorua.* Geological map of New Zealand 1:250 000. Map(1 sheet). Wellington, New Zealand. DSIR.

2 Houghton, B F and Hegan, B D 1980. *A preliminary assessment of geological factors influencing slope stability and landslipping in and around Tauranga City.* NZ Geological Survey Engineering Geology Report EG 348.

3 Wesley, L D 1992. Some Residual Strength Measurements on New Zealand Soils. *Proceedings 6th Australia New Zealand Conference on Geomechanics*: 157 -162.