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A landslide investigation at the Pleasant Hills subdivision, Legana

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Abstract

A landslide on Stage III of the Pleasant Hills subdivision at Legana has been delineated. The morphology of the area suggests that a re-activation of an ancient landslide has occurred. Subsurface investigations indicate that a laterite gravel layer which contains water under pressure plays a significant role in the failure mechanism. Areas in which building is not permitted have been delineated.

INTRODUCTION

Following a period of high rainfall in mid-September 1989 a landslide of substantial proportions (fig. 1) developed on Stage III of the Pleasant Hills subdivision. The extent of the landslide has been delineated by survey. Investigations were conducted to determine the reason for the landslide.

GEOLOGY

Most of Stage III of the Pleasant Hills subdivision is covered by a veneer of Quaternary age, high plasticity clayey soil. This soil is largely derived from the weathering of Tertiary age basalt which occurs on the higher ground of the subdivision. The area has been mapped by Longman (1964) as basalt talus. Test pitting by Weldon (1987a, 1988), and holes associated with this investigation, indicates that the basalt talus is variable in thickness. It is underlain, in places at shallow depth, by Tertiary age sediments known as the Launceston Beds. It is within these beds that the landslide has occurred. Jurassic age dolerite occurs on the south and south-west extremity of Stage III. In places discontinuous sheets of limonitic material are present at the surface.

SITE INVESTIGATIONS

A trailer-mounted auger drill rig was used to investigate the landslide area. In general, the auger holes (fig. 1) encountered grey-brown silty topsoil from the surface to about 0.4 m depth, yellow-brown clay to about 1.6 m depth, and brown clay to the maximum depth of investigation (7.5 m). Grey clay layers were occasionally intersected between 1.6 and 2.5 m depth. A limonite gravel was intersected at about 6.5 m depth. This was underlain by brown clay.

The clay immediately above and below the limonite gravel was described by the driller as soft and easy to auger. The limonite gravel was wet. Undisturbed square tube samples were obtained from the soft zone for laboratory shear box testing. Open standpipe piezometers were placed into the soft zone to obtain data for pore pressure ratio determinations.

The extent of the landslide was determined by survey. Several pegs were placed on either side of the landslide scarp as reference points for further monitoring.

PIEZOMETERS

Open standpipe (PVC) piezometers were installed at three locations within the body of the landslide. The lower four metres of the standpipes was slotted and installed to the bottom of the auger holes. Fine gravel filter material was used as packing between the standpipe and the walls of the auger hole. The piezometers were installed to measure the head of water in the limonite gravel layer.

The water level in piezometer holes 2 and 3 was measured at between 1.6 and 1.87 m below ground level in the months following the failure. In piezometer hole 1 the water level was between 3.6 and 3.8 m below ground level.

LABORATORY TESTING

The undisturbed tube samples consisted of a yellow-brown sandy clay. The sandy clay (SC) contained sub-angular, fine sand grains which were dominantly composed of feldspar with some quartz. Mica flakes and fragments of fossilized plant remains were also present. Two distinct moisture content zones (an upper moist zone and a lower wet zone) were visible in two of the samples. Shear box tests were performed on samples taken from each of these zones. The results are summarised in Table 1. X-Ray diffraction tests were performed to identify the clay minerals present.

Table 1

Hole	Depth	MC	LL	PL	LS	PHI	c'	Kaol	Goeth	Mont
3	6.4-6.5	38	83	27	18	18	7	80	20	-
3	6.5-6.6	62	101	28	22	13	6	80	15	5
5	6.8-6.9	39								
5	6.9-7.0	54	90	29	20	14	6	80	10	10

MC - moisture content (%)

LL - liquid limit (%)

PL - plastic limit (%)

LS - linear shrinkage (%)

PHI - effective residual angle of internal friction (°)

c' - effective residual cohesion (kPa)

Kaol - kaolinite (%)

Goeth - goethite (%)

Mont - montmorillonite (%)

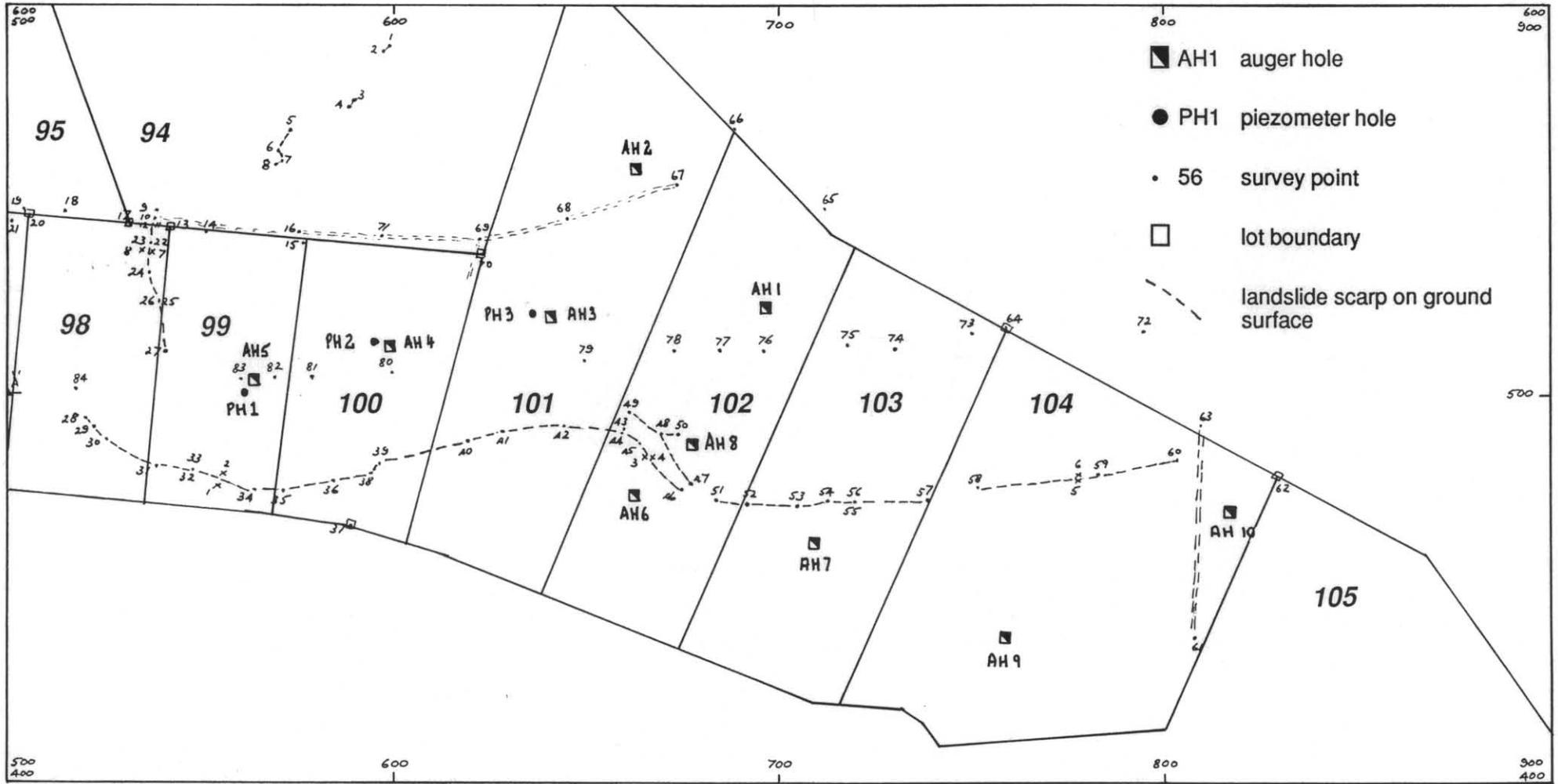
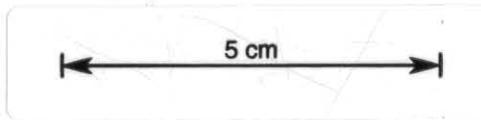


Figure 1. Extent of landslip area, Pleasant Hills subdivision Stage III, showing auger and piezometer holes and survey points.



The undisturbed tube samples also showed slickensides which were coated with a thin layer of darker brown clay. Sub-vertical fissures were clearly evident in the samples (particularly when these were allowed to air dry). Iron staining, up to 3 mm in width, was common about the fissures. Some black staining, probably manganese, was also present in places.

STABILITY ANALYSIS

Slope stability analysis is performed using the Factor of Safety concept. A commonly used definition is:

$$\text{Factor of Safety} = \frac{\text{Sum of restoring forces on slope}}{\text{Sum of driving forces on the slope}}$$

The restoring forces include the strength parameters (cohesion and friction) and the mass of that part of the soil resisting movement. The driving forces include pore pressure, load on the soil, and the mass of that part of the soil provoking movement. It can be seen from the above equation that the driving forces and the restoring forces are equal when the Factor of Safety (FS) equals one. The FS becomes less than one when the driving forces predominate and failure occurs.

Stability analysis is usually simplistic and does not take into account the variability of natural materials.

The shear box tests indicate that the effective residual cohesion on the presumed failure plane is 6 kPa. The effective residual angle of internal friction varies from 13–14°. The depth to the presumed failure plane at piezometer holes 1–3 is between 6.0 and 7.5 metres. Assuming a density of 20 kN/m³ for the materials above the failure plane, the pore pressure ratio (as indicated by the water depth measurements) varies from 0.344–0.393 at piezometer holes 2 and 3, to 0.183–0.260 at piezometer hole 1. The overall natural slope of the failed area varies from 9–10° but locally may be steeper than this over short slope segments.

Planar failure stability analysis was performed using the computer program PLANAR (Weldon, 1989). The results for the parameters mentioned above are shown on Figure 2 as a plot of pore pressure ratio versus factor of safety. The pore pressure ratios at which failure occurs (i.e. FS = 1.0) are greater than 0.4. This is higher than that indicated by observation after the actual failure.

The depth to the presumed failure plane measured at piezometer holes 1–3, the head scarp, and the presumed toe (at the tributary of Muddy Creek) determines the position for any circular failure arc. A large radius circle (i.e. Yc in the range 1200–1400 m) is therefore required to meet these requirements. Figure 3 shows a long section selected for circular stability analysis using the computer program SLIPCIRC (Weldon, 1987b). The results of the analysis are presented in Figure 4 as a plot of pore pressure ratio versus factor of safety. For a fully saturated slope (i.e pore pressure ratio equal to 0.5), the FS with Yc = 1200 m is 1.09, and for Yc = 1400 m FS is 1.163.

In summary, stability analysis shows that for the planar failure model, failure occurs at higher pore pressure ratios than that observed from field observations after the failure. The failure may have therefore resulted in a reduction of pore pressures after the event with some drainage occurring. The circular failure model technically does not produce failure (i.e. where FS is equal to or less than 1.0). This could be attributed to the fact that the tested samples may not have been from the actual failure zone. The failure zone may have been deeper than 7.5 m, the maximum depth of the auger holes.

FAILURE MECHANISM

During on-site discussions, Mr P. Phelps and Mr M. Vos mentioned that during the construction of the roadway located upslope from the failed area, a laterite gravel layer had been encountered. The gravel was wet and was both overlain and underlain by a soft clay. It is possible that this gravel is the same layer encountered during the investigation of the failed mass. In this event its relative position suggests that the present landslide is a re-activation of a pre-existing one. This is supported by the subtle hummocky surface morphology of the landslide area. Heavy blackberry and low scrubby vegetation hid this morphology during the earlier examination of the subdivision by Weldon (1988).

The slope of the laterite layer encountered in piezometer holes 1–3 is about 10°, approximately the same as the overall natural surface. This would tend to indicate a planar failure mechanism in preference to a circular failure arc.

The laterite gravel layer appears to be a key element in the instability of the area. It is probably continuous at least between piezometer holes 1 and 3. The gravel layer is apparently confined on both sides by clayey materials and may be lens shaped. The moisture content of the clays has probably been increased by water contained in the gravel. The clays would be softened by the presence of water, with a subsequent loss in strength. As excess pore pressures build up, the clay is unable to support the overlying materials and failure occurs.

IMPACT ON SUBDIVISION

The landslide head and side scarps are outlined in Figure 5. Obviously building cannot be located on the mass which is subject to movement. Once failure has occurred there is a possibility of further movement, particularly in the head and side scarp regions. These regions have a risk of instability caused by removal of support previously provided by the materials which have now failed. There is also a possibility of increased pore pressure due to aquifer sealing caused by disruption of the gravel layer which contains the groundwater.

Due to the irregular nature of the landslide scarp, areas of high instability have been indicated as a dotted area on Figure 5. A buffer zone, 20 m in width, has also been delineated around the landslide area for reasons of potential instability described above. This buffer zone is a region where building should not be permitted, and is included in the hatched area shown on Figure 5.

It is therefore concluded that lots 98, 99, 100, 101 and 102 are unsuitable for development. There may be sufficient area on lot 103 to construct a dwelling. It is advised that the house be sited as close to the roadway as possible. Lot 104, although steeper than the land which has failed, can be built on in the area not hatched on Figure 5. Auger holes on the steeper part of this lot indicate that the underlying material is dolerite (which in this case is considered stable) and not the Tertiary sediments which have failed elsewhere on the subdivision.

Lot 94 is sufficiently large that suitable house sites exist on the lot. Preferred sites occur adjacent to the long access track. Building should not be permitted on this lot within the hatched area shown on Figure 5.

Test pits on lot 97 indicate that the risk of landslide is lower on the front half of this lot than the rear half. A house can be located on this lot provided it is not sited within the hatched area shown on Figure 5.

REMEDIAL MEASURES AND MAINTENANCE

The landslide is of considerable dimensions, with the toe presumed to be along the line of an un-named tributary of Muddy Creek. The opposite bank of the creek is steep in most places and is acting as an effective toe buttress to restrain the landslide from enlarging in the downslope direction. With time, the head and side scarps of the landslide will become more pronounced and some bulging may occur in the toe area. Attempts to further restrict the movement are likely to be extensive, extremely expensive and impracticable, as the land is not suitable to build on. The head and side scarps could be graded from time to time to improve aesthetics and to seal off any cracks.

The developer has already mentioned the possibility of combining lots to form larger land parcels. Under no circumstances should building be permitted in the hatched area shown on Figure 5. Part of the hatched area could be given over to public open space and extensively planted with trees. This will have a stabilizing influence on the area.

Several council services traverse the landslide area, and constant attention will be required to ensure that those services carrying water are properly maintained. Cracked and damaged pipes will allow water to escape into the trench back-fill which, because it is a uniform-size crushed gravel, will tend to act as a large french drain. It may also act as a potentially

long subsurface wetting front. Where the trench is at right angles to the contours this has little consequence but where the trench is at shallow angle to the contours, a potential hazard may arise.

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[25 June 1990]

Figure 2. Results of planar failure stability analysis plotted as pore pressure ratio versus factor of safety.

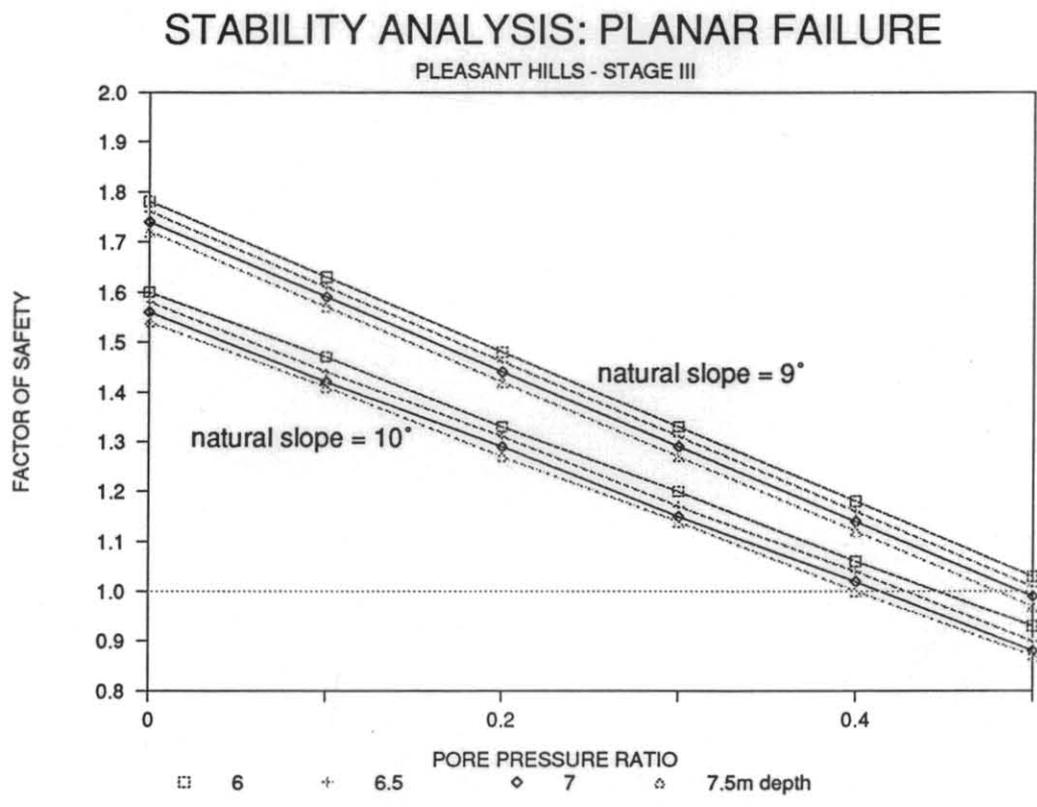


Figure 2a. Effective residual angle of internal friction = 13°, effective residual cohesion = 6 kPa.

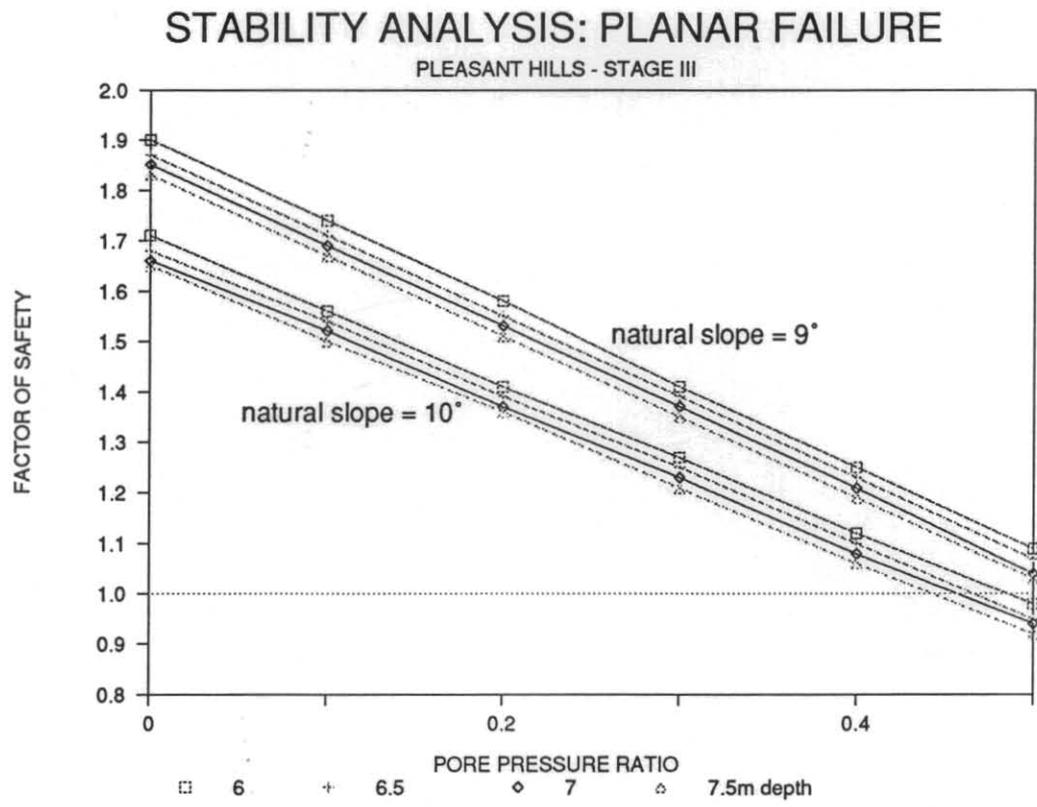


Figure 2b. Effective residual angle of internal friction = 14°, effective residual cohesion = 6 kPa.

STABILITY ANALYSIS: CIRCULAR FAILURE

PLEASANT HILLS - STAGE III

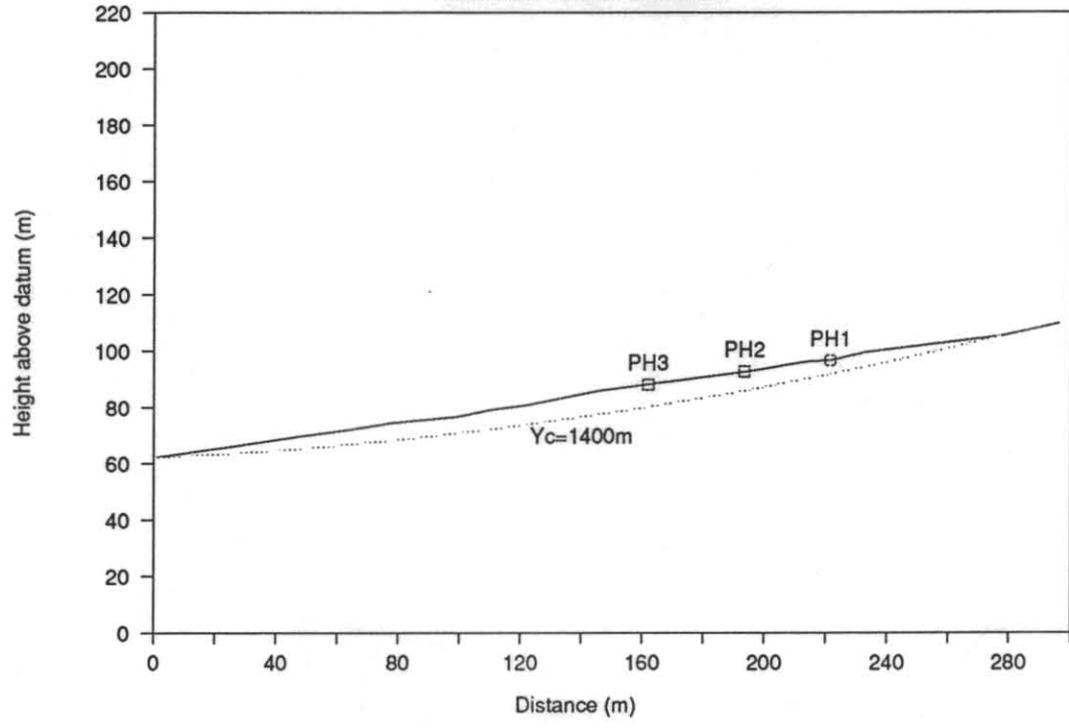


Figure 3. Cross section for circular failure stability analysis. Approximate location of piezometer holes shown. Failure surface for $Y_c = 1400$ m is shown.

STABILITY ANALYSIS: CIRCULAR FAILURE

PLEASANT HILLS - STAGE III

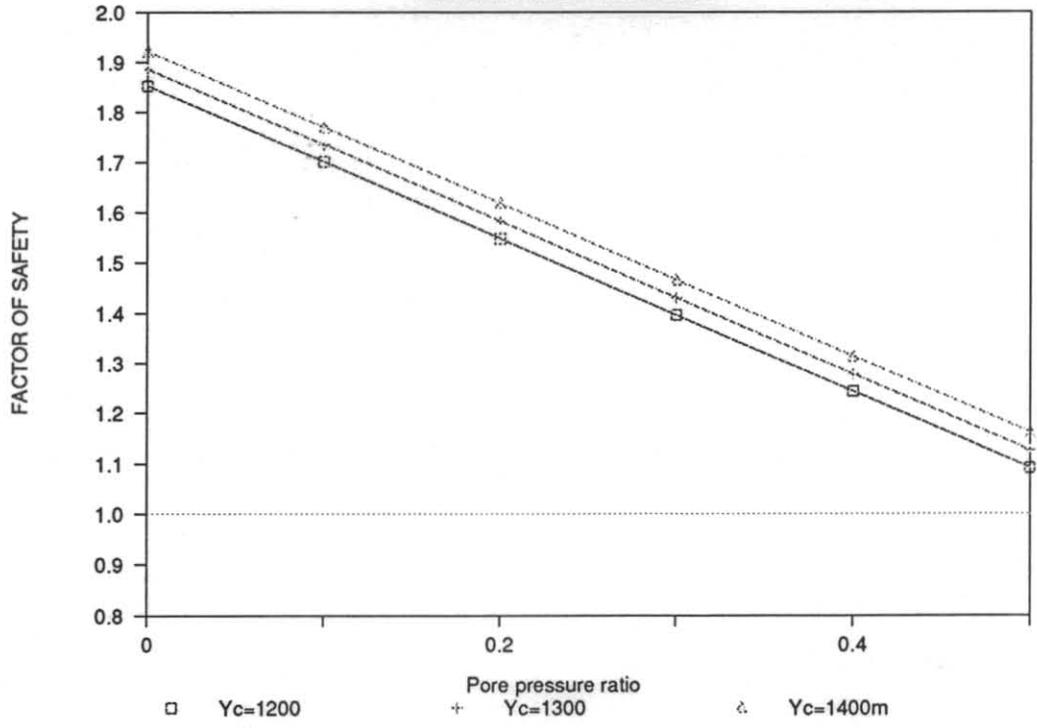
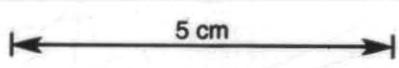


Figure 4. Results of circular failure stability analysis plotted as pore pressure ratio versus factor of safety.
 Effective residual cohesion = 6 kPa
 Effective residual angle of internal friction = 13°
 Assumed density = 20 kN/m³



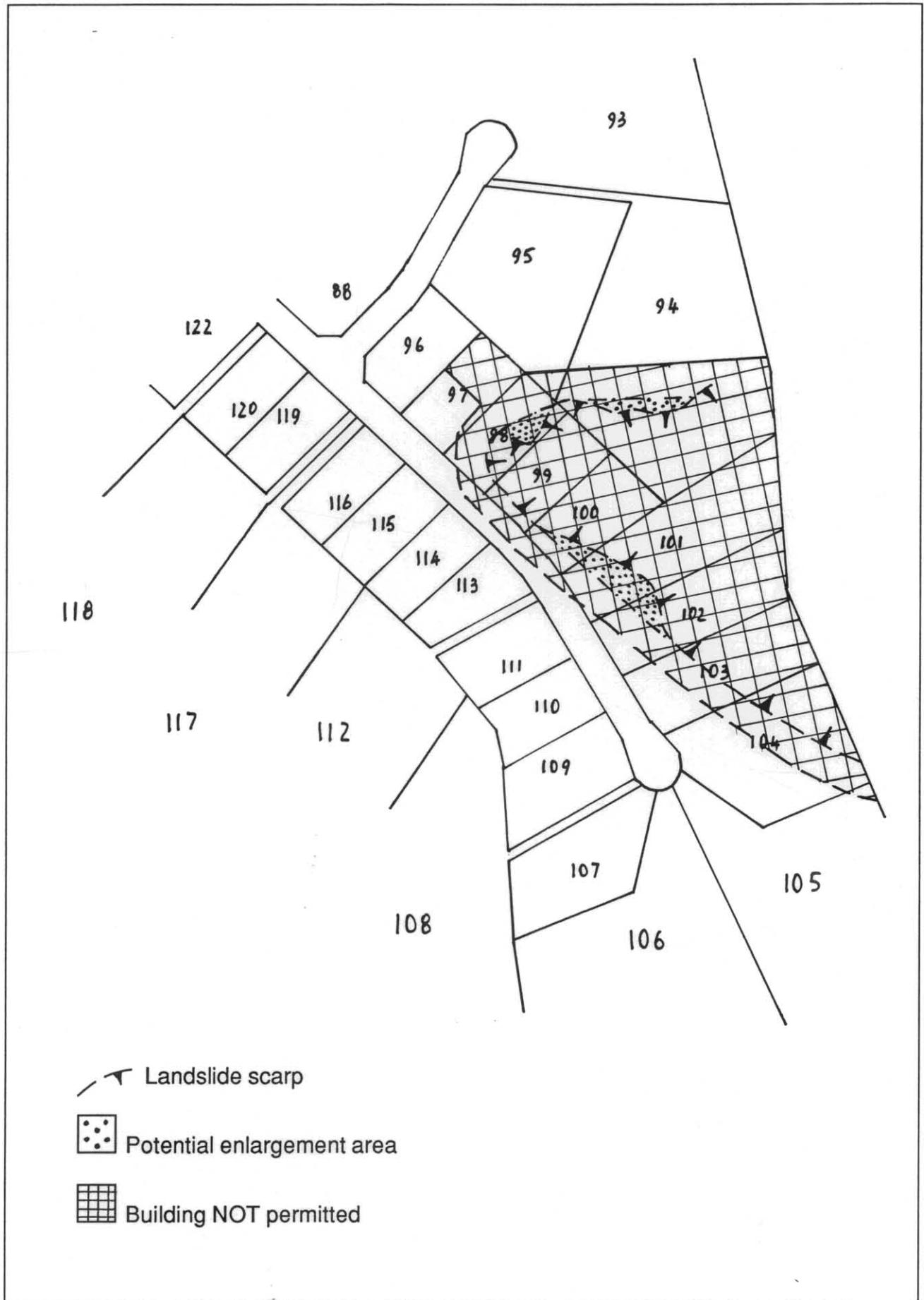


Figure 5. Areas considered NOT suitable for building, Pleasant Hills subdivision Stage III.