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The assessment of land stability at Panorama Heights Subdivision, Devonport

by P. C. Stevenson

This report is the latest of three reports on this subdivision. The first 'An examination of the Landslip Potential at the Panorama Heights Sub-division, Devonport' (Stevenson, 10 February 1972) raised doubts of the stability of the area on the evidence of trial pits and recommended measurement of soil strength properties and the calculation of a stability analysis.

The second report, the results of the tests and calculations by Mr V. Kowaluk of the Public Works Department was sent from the Department to the Director of Mines on 16 June 1972 and forwarded to the Devonport Municipal Council on 19 June 1972. This report is given in Appendix 1.

This report applies the results of the calculations to the geological and climatic environment of the area, assesses the risk of landslip and proposes the boundary of a proclaimed landslip area.

LABORATORY TEST RESULTS

The significant feature of these results is the variability of soil strength parameters. The reason for this variability is to be found in the origin of the clay, the weathering of basalt rock. This rock when extruded from its volcanic source is variable in composition and texture, and it has then been subjected to a deep weathering process which not only varies with depth below surface, but also enhances the compositional and textural variations of the original lava. The process of soil formation introduces its own variation and if as has been suspected, some parts of the subdivision have landslipped previously, any regular pattern of variation will have been disturbed.

Any sampling process carries with it limitations and it cannot be assumed that the values for cohesion and friction angle represent extremes.

STABILITY ANALYSIS

The assumptions in Mr Kowaluk's report may be regarded as reasonable on geological grounds. Assumption 1, of a uniform infinite slope is realistic in that the slope at Panorama Heights appears to have failed in the past neither at the top or bottom but in a position where the slope continues both uphill and downhill of the failure. The uniformity of slope over the area of failure is reasonably approximated in practice to within 2 or 3°, well within the range of variation used in the calculations.

The second assumption that seepage forces act only on a layer, below 20 ft is probably a conservative one, and the real situation may well be more dangerous than this after heavy and prolonged rainfall.

The third assumption is normal in calculations of this kind, with the added force that slope failure has been asserted in the writer's first report, on geological evidence, with an implication of zero cohesion.

A stability analysis is a mathematical model of a complex geological situation in which the factor of safety or output is seen to change when the slope angle, soil strength parameters and groundwater conditions are varied. A factor of safety less than unity indicates that failure would take place under the conditions set by the input. As a simplified model of a complex situation the analysis has its limitations and no undue weight should be given to the exact value of the factor of safety. It can perhaps be put in perspective by regarding it as the number of times stronger than necessary than the foundation material is. In normal house building practice an estimate of such a factor is rarely attempted as the material on which the house is built is many times stronger than necessary. It will therefore be plain that the attempt to do so at Panorama Heights is an exceptional process, only made necessary by the unusually unfavourable soil conditions there.

In a situation where so much is at risk an average friction angle is not an acceptable concept. If any part of the area has a lower value and is therefore more unstable than the average this may fail, and the object of the investigation is to ensure that no part fails.

Additional factors not included in the analysis

With the above in mind it will be apparent that even a factor of safety as high as 2 must be regarded as unusually low for this land use.

Two other items have not yet been considered. No attempt has been made to introduce the weight of the house into the analysis. The stability has only been that of the bare ground. The weight of the house contributes nothing to the strength of the ground and only contributes to the unfavourable load on it.

The second item is the unpredictable stress that will be thrown on the ground by the use of the area as a housing subdivision. Some examples may help to illustrate this. A garden sprinkler giving 200 gallons per hour, left on for 4 days, a situation not unusual over a long week-end, adds 64 tons of water to the area that it covers, and at the same time softens the clay soil beneath the surface and makes it less able to sustain the load. By comparison 5 in of rain adds 70 tons to a 60 ft x 100 ft block. It is inevitable that trenches for sewer pipes will be cut along the contours. When backfilled they will form lines of high stress in the soil near the surface, for the cohesive strength of the soil in them will be almost nil and they will at the same time act as collectors for any run-off and will carry it into the most highly stressed region. Such a stress concentration could initiate slope failure with progressive overstressing of the surrounding area.

The attached drawing (fig. 1) illustrates a slope analysis of the area, based on contours supplied by the Devonport Council. The outer line contains all areas steeper than 14° while the inner line contains all areas steeper than 19°. The slope angles are those used in the stability analysis, and in view of the preceding discussion the area of 14° and steeper must be regarded as the area at risk. An apparent anomaly should be pointed out, that the area suspected of having failed already does not come within the risk area. This however, is logical, because the act of failure flattens the angle of slope and so increases the future stability.

It must be pointed out that landslips tend to migrate upslope, because the land immediately uphill from a slip is left oversteepened and under-supported. The risk area has therefore been rationalised to some extent and the area it is proposed to proclaim as a landslip area under Section 431A of the Local Government Act 1962 is indicated in Figure 1.

PREVENTIVE MEASURES

The land within the proclaimed area, if left undisturbed can be expected to remain stable for periods of some years, only becoming active after prolonged wet weather. A slip then taking place would be destructive of landscape and could menace houses downhill of the slope. It is recommended that trees be planted in the area as they are at all points the best means of landslip prevention. They provide shade and soil cover to prevent excessive dessication, remove excess moisture and mechanically bind the soil layers to considerable depth and have the advantages of lack of maintenance, cheapness and beauty. Physical reshaping of the ground by bulldozing should be avoided as the present slope will be one of at least temporary stability.

The same recommendations apply throughout the subdivision outside the proclaimed area. In addition sewer and stormwater pipes should be constructed so that slight local soil failures at high stress areas will not cause failures in tension.

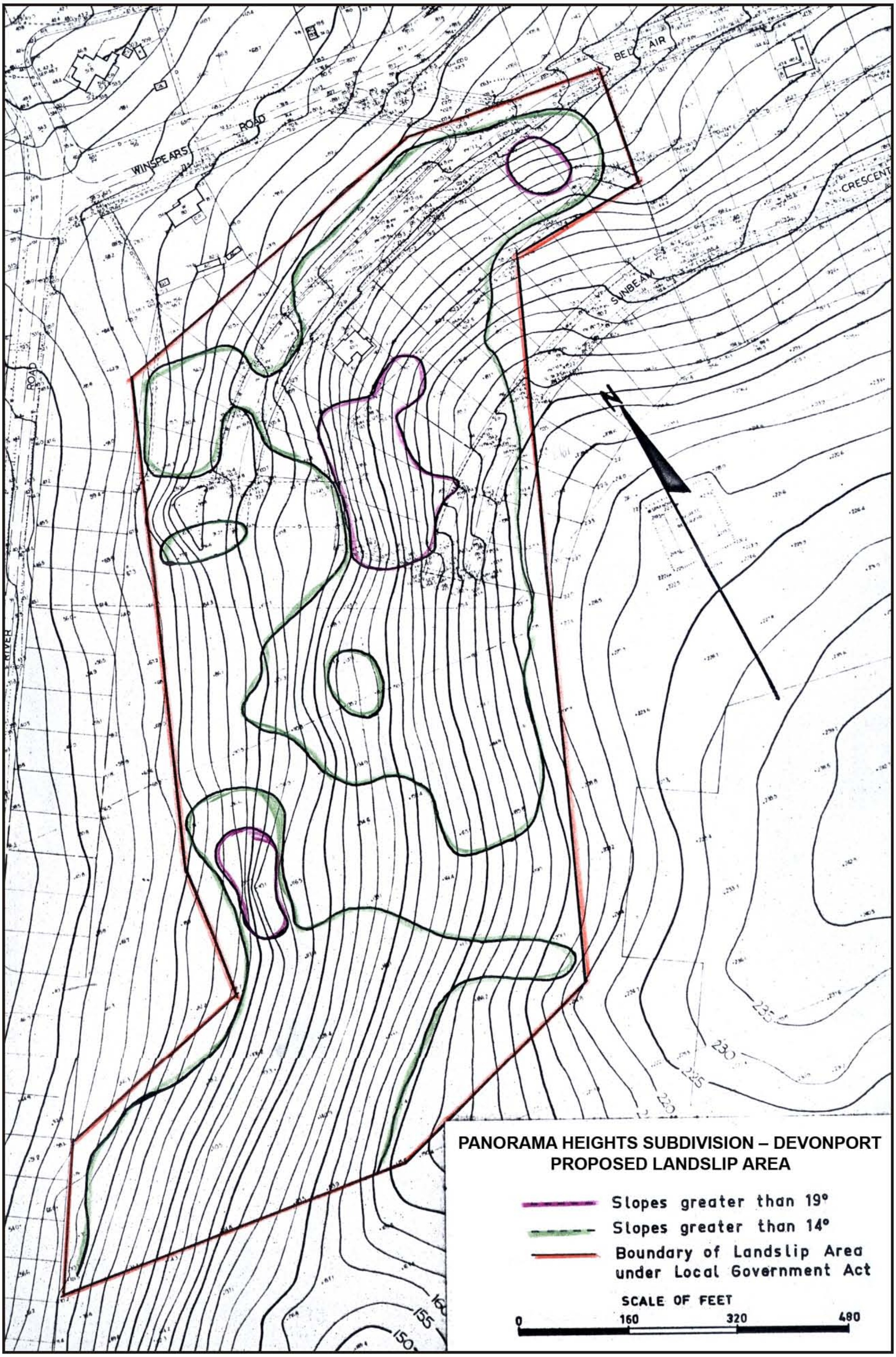
PANORAMA HEIGHTS SUBDIVISION – DEVONPORT
PROPOSED LANDSLIP AREA
Slopes greater than 19°
Slopes greater than 14°
Boundary of Landslip Area under Local Government Act
SCALE OF FEET
0
160
320
480
WINSPEARS ROAD
BEL AIR
CRESCENT
SUNBEAM
ROAD
RIVER
235
230
225
220
155
150

Figure 1

Appendix 1

Tests to assess the possibility of landslip developing on the Panorama Heights Subdivision.

W. Kowaluk

The southern part of the sub-division, where dolerite outcrops was considered safe from landslips and was not investigated. The investigation was limited to the central part of the sub-division, featuring red basaltic soils and steepest grades.

TEST HOLES

The locations of the test holes were selected by the writer in consultation with Mr P.C. Stevenson and are shown in Figure 2.

Drilling was carried out by Department of Mines 'Gemco' drilling rig using 4" continuous flight twist auger. Undisturbed 1½" and 3" samples were obtained for laboratory testing. Bore logs are given below.

Bore hole 1

Depth (ft)	*Description*	*Remarks*
6′ 1"	Basaltic red clay	
6′ 7"	Firm red clay	1½" sample
9′ 0"	Firm red clay	
9′ 6"	Firm red clay	1½" sample
13′ 3"	Firm red clay	
13′ 9"	Firm red clay	1½" sample
17′ 8"	Firm red clay	
18′10"	Firm red clay	3" sample
22′ 2"	Red clay changing to yellow clay	Free water
25′ 3"	Too rocky to sample	
26′ 4"	Firm red clay	3" sample
30′ 3"	Firm red clay	
31′ 1′	Hard red and grey clay	3" sample

Bore hole 2

Depth (ft)	*Description*	*Remarks*
6′ 0"	Red basaltic clay	
6′ 7"	Firm red clay	3" sample
9′ 0"	Firm red clay	
9′ 6"	Firm red clay	1½" sample
13′ 0"	Firm red clay	
13′ 6"	Firm red clay	1½" sample
15′ 0"	Firm red clay	
16′ 4"	Firm red clay	3" sample
20′ 2"	Firm red clay	
21′ 7"	Firm red clay	3" sample
25′ 2"	Firm red clay	
26′ 0"	Firm red clay	3" sample
29′ 0"	Basaltic sandy gravel	Free water
32′ 0"	Basaltic sandy gravel	Free water

Bore hole 6

Depth (ft)	*Description*	*Remarks*
10′ 9"	Red-brown clays	
11′ 9"	Firm brown clay	3" sample
13′ 0"	Firm brown clay	
14′ 0"	Friable brown-yellow clay	3" sample
16′ 3"	Friable brown-yellow clay	
17′ 3"	Friable brown-yellow clay	3" sample
19′ 6"	Friable brown-yellow clay	
20′ 9"	Firm red-brown clay	3" sample

Bore hole 8

Depth (ft)	*Description*	*Remarks*
9′ 0"	Red basaltic clays and some loam	
9′ 6"	Firm red clay	1½" sample
11′10"	Firm red clay	
12′11"	Firm red clay	3" sample
17′ 9"	Soft red clay	
18′11"	Hard red clay	3" sample
22′ 0"	Soft red clay	
22′ 8"	Soft brown clay	3" sample
60′ 6"	Soft brown clay	
41′ 6"	Hard yellow clay	3" sample
52′	Soft brown clay	Damp faces

All holes to the depth drilled were in basaltic clay and no sediments or other materials were found.

The clays varied in colour from red to brown and yellow. The colour variation is due to different degree of oxidation, leaching and perhaps differences in chemical composition of the original basalt mass, and does not necessarily signify variation in physical properties of the clay.

Free water was encountered in some of the holes. This is due to local seepages, but no definite continuous water table exists.

LABORATORY TESTS

Laboratory tests consisted of classification tests (sieve analysis and Atterberg tests) and triaxial tests.

The classification tests are summarised in Table 1. All samples fall within the CH (heavy clay) designation; sample D1 showing higher proportion of fine sand, probably due to leaching.

Table 1. CLASSIFICATION TESTS

	Sample No.						
	D2	*D1*	*D5*	*D3*	*D7*	*D4*	*D6*
B.S. sieve No.	*Percentage passing*						
14	100	100					
25	99	99					
36	98	97		100	100	100	
52	97	92	100	99	99	99	100
100	93	82	98	99	99	99	99
200	87	68	95	98	97	98	98
L.L.	86	86	74	96	111	87	75
P.I.	57	55	51	61	76	46	31
L.S.	21	20	20	23	27	19	17

Triaxial tests were of the multistage consolidated undrained type, permitting the evaluation of the two soil shear strength parameters, cohesion (C′) and friction angle (4′), required for stability analysis. The triaxial test results are shown in Table 2. The remoulded samples were obtained by breaking up and remoulding the original undisturbed samples to the initial density and moisture content, as close as it could be achieved.

Table 2. TRIAXIAL TESTS

Test hole No.	Depth (ft)	Sample No.	M.C.	Wet density lb/ft^3	*Shear strength*			
					Undisturbed		*Remoulded*	
					C′ lb/ft^2	C′ ′	lb/ft^2	′
1	18′0"-18′6"	D2	50	104	1150	14.5°	-	-
	25′9"-26‘3"	D1	53	107	540	31.0°	-	-
2	6′0"- 6′6"	D5	41	111	1150	17.0°	575	24°
	15′9"-16′3"	D3	55	105	1224	21.0°	-	-
	21′0"-21′6"	D7	55	107	1300	20.0°	260	25°
6	16′8"-17′2"	D4	66	103	720	28.0°	-	-
8	18′4"-18′10"	D6	63	103	980	24.0°	-	-

Triaxial test results show a wide variation in soil strength parameters, the cohesion varying between 540 lb/ft^2 and 1300 lb/ft^2, and the friction angle between 14.5° and 31°. Upon increase in the value of the friction angle, variations, particularly in moisture content, were also found to exist within any one sample.

No definite soil layers could be identified having identical or similar strength properties, the variation in strength is distributed at random in depth and location. This is not inconsistent with the type of soil being investigated.

STABILITY ANALYSIS

Due to variation in soil strength properties and topography of the area (changing slopes), analysis based on the actual existing set of conditions could not be made. Several simplifying assumptions had to be made. The assumptions were aimed at reproducing natural conditions which could arise in future.

The assumptions were:

(1) Uniform infinite slope. Two cases were considered, 14° slope and 19° slope.

(2) Two soil layers were assumed to exist 0–20 ft in depth and below 20 ft, the latter subject to seepage forces. Soil in each layer was assumed to be homogeneous with respect to density and strength.

(3) At incipient failure, the cohesion was considered zero and only the friction forces resisting sliding.

Analysis was carried out using ‘Stabilan’ computer program. Friction angles used were 15°, 20°, 25° and 30° covering the range of friction angles found during tests. Other variables were, slip circle radius and the location of the centre of rotation. In each case the factor of safety was obtained to assess the degree of stability. A factor of safety of 1.00 indicates incipient failure i.e. the actuating forces are balanced exactly by the resisting forces, in this case the friction within the soil.

Results of stability analysis

Slope angle, 19°

Soil friction angle	15°	20°	25°
Factor of safety (min.)	0.90	1.09	1.14
Factor of safety (max. analysed)	0.98	1.32	2.01

The minimum factor of safety occurring at shallow depth (up to 6 ft) and the maximum at depth of 40 ft.

Slope angle, 14°

Soil friction angle	15°	20°	25°	30°
Factor of safety (min.)	1.07	1.46	1.87	2.31

The minimum factor of safety again occurring at shallow depth and increasing appreciably with depth.

DISCUSSION AND CONCLUSIONS

It can be seen from the above analysis that the factors of safety against sliding are generally low, reaching satisfactory levels only on gentle slopes and where the soil friction angle is high.

Very low and high friction angles were found only in limited areas, most samples having friction angles in the 20° to 25° range, which could be regarded as average in most areas.

On this basis the possibility of a landslide developing on slopes of 19° and steeper, with a safety factor of only 1.1 approximately, is quite high. Below 19°, the danger diminishes and on slopes less than 14° the possibility of a landslide developing is considered to be remote, but cannot altogether be ruled out.

In all cases, shallow slides are more likely to occur than deep ones, unless under special circumstances, a weak layer develops at some depth. Such a case could arise if water was allowed to reach and percolate along the interface of clay and decomposed basalt that was encountered at the bottom of some test holes.

Development such as terracing, benching, excavation for garages and similar earthworks which tend to change the natural slopes, as well as intensive irrigation, broken stormwater drains, digging of trenches for pipe-laying, which would subsequently collect and concentrate water, increase the possibility of a landslide.

[22 June 1972]

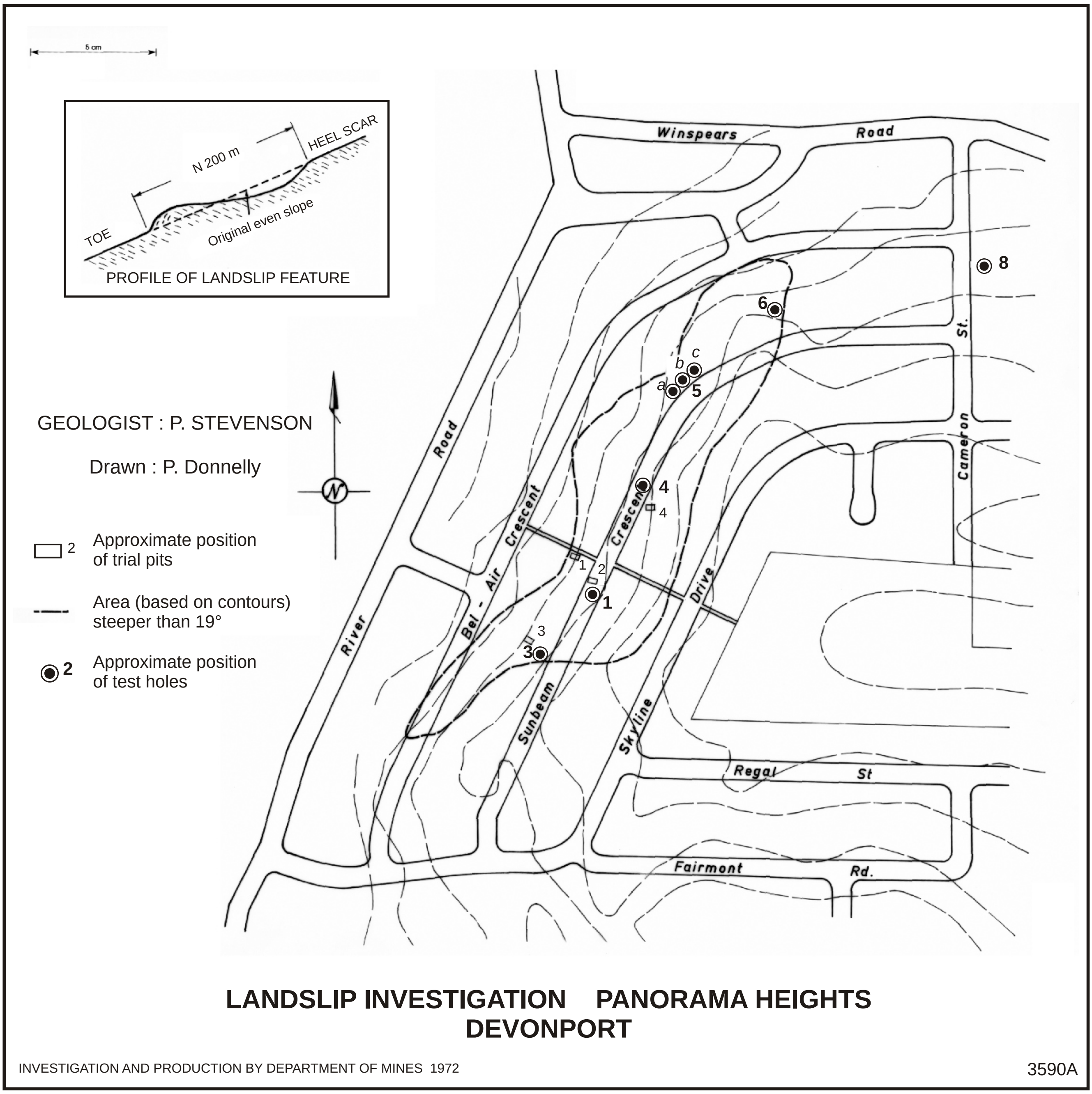

Figure 2