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CLIFF LINE HAZARD DEFINITION STUDY
at
TUDIBARING HEADLAND
COPACABANA NSW

Undertaken on Behalf of

GOSFORD CITY COUNCIL

Our Ref: 210896RN.DOC\X017-5

Prepared: 30 September, 1996

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1. EXECUTIVE SUMMARY

This report describes a geotechnical evaluation of the previously classified "High Risk" area of the Tudibaring Headland Area at Copacabana, north of MacMasters Beach.

Predicated on a management period of 50 to 100 years, the work has predicted the recession of the cliff area and identified areas of Immediate High Hazard, High Hazard and Medium High Hazard to residential properties.

The work has concluded that:

1. Whilst the majority of the cliff area instability is natural and engineering works of little avail, there are some items [principally drainage & inappropriate earthworks] which are aggravating the instability process; these items thus require prompt attention.
2. By the implementation of suitable "building restrictions", all existing lots within the study area may be used for residential building purposes.
3. Land resumptions are not presently required.
4. Parts of some allotments are unsuitable for new development.
5. There is a significant "public risk" in the lower wave cut platform area due to falling of large individual rocks and the possibility of a large rock fall near areas of major rock undercut. Suitable public warning signs are thus necessary in the area; security fencing and cliff access restrictions may also be necessary in some locations.
6. Specific engineering advices are required to identify the locations of security fencing and warning signs.
7. The cliff recession at Copacabana should be monitored by conventional survey, assisted by geotechnical personnel, at intervals of approximately 5 years.

2. INTRODUCTION

In 1995, *GOSFORD CITY COUNCIL* engaged WBM Oceanics Australia to prepare a Coastal Management Study to identify the various coastline hazards in the Central Coast area; this study identified a number of headland areas which were considered to have a moderate to high risk. Following review of this Study Management Plan, in October, 1995 ***Gosford City Council*** invited Shirley Consulting Engineers Pty Ltd [SCE] to submit a proposal to carry out a detailed hazard assessment of the "High Risk" Area of Tudibaring headland at the northern end of MacMasters Beach [being Lots No. 873 to 896 Del Monte Place, Copacabana], the area previously identified in the WBM Oceanics Australia study.

In April, 1996 The Council accepted SCE's proposal for the Cliff Line Hazard Definition Study to report to the Council on the following:

a) The present cliff / headland stability and possible extent of cliff recession over the next 50 to 100 years.

b) Those parts of the study area considered to be at serious risk.

c) Outline suggestions as to possible methods of mitigating the risk.

d) Recommendations as to limitations on further development [including restrictions on future building works] on the headland area.

It is also understood that the overall purpose of the work was to provide a ***"geotechnical"*** input into the long term planning processes in the Copacabana area.

To assist in our work, the Council provided copies of extracts from relevant building application files, a base cadastral plan of the area, the "Coastal Management Study and Coastal Management Plan" prepared by WBM Oceanics & Planning Workshop in August, 1995 and colour photocopies of photographs of the headland area taken from a helicopter in 1995.

3. INVESTIGATION METHOD & WORK UNDERTAKEN

The investigatory method adopted in this study was one of assembly of available geotechnical data on the area, detailed site observations and geotechnical mapping [including detailed examination of the cliff area], engineering analysis and review of historical records of coastal cliff retreat. As a first step in this process, job file records from previous geotechnical investigations in the area carried out by this firm were reviewed and pertinent geotechnical data extracted. Subsequently, a conference was held with Council Officers on 9 May, 1996. After this conference, the available geological data was assembled from the firm's "in house" geological database and the Department of Mineral Resources [Geological Survey] of NSW. In addition, aerial photographs of the headland area were obtained for the period 1954 to 1996 from the Central Mapping Authority and reviewed. The material obtained during the course of this "literature review" is listed in the "References / Documents" section of the Appendix.

Subsequently, an inspection of the area for this project was carried out by our *Messrs A F Shirley & J P Lewis [Geotechnical Engineer]* on 21 May, 1996 to establish project planning constraints. After this inspection, an extended detailed area inspection to establish pertinent geological features and cliff access constraints was carried out by our *Messrs. Shirley & Imlay [Senior Engineering Geologist] in company with Vertigo High Access [Specialist Access Contractor]* on 1 June, 1996.

Following this inspection detailed cliff face inspections were carried out by Mr A F Shirley, assisted by Vertigo High Access on 19, 20, & 21 June 1996 using abseiling techniques. During the course of the these inspections:

- The cliff face was measured and the various geological units exposed at the cliff face observed and mapped.
- A number of samples of the various materials on the cliff face collected for later examination.
- Measurements were made of a number of major cliff features [particularly the two large caverns and zones of large open jointing].

The geotechnical and engineering studies undertaken for this project have thus embraced:

a) Review & determination of the regional and local geology, from published & "in-house" sources and field checking.

b) Site geological mapping, interpretation & assessment.

c) Evaluation of the weathering & stability of the various rock units and rock blocks on the cliff face.

d) Documentation of the site features by cliff mapping and site photography [both still 35 mm and Hi8 Video].

e) Engineering and geological assessment of the properties of the various site rock units and their potential impact on cliff top development.

f) Consideration of the pattern of recession in the study area, review of the historical record of recession and possible impact of such recession on development.

g) Establishment of "hazard" zones for the cliff top area to assist in the planning process.

h) Formulation of advices on preferred methods of building construction in areas of Low to Medium hazard.

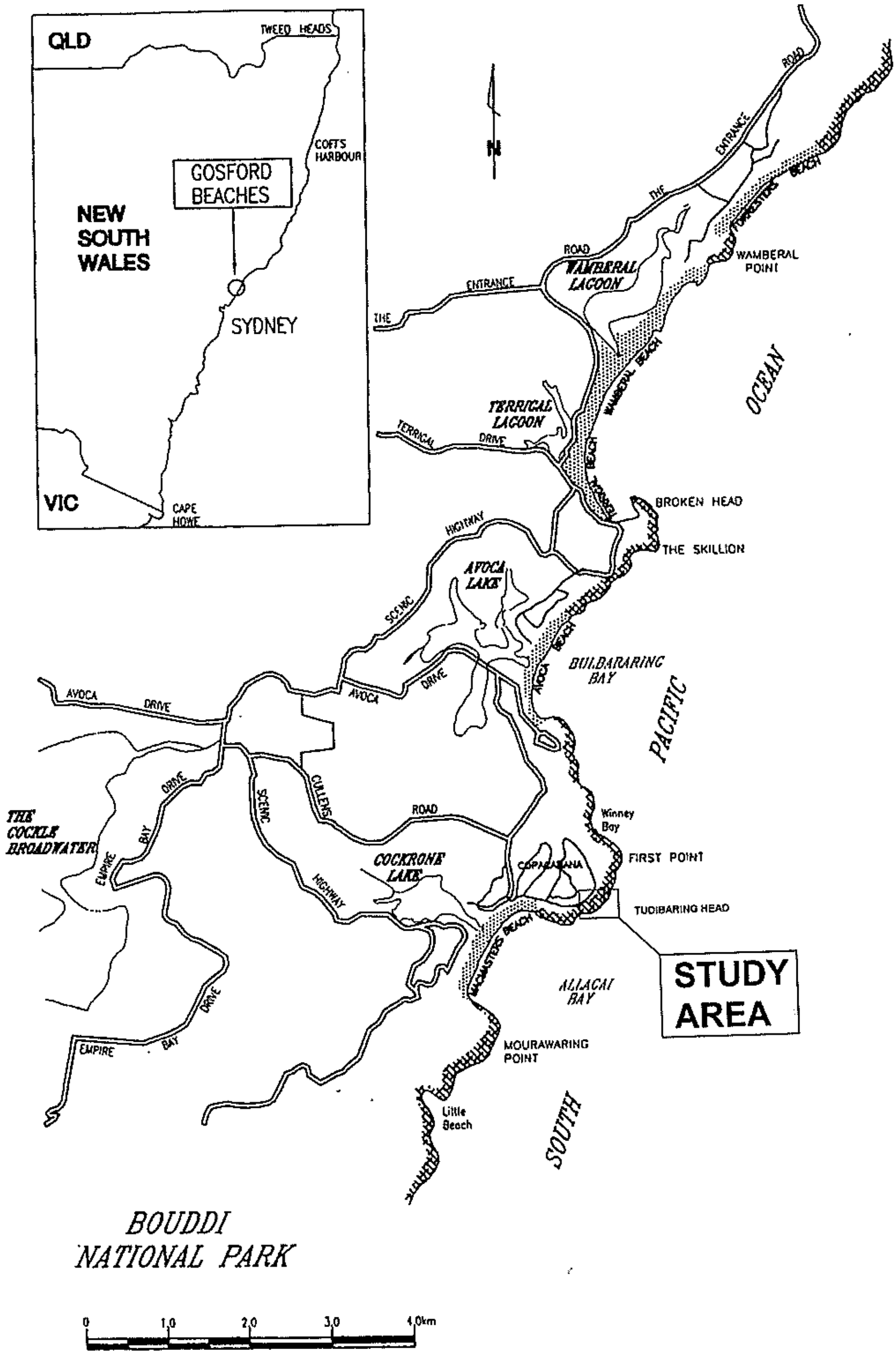
The work was carried out by our Messrs. Shirley, Imlay & Lewis during the period May to September, 1996.

4. SITE CONDITIONS

The area of geotechnical & geological study is indicated on Drawing No. X017G1 and the local area site geology is presented on Drawing No X017G2. The detailed plan of the study area is presented as Drawing No. X017G3 in the Appendix [viz: the Hazard Zoning Plan]. It is also to be noted that this Hazard Zoning Plan is a compilation of features from a number of sources, including an enlargement of the 1973 area 1:4,000 orthophotomap. In regard to the orthophotomap, during our site work it became apparent that whilst the contours shown on the orthophotomap were correct "in form", they had local height errors of up to 6 metres; these errors are possibly due to the enlargement of the orthophotomap and proximity of cliff area. The contours shown on the Hazard Zoning Plan should thus only be considered as "indicative" of the land form, but not necessarily correct in reduced level.

Some of the features noted during our inspections of the cliff and cliff top areas are presented on the Hazard Zoning Plan and Geological Sections [Drawings No. X017G4 to G7 (incl.) in the Appendix]. Also, colour photocopies of a number of the photographs / video taken during the investigation are presented in the Appendix to illustrate some of the stability

TITLE:	DRAWING NO.	REV:
SITE LOCALITY PLAN & STUDY AREA **CLIFF LINE HAZARD DEFINITION STUDY** **TUDIBARING HEADLAND, COPACABANA**	**X017G1**	**0**



BASE PLAN FROM REPORT BY WBM
OCEANICS AUSTRALIA – AUGUST 1995

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	CLIENT:		
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problems identified by this study. An edited video tape of the cliff area is also provided separately.

The site and cliff area is described in the following paragraphs:

4.1. Land Generally

The study area is located on the southern side of Del Monte Place, Copacabana and forms much of the southern headland area of Tudibaring Headland; the headland area First Point [see Drawing No. X017G2] is also several hundred metres to the north of the study area. The various allotments of land the subject of this study are generally orientated in a north-west / south-east direction with the southern [cliff side] boundaries being some 5 to 40 metres from the cliff area.

Within the northern part of the study area, residential properties are located well back from the cliff line [generally about 30 to 40 metres] and there is a right of way between Lots No. 890 & 891 which traverses on the seawards side of Lots No. 891 to 896. Observation of this right of way indicated that extensive soil erosion was occurring and that run-off was being concentrated near the rear of Lot No. 890.

Also, during the course of our inspections it was noted that:

a) The vegetation evident in the Council Reserve area of the study area consisted of primarily a large grassed area over the northern half [from Lot No. 886 northwards]; within the southern section however, as well as grassed areas there were extensive native shrubs [including coastal banksia & bottle brush] as well as the exotic bitou bush.

b) Some of the property owners [particularly near Lots No. 885 to 887, and 876 / 877] had extended garden planting activities well onto the Council reserve.

c) Some areas of localised filling near the cliff top area were noted which are creating an 'over steep' slope to some parts of the cliff.

d) Public access across the reserve was being denied by a fence near the Lot No. 876 / 877 boundary.

4.2. Cliff Area

Inspection of the cliff area from:

- the wave cut rock platform below much of the cliff adjacent to the site, and
- local rock ledges and caverns in the cliff near the high tide mark, and
- the cliff itself during physical descent of the cliff face at four locations,

indicated that the cliff headland in the vicinity of the study area consists of two distinct areas, viz:

a) An upper steeply sloping area [defined by the "cliff boundary" shown on the Hazard Zoning Plan], and

b) A near vertical cliff [defined by the "major cliff line boundary" shown on the Hazard Zoning Plan] of between 52 and 62 metres high which rises from the rock platform at a slope angle of between 75° to 80°.

Some parts of the near vertical cliff are, in places, vertical / over vertical and extensively undercut; there are also two large cavern areas in the proximity of Lots No. 883 to 888 Del Monte Place.

In the vicinity of Lots No. 880 - 887 the upper portion of the cliff line has a moderate to steep slope [approximately 35°] formed by the weathering pattern of the bedrock materials exposed in this area [viz: siltstone with interbedded claystone]. This area of the cliff slope was observed to be in active recession with soil slumps occurring over much of the slope. Further, during the course of our study two soil slumps occurred in the vicinity of Lot No. 881 between June & August, 1996.

Inspection of the main cliff area also revealed that:

i] the strength stability of the rock face & cliff varies considerably over the exposed rock face and cliff line, and

ii] there were many "near vertical" open joints trending north west, with some open joints being as wide as 150 mm, and

iii] recent [i.e. last 12 months] rock falls had occurred within the caverns and along the cliff face area, and

iv] there was an area of extensive seepage over the cliff near Lot No. 887 [see heavy seepage noted on the Hazard Zoning Plan].

In addition, during the course of our fieldwork, a large rock mass fell from the cliff area near Lot No. 876 overnight on 19 June, 1996 [see Plate No. P18].

All the above observations confirmed that the cliff face area was undergoing active coastal recession. This recession process is discussed further in the ***COMMENTS*** section of this report [Section 9].

4.3. *Northern Area - Concentrated Drainage & Erosion*

As noted above, in the vicinity of Lots No. 893 to 896, significant soil erosion was observed within the right of way on the seaward side of the properties. Whilst it would appear that the majority of this erosion is caused by concentrated drainage from the building works on Lot No. 896, much of the erosion is the result of the earthworks for the right of way. Significant silt transport from this area was also noted, with an extensive silt deposit near the rear of Lot. No. 890.

Near the rear of Lots No. 889 & 890 [see seepage area noted on the Hazard Zoning Plan] the ground conditions during the entire study period were wet underfoot and considered symptomatic of both:

- the concentration of surface run-off from the right of way, and
- subsurface water seepages from within the rock mass.

It was also apparent that the concentrated run-off from the drain on the right of way was being channelled in the direction indicated on the Hazard Zoning Plan; as such, this concentrated run-off could be having a significant effect upon the long term stability of the cliff line in this area.

5. REGIONAL GEOLOGY

The geological units outcropping in the general region of the site have been identified as being of the *Terrigal Formation* and are designated as ***Rnt*** on the Local Geology Plan [Drawing No. X017G2] included in this report. This geological formation is of lower Triassic Age and comprises the uppermost 212 metres of the Narrabeen Group. The rock units comprise an interbedded sequence of sub-horizontally bedded and interbedded quartz-lithic sandstones, laminated siltstones and claystones. These materials have been interpreted to be of alluvial in-channel and floodplain depositional origin. The in-channel depositional environments include channel floor, channel bar, point bar and abandoned channel. The floodplain environments are less differentiated but include levee, backswamp, marsh and crevasse splay. [Ref: 2]

The upper Terrigal Formation sandstones tend to be quartz-rich, with a high lithic content that increases toward the base of the unit. The sandstones characteristically have a sideritic or calcite cement. Common lithic fragments include feldspar, devitrified acid and intermediate volcanics, silicic and shaley cherts, reworked silts and shales. The presence of the iron mineral haematite stains the rocks and clays a brown / red colour.

Structural and joint orientation measurements by this firm at outcrops of the Terrigal Formation, indicate the presence of the prominent, regional North-East trending joint set (common to the Sydney basin) with secondary joint sets which are more variable in strike direction mainly trending 90°, 110°, 130° and 165°. These general joint trends were confirmed at this site. Areas of highly fractured and closely jointed outcrop are common within the Terrigal formation with many locally porous zones occurring within the rock units, as well as along joint planes, natural fracture zones, etc..

6. SITE GEOLOGY

The Tudibaring Headland consists of a highly variable sequence of interbedded sandstones siltstones and claystones. The rock exposure observed in the cliff is between 55 and 80 metres in thickness and exhibits numerous joints and bedding plane fissures. Differences in rock strength and weathering rates of the various stratigraphic units, combined with the rock discontinuities [viz: joints and bedding planes] create numerous zones of active and potential instability within the cliff area. The major cliff re-entrants also exhibit substantial undercutting which has severely jeopardised the re-entrant stability.

In addition, within the central area of the study, two large caverns [see Plates No. P2, P4, P5, P5 & P9] exist as a result of the soft strata near the wave impact zone; these caverns are 20 to 25 metres wide, some 12 to 14 metres high and have developed to some 14 to 16 metres from the main cliff line. Inspection of the northern cave [the smaller of the two] indicated that there was a wide [approx 300 mm] prominent north west [approx 110°] joint set which was being aggressively eroded by wave action. From debris in the caverns [see Plates No. P12 & P13], it would appear that they are enlarging under the wave action at a relatively rapid [tens of years, rather than hundreds of years] rate. The enlargement of the caverns also presently appears to be proceeding more vertically than horizontally.

6.1. Stratigraphy

The stratigraphy of the cliff materials was measured during the cliff descents and is detailed within cliff face sections detailed on Drawings X017G4 to G7 [in the Appendix]. In the following geology, the descriptions of the rock units exposed are broadly correlated to the previous work undertaken by K L McDonnell [Ref. 2].

From our site observations, the geological units at the site have been identified as lying between a capping sandstone unit [Unit H] exposed towards the top of the sequence and a basal sandstone / siltstone [Unit C] which forms the majority of the wave cut platform. Whilst descriptions of each unit are set out below, due to variations in bedding orientation, the thickness of each unit varies along the cliff face exposure.

TITLE:

LOCAL GEOLOGY PLAN
CLIFF LINE HAZARD DEFINITION STUDY
TUDIBARING HEADLAND, COPACABANA

DRAWING NO: X017G2

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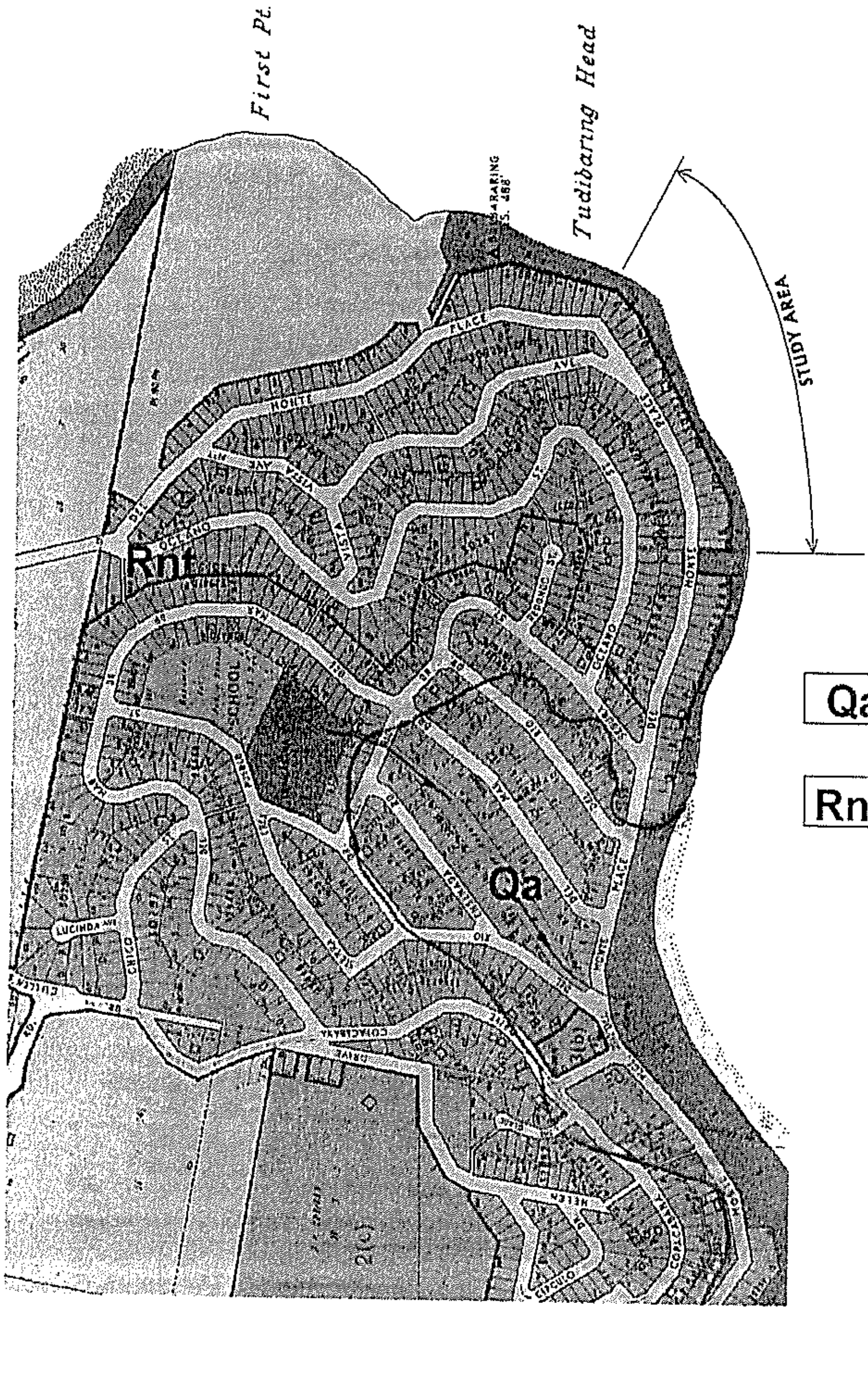

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Further, due to spatial differences between depositional environments, the thicknesses vary from those set out in the stratigraphy described by McDonnell.

6.1.1. Unit Rnt - H (including G)

This unit is exposed towards the north end of the study area and forms the capping unit which is exposed at the rear of Lot No. 889 Del Monte Place. The sandstone is highly to moderately weathered and yellow brown [due to weathering] in colour.

The unit consists primarily of a massive sandstone, with minor siltstones, which in the fresh state is grey in colour. In the lower levels of the unit, softer beds and chert is encountered. The unit thickness observed was approx. 15 - 20 m.

A channel bar depositional environment has been postulated for this unit.

6.1.2. Unit Rnt - E (including F)

This unit consists of interbedded siltstones and sandstones, in the lower level with alternating bands of siltstone and claystone [up to 300 mm thick] occurring in the upper portion of the unit [see Plates No. P3 & P4]. There are also thinly bedded siltstones towards the base of the unit. The unit thickness is approx 14 to 16 metres [Ref: 2] which accords with site observations.

This unit was observed to be highly jointed and weathered to a shallower slope line than the capping and underlying sandy units. The claystone in this unit also weathers rapidly on exposure and the majority of the unit exposed at the site was in a state of active recession. Some recent [within the period June - August 1996] localised surficial soil slumps were noted within this unit [see Drawing No. X017G3]. The depositional environment is thought to have included floodplain and minor channel bar zones.

6.1.3. Unit Rnt - D

At this site, Unit D consists of a grey, fine to medium grained sandstone, with some minor siltstone inclusions [see Plates No. P5 & P6]. The unit forms the primary cliff line in the study area and was measured as having a thickness of between 32 & 38 m. The pattern of cliff line is also dominated by the joint orientations of this unit.

As suggested by McDonnell, Unit D is predominantly a sandstone unit; however, in the lowermost portion, the unit has both interbedded sandstone / siltstone units and minor siltstone beds. The depositional environment has been postulated as a combination of floodplain and point bar.

6.1.4. Unit Rnt - C

Unit C consists of a dark grey siltstone, with minor claystone and some thin sandstone beds; the unit is the basal unit observed of the cliff over the length of the study area. Whilst the unit thickness varies over the study area from 16 to 20 metres, it rises [due to the south west regional dip] in the cliff face towards the north.

The unit is weak to very weak and weathers rapidly. In many locations weathering of the claystone / siltstone profiles of up to four [4] metres were noted. In addition, two large caverns have formed near Section 3 [Drawing No. X017G6] which have weathered out depths of between 14 & 16 metres. The weakness of this unit is responsible for the main failure mechanism of the cliff face and the cliff line recession [or regression] process.

Sandstone and siltstone components of this unit make up the wave cut platform itself.

The depositional environment has been postulated as a combination of Marsh (for the Claystone unit) and Floodplain and point bar (for the siltstone and sandstone units).

6.2. Geological Structures & Discontinuities

The geological units at the site are mainly sub-horizontal units with shallow regional dips toward the south west. Zones of local dip variation were observed and where cross bed sets occur, some areas exhibit dips of up to 40° in variable directions.

The combination of the near vertical joint sets [generally 80°] described above, local areas of high dip, the cliff line trend as dictated by the regional joint pattern, and horizontal undercutting through differential rates of weathering, has led to localised zones of instability. Observations of the cliff during the field inspections noted some areas with the potential for immediate failure; such a failure actually occurred during the course of the study [see Plate No. P18].

Whilst a number of measurements of the joint structure within the site sandstone bedrock were made during our site inspections, a statistical joint determination was not carried out; the main joint orientations were determined by reference to exposed cliff line features and plotting of joint patterns on the orthophoto maps. Suffice to note that unfavourable joint orientations were noted at the site, with the north east trending cliff line being joint controlled. There were also north west / south east joint sets and north eats / east trending joints which linked to create potential failure wedges.

7. ENGINEERING GEOLOGY

The way in which the site geology influences the recession of the cliff line and affects residential development within the "Zone of Influence" of the cliff line is discussed in the following paragraphs.

7.1. Cliff Line Recession Mechanism

The mode of recession [or regression] of the cliff line is primarily controlled by a differential weathering process, aggravated near sea level by wave action. In the postulated mechanism, the softer claystone and siltstone units are more easily weathered than the harder sandstone units and give rise to undercutting of the overlying sandstone units. This mechanism was observed throughout the exposed cliff face exposure, particularly near the wave cut platform area where a number of shallow caves and two deep caverns have formed within the basal siltstone / claystone unit.

The faster weathering units undercut the sandstone units and eventually, when a vertical or near vertical joint plane is encountered, the sandstone blocks break off in a line which is roughly parallel to the existing cliff face. Also, the unfavourable orientation of joint lines can combine with the bedding plane orientations to form an unstable rock wedge which then results in blocks of material falling from the cliff face. Both of these mechanisms were observed at the site and Plates No. P10, P11, P17, P19, P20 & P21 are included in the Appendix to illustrate this process.

In addition, there are two other mechanisms evident in the study area, viz:

- seepages through rock joints and fissures [See Plates No P7, P10 & P11], and
- surficial soil slumping in the upper cliff area;

these are discussed as follows:

7.1.1. Seepages Through Rock Mass

In addition to the rock block instability occasioned by weathering and unstable joint blocks, the joints systems and bedding planes within a rock mass in close proximity to a coastal cliff line are normally more open and the rock more weathered than the parent 'unaffected' rock mass itself. Thus, as a cliff line "recesses" the joints open due to a reduction in the restraint of the internal rock stresses and cause a higher joint permeability in the cliff top area [this matter is further discussed in Section 7.3].

In view of the foregoing, it is common in cliff areas for significant seepages to be observed both within and through the rock mass. At this site, significant seepages were noted immediately north of Section 3 [near the northern cavern] and near the interface of Unit H / Unit E [which crops out near Lot No. 890].

Therefore, as the effect of seepage is to increase the rate of weathering in the rock & joints, it has the consequence of accelerating the rate of cliff recession in a particular area. In addition, because of the open joints near a cliff top area, the rate of cliff recession can be increased by the inappropriate discharges of stormwater near the cliff top.

7.1.2. Surficial Soil Slumping

Generally, the surficial soils overlying the bedrock materials were observed to be shallow and sandy in nature; they thus generally do not give rise to soil instability problems. However, in the vicinity of Lots No. 880 - 887 [see Plates No. P3 to P6], the strata exposed within the cliff line is an interbedded siltstone and claystone which gives rise to a soft 'weathered rock' and thin soil cover; this thin soil cover is prone to significant 'soil slumping', particularly if wet. Thus, in this area the general rate of cliff recession is greater than the average due to soil slumps within the weathered siltstone and claystone strata.

7.2. Rate of Cliff Recession.

It is firstly to be noted that the recession of a particular cliff line is a function of many things, but mainly:

- The softness and variability of the rock materials and likelihood of extensive localised weathering.
- The development of "caverns" in the cliff as a result of wave action, local variations in rock strength and weathering pattern.
- Joints, joint orientations and geological discontinuities within the cliff area.
- Seepage through the joints, bedding planes & rock mass.
- Susceptibility of area to earth tremors and intensity of storm activity.

Also, because of the way in which these factors can combine is random, a cliff top area can maintain considerable "apparent stability" for many years and then suddenly recess in a series of major rock falls within a very short period. For example:

i] Between Sections 3 & 4, there is a major cliff undercut area [see Hazard Zoning Plan & Plates No. P8, P9, P14 & P15] where a very large section of the cliff above the undercut is theoretically unstable. It could fall in the immediate future, or its fall could be delayed for hundreds of years.

ii] The undercut cliff areas north of Sections 1 & 4 have a well developed joint system, stress fractures and many open joints; as such, parts of the cliff area above the undercut can fall at any time [see Plate P11].

iii] In the vicinity of Lot No. 875, it is likely that a recent major cliff line rock fall has taken place [say within the last 20 - 50 years] which has left the upper "soft rock" area of the cliff near vertical and now theoretically unstable. [See Section 1 - Drawing No. X017G4 & Plate No. P16].

Specifically, in regard to the rate at which the cliff recedes, from other work in the Central Coast Area [Ref: 6], extensive photogrammetric studies found that the coastal retreat in the Norah Head area averaged 2 to 3 metres for the period 1954 to 1993, although in places a retreat of 4 metres was recorded. This information also suggested that the average retreat in that area would be 4 to 6 metres for a 50 year period.

In addition, a comparison of the surveyed location of the cliff top area in 1959 to the present location indicated localised areas which had receded up to 9 metres in the period. This amount of "measured" recession is however the subject of debate due to inconsistencies in the definition of the "cliff top" originally surveyed.

When all the above is considered "in concert", it is considered that the available evidence suggests that a cliff top recession rate of 4 metres per 50 year period would not be unreasonable.

Also, to assist in future monitoring of the rate of cliff recession, Table 2 [in Appendix] has been prepared which sets out the distance from the lot boundaries of the cliff line observed in this study.

7.3. *Effect of Cliff Recession on Area behind Cliff*

In addition to the area of cliff directly affected [i.e. by blocks of rock falling off and / or a soil slump occurring] by the recession process, the area behind the cliff face is affected by:

- the "zone of influence" of the rock joint system and defects,
- the predicted amount of recession of the rock face for the management period,
- progressive release of the "locked in rock stresses" in the parent rock mass.

Thus, it is usual to assess the effects of cliff recession [and thus the effect on the land near the top of the cliff] on the basis of comparison of equivalent geological sections and a prediction of the amount of recession for the management period. This analysis is presented in terms of "cliff recession influence lines" [***CAT 1 / CAT 2*** lines, etc.] on the Geological Sections [Drawings No. X017G4 to G7] included in the Appendix.

The reasoning behind the "influence lines" shown on the sections, and consequent "Hazard Category" is discussed in a later section of this report.

8. CLIFF AREA STABILITY & HAZARD ZONING

To implement a hazard management strategy, it is necessary to identify and classify the hazard, and to then determine what controls / constraints need to be applied to areas of equal hazard. Such a process inherently involves some 'qualitative' assessment of the site conditions, in parallel with the usual 'quantitative engineering' approach.

In this study, because of the difficulties in predicting the rate at which the cliff line might recess [because there is little, or no reliable statistical data of sufficient time period available], the hazard zoning strategy has been determined from an assessment of the geotechnical processes, in combination with theoretical considerations. The classification system and its methodology is described in the following paragraphs:

8.1. Hazard Classification System

The classification, or identification, of a land area considered to have some "Hazard" or "Risk of Instability" has been the subject of much technical discussion, with several authors developing various classification systems [eg. Chesnut 1974 (Ref: 1) and Shirley 1975 & 1982 (Ref: 3 & 4)]. However, for the purposes of this study, it has been decided to adopt the following hazard classification system.

Table 1 - HAZARD CLASSIFICATION SYSTEM

HAZARD CATEGORY	DEFINITION	DESCRIPTION
Category 1	**50 Year Low Hazard area**	Areas where, it is predicted that within 50 years, development would not be impacted on adversely by coastal or cliff instability processes.
Category 2	**50 Year Medium Hazard Area**	Land areas comprising lots, either whole or in part, which, it is predicted that within 50 years, could be impacted upon by ongoing coastal retreat processes, cliff joint opening due to cliff recession, and where development could be secured against adverse impacts with appropriate foundation design.
Category 3	**50 Year High Hazard Area**	Land areas comprising lots, either whole or in part, which, it is predicted that ***within 50 years***, could be subject to high hazard in respect of erosion, landslip, rockfall, rock destress and / or tidal inundation.
Category 4	**Immediate High Hazard Area**	Land areas comprising lots, either whole or in part, which, ***at present***, are subject to high hazard in respect of erosion, landslip, rockfall, rock destress and / or tidal inundation.

The reasons for adopting such a classification system was that the system:

- is compatible with the usual zoning system adopted for coastal areas where storm activities threaten the typical beachfront environment, and
- can be used as a base for subsequent more detailed studies to delineate development constraints and possible remedial action.

It is also to be noted that the hazard classification of the area in 1974 [by the GSA Ref. 1] is compatible with the latest work performed by SCE, with the GSA identifying the cliff top area as having a "Bad Risk".

8.2. Methodology for Determining Hazard Category

In this study, the various ***hazard categories*** were determined from a consideration of the geology and other matters discussed in the "Engineering Geology" section of this report, using the following methodology:

a) The various cliff and slope formation processes were assessed on the basis of the underlying geology and field observations, in conjunction with observations carried out in the area over a number of years [see list of previous work by SCE in the area listed in the Appendix].

b) Viewing of aerial photographs over an extended time period [1954 - 1995].

c) Site measurement of the present cliff location from individual lot boundaries and comparison of these measurements to the cliff line surveyed at the time of subdivision in 1959.

d) On the basis of the comparative sections, an estimate was made of the likely further recession of the coast line over the next fifty to one hundred years using a "cliff line mimic" process to predict the alignment of the cliff line in 100 years.

e) The cliff "influence lines" were determined and projected in plan view at the cliff top level.

f) Finally, the "hazard category" boundaries were reviewed in the light of all the available data and rationalised into the Hazard Zoning Plan.

8.3. Hazard Map & Cliff influence Lines

After assembly of the geological sections and a consideration of the likely development of instability in the study area, the Hazard Zoning Plan [Drawing No. X017G3 - in the Appendix] was developed. The map does not however strictly follow the "categories" determined by the cross sections, as local adjustments were considered necessary due to other factors such as the extent of caverns, local soil slumps observed and "oversteep nature" of the cliff area near Section 1 [Lot No 875]. It is also noted that as further cliff line recession data on the area becomes available with time, the Zoning Plan could be refined.

The boundaries between the various hazard categories was determined as follows:

8.3.1. Category 3 / Category 4

This boundary was assessed from a consideration of the observed rock defects, major joint orientations, and the 'over steep' nature of some of the site slopes observed.

The underlying principal in the prediction of the areas subject to 'immediate hazard' was determined on the basis of existing undercutting of the rock cliff face and the prediction that the zone of cliff face so affected by the undercutting [either by small caves or the large caverns] had already destabilised the cliff face area above the undercut itself.

Whilst it could be said that the two large caverns only represent a "localised feature" and that the next step in the cliff recession process near Section No. 3 will be by major rock fall between Sections No. 3 & 4, it is considered that:

- the amount of debris observed in the southern cavern, and
- extent of wave action within the large open joints in the northern cavern,

indicates that the caverns are expanding at a "significant rate". As such, the caverns have sufficiently destabilised the surrounding joint system to classify the area above the caverns as being of "immediate high hazard".

8.3.2. Category 2 / Category 3

The boundary between category 2 & category 3 zones was determined on the basis that the cliff line as a whole could recede by the general rate of coastline recession [see Section 6.2 above], and then mimicking the receded cliff face on the basis of this four [4] metre recession.

Whilst it is appreciated that the face of the cliff face so mimicked may not represent the real situation in 50 years time, it is considered that the resultant category boundary near the cliff top area represents a reasonable estimate of the likely result of cliff line recession.

8.3.3. Category 1 / Category 2

In regard to the boundary between category 1 & category 2 Hazard Zones, as well as the area directly affected by the predicted period recession, rock "stress release' occurs well behind the cliff line in an ongoing process. In this "stress release" process, the natural high horizontal stresses within the rock mass are progressively released as the cliff recedes and causes a small opening of the near vertical joints in the rock mass behind the cliff face. For the purposes of this study, the definition of the boundary between the stable [Category 1] zone and the "joint affected" [Category 2] zone, was determined from a consideration of the stress patterns near excavations and resulted in a boundary line drawn upwards from the base of the cliff at an angle of 50°.

In view of the foregoing, and as this joint opening process has the potential to affect building developments, it is considered appropriate that any building works in the area where "opening of joints" may occur should be "flexible" and designed to accommodate the opening of joints. In this regard, as the "opening of joints" or very small horizontal strains, are analogous to the ground movements associated with underground mining activities [sometimes called "mine subsidence"]. Therefore, in the Category 2 area, building works should be planned as if they were in an area subject to "mine subsidence".

9. COMMENTS

The technical background to the recommendations of this report is presented in the following paragraphs:

9.1. Existing Cliff Top Development

During the course of our site inspection, it was observed that many property owners were using part of the Council Reserve for gardening, planting and other activities. Whilst the majority of these activities were in sympathy with appropriate preservation and usage of the cliff top area, in some areas this was not the case. The particular problems identified were:

- concentrations of drainage, and
- fill placement and fencing near the cliff top.

These are discussed as follows:

9.1.1. Concentration of Drainage

The concentration of stormwater and seepage near Lot No. 890 from the right of way further to the north is concentrating run-off as indicated on the Hazard Zoning Plan towards the most critical area of the cliff [viz: the northern cavern] where considerable natural seepage and major open joints were evident.

As this concentrated run-off will inevitably aggravate the rate of cliff recession in this area [see Section 7.1.1 above] it is important to carry out appropriate drainage works in this area as a matter of priority. In this regard, it would appear that the simplest solution would be to provide a stormwater drainage inlet within the concrete driveway at the rear of Lot No. 149 and to pipe this drainage to the Council system in Del Monte Place. Subsequently, it would be possible to direct the drainage from the right of way further north to this drainage inlet.

9.1.2. Fill Placement & Fencing near Cliff top

During our site inspections, several areas were observed where fill materials had been placed near the cliff top; the majority of the filling was observed to be garden refuse and small quantities of soil. As the effect of this filling is to create an 'over steep' slope on a weak, "soil like" material [actually both soil and extremely weathered bedrock] the filling ***and*** the natural underlying materials undergo soil slumping. Thus, the fill placement causes a more rapid erosion of the upper area of the cliff, because it loads the "meta-stable" slope area and thus induces a more rapid failure of the natural, weathered cliff top area.

In view of the foregoing, the dumping of fill near the top of the slope should be prohibited and existing fill materials removed.

In addition, during our fieldwork access to the cliff top area was hampered by the fence over the Council Reserve near the Lot No. 876 / 877 boundary; as the fence did not appear to have any public purpose, it should be removed at an early date.

9.2. Building & Development in the Cliff Top Area

In accordance with the hazard categorisation scheme, it is clear that building structures within the Category 4 [Immediate High Hazard] area is not appropriate; this is because substantial damage and loss could occur within the lifetime of the structure.

In the area designated as Category 3 [High Hazard], the most probable effect of the predicted recession for the management period of 50 to 100 years will be ***significant*** rock joint opening and consequent relatively large horizontal strains that can damage buildings. There is also some risk [albeit small] that the cliff recession process may in fact extend to within this zone during the management period. As such recession could cause serious structure damage, it would not be prudent to permit new buildings, structures or extensions in this area.

In areas designated as Category 2 [Medium Hazard] the predicted rock joint opening over the management period is considered relatively small and well able to be combated by suitable footing / structure design. Further, as it is extremely unlikely that the cliff recession process will extend to this area during the management period, new buildings / extensions could be permitted in this area, subject to appropriate footing and structure design [see Section 9.4.2 below].

Whilst the allotments within the Low Hazard [Category 1] area do not require any special precautions as a result of the "cliff recession" process, these allotments generally have moderate to steep slopes and present a number of building difficulties arising from the substrata in the area. These lots should thus be the subject of specific geotechnical studies for any building / development works.

It is also important that any drainage, mining or earthworks activities in the Category 2, 3 & 4 areas, be very carefully considered from a cliff stability / hazard point of view; therefore, any proposal to carry out significant earthworks, drainage or mining under, or near, the area should be the subject of most careful investigation.

9.3. Existing Structures

During the course of our site inspections, it was observed that the majority of site structures had a significant 'degree of flexibility' and that only a few structures were of the 'inflexible' kind. Also, only one structure [viz: the balcony to the house on Lot No. 883] was located within the "immediate high hazard" zone. Thus, all the existing houses in the study can be allowed to remain. It is suggested however that the balcony on Lot No. 883 be demolished in the near future.

In regard to the remaining site structures, and bearing in mind the remarks under Section 9.2 above, it is possible that any "inflexible structures" located within the Category 2 & 3 areas could be damaged as a result of the horizontal strains within the parent rock mass. Thus, it would be prudent for property owners with inflexible buildings / structures to make suitable adjustments to avoid significant future damage, possibly in collaboration with a suitably experienced engineer.

Whilst detailed comments on individual properties are beyond the scope of this report, the brick jointing in the house erected on Lot No. 881 has been rendered ineffective by the manner in which the brick window sill / roof structure has been constructed. As this house is constructed over the Category 2 / 3 zone boundary, the lack of an effective joint could be a serious problem in the future.

9.4. *New Buildings or Extensions to Existing Buildings*

9.4.1. Hazard Category 3

As noted in Section 9.2 ***new*** building works within Hazard Category 3 should be prohibited.

However, as a number of buildings, or parts of buildings are located within the Category 3 area, it is suggested that the buildings be allowed to remain if they are suitably modified to account for the possible 'horizontal strain' of the rock strata as set out in Section 9.4.2. Also, if extensions are planned [in the Category 2 area] to these buildings, then an appropriate 'structural slip joint' should be provided between the extension and the existing structure.

9.4.2. Hazard Category 2

In regard to new building works within the Hazard Category 2 areas, such buildings / structures will need to be designed to allow for the potential horizontal movement [and strain] in the foundation strata that will result from the predicted "opening of rock joints" during the management period. Whilst it is recognised that the degree of such rock joint opening is very difficult to quantify, it is considered that a total joint opening equivalent to 0.1% strain would be a conservative estimate.

In regard to a suitable design methodology, as a considerable body of knowledge has been built up in areas of "mine subsidence" [where special designs have been developed to combat the "lateral strains" that occur in a mine subsidence area] such a design approach is considered appropriate to the headland at Copacabana. An example of this knowledge is contained in Ref. 7; to assist in the appreciation of such mine subsidence design, a copy of Ref. 7 is included in the Appendix to this report.

A feature of mine subsidence design is also the concept of "structure flexibility" and ability to tolerate small ground movements; thus, where flexible structures already exist in the cliff top area, little or no modifications to such structures will be required. However, where rigid, inflexible structures exist, the same will need to be modified if long term damage is to be avoided.

It is therefore suggested that any new buildings constructed with the Category 2 area of the cliff at Copacabana be designed / constructed in a manner analogous to "mine subsidence" design principals.

9.5. *Hazard Management Strategy*

Whilst reduction of the natural cliff hazard is not considered technically feasible, efforts should be made to ensure that the ever present natural hazard is not exacerbated and the public made aware of the potential risk in the area. In this regard, it is clear that a management program is required to:

a) Warn the public as to the "Risk" of rocks falling.

b) Control / prevent the placement of filling near the cliff top.

c) Control / prevent concentrated stormwater discharges over the cliff area.

d) Long term monitor the rate of cliff recession to provide more accurate data for future investigations.

In addition, because of the generally steep slope of the allotments on the Del Monte Place [landward] side, all building works carried out on any allotment should be the subject of specific geotechnical advice. Whilst this report should address the usual geotechnical matters of building works, site excavations, erosion control, etc., the provision of detailed comments on cliff stability, etc. are not considered necessary if the development falls within the guidelines contained in this report.

In regard to item a), the posting of adequate warning signs in the areas in combination with some limited security fencing of particular areas would appear to be sufficient as it is obvious to the casual observer that the cliff area is potentially unstable. Details of this fencing & warning signs are however beyond the scope of this report.

In regard to items b) & c), these can be managed by combined actions by Council & residents; an appropriate "plan of action" does however need to be prepared.

In regard to item d). [viz: the suggested "monitoring" of the rate of cliff recession], because of the relatively short observational 'historical base' for the cliff line recession, it is considered that the future recession of the cliff should be monitored so that more accurate predictions can be made in the future as to the impact on site development. Also, because the rate is likely to be low, measurements every 5 years would appear to be adequate, unless a major rock fall takes place. Further, should a major rock fall be recorded, detailed measurements at that time would be of considerable benefit to future investigators.

To facilitate future cliff line monitoring, during this study a series of measurements of the distance between the cliff area and the property boundaries were made [see Table 2 in Appendix] by measuring the distance between the seaward lot boundary and the upper 'cliff boundary'.

9.6. Additional Studies

At this time the various hazard category areas appear to be technically well founded; therefore, additional work is unlikely to produce significant adjustment to the hazard categories determined by this work.

However, should individual lot owners consider that some adjustment to the Hazard Categories is warranted, then it is suggested that the additional geotechnical studies for a particular lot be at as least extensive as the studies carried out for this work. This is because each individual lot must be seen as part of the overall headland area.

In regard to the implementation of a number of suggestions made in this report [e.g. drainage, erosion control near the cliff top area, warning signs & security fencing], detailed advices on these items will be required by appropriate specialists [e.g soil conservationists, engineers, etc.].

10. RECOMMENDATIONS

On the basis of the available information and work undertaken as part of this study, the following recommendations are made to mitigate the risk to persons / property in the study area [viz: Lots No. 873 to 896 Del Monte Place, Copacabana]:

10.1. General Hazard Management Strategy

a) A management strategy, which includes warning of the public of the stability risk in the area together with guidelines for development and building controls, be developed for the land areas identified in this study [see Drawing No. X017G3] as having a Hazard Category 2, 3 & 4.

b) The management strategy be developed in consultation with the residents of the area, and provide for:

- building / structure restrictions on the various allotments, and
- restrictions on earthworks, filling and drainage, both on the allotments themselves and on the Council Reserve, and
- limitations on cliff area access.

c) The strategy include the requirement for ***"specific lot"*** geotechnical advices for any new building / development works on a particular allotment within the study area. Such "specific lot" report need not however address cliff stability issues, if the development implements the guidelines of this report.

10.2. Specific Strategy for Warning of Stability Risk & Cliff Access

a) Warning signs be placed in strategic locations to warn the public of the risk of falling rocks from the cliff area and a major rockfall; the actual locations of the signs to be determined in consultation with residents, Council and geotechnical personnel.

b) Suitable security fencing be erected to prevent public access particular areas where the "risk" is considered to be higher than the Immediate High Hazard [e.g. the southern cavern].

c) Access to the cliff area [e.g. for abseiling, viewing, etc.] be limited to suitable authorised persons.

10.3. Specific Strategies for Development within Various Hazard Categories

10.3.1. Hazard Category 4

a) The owners of any existing structure located within Hazard Category 4 be advised to demolish the structure as soon as practical. Presently, this only affects the timber deck of Lot No. 883.

b) Suitable soil conservation practices, principally planting & erosion control works, be implemented on the cliff top areas to minimise the rate of erosion / recession.

c) The existing filling near the cliff top boundary be removed and erosion control measures implemented on the regraded areas.

d) No further placement of filling near the cliff top boundary be permitted.

e) The fencing across the Council reserve near Lots No. 876 & 877 be removed and public access across this area restored.

10.3.2. Hazard Category 3

a) No new building structures be approved in this Hazard Category.

b) The owners of any existing building structures within this zone be advised to renovate / alter their buildings so that the structures embody the building recommendations for the Hazard Category 2 area.

10.3.3. Hazard Category 2

a) New buildings / structures in this zone be in accordance with recognised "mine subsidence" practice to accommodate lateral strains of 1 in 1,000 [0.1%].

b) Any extension to an existing structure be "structurally separated" and a 'slip joint' provided if required between the two structures.

10.4. Specific Strategy for Earthworks, Erosion Control & Drainage

a) The concentrated run-off from the right of way near the northern part of the site be controlled and piped to the Council system in Del Monte Place.

b) Appropriate siltation and erosion control works be implemented on the right of way area near Lots No. 892 to 896.

c) Advices be obtained from the Soil Conservation Service of NSW in respect of suitable methods of planting / erosion control for the upper cliff area near Lots No. 880 to 888.

d) The site surface drainage near Lots No. 887 & 888 be "re-contoured" so that surface water is not concentrated over the cliff top area.

10.5. Significant / Major Development in Area

Any proposal to carry out significant earthworks, drainage or mining under, or near, the headland area should be the subject of a most careful investigation.

10.6. Cliff Line Monitoring & Additional Studies

a) The rate of recession of the cliff line be monitored by measuring the distance between the southern boundary of a particular lot and the cliff boundary / major cliff line every 5 years. To ensure consistency of the "cliff top", such work to be performed in company with geotechnical personnel.

b) Should further "specific lot" geotechnical studies be required to reassess the Hazard Category of a particular lot, then the scope of work carried out extend to the entire headland area and include detailed examination of the complete Tudibaring Headland cliff area.

c) Geotechnical advices be obtained in respect of the locations of warning signs, security fences and drainage control measures, prior to construction.

Finally, it is to be noted that whilst the conclusions & recommendations in this report are based on an extensive amount of geotechnical investigation & analysis; the work has been limited by budget and area knowledge constraints. Therefore, as more information becomes available, it may be possible to amend and / or reclassify some of the hazard zone boundaries.

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Encl: Technical Appendix

TECHNICAL APPENDIX

Listing Of References & Documents

Colour Copies of Site / Cliff Photographs

Plates No. P1 to P21 [incl.]

Drawings & Tables:

Included in Text		*Inserted After Page.*
X017G1	Site Locality Plan & Study Area	5
X017G2	Local Geology Plan	8

Included in Appendix

X017G3	Hazard Zoning Plan
Table 2	Hazard Zone Boundary Dimensions
X017G4 - G7	Geological Sections Through Cliff

Technical Reference:

Design of Buildings for Mine Subsidence

LISTING OF REFERENCES & DOCUMENTS

REFERENCES

1. CHESNUT, W. S. [1974] "Engineering Geological Considerations which could Influence Urban Development in the Gosford - Wyong Region" GSA of NSW Report No. 1974/11, November, 1974 [unpublished].
2. MCDONNELL, K. L. [1974] "Depositional Environments of the Triassic Gosford Formation, Sydney Basin" - Geol. Soc. Aust. Vol. 21, Part 1 March 1974.
3. SHIRLEY, A. F. [1982] "The Scope & Extent of Geotechnical Reports for Residential Building & Subdivisions", Proceedings of LGEA Conference Sydney 1982.
4. SHIRLEY, A. F. [1975] "The Theoretical and Practical Aspects of Land Stability Classification". Proceedings of 2nd ANZ Conference on Geomechanics, Brisbane 1975 pp 10-15 I.E. Aust..
5. WBM OCEANICS AUSTRALIA & PLANNING WORKSHOP - "Coastal Management Study and Coastal Management Plan" for Gosford City Open Coast Beaches - August 1995
6. AUSTRALIAN WATER & COASTAL STUDIES [in conjunction with Shirley Consulting Engineers & Southern Aerial Surveys] - "Coastline Hazard Definition Study - The Entrance & Noraville" Report No. 95/47 to Wyong Shire Council, August, 1996.
7. BRAY I J - [1988] "Design of Buildings for Mines Subsidence". Proceedings of Conference on Buildings & Structures subject to Mine Subsidence - 28 - 30 August, 1988, Maitland NSW. I.E.Aust. & Mine Subsidence Board.

DOCUMENTS

1:4,000 Orthophoto Map "AVOCA 3690-5" [1973 & 1988] Central Mapping Authority

1:50,000 Provisional Geological Map MANGROVE - GOSFORD - NORAH HEAD Ref: No. 8672 by R E Uren [1977] - Geological Survey of NSW

1:25,000 Provisional Geological Map "GOSFORD 9131-2-S" [1995] - Geological Survey of NSW.

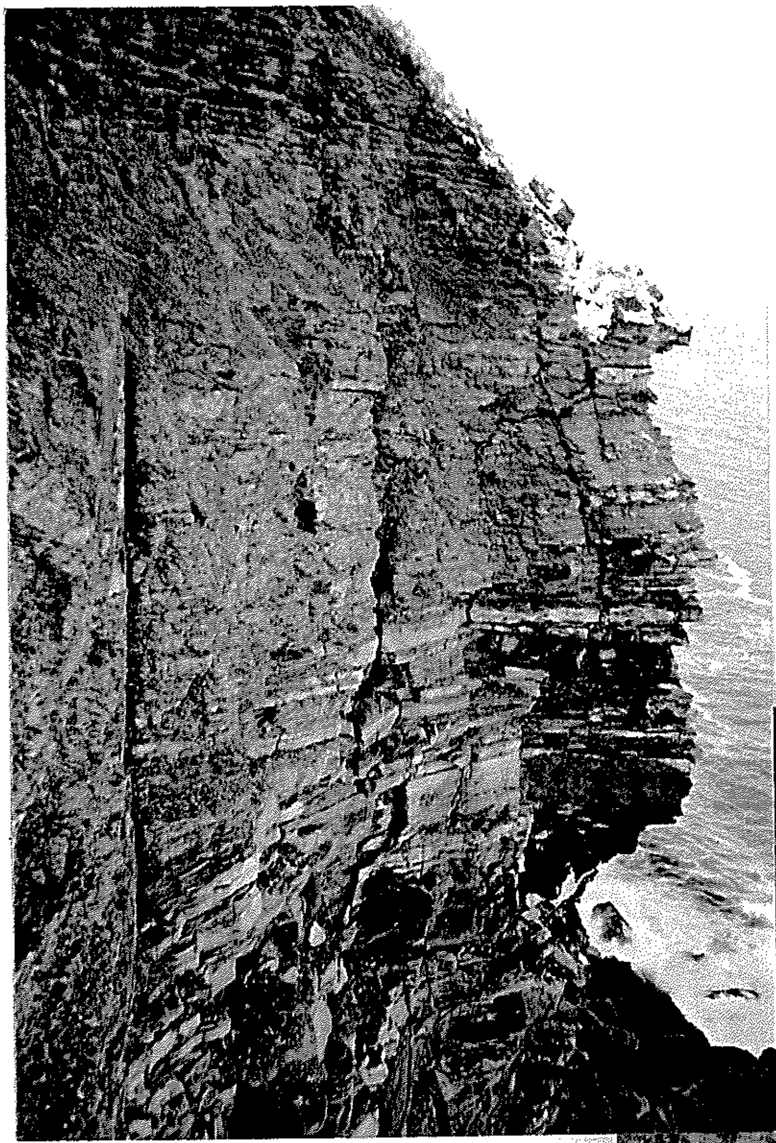
1:9504 Shire of Gosford Planning Map - Sheet 8 [1968] - State Planning Authority of NSW

GEOTECHNICAL REPORTS FROM SCE JOB FILES

Job Number	Location
W051	Lot 875 Del Monte Place, Copacabana
I103	Lot 792 Oceano Street, Copacabana
J009	Lot 804 Oceano Street, Copacabana
K037	Lot 798 Oceano Street, Copacabana
I125	Lot 996 Oceano Street, Copacabana
J019	Lot 948 Oceano Street, Copacabana
M110	Lot 970 Vista Avenue North, Copacabana
R028	Lot 970 Vista Avenue North, Copacabana

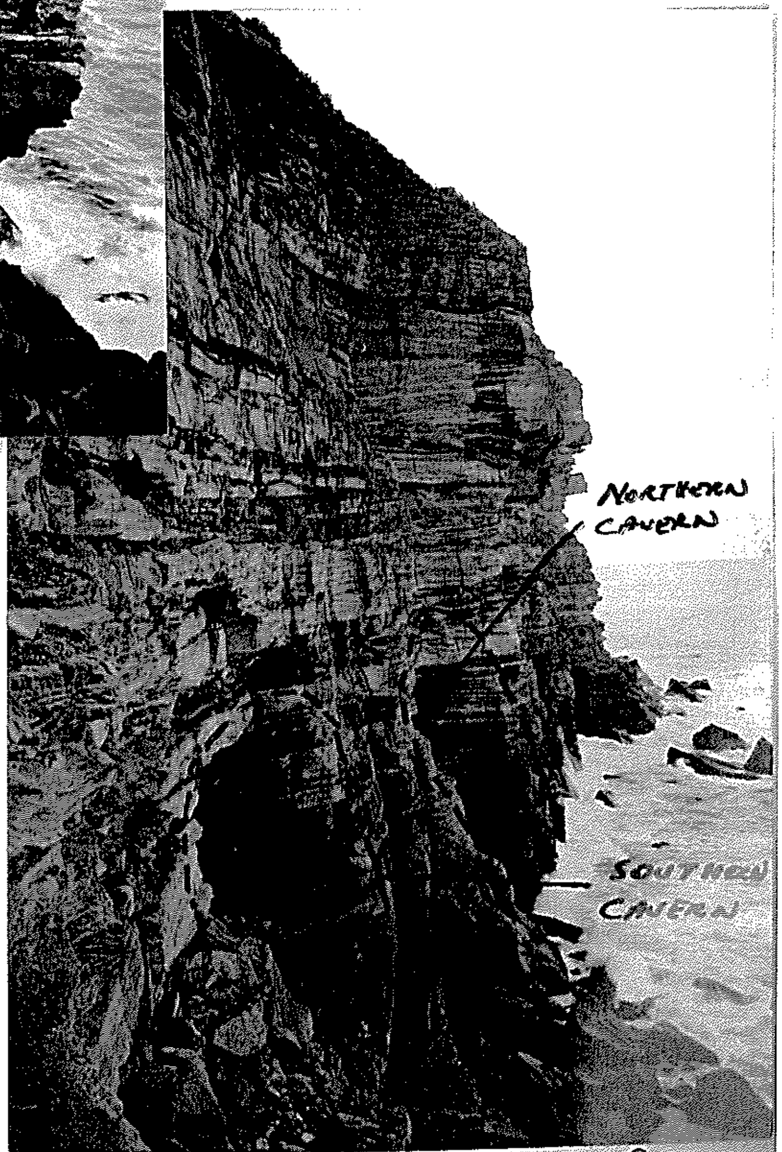
CLIFF LINE HAZARD DEFINITION STUDY - TUDIBARING HEADLAND

HEADLAND & CLIFF AREA PHOTOGRAPHS



P.1.

UNDERCUT HEADLAND AREA
NORTH OF SECTION 1.

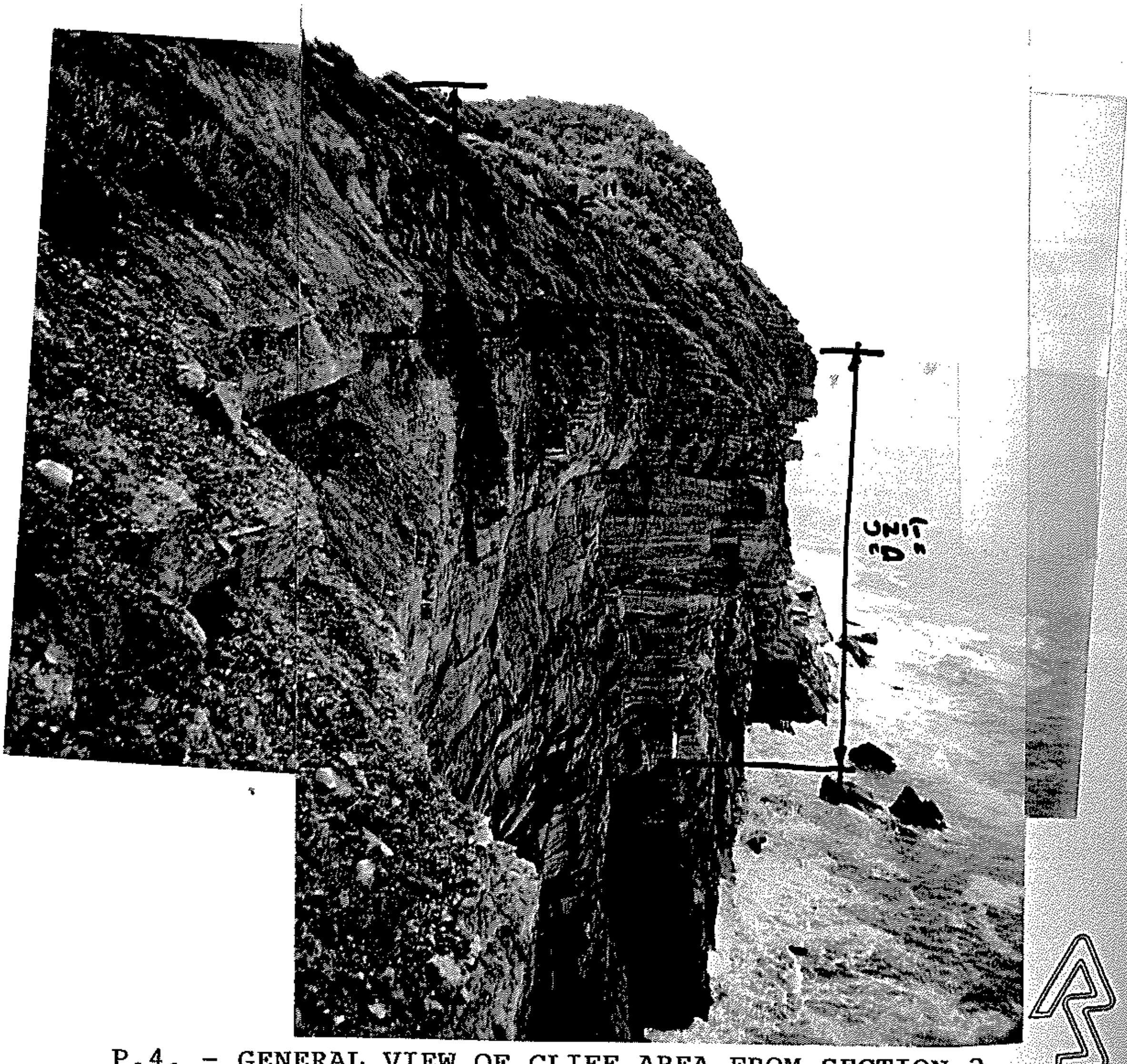


P.2.

GENERAL VIEW OF CAVERNS
IN CLIFF AREA FROM
SECTION 2.

HEADLAND & CLIFF AREA PHOTOGRAPHS

P.3. - SOFTER SILTSTONE / CLAYSTONE UNIT "E"
VIEW LOOKING NORTH FROM SECTION 2.

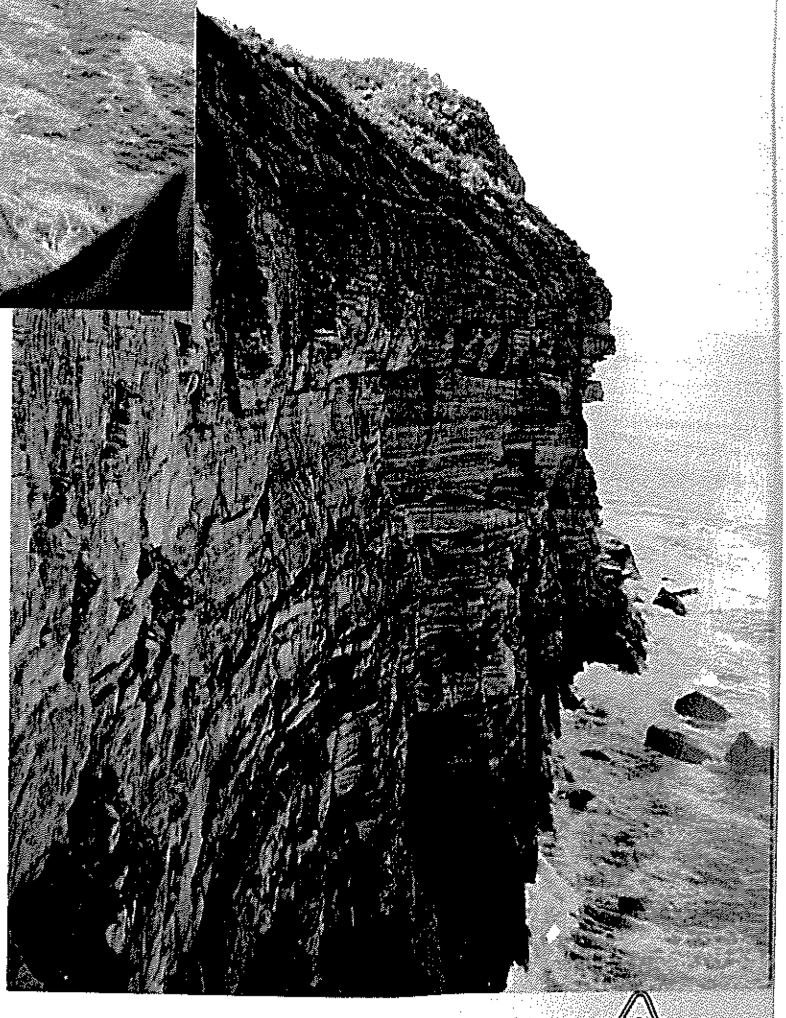


P.4. - GENERAL VIEW OF CLIFF AREA FROM SECTION 2
LOOKING NORTH

HEADLAND & CLIFF AREA PHOTOGRAPHS

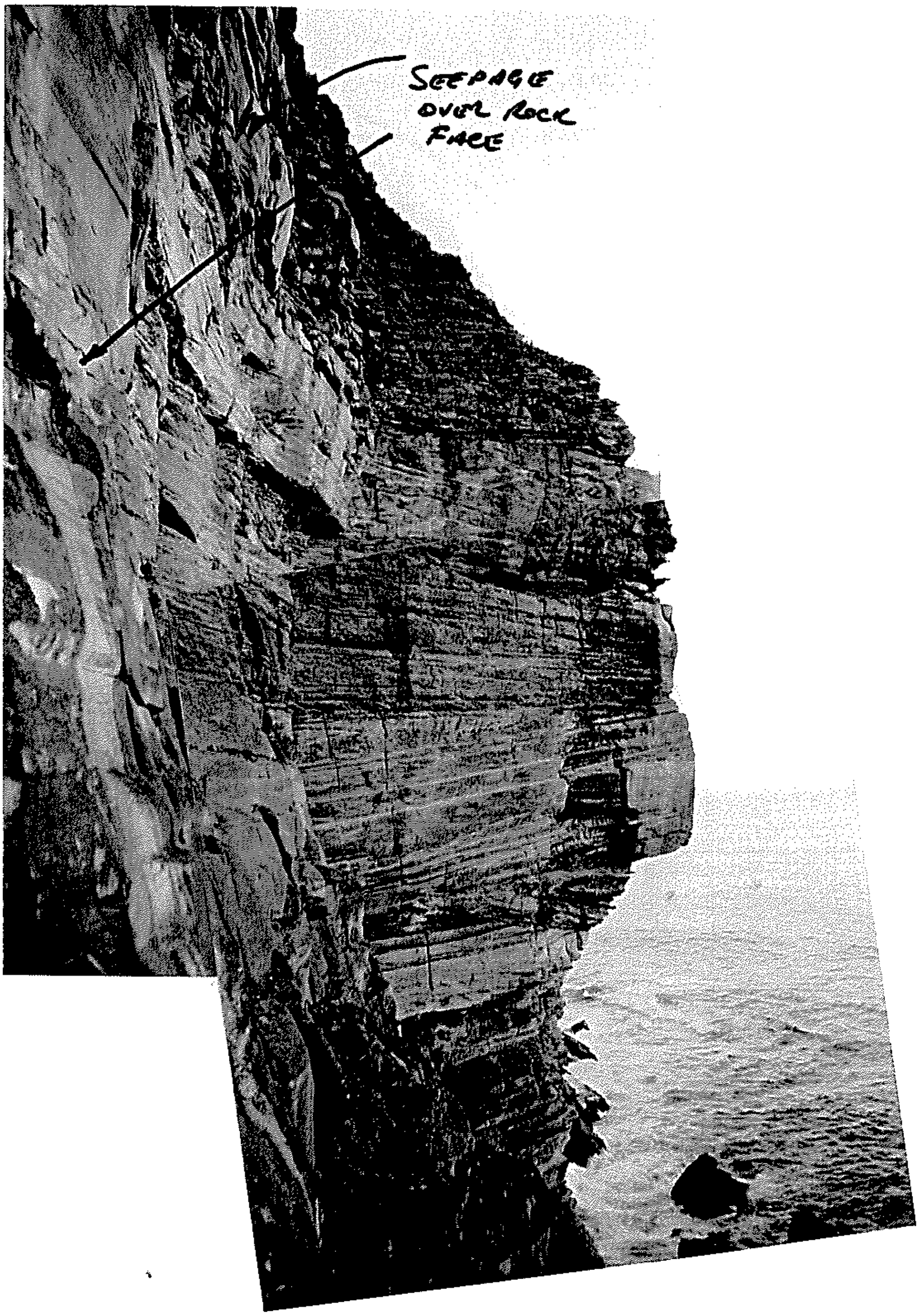
P.5.

VIEW OF MAIN SANDSTONE UNIT "D" & NORTHERN CAVERN FROM SECTION 2.



P.6.

VIEW OF MAIN SANDSTONE UNIT "D" FROM SECTION 2.



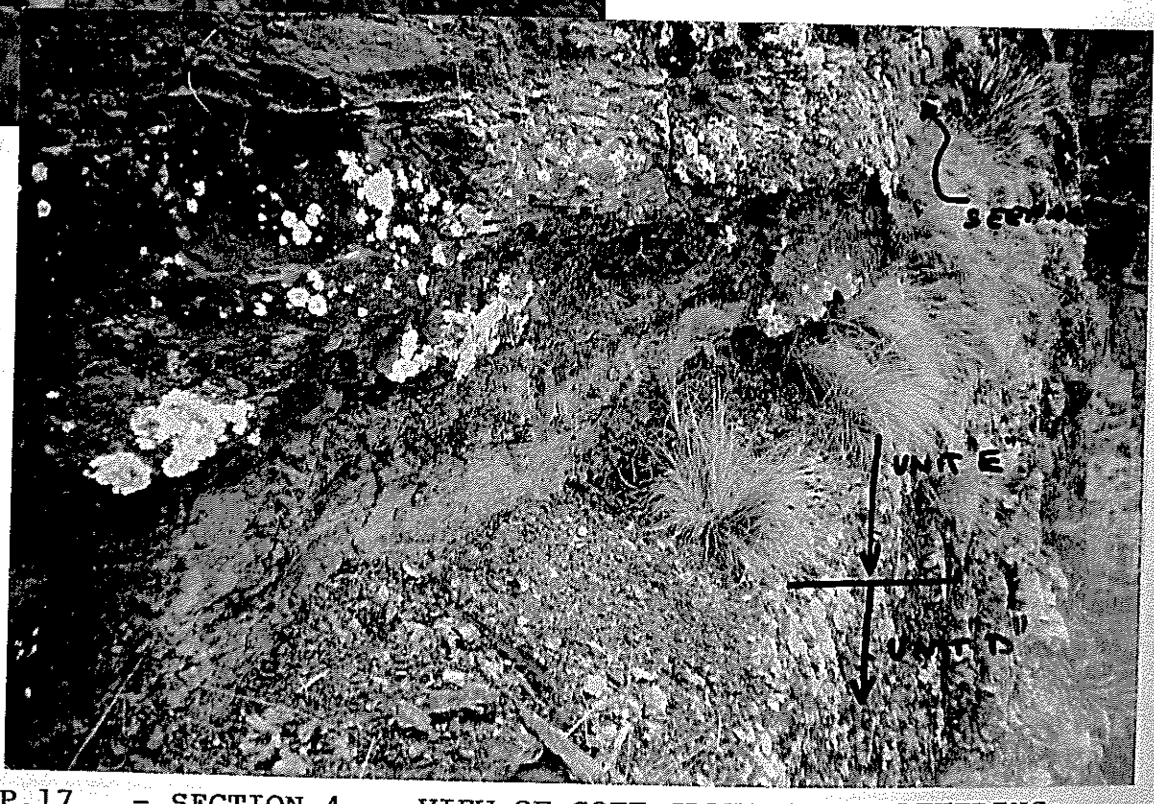
P.7. - VIEW OF MAIN SANDSTONE UNIT "D" & SEEPAGE OVER CLIFF FACE NEAR SECTION 3.

NOTE: CLIFF UNDERCUT AREA NORTH OF SECTION 3.

HEADLAND & CLIFF AREA PHOTOGRAPHS

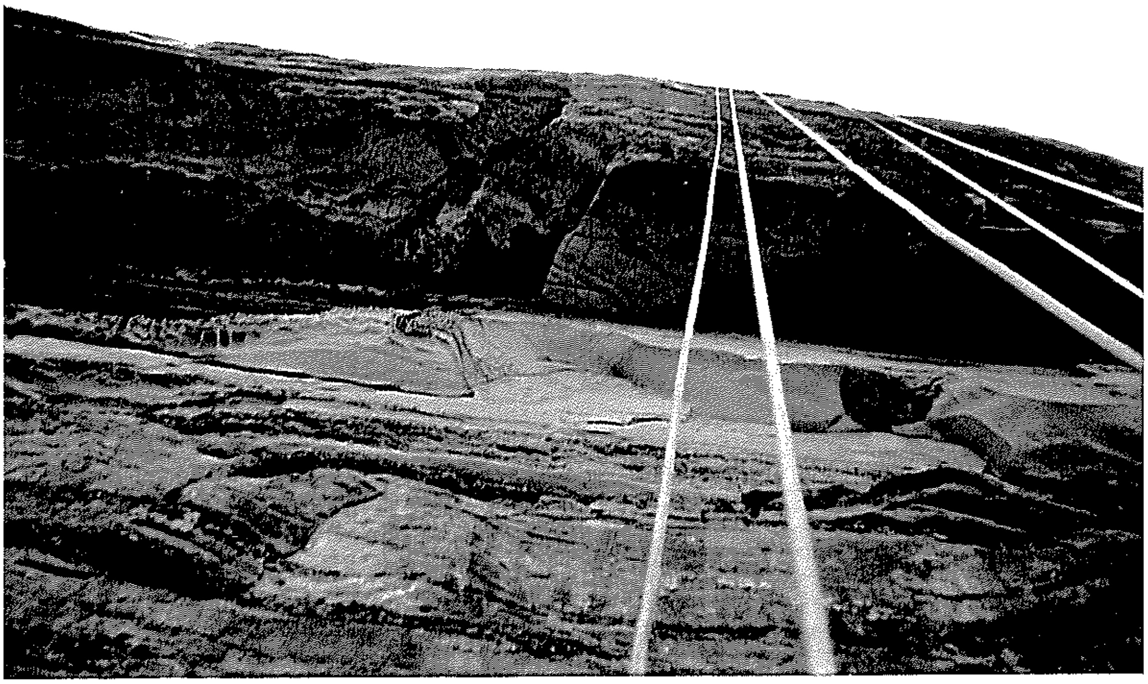
P.16.

SECTION 1. - VIEW OF SURFICIA
SOIL UNDERCUT BY SOFT ROCK
WEATHERING.



P.17. - SECTION 4. - VIEW OF SOFT CLAYSTONE WEATHERING
ABOVE UNIT "D".
NOTE: ALSO SEEPAGE.

HEADLAND & CLIFF AREA PHOTOGRAPHS

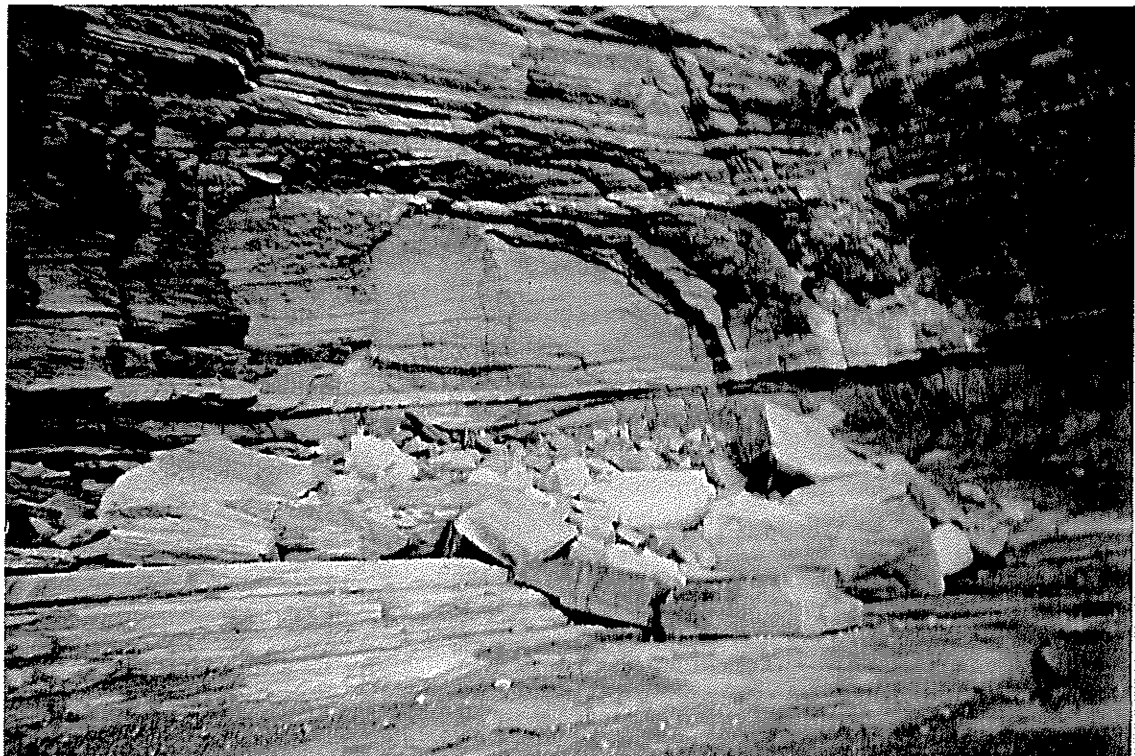


P.14. - SECTION 4. - UNDERCUTTING OF MAJOR SANDSTONE UNIT "D".

P.15. - SECTION 4. - UNDERCUTTING OF MAJOR SANDSTONE UNIT "D" LOOKING SOUTH TO SECTION 3.

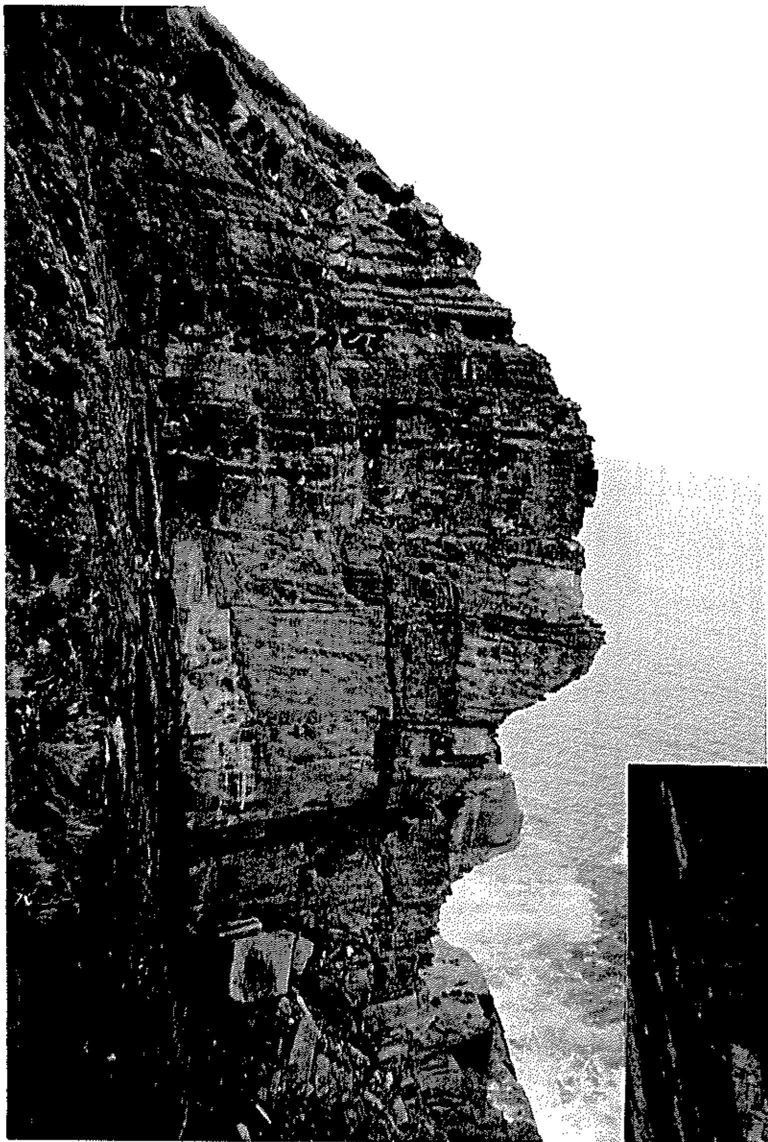
HEADLAND & CLIFF AREA PHOTOGRAPHS

P.12. - STRESS RELIEF ROCK SPALLING IN ROOF OF SOUTHERN CAVERN.



P.13. - ROCK DEBRIS ON FLOOR OF SOUTHERN CAVERN.

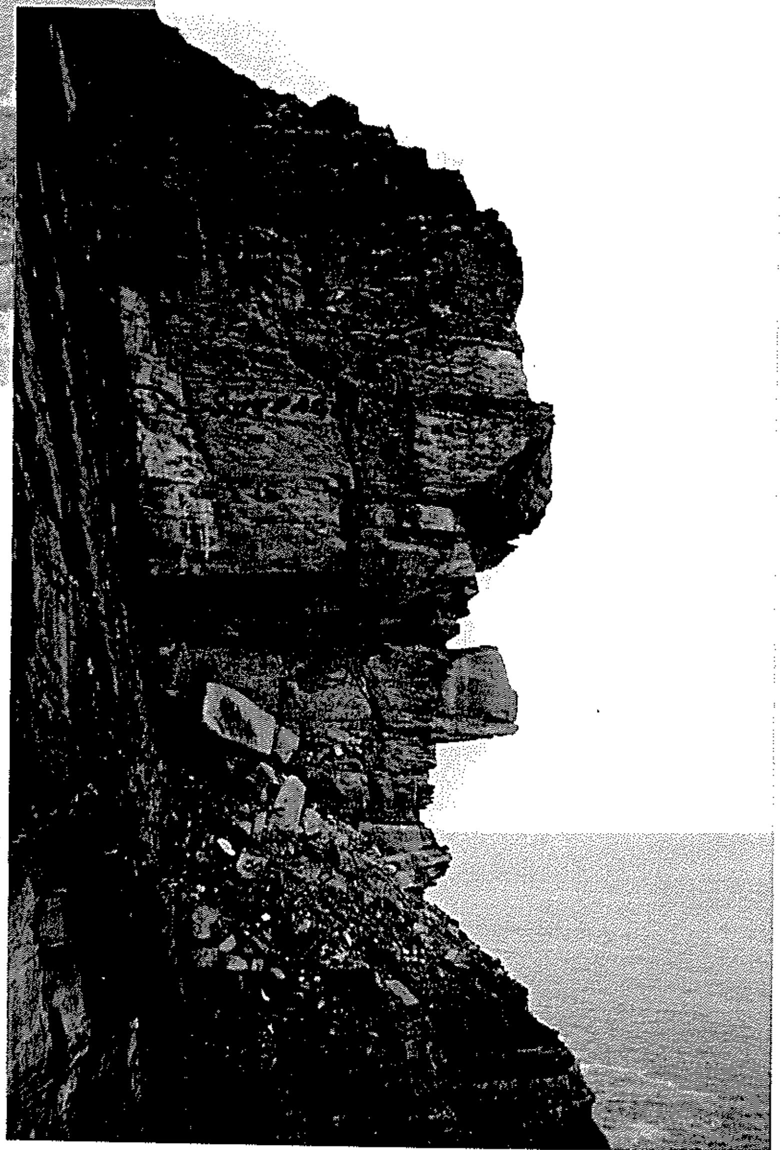
HEADLAND & CLIFF AREA PHOTOGRAPHS



P.10.

VIEW OF MAJOR SANDSTONE UNIT "D" & UNDERCUT CLIFF AREA NORTH OF SECTION 4.

NOTE: SEEPAGE.

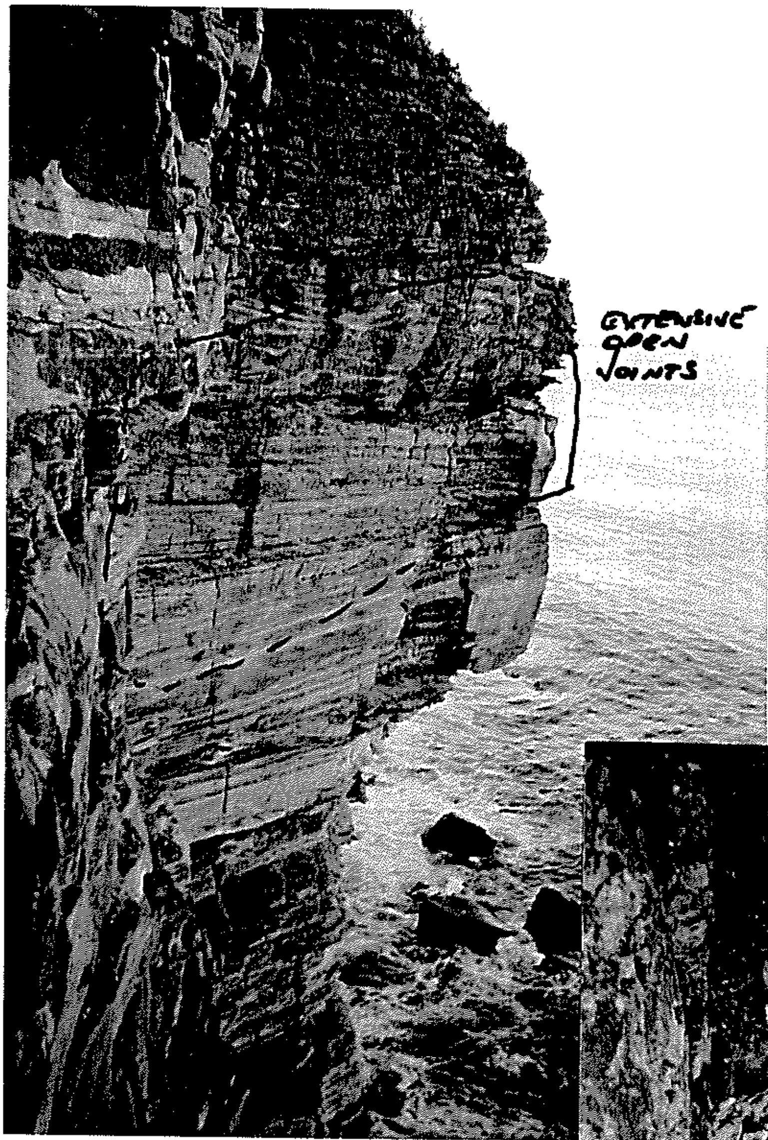


P.11.

VIEW OF UNDERCUT CLIFF AREA NORTH OF SECTION 4.

NOTE: SOFTER ROCK WEATHERING & JOINT BLOCK INSTABILITY.

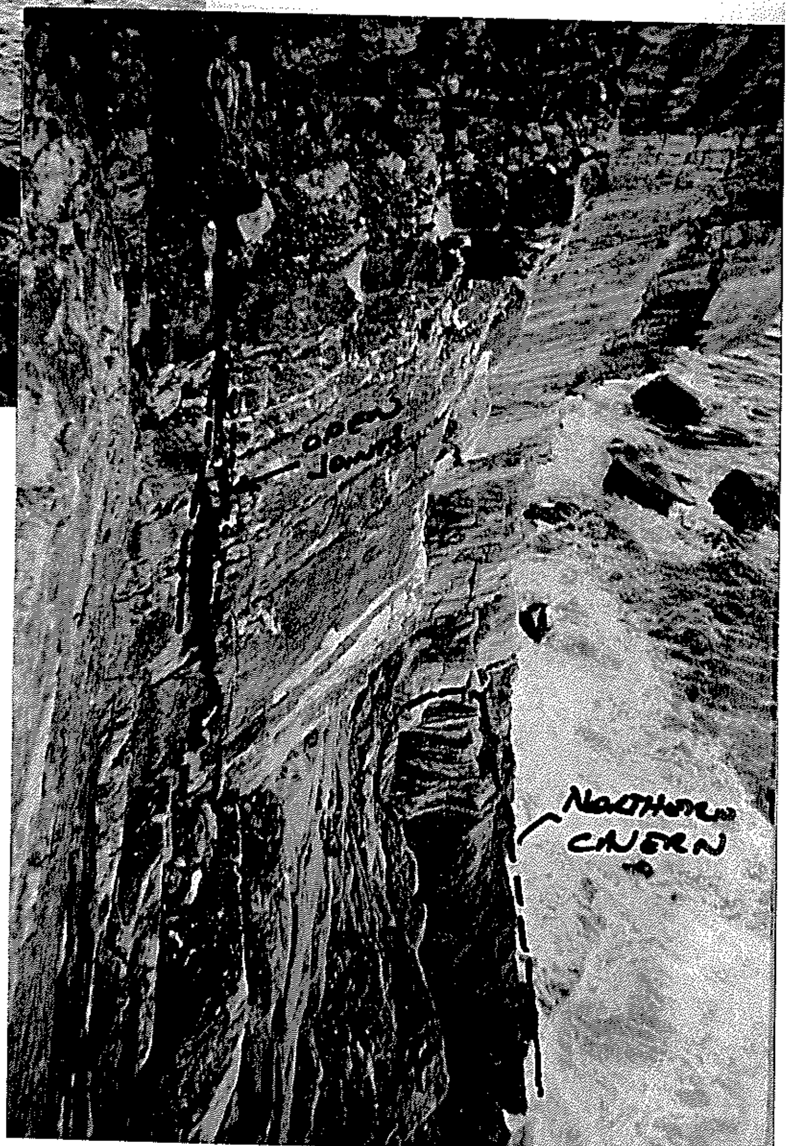
CLIFF LINE HAZARD DEFINITION STUDY - TUDIBARING HEADLAND
HEADLAND & CLIFF AREA PHOTOGRAPHS



P.8.

VIEW OF EXTENSIVELY UNDERCUT CLIFF AREA NORTH OF SECTION 3

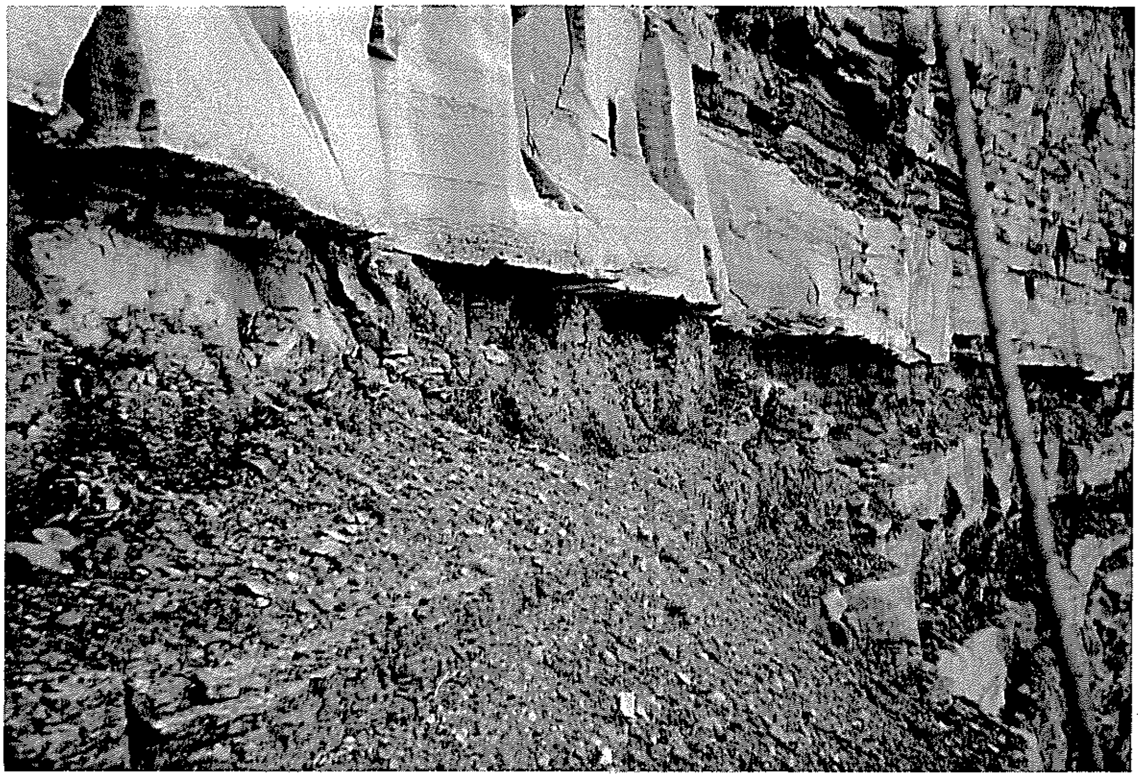
NOTE: OPEN JOINTS IN OVERHANG.



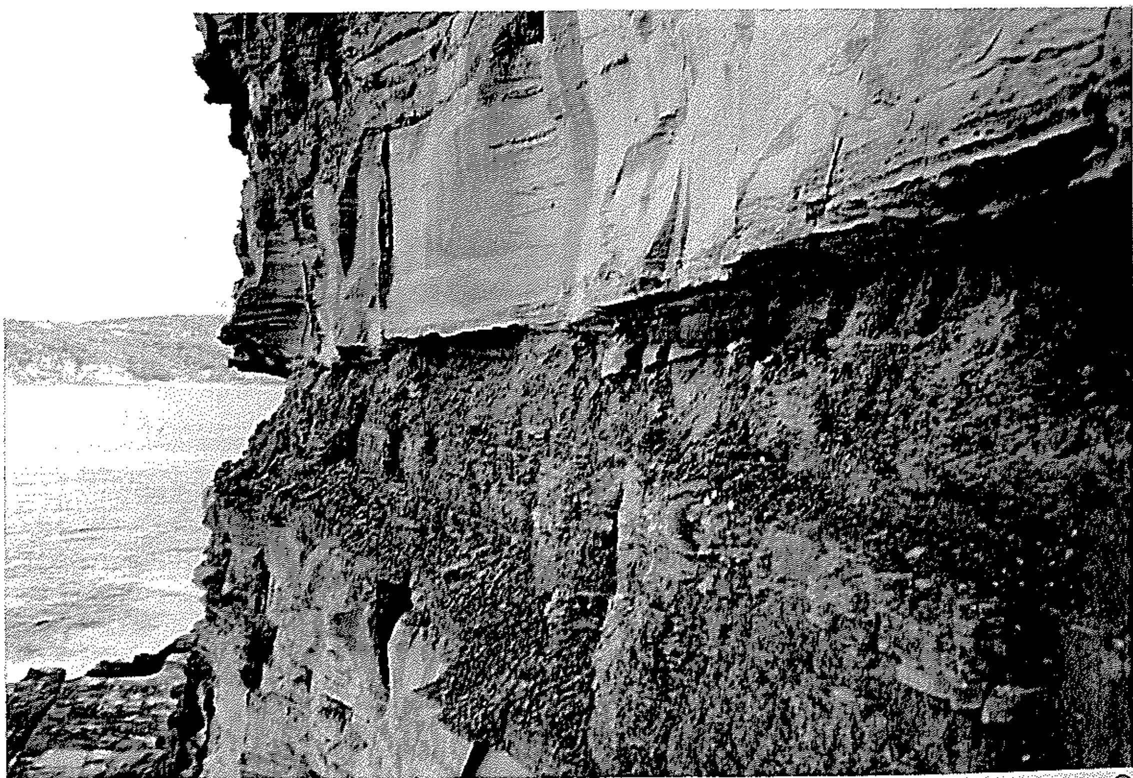
P.9.

VIEW OF CLIFF UNDERCUT & NORTHERN CAVERN FROM SECTION 3.

NOTE: OPEN JOINTING IN CLIFF.

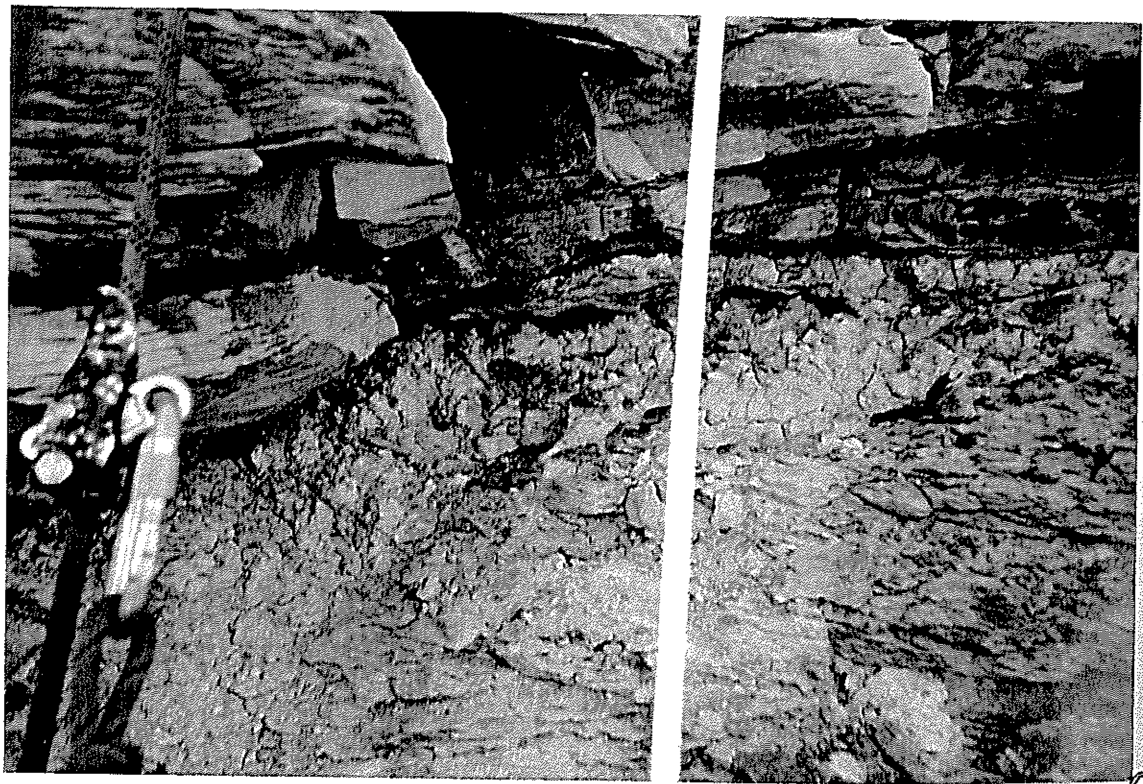


P.20. - SECTION 2. - WEATHERING OF UNIT D9 LOOKING NORTH.



P.21. - SECTION 2. - WEATHERING OF UNIT D9 LOOKING SOUTH.

P.18. - VIEW OF ROCK DEBRIS FROM ROCK FALL NEAR LOT No. 876.

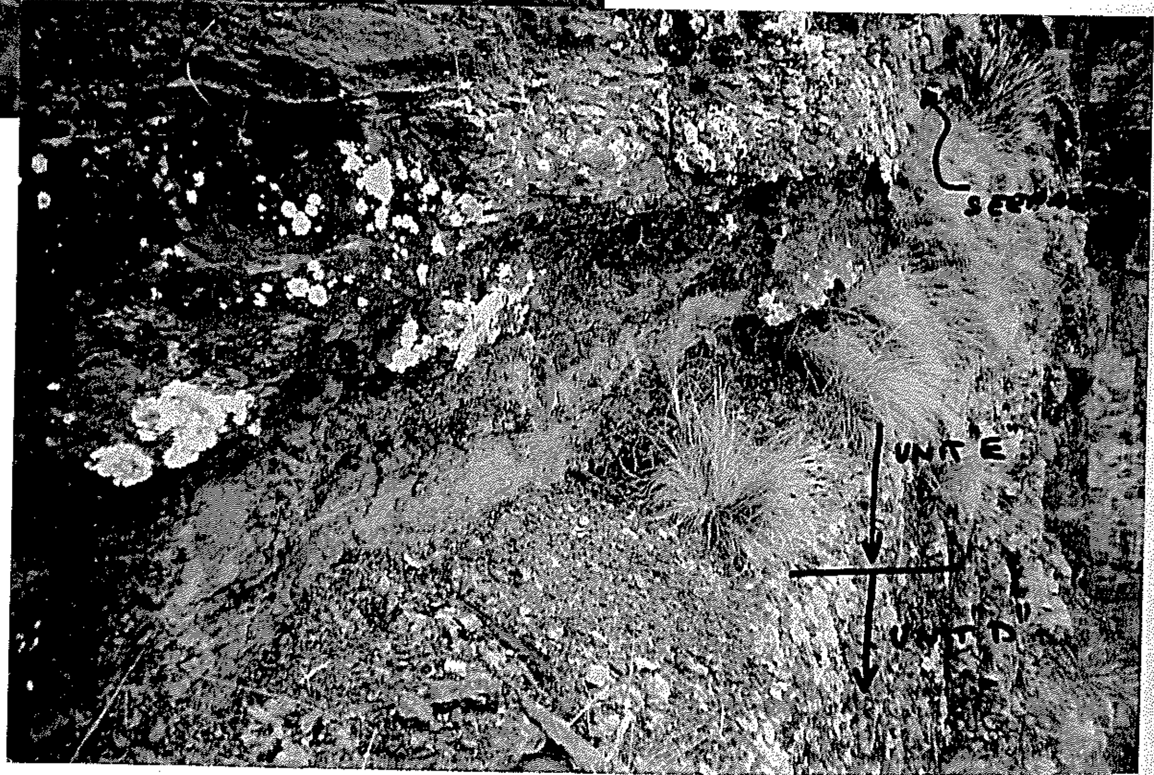


P.19. - SECTION 1. - WEATHERING OF SOFT CLAYSTONE NEAR BOTTOM OF UNIT "E".

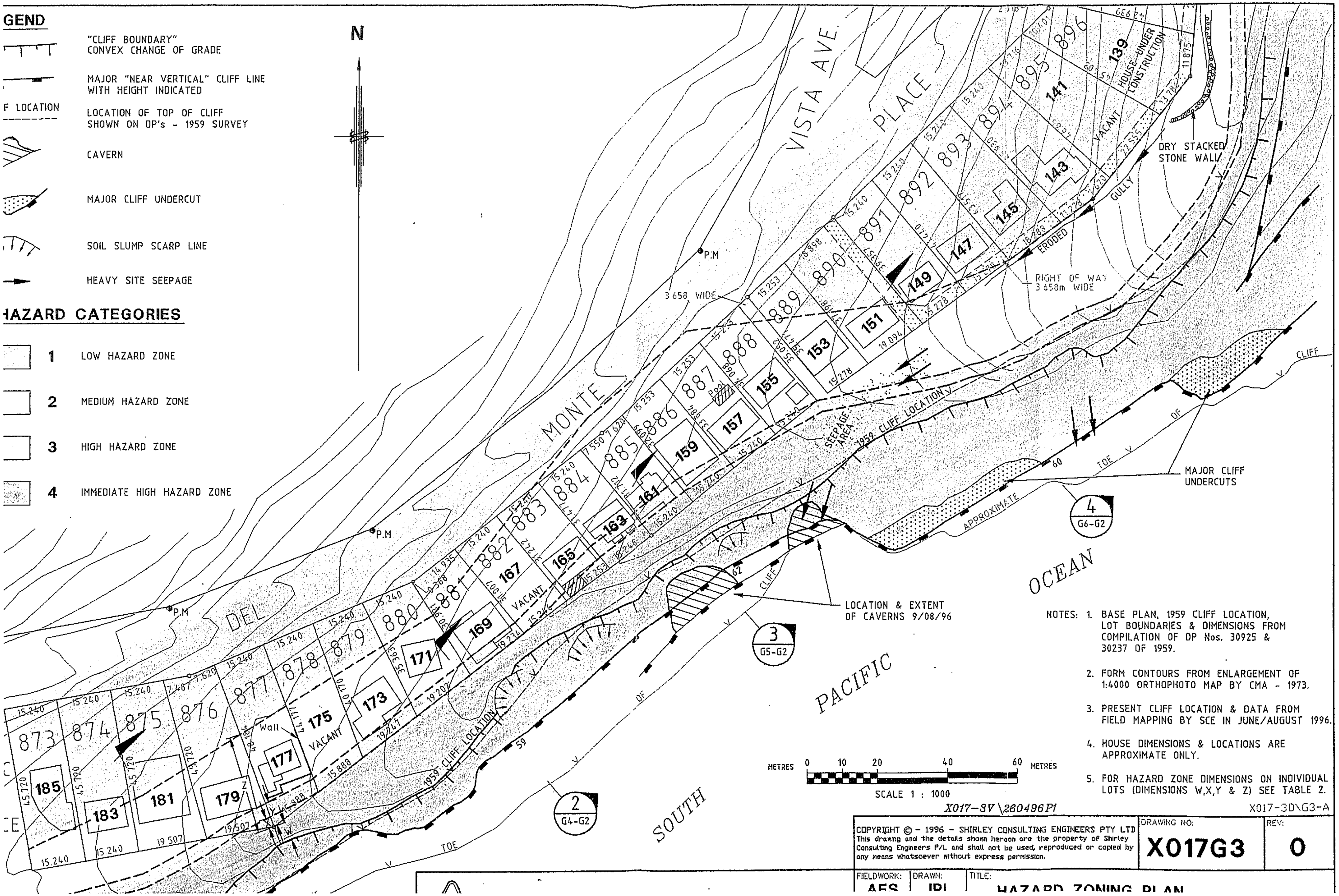
CLIFF LINE HAZARD DEFINITION STUDY - TUDIBARING HEADLAND
HEADLAND & CLIFF AREA PHOTOGRAPHS

P.16.

SECTION 1. - VIEW OF SURFICIA
SOIL UNDERCUT BY SOFT ROCK
WEATHERING.



P.17. - SECTION 4. - VIEW OF SOFT CLAYSTONE WEATHERING
ABOVE UNIT "D".
NOTE: ALSO SEEPAGE.



GEND

"CLIFF BOUNDARY" CONVEX CHANGE OF GRADE

MAJOR "NEAR VERTICAL" CLIFF LINE WITH HEIGHT INDICATED

F LOCATION — LOCATION OF TOP OF CLIFF SHOWN ON DP's - 1959 SURVEY

CAVERN

MAJOR CLIFF UNDERCUT

SOIL SLUMP SCARP LINE

HEAVY SITE SEEPAGE

HAZARD CATEGORIES

1 LOW HAZARD ZONE

2 MEDIUM HAZARD ZONE

3 HIGH HAZARD ZONE

4 IMMEDIATE HIGH HAZARD ZONE

NOTES:
1. BASE PLAN, 1959 CLIFF LOCATION, LOT BOUNDARIES & DIMENSIONS FROM COMPILATION OF DP Nos. 30925 & 30237 OF 1959.
2. FORM CONTOURS FROM ENLARGEMENT OF 1:4000 ORTHOPHOTO MAP BY CMA - 1973.
3. PRESENT CLIFF LOCATION & DATA FROM FIELD MAPPING BY SCE IN JUNE/AUGUST 1996.
4. HOUSE DIMENSIONS & LOCATIONS ARE APPROXIMATE ONLY.
5. FOR HAZARD ZONE DIMENSIONS ON INDIVIDUAL LOTS (DIMENSIONS W,X,Y & Z) SEE TABLE 2.

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X017G3	0

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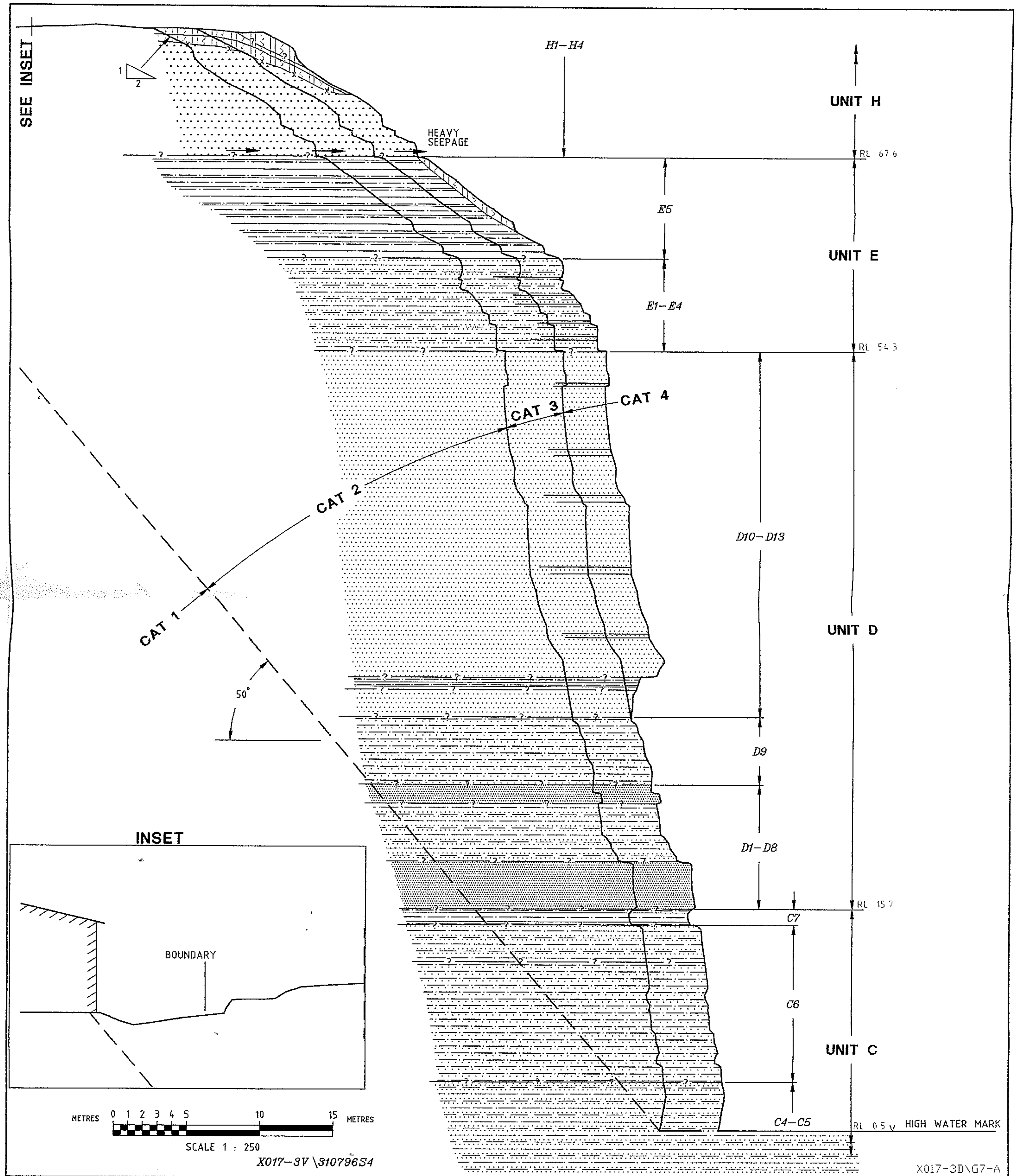

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210896\X017-5

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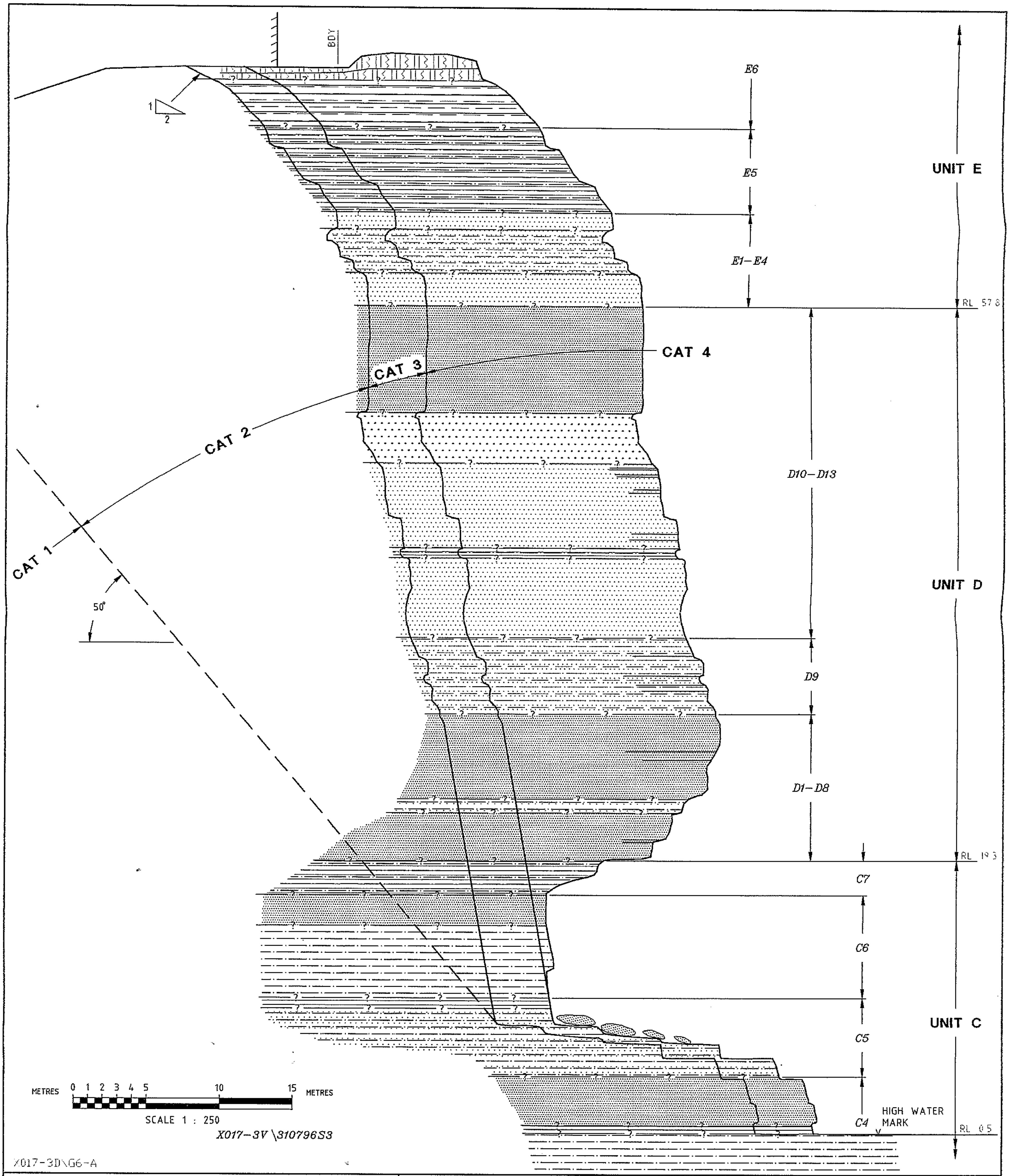
GEOLOGICAL SECTION 4
CLIFF LINE HAZARD DEFINITION STUDY
TUDIBARING HEADLAND, COPACABANA

CLIENT:
GOSFORD CITY COUNCIL



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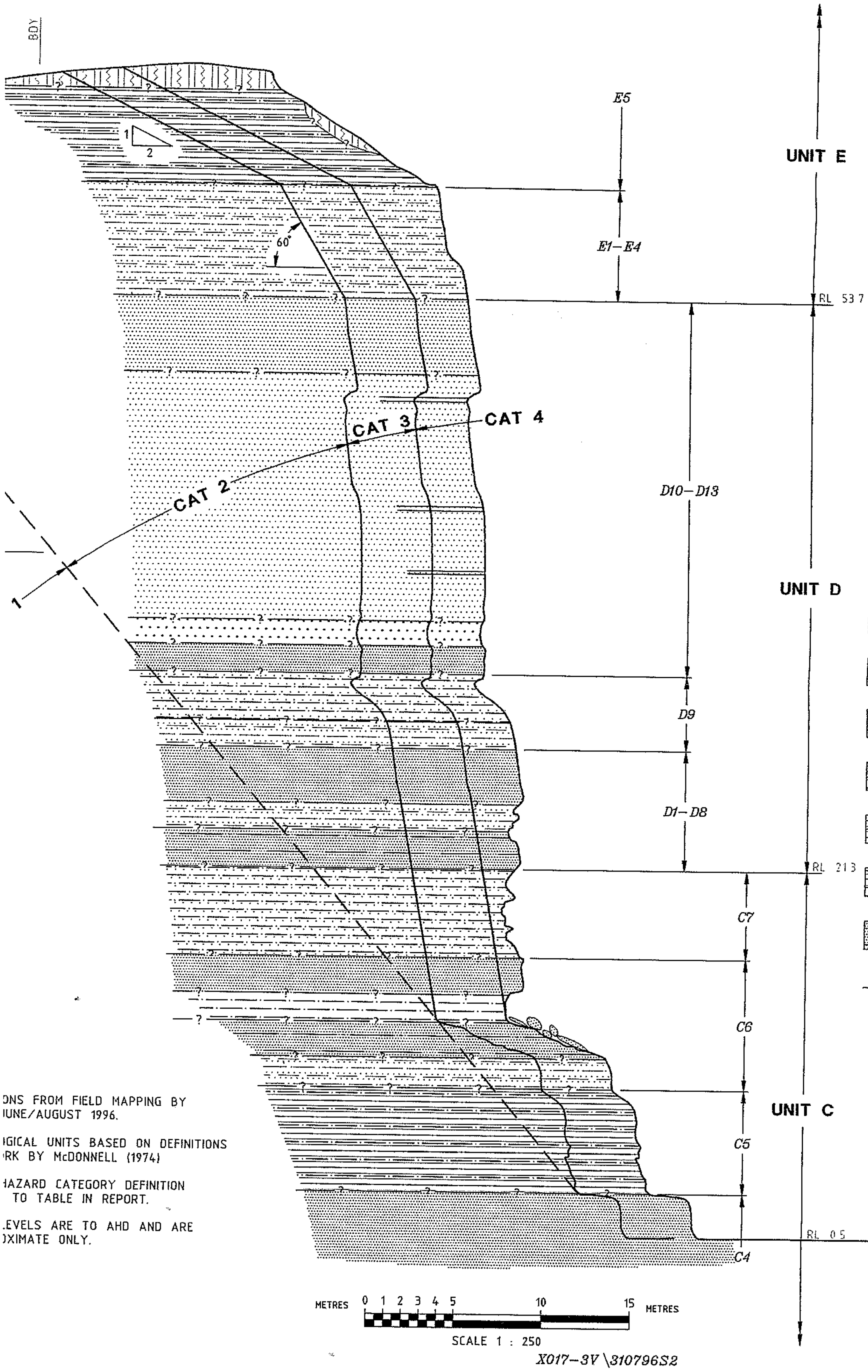

LEGEND

- SURFICIAL SOILS (LOAM, SANDY CLAY)
- FINE - FINE/MEDIUM GRAINED SANDSTONE
- MEDIUM GRAINED SANDSTONE
- MEDIUM/COARSE - COARSE GRAINED SANDSTONE
- SILTSTONE
- CLAYSTONE
- INTERBEDDED SANDSTONE & SILTSTONE
- SILTSTONE WITH MINOR SANDSTONE BEDS
- CLAYSTONE WITH MINOR SILTSTONE BEDS
- SILTSTONE/CLAYSTONE
- SILTSTONE WITH MINOR CLAYSTONE BEDS
- SANDSTONE WITH MINOR SILTSTONE BEDS
- —?— INFERRED GEOLOGICAL BOUNDARY

ONS FROM FIELD MAPPING BY
IUNE/AUGUST 1996.

IGICAL UNITS BASED ON DEFINITIONS
RK BY McDONNELL (1974)

HAZARD CATEGORY DEFINITION
TO TABLE IN REPORT.

EVELS ARE TO AHD AND ARE
XIMATE ONLY.

X017-3D\G5-A

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210896RN\X017-5

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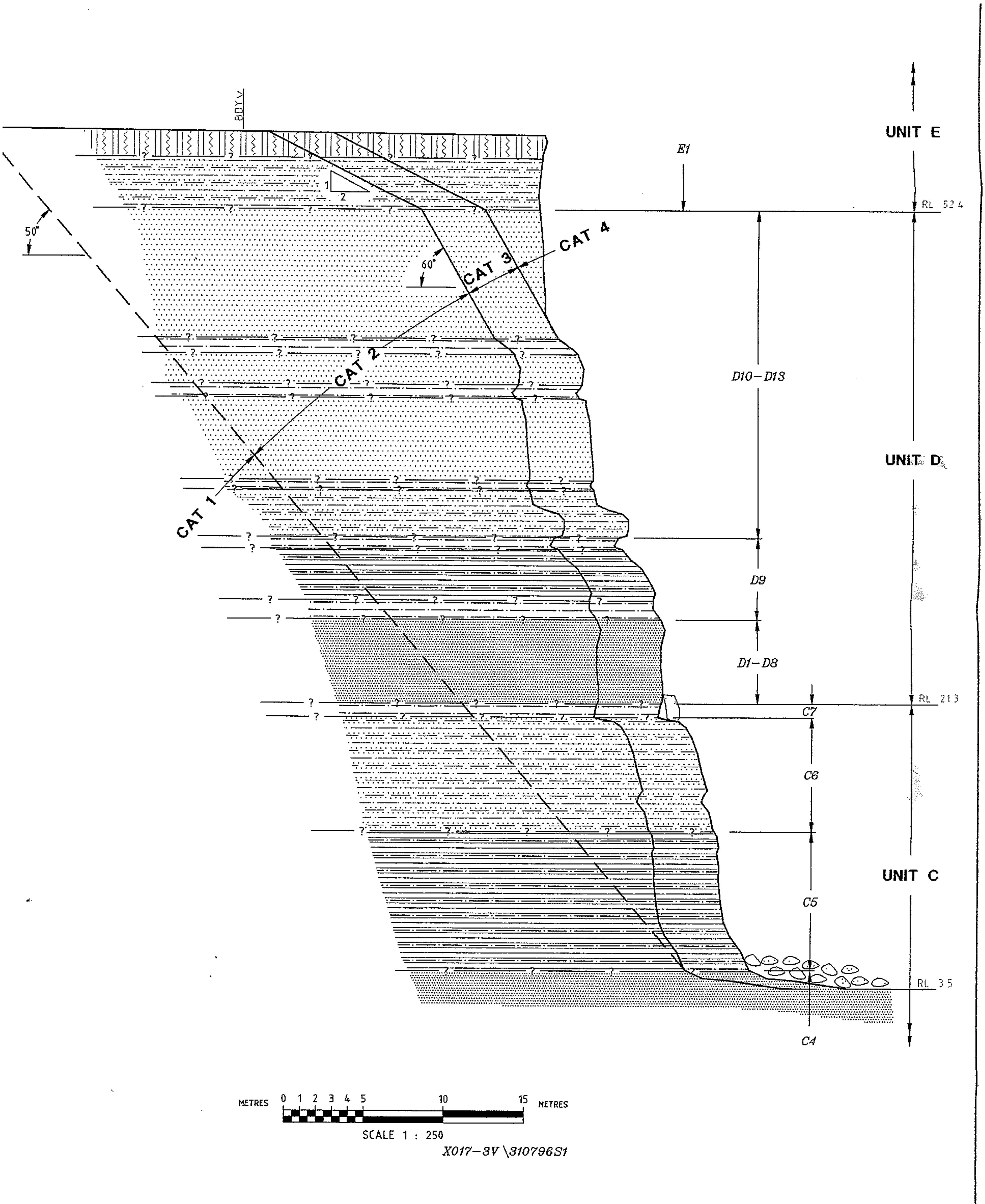
DATE: 07-08-1996		CLIENT: GOSFORD CITY COUNCIL



X017-3D\G4-A

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	DATE: 07-08-1996		CLIENT: GOSFORD CITY COUNCIL

CLIFF LINE HAZARD DEFINITION STUDY - TUDIBARING HEADLAND, COPACABANA

TABLE 2 - HAZARD CATEGORY ZONES & CLIFF DIMENSIONS

LOT No.	*STREET No.*	DIMENSIONS (m) FROM SEAWARD LOT BOUNDARY TO -			
		CLIFF BOUNDARY (w)	CATEGORY 1/2 (z)	CATEGORY 2/3 (y)	CATEGORY 3/4 (x)
Park					
		25.5	(6.0)	9.0	13.0
873	*185*				
		24.3	(10.0)	7.6	11.6
874	*183*				
		24.0	(14.0)	5.4	9.4
875	*181*				
		15.2	(20.5)	0.0	4.0
876	*179*				
		5.2	(27.3)	(6.8)	(2.8)
877	*177*				
		9.8	(25.6)	(4.0)	0.0
878	*175*				
		16.0	(24.0)	(4.0)	0.0
879	*173*				
		18.8	(21.5)	(4.0)	0.0
880	*171*				
		11.5	(19.5)	(4.0)	0.0
881	*169*				
		4.5	(24.0)	(4.0)	0.0
882	*167*				
		6.1	(27.7)	(6.0)	(2.0)
883	*165*				
		9.1	(31.7)	(8.0)	(4.0)
884	*163*				
		9.0	(35.4)	(8.0)	(4.0)
885	*161*				
		9.0	(37.4)	(7.9)	(3.9)
886	*159*				
		15.0	(37.9)	(5.5)	(1.5)
887	*157*				
		17.0	(38.2)	(3.4)	0.6
888	*155*				
		17.8	(28.7)	0.0	4.0
Easement					
		18.3	(26.6)	1.5	6.5
889	*153*				
		18.5	(18.6)	8.2	12.2
890	*151*				
		25.6	(10.0)	15.2	19.2
Right of Way					
		26.7	(7.8)	16.3	20.3
891	*149*				
		29.5	(3.8)	20.8	24.8
892	*147*				
		33.0	0.0	22.2	26.2
893	*145*				
		35.0	0.0	25.2	29.2
894	*143*				
		32.5	0.0	23.4	27.4
895	*141*				
		29.5	0.0	20.7	24.7
896	*139*				
		20.7	0.0	10.8	14.8
897	*138*				

Notes: 1. Dimensions are to the south, or in a seaward direction. Dimensions shown in brackets are to the north or towards Del Monte Place.

2. Dimensions shown are along the line of the common Boundary of adjoining Lots.

3. Refer to Dwg. No.X017G3 for explanation of terms w,x,y & z used in this table.

Our Ref: 020996TB.XLS/X017-5
Prepared:27/09/96

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DESIGN OF BUILDINGS FOR MINE SUBSIDENCE

FROM: "Conference on Buildings & Structures subject to Mine Subsidence
28-30 August, 1988 - Maitland NSW
I E Aust (Newcastle) & Mine Subsidence Board of NSW

Design of Buildings for Mines Subsidence

SUMMARY

Mine Subsidence is a factor in the design of building in many areas of New South Wales. This paper examines damage to buildings which have not been designed for mine subsidence and presents field data as the basis for design criteria. Procedures for the design of mine subsidence resistant buildings are developed and typical details presented. For the case of low rise buildings in areas where ground curvature associated with mine subsidence is gentle, the most important aspect of design is shown to be the isolation of the superstructure from horizontal ground strains. Designers are encouraged not to impose restrictions on the use of conventional building materials and construction techniques, but rather to concentrate on design of the structure to minimise forces induced by ground strains and detailing of finishes to accommodate ground curvature.

1 INTRODUCTION

Successful building design involves the understanding and consideration of all factors which affect a buildings' function. Mine subsidence is one factor which is important in many areas of New South Wales.

This paper examines damage to buildings which have not been designed for mine subsidence and presents data from field investigations which is used as the basis for design criteria.

Design procedures for mine subsidence resistant buildings are developed and typical details examined. Particular reference is made to the design of a single storey institutional building for mine subsidence and the associated cost premium.

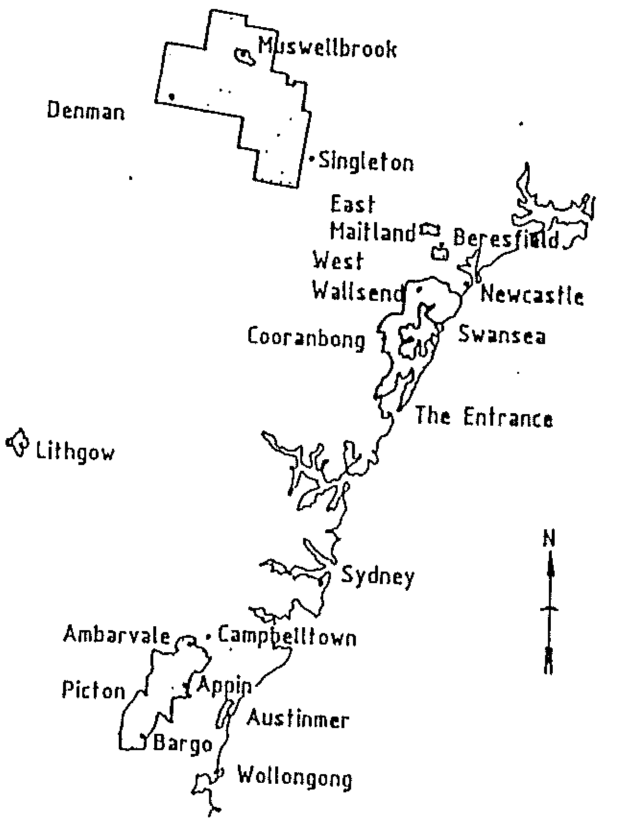
2 MINE SUBSIDENCE IN NEW SOUTH WALES

The two government bodies responsible for the control of underground mining and mine subsidence in New South Wales are the Department of Mineral Resources and the Mine Subsidence Board. The Department of Mineral Resources is responsible for the management of the development of coal resources in New South Wales, including the granting of mining leases. The Mine Subsidence Board, which is constituted under the Mine Subsidence Compensation Act, 1961, is responsible for providing compensation where buildings, roads and other surface structures are damaged by mine subsidence.

The Board has the ability to limit its liability by the declaration of mine subsidence districts within which it has control over the extent and nature of development. At present, there are 21 declared Mine Subsidence Districts in New South Wales. The location and extent of these districts are shown in figure 1.

Any proposed surface development located within a Mine Subsidence District requires formal approval by the Mine Subsidence Board, to qualify the development for compensation for damage caused by

For small structures, such as houses, the Board will recommend acceptable footing and superstructure details to assist the owner gain approval for the proposed development. For larger more complicated structures the Board will supply the necessary design subsidence parameters to allow designers of the proposed development to accommodate surface movements caused by mine subsidence and thereby gain Board approval.



3 BUILDING DAMAGE

3.1 General

Mine Subsidence is movement of the ground surface as a result of underground mining of coal or other materials. Parameters associated with these movements are subsidence, tilt, curvature, horizontal displacement and horizontal ground strain (refer Figure 2). Of these five subsidence parameters it is horizontal ground strains and ground curvatures which will generally cause damage to buildings.

Subsidence, tilt, and horizontal displacement are rigid body movements of the ground which do not induce significant stresses in the footings or superstructure. Ground tilt does not damage normal buildings but may affect the function of particular buildings or building elements. Tilt, for example, may result in doors not remaining open or closed without being restrained. Hydraulic services are affected by ground tilt as it causes a change in the effective fall or slope of those services.

3.2 Horizontal Ground Strains

Horizontal ground strains are the cause of most mine subsidence damage.

Tensile strains are characterised by open cracking in brickwork and concrete, tearing of wall and roof sheeting and fracturing of in-ground services.

Compressive strains result in shear failure of brickwork and concrete where one portion of a wall or slab is pushed above or below a restrained element, buckling of wall & roof sheeting and squeezing in of voids such as wall cavities and under floor drainage spaces.

The degree of damage caused by horizontal strains is dependent on the materials, shape, age and design of buildings concerned, and to date only general rules have been formulated from available field investigations. As the result of experience gained over many years and from the compilation of many field investigations, the Subsidence Engineer's Handbook (10) presents the classification of subsidence damage reprinted in this paper as Table I.

The factors of intensity of strain and the length of structure used in compiling the classification have been found to give a general guide in the prediction of damage intensity. The graph presented as Figure 3 may be usefully employed in conjunction with Table I when attempting to predict damage which may occur as the result of horizontal ground strains.

Buildings are damaged by the horizontal forces acting on them as a result of the horizontal ground strains. Horizontal forces act on a structure in two basic ways. Firstly, horizontal forces are generated in the footings of a building due to friction and adhesion between the ground and the footings. Secondly, passive earth pressures act on the side of the structure in the ground. The greater the embedment of the structure in the ground, the greater the forces acting on it and the greater their eccentricity (Refer to Figure 4). When the horizontal forces acting on the building exceed its capacity to resist those forces, severe cracking and distress will result.

As stated previously horizontal displacements are rigid body movements and, as such, do not induce stresses in individual buildings. It is, however, worth considering the effects of horizontal movements on structures such as pavements and covered ways which link individual buildings.

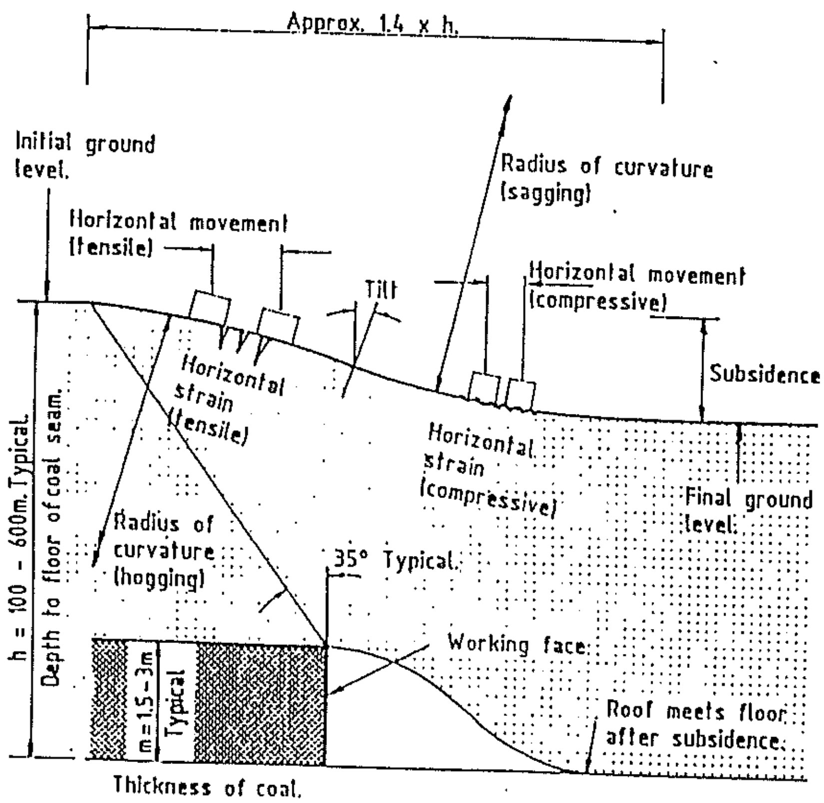


Figure 2 Mine Subsidence and Subsidence Parameters

TABLE I

NATIONAL COAL BOARD
CLASSIFICATION OF SUBSIDENCE DAMAGE

Class of Damage	Description of Typical Damage
1 Very slight	Hair cracks in plaster. Perhaps isolated slight fracture in the building, not visible on outside.
2 Slight	Several slight fractures showing inside the building. Doors and windows may stick slightly. Repairs to decoration probably necessary.
3 Appreciable	Slight fracture showing on outside of building (or one main fracture). Doors and windows sticking; service pipes may fracture.
4 Severe	Service pipes disrupted. Open fractures requiring rebonding and allowing weather into structure. Window and door frames distorted; floors sloping noticeably; walls leaning or bulging noticeably. Some loss of bearing in beams. If compressive damage, overlapping of roof joints and lifting of brickwork with open horizontal fractures.
5 Very Severe	As above, but worse, and requiring partial or complete rebuilding. Roof and floor beams lose bearing and need shoring up. Windows broken with distortion. Severe slopes on floors. If compressive damage, severe buckling and bulging of the roof and walls.

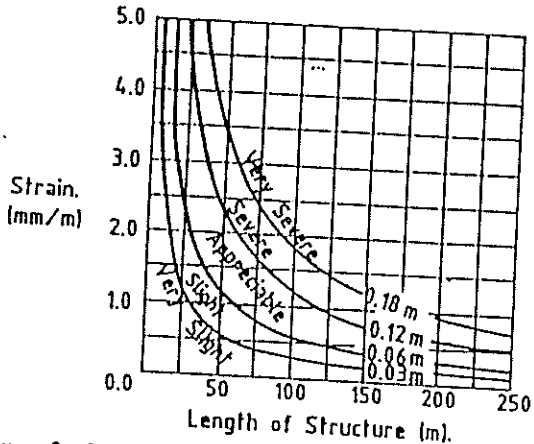


Figure 3 Relationship of Damage to Length of Structure and Horizontal Ground Strain.

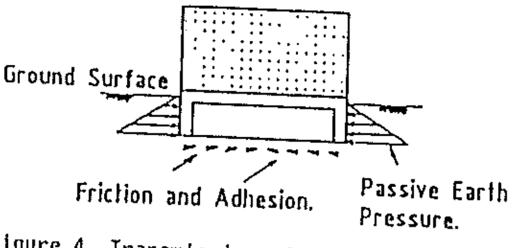


Figure 4 Transmission of Horizontal Forces

Figure 5 shows how buildings tend to move toward each other as a result of compressive ground strains and clearly indicates problems which may occur.

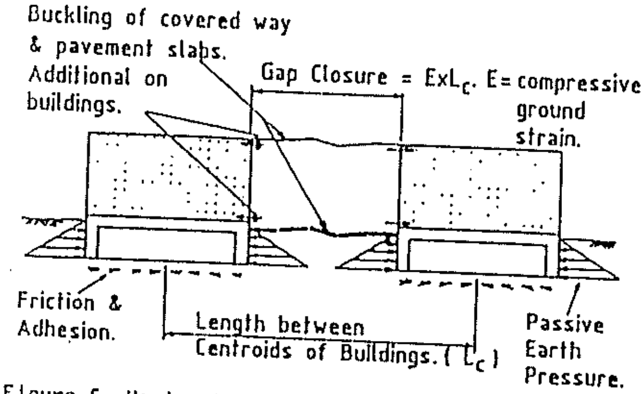


Figure 5 Horizontal Movement of Buildings due to Compressive Ground Strains.

3.3 Ground Curvature

Mine subsidence ground curvatures cause damage to buildings which is similar to that caused by better known examples of ground curvature such as foundation settlement and swelling/shrinking of reactive foundations. Differential movements associated with ground curvatures generally do not damage the structural elements of a building. However, non-structural elements such as external cladding and internal partitions are particularly prone to damage because they have limited flexibility to accommodate in-plane movements.

The analysis of a large amount of field data on damage due to all kinds of ground curvature has resulted in the formulation of relationships between the occurrence of damage in various types of wall construction and the deflection ratio (Refer Figure 6 for definition).

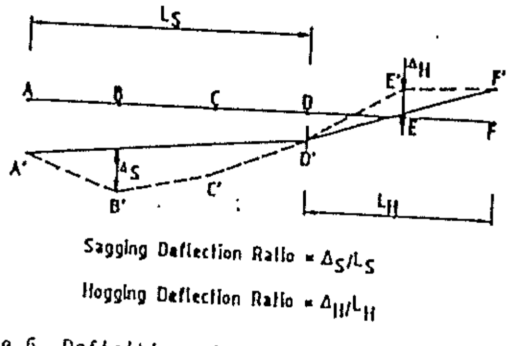
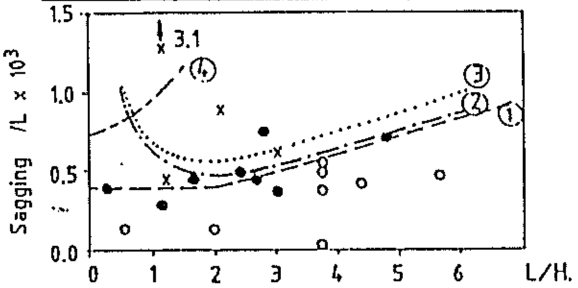


Figure 6 Definition of Deflection Ratio Δ/L

Polshin and Tokar (11) in 1957 introduced the concept of relating the onset of visible cracking in walls to a specific tensile strain, dependent on the material involved. They suggested a value of 0.05 per cent for unreinforced brick walls. Burland and Wroth (2) in 1975 extended this principle to take account of bending and shear strains, and adopted a critical tensile strain of 0.075%. The resulting relationships between deflection ratio, the Length/Height ratio and the onset of visible cracking shows some agreement with the observed cases of cracked and uncracked walls as shown in Figure 7.

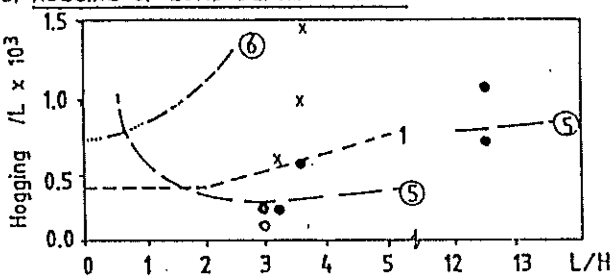
a) SAGGING OF LOAD BEARING WALLS



①--Polshin & Tokar
②---Burland & Wroth - Point load midspan - Bending Mode
③·····Burland & Wroth - Uniformly distributed load - Bending mode
④---Burland & Wroth - Point load midspan - Shear Mode

Δ/L = Deflection Ratio.
L/H = Length/Height of wall

b) HOGGING OF LOAD BEARING WALLS.



①--Polshin & Tokar
⑤---Burland & Wroth - Point loads at ends -Bending Mode
⑥---Burland & Wroth - Point loads at ends -Shear Mode.

Key
o No Damage.
• Slight Damage.
x Substantial Damage.

Figure 7 Relationship between Δ/L & L/H for Buildings showing various degrees of Damage (after Burland & Wroth, 1975).

Burland and Wroth also made the important distinction between cracking due to hogging and sagging curvatures, allowable deflection ratios for hogging curvatures being approximately half those for sagging curvatures. This conclusion, based on evidence from a small number of field cases, is explained by the fact that in a sagging mode the tensile effects at the lower part of the structure (footings) are to some extent restrained by shear stresses at the footing/brick wall interface whilst in a hogging mode such restraint is absent. Their work has produced allowable deflection ratios of 1/2000 for a length/height ratio of 2 and 1/1250 for a length/height ratio of 5.

All the foregoing criteria related to visible cracking and damage must be viewed in relation to aesthetic, serviceability and functional requirements of a structure. It must be remembered that the onset of visible cracking does not necessarily represent a limit state and thus, the design requirements for a structure will vary in different situations.

4 DESIGN OF BUILDINGS

4.1 Design Data and Philosophy

In New South Wales the Mine Subsidence Board will supply the following subsidence parameters for the design of buildings in declared mine subsidence districts:

Maximum Subsidence
Maximum Tensile Strain
Maximum Compressive Strain
Maximum Ground Slope
Subsidence and Strain Profiles

From the subsidence profile the designer can easily calculate the maximum ground curvature for design purposes.

The basic design philosophy for buildings subject to mine subsidence is that the structural and non-structural elements of the building must be designed to either resist or accommodate the movements and forces generated by subsidence.

The importance of each subsidence parameter must be assessed for the particular building under consideration and appropriate design action taken.

4.2 Ground Subsidence

Ground subsidence is a rigid body movement and as such will generally not influence the design of individual buildings.

This movement becomes an important design parameter only when a development is sensitive to relative vertical displacement of its elements. The performance of installations such as water & sewerage drainage systems are particularly sensitive to relative ground levels and care needs to be taken in the selection of minimum design falls for gravity operation of the system.

4.3 Ground Strains

Horizontal ground strains, both tensile and compressive, are particularly important in the design of individual buildings.

If a building is rigidly connected to footings which are embedded in the ground, then it is possible to analyse the effect of horizontal ground strains on the building. Such analysis generally shows that the building will suffer extensive damage and possible collapse as a result of these strains. Figure 8 graphically illustrates the problem of transmitting horizontal ground strains to the structure.

Experience has shown that two successful approaches to design of a building for minimal damage due to horizontal ground strains are:

1) Isolate the superstructure from the horizontal ground strains by designing the substructure to resist the forces imposed by the ground strain.

2) Divide the building into a number of discrete units to minimise the effect of the ground strains on each unit.

In most cases a combination of these two approaches will produce the most economical solution to the problems associated with horizontal ground strains.

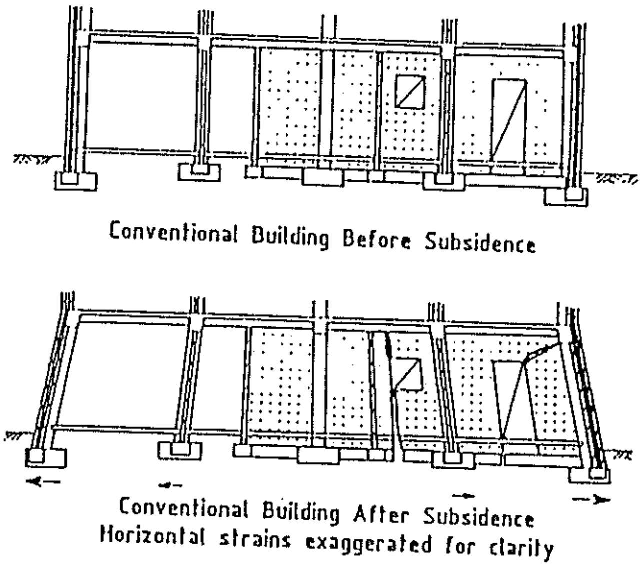


Figure 8 Diagram showing the effects of Horizontal Ground Strains on a Conventional Building.

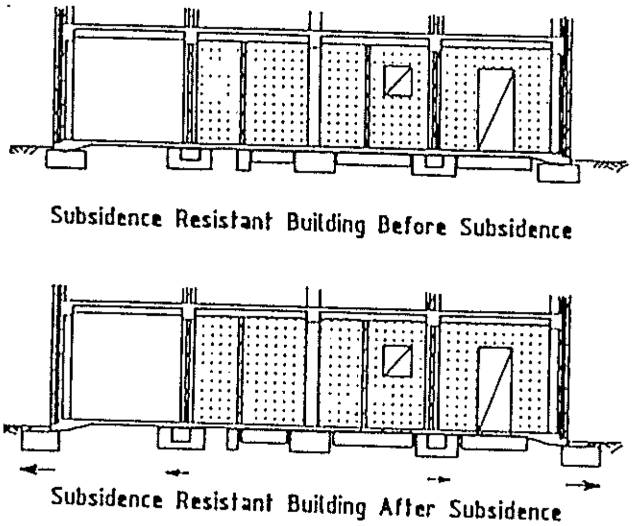


Figure 9 Diagram showing the effects of Horizontal Ground Strains on a Subsidence Resistant Building.

Final design will depend on the relative importance of the following factors:

1) The type of building. For example, is the building a one/two storey brick residential building or a multi-storey framed building?

2) The building's function and thus the sensitivity of the occupants or activities of the building to damage.

3) The site conditions, i.e. are the foundations stable or unstable and is the site sloping or level?

4) The intensity of the horizontal ground strains.

5) Cost.

The application of dividing a building into discrete units is illustrated by the following. Recently, our office was engaged to design and document a number of two storey loadbearing brick buildings for a residential development located in a Declared Mine Subsidence District on the Central Coast. The typical block consisted of two units on each floor located each side of a central stair well. The blocks had a maximum overall length of approximately thirty metres.

The design horizontal ground strain for the site was ± 0.5mm/m, which would produce a maximum differential horizontal movement at each end of the roughly symmetrical buildings of 30/2 x 0.5 = 7.5mm. We believed this movement to be undesirable for a loadbearing brick building supported on strip footings, and decided to split the buildings into two, by providing a joint through the entire structure on one side of the stair well, reducing the maximum differential horizontal movement to 4mm which we regarded as acceptable. The joint was simple to document both structurally and architecturally and consequently simple and inexpensive to construct, yet significantly reduced the risk of damage due to mine subsidence horizontal ground strains.

Where it is not practical to break the building into small units, it is necessary to isolate the superstructure from horizontal ground strains and to design the substructure to resist the forces induced by these strains.

The most effective design will minimise the magnitude of the horizontal forces and the eccentricity of those forces on the building. This can be achieved by:

1) Eliminating or significantly reducing the magnitude of the passive earth pressures acting on the building by:

   (a) Not having building elements deeply embedded in the ground, or by allowing elements in the ground to move with the ground.
   (b) By allowing the ground to move relative to elements in the ground without developing significant passive forces.

2) Minimising the frictional forces between the ground (or elements in the ground) and the structure by either:

   (a) Provision of a sliding layer between the building and the ground or elements in the ground.
   (b) Dividing the building into sections and thereby reducing the overall weight of each section and hence the frictional forces.

The simplest and most cost-effective method of satisfying these objectives is to support the building on a 'continuous' slab which is designed to resist the forces induced by horizontal ground strains. The effectiveness of this method in eliminating damage due to horizontal ground strains is illustrated in Figures 8 and 9.

Use of a 'continuous' slab system is not limited to good foundation conditions as it can be easily adapted to most conventional footing systems. Figures 10 and 11 present a comparison of structural details for conventional footings and mine subsidence resistant footings.

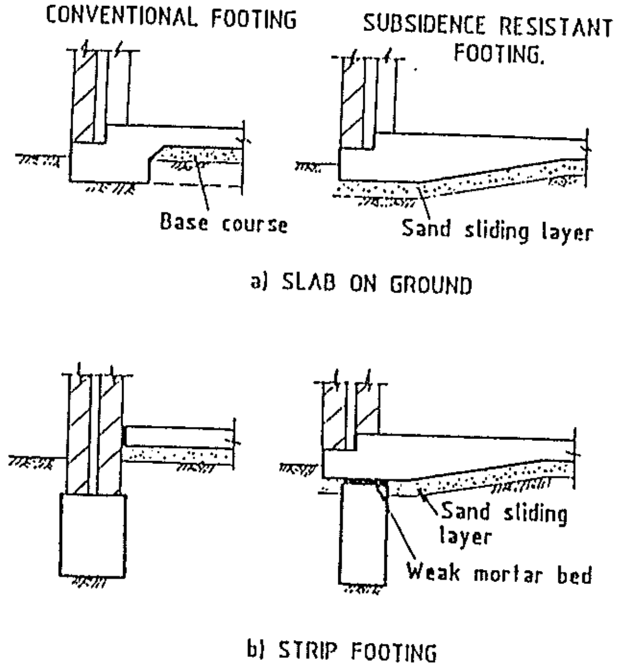


Figure 10
Comparsion of Conventional Footings and Subsidence Resistant Footings - Non-Reactive Ground Conditions

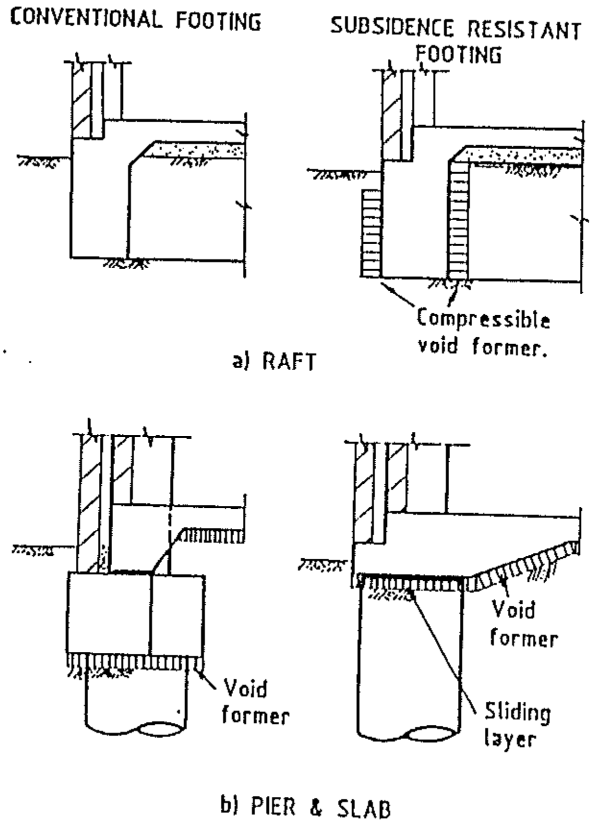


Figure 11
Comparsion of Conventional Footings and Subsidence Resistant Footings - Reactive Ground Conditions

The design of the 'continuous' slab requires consideration be given to:

1) Design for Horizontal Ground Strain Induced Forces
2) Design for Control of Thermal and Shrinkage Cracking

Tensile and compressive forces are induced in the slab by friction. The maximum force which can be developed is $\mu W/2$ where μ is the coefficient of friction and W is the total design load of the building. The design coefficient of friction for a sand sliding layer and a concrete slab without projections is 0.67. It is important to note that for practical purposes the design force is independent of the horizontal ground strain intensity. Geddes (7) provides a complete explanation of the development of friction forces. The concrete in the continuous slab will resist the compressive forces and buckling of the slab will be prevented by self weight and other gravity loads. The slab will need to be reinforced or prestressed to resist tensile forces.

For a 'continuously reinforced' slab simple equations are used to determine the reinforcement required to resist ground strain induced tensile forces, and to control thermal and shrinkage crack widths as outlined in Reference 4.

The design of a prestressed slab to resist stresses due to ground strains, applied loads, concrete shrinkage and differential temperatures can be performed using the principles outlined in Reference 12.

4.4 Ground slope

Ground slope associated with mine subsidence is generally a small and temporary problem to surface developments.

As mining progresses and the subsidence wave passes any point, the ground surface will return to its original slope. While the development is on the active portion of the subsidence wave (Refer Figure 2) it will slope and steps may need to be taken to accommodate the slope.

For buildings and functions which are particularly sensitive to variation in slope such as satellite earth stations and terrestrial radio systems, it may be necesary to provide mechanisms for re-alignment.

4.5 Ground Curvature

In the design of any building it is necessary to ensure that the foundation movements transmitted to the superstructure are within the allowable deformation limits of building elements. Mine subsidence ground curvature is one additional factor which contributes to foundation movements.

Previous discussion of building damage caused by ground curvature showed that relationships exist between the type of wall construction and allowable deflection ratios for satisfactory performance of that type of wall. Designing for ground curvature involves relating the mine subsidence and foundation ground curvature information, to the size of the building and the allowable deflection ratio for a particular type of wall construction.

Where footing movements due to foundation settlement or reactive ground are small relative to the mine subsidence induced ground curvatures it is possible to relate the minimum radius of mine subsidence ground curvature to the maximum tolerable deflection ratio (Δ/L) for different types of wall construction and different lengths of structure. Deflection ratio is related to the radius of curvature in accordance with the formula presented in Figure 12.

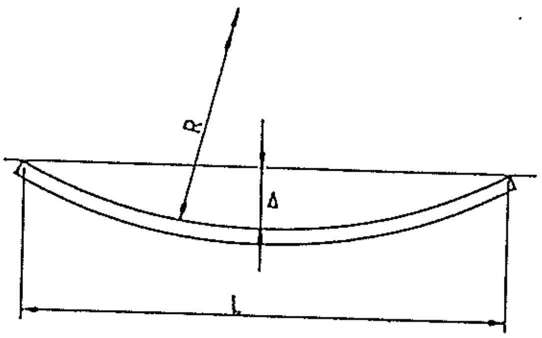
In Table II this formula has been used to relate allowable deflection ratios for various types of wall construction to limiting radii of curvature for 15m and 30m long structures.

Samples of predicted mine subsidence radii of curvature are:

Rosemeadow (South Campbelltown Mine Subsidence District)
- Radius of Curvature 8km

Appin (Appin Mine Subsidence District)
- Radius of Curvature 9km

By comparing these sample radii of curvature with the limiting radii of curvature for different types of wall construction it can be seen that mine subsidence induced ground curvature will often, by itself, not be a significant factor in the design of buildings.



$\frac{\Delta}{L} = \frac{L}{8R}$ (2)   Δ = Deflection
L = Length of Structure.
R = Radius of Curvature.

Figure 12 Deflection Ratio due to Ground Curvature

TABLE II

ALLOWABLE DEFLECTION RATIOS AND LIMITING RADII OF CURVATURE (R)

WALL CONSTRUCTION	ALLOWABLE DEFLECTION RATIO (14)	LIMITING R (km) L = 15m	L = 30m
A. <u>LOAD BEARING</u>			
SOLID MASONRY			
- Rendered	1/4000	7.5	15.0
- Face	1/3000	5.6	11.3
B. <u>NON-LOAD BEARING (or lightly loaded)</u>			
SOLID MASONRY			
- Rendered	1/2000	3.8	7.5
- Face	1/1500	2.8	5.6
ARTICULATED MASONRY			
- Rendered	1/800	1.5	3.0
- Face	1/500	0.9	1.9
MASONRY VENEER			
- Rendered	1/500	0.9	1.9
- Face	1/300	0.6	1.1
NON-MASONRY			
- Timber or prefab.	1/200	0.4	0.8

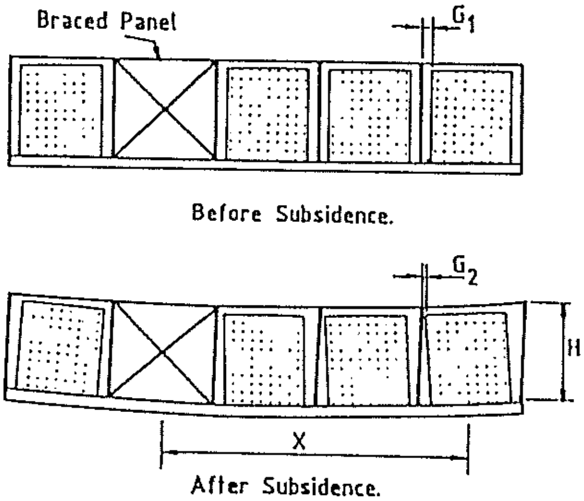
For buildings in areas where foundation movements caused by soil mosture variations or settlement occur, the mine subsidence ground curvature movements and foundation movements will be additive.

Framed structures, incorporating masonry infill panels, require special consideration to avoid damage due to 'racking'. Figure 13 illustrates this problem and relates to differential horizontal movement occuring in a panel to:

- The radius of ground curvature
- The distance between the braced panel and the panel under consideration
- The height of the panel, as shown in Figure 13.

The maximum differential horizontal movement for a building of length L, with a central braced panel is

$$G = \frac{LH}{2R} \quad (1)$$



Differential Horizontal Movement $= \Delta_1 = G_1 - G_2 = \frac{XH}{R}$

Figure 13 Differential Horizontal Movements in a Braced Frame due to Ground Curvature.

In a building 30m long, with a three metre storey height and a braced panel located in the centre of the structure, subjected to ground curvature with a radius of 9km, the maximum differential movement would be 5mm. This movement is additional to diffential movements due to 'normal' factors such as temperature, shrinkage and axial shortening, and it would be necessary to introduce simple articulation into the building by:

1) Providing flexible masonry ties between the columns and masonry.

2) Supporting the ceiling from the roof structure, and detailing the ceiling to allow movement of the ceiling relative to the walls.

Our experience has shown that gentle ground curvatures do not generate significant bending moments in concrete framed structures and special detailing of concrete joints is not required under these conditions.

5. CASE STUDY

5.1 General

In 1985, our office was engaged by the Public Works Department N.S.W., to design and document a special purpose school located in the South Campbelltown Mine Subsidence District. The School is now complete and to our knowledge no particular difficulties were encountered during construction of the subsidence resistant aspects of the buildings.

The following subsidence parameters were supplied by the Mine Subsidence Board:

Maximum Subsidence = 3 metres
Maximum Tensile Strain = 3mm/metre
Maximum Compressive Strain = 2.5mm/metre
Maximum Ground Slope = 1: 100
Ground curvature = 8km

The buildings are of single storey brick veneer construction with a braced steel roof frame supporting metal sheeting. The sloping site consisted of 1.5 metres of reactive clay soil overlaying siltstone. Due to the proposed function of the building the design brief specified the use of a concrete floor slab with limited variation in floor level. Balanced cut and fill was used resulting in the floor slab being located between zero and two metres above the siltstone.

5.2 Subsidence Resistant Design

A continuous concrete slab was provided to isolate the superstructure from the horizontal ground strains. Slabs on weak shale and moderate depths of reactive clay were designed as slabs on ground and the remainder were designed as suspended, supported on piers and poured on 50mm collapsible paperboard formwork. Refer Figure 14 & 15 for details. The slabs were generally 12m wide and a maximum of 30m long continuously reinforced with fabric to control shrinkage and temperature cracking and resist ground strain induced forces.

The superstructure was jointed at the slab control joints, and on each side of the joint was treated as an independent block: with independent walls on each side of the joint, cover plates provided to slab and walls, and flashing to the roofs to accommodate differential horizontal movements of each block.

Ground curvature was not critical. Based on a structure length of 30m and a radius of curvature of 8km the Δ/L for the structure was 1/2130, which compares favourably with an allowable deflection ratio of 1/500 for non-load bearing masonry veneer wall construction (Refer Table II).

For a structure height of 3m, and a braced structure length of 20m the maximum racking of the structure was 7.5mm. Brick joints were provided at 8 metre maximum centres and ceilings were detailed to be free of the walls to allow racking of the steel frame to occur without causing distress to the ceiling/wall junction.(Figure 16)

5.3 Cost Premiums

The Quantity Surveyor for the project estimated that the structural cost premium for the mine subsidence resistant design over a conventional design was one per cent of the total cost of the buildings. For buildings supported on a slab on ground the cost premium was only 0.6%. The main components of the premium being additional slab reinforcement, additional walls, cover plates and flashings at control joints. The cost premium for buildings supported on a suspended slab on piers was 1.6%.

The functional requirements of the school required a solution suited in many ways to a mine subsidence resistant design which resulted in the provision of a mine subsidence resistant building at a very small cost premium. Our office has undertaken other studies including two storey reinforced concrete framed buildings which indicate that a cost premium of approximately three per cent of the total cost of the building is typical for mine subsidence resistant design.

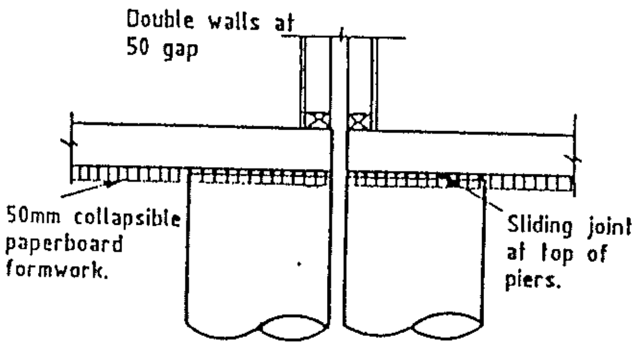


Figure 14 Pier & Suspended Slab at Control Joint

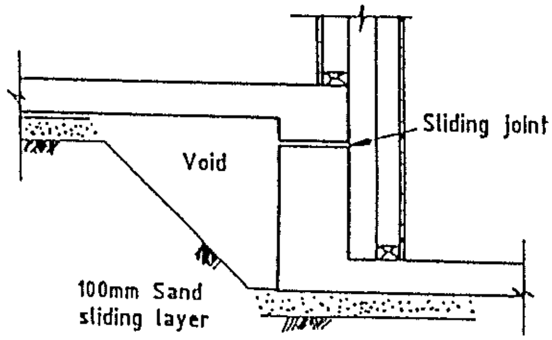


Figure 15 Slab on Ground at Stepped Control Joint

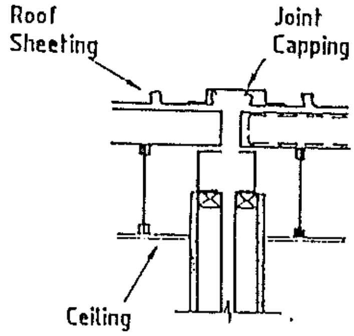


Figure 16 Roof Structure & Suspended Ceiling

6. CONCLUSION

Mine subsidence is a factor in the design of buildings in many areas of New South Wales. Areas which are planned to be developed and which will suffer ground movements as a result of coal mining have been proclaimed Mine Subsidence Districts. In these areas, in accordance with the Mine Subsidence Compensation Act 1961, proposed buildings require the approval of the Mine Subsidence Board to qualify for compensation for damage caused by mine subsidence.

Damage to buildings which have not been designed for mine subsidence is primarily the result of horizontal ground strains. These strains will cause cracking of walls and damage to finishes and services, and may lead to catastrophic failure of structures which have limited ductility.

Based on our experience in the design of one and two storey institutional type buildings in areas where the ground curvature associated with mine subsidence is gentle, the most important aspect of design is the isolation of the superstructure from horizontal ground strains. This can be achieved by supporting the superstructure on a continuous ground slab designed to resist friction forces induced by horizontal ground strains. Differential movements associated with ground curvature can be accommodated by simple detailing of walls and finishes.

It is recommended that designers do not impose restrictions on the use of conventional building materials and construction techniques. Benefit is obtained by concentration on design of the structure to minimize forces induced by ground strains and detailing of finishes to accommodate ground curvatures. The premium for one and two storey institutional type buildings in mine subsidence areas where the ground curvature is gentle is less than three per cent of total cost.

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